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Shear Behavior of Concrete Structures Strengthened with Ultra-High Performance Fiber-Reinforced Concrete (UHPFRC)

Yitao Huang

黄义涛

Invitation

You are cordially invited
to attend the public defence
of my PhD dissertation entitled

**Shear Behavior of Concrete
Structures Strengthened with
Ultra-High Performance
Fiber-Reinforced Concrete
(UHPFRC)**

on
Wednesday, June 25th, 2025

at
15:00

in
**Senate Hall of Aula
TU Delft Campus**

At 14:30, a brief introduction
on the topic of my thesis
will be given.

A reception will be held
after the defence.

Hope to see you there.

Yitao Huang

Shear behavior of concrete structures strengthened with ultra-high performance fiber- reinforced concrete (UHPFRC)

Yitao HUANG

Shear behavior of concrete structures strengthened with ultra-high performance fiber-reinforced concrete (UHPFRC)

Dissertation

for the purpose of obtaining the degree of doctor

at Delft University of Technology

by the authority of the Rector Magnificus, Prof.dr.ir. T.H.J.J. van der Hagen,

chair of the Board for Doctorates

to be defended publicly on

Wednesday, 25 June 2025, at 15:00 o'clock

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Keywords: Shear strengthening, ultra-high performance fiber-reinforced concrete (UHPFRC), interface behavior, active infrared thermography, electromagnetic method, numerical analysis.

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To my dear parents and elder brother

献给我亲爱的父母和哥哥

Summary

Due to aging, increase in service loads, and/or upgrade of design codes, many existing concrete structures do not, or soon will not satisfy the required load-bearing capacity. Therefore, measures such as reconstruction or repair are necessary. Among various strengthening methods, ultra-high performance fiber reinforced concrete (UHPFRC), due to its high mechanical properties and superior durability, has emerged as a promising solution for strengthening concrete structures by enhancing load and deformation capacity, and improving durability. While extensive research has been conducted on the use of UHPFRC for flexural strengthening, studies focused on shear strengthening remain limited. After strengthening, the interface between UHPFRC and traditional reinforced concrete (RC) structures is formed. However, interface behavior has not been thoroughly investigated, further posing a gap in understanding the optimal strengthening design of UHPFRC in hybrid UHPFRC-concrete systems.

This dissertation focuses on the shear performance of concrete structures strengthened with UHPFRC, with a focus on the interface behavior between UHPFRC and normal concrete, and the mechanical properties of UHPFRC.

To assess the shear strengthening efficiency of UHPFRC, this dissertation starts with a literature review (Chapter 2) that addresses three major aspects. The first aspect focuses on the shear performance of hybrid UHPFRC-RC structures. Strengthening applications of UHPFRC in shear, and current analytical and numerical methods to predict the shear capacity of RC structures strengthened with UHPFRC are critically analyzed. The second aspect is focused on the UHPFRC-concrete interface behavior which is governing the response of the hybrid beams. From the review, the role of governing parameters on the interface behavior, including the effects of bonding technique, moisture exchange between the two materials, differential shrinkage and the role of coupled environmental and mechanical loads, are discussed. The final aspect deals with the application of non-destructive techniques (NDTs) to assess the strengthening efficiency of UHPFRC, focusing on evaluation of (i) UHPFRC material properties and (ii) UHPFRC-concrete interface performance in hybrid structures.

Experimental design is presented in Chapter 3, where the material and structural tests are systematically introduced. This chapter provides a series of material tests to, among others, characterize the workability, shrinkage and mechanical properties of both UHPFRC and normal concrete (NC). It also introduces the design and setup of a comprehensive structural test to evaluate the shear performance of UHPFRC-strengthened RC beams. Following the experimental methodology from Chapter 3, Chapter 4 presents the results of the material and structural tests. Through comparative analysis, this chapter examines the material and structural behavior of UHPFRC-strengthened beams and its constituents

by varying different parameters, thereby setting a basis for evaluating the shear improvement in strengthened RC beams.

In order to provide a deeper analysis on the UHPFRC-concrete interface quality in strengthened beams, Chapter 5 focuses on the assessment of possible delamination between UHPFRC and existing concrete by applying active infrared thermography. Combined with both experimental and numerical analysis, a systematic procedure is developed to detect subsurface delamination in hybrid UHPFRC-NC specimens. Besides the interface properties, another governing parameter, namely the material properties of UHPFRC, is examined in Chapter 6. This chapter investigates the fiber distribution and orientation within UHPFRC elements, important factors influencing material properties of UHPFRC, and therefore further affecting its shear strengthening efficiency. Using a calibrated electromagnetic method and validated through x-ray computed tomography (CT scanning), the effect of governing parameters, including casting direction and vibration time, on fiber distribution and orientation in UHPFRC elements is investigated. This chapter helps to clarify the connection between the material properties of UHPFRC and its structural performance.

Based on the structural test results (Chapter 4), evaluation of interface properties (Chapter 5) and material properties of UHPFRC (Chapter 6), in Chapter 7, a numerical model is developed to simulate shear performance of UHPFRC strengthened concrete structures, validated by the experimental results. A parametric study is further conducted to investigate key factors including interface properties, UHPFRC tensile behavior and non-uniform fiber distribution in UHPFRC. The numerical analysis offers insights on the influence of these parameters on shear strengthening efficiency.

In the final Chapter 8, the findings regarding the overall shear strengthening performance of UHPFRC in RC beams are given, providing practical insights for optimizing UHPFRC applications in concrete structures. Finally, suggestions for future research are given.

Samenvatting

Door veroudering, toename van de gebruiksbelastingen en/of aanpassing van ontwerpcodes voldoen veel bestaande betonconstructies niet of binnenkort niet meer aan het vereiste draagvermogen. Daarom zijn maatregelen zoals reconstructie of reparatie noodzakelijk. Van de verschillende methodes voor constructieve versterking is ultrahoogwaardig vezelversterkt beton (UHPFRC), vanwege zijn hoge mechanische eigenschappen en superieure duurzaamheid, naar voren gekomen als een veelbelovende oplossing voor het versterken van betonconstructies door het verbeteren van de belastings- en vervormingscapaciteit en het verbeteren van de duurzaamheid. Er is uitgebreid onderzoek gedaan naar het gebruik van UHPFRC voor het verhogen van de buigcapaciteit, maar studies gericht op het verhogen van de afschuifsterkte blijven beperkt. Na versterking ontstaat er een interface tussen UHPFRC en traditioneel gewapend beton (RC). Het gedrag van de interface is echter nog niet grondig onderzocht, waardoor er een leemte ontstaat in het begrijpen van het optimale versterkingsontwerp van UHPFRC in hybride UHPFRC-beton-systemen.

Dit proefschrift richt zich op de afschuifprestaties van betonconstructies versterkt met UHPFRC, met een focus op het gedrag van de interface tussen UHPFRC en normaal beton en de mechanische eigenschappen van UHPFRC zelf.

Om de efficiëntie van het verhogen van de afschuifsterkte met UHPFRC te beoordelen, begint dit proefschrift met een literatuuronderzoek (Hoofdstuk 2) dat drie belangrijke aspecten behandelt. Het eerste aspect richt zich op de afschuifprestaties van hybride UHPFRC-RC-constructies. Versterkende toepassingen van UHPFRC in afschuiving en huidige analytische en numerieke methoden om de afschuifcapaciteit van RC constructies versterkt met UHPFRC te voorspellen worden kritisch geanalyseerd. Het tweede aspect is gericht op het gedrag van de interface tussen UHPFRC en beton, die de respons van de hybride liggers bepaalt. De rol van bepalende parameters op het gedrag van de interface, inclusief de effecten van de verbindingstechniek, vochtuitwisseling tussen de twee materialen, differentiële krimp en de rol van gekoppelde omgevings- en mechanische belastingen worden besproken. Het laatste aspect betreft de toepassing van niet-destructieve technieken (NDT) om de versterkingsefficiëntie met UHPFRC te beoordelen, waarbij de nadruk ligt op de evaluatie van (i) de materiaaleigenschappen van UHPFRC en (ii) de prestaties van de interface tussen UHPFRC en beton in hybride constructies.

Het experimenteel ontwerp wordt gepresenteerd in hoofdstuk 3, waar de materiaalkundige en constructieve experimenten systematisch worden geïntroduceerd. Dit hoofdstuk voorziet in een reeks materiaalproeven om onder andere de verwerkbaarheid, krimp en mechanische eigenschappen van zowel UHPFRC als standaard beton (NC) te karakteriseren, en introduceert het ontwerp en de opzet van een uitgebreide constructieve test om

de afschuifprestaties van UHPFRC-versterkte RC-balken te evalueren. Na de experimentele methodologie uit hoofdstuk 3, presenteert hoofdstuk 4 de resultaten van de materiaalkundige en constructieve testen. Door middel van een vergelijkende analyse onderzoekt dit hoofdstuk het materiaal- en constructieve gedrag van UHPFRC-versterkte liggers en hun onderdelen door verschillende parameters te variëren, waardoor een basis wordt gelegd voor het evalueren van de afschuiving van versterkte RC liggers.

Om een diepere analyse te geven van de kwaliteit van de UHPFRC-beton interface in versterkte liggers, richt hoofdstuk 5 zich op de beoordeling van mogelijke delaminatie tussen UHPFRC en bestaand beton door het toepassen van actieve infrarood thermografie. Gecombineerd met zowel experimentele als numerieke analyses wordt een systematische procedure ontwikkeld om delaminatie onder de oppervlakte te detecteren in hybride UHPFRC-NC proefstukken. Naast de interface-eigenschappen wordt in hoofdstuk 6 een andere bepalende parameter onderzocht, namelijk de materiaaleigenschappen van UHPFRC. Dit hoofdstuk onderzoekt de vezeldistributie en -oriëntatie binnen UHPFRC elementen, belangrijke factoren die de materiaaleigenschappen van UHPFRC beïnvloeden en daardoor ook de efficiëntie van het verhogen van de afschuifsterkte. Met behulp van een gekalibreerde elektromagnetische methode en gevalideerd door middel van röntgen computertomografie (CT-scanning), wordt het effect van bepalende parameters, inclusief stortrichting en trillingstijd, op de vezeldistributie en -oriëntatie in UHPFRC elementen onderzocht. Dit hoofdstuk helpt het verband te verduidelijken tussen de materiaaleigenschappen van UHPFRC en de constructieve prestaties ervan.

Gebaseerd op de constructieve testresultaten (Hoofdstuk 4), de evaluatie van de interface-eigenschappen (Hoofdstuk 5) en de materiaaleigenschappen van UHPFRC (Hoofdstuk 6), wordt in Hoofdstuk 7 een numeriek model behandeld dat is ontwikkeld om de afschuifprestaties te simuleren van UHPFRC versterkte betonconstructies, gevalideerd door de experimentele resultaten. Een parametrische studie wordt verder uitgevoerd om belangrijke factoren te onderzoeken, waaronder interface-eigenschappen, UHPFRC trekgedrag en niet-uniforme vezeldistributie in UHPFRC. De numerieke analyse biedt inzicht in de invloed van deze parameters op de efficiëntie van verhoging van de afschuifsterkte.

In het laatste hoofdstuk 8 worden de bevindingen met betrekking tot de algemene prestaties van UHPFRC voor het verhogen van de afschuifsterkte in RC balken gegeven, die praktische inzichten verschaffen voor het optimaliseren van UHPFRC toepassingen in betonconstructies. Tot slot worden suggesties gegeven voor toekomstig onderzoek.

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Chapter 1 Introduction

1.1 Research background

Concrete is the most used construction material in the world. However, due to increase of loads, ageing and possible deterioration, and more stringent requirements in design codes, the bearing capacity of existing concrete structures decreases and/or does no longer satisfy service demands [1,2]. Rehabilitation of existing concrete structures, usually implying a strengthening method to upgrade the capacity or repair to improve the durability, and therefore extend the service life of existing structures, is favorable to government and industries. Advantages include short 'out-of-use periods' of structure, limited traffic hindrance and cost effectiveness [3]. A common strengthening measure is to cast a layer of new concrete in the regions where principal stresses are the largest to improve the structural resistance. However, structures strengthened with normal concrete (NC) can suffer a decrease in their resistance again after only a short time [4]. Drawbacks also exist in other popular strengthening methods, such as difficulty in shaping of steel plate and its susceptibility to corrosion by applying steel jacketing, and premature debonding failure at the interface in strengthening with FRP [5]. These issues prompted researchers to look into the application of advanced cementitious materials for strengthening of structural concrete.

UHPFRC is a relatively new construction material with high compressive and tensile strength, as well as superior durability that is attractive for strengthening applications. Compared to normal concrete, UHPFRC has a dense and compact microstructure with low permeability due to the use of fine particles like silica fume, a low water-binder ratio and the addition of superplasticizer [6]. Consequently, UHPFRC shows high resistance against degradation processes such as carbonation, chloride ingress, sulfate attack and ingress of corrosive liquids [7–9], freeze-thaw cycles and thermal loads [10]. The low amount of coarse aggregates (in size and volume) and a high fiber content (normally 2 or 3% by volume) cause increased strength, stiffness and ductility. Moreover, its excellent rheological properties facilitate casting when the material is in the fresh state [11]. In terms of compressive strength, UHPFRC should have a characteristic cylinder compressive strength of at least 120 MPa according to Swiss norm [12] (or 150 MPa according to French recommendations [13]), which is significantly higher than that of NC (≤ 50 MPa). UHPFRC also has higher tensile strength and sometimes shows strain (and deflection) hardening behavior due to the effect of fibers, subsequently accompanied by a softening plateau and pullout of the fibers. Therefore, multiple cracks will be developing prior to a localized crack, which results in a ductile behavior with a significantly higher fracture energy compared to NC (i.e., generally several times higher, dependent on the UHPFRC mixture design). In addition, UHPFRC becomes a promising material to resist blast/impact loads due to its outstanding energy absorption capacities [14]. It has to be noted that although the higher cement content in UHPFRC compared to NC increases the CO₂ footprint of the material itself, the superior performance of UHPFRC may enable a longer

service life [15], the construction of slender structures, reduction of traffic hindrance [16] and can greatly reduce the needed amount of construction material, resulting in improved sustainability aspects [17]. Furthermore, in spite of higher material costs, the overall construction costs when UHPFRC is applied (Strategy A) can be even lower than that of NC as strengthening material (Strategy B) when reduction of material demand and traffic intervention, the long-term serviceability and decrease of future maintenance are taken into account (Figure 1.1 [4]).

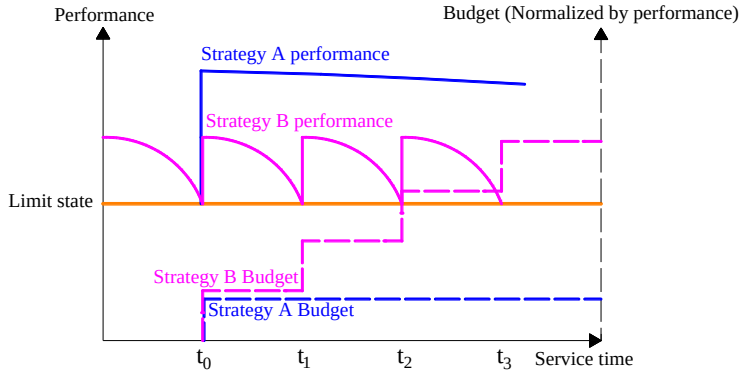


Figure 1.1 Two different strengthening strategies for service life maintenance: Strategy A (Structures strengthened with UHPFRC) and Strategy B (Structures strengthened with NC) (adapted from [4]).

Although UHPFRC is a promising strengthening material with numerous qualities, a few shortcomings limit its wider practical application. One of them is the lack of design codes [18]. Though efforts have been made to establish design standards such as AFGC [13] and JSCE [19] to facilitate the application of UHPFRC, most of available codes focus on the material properties characterization and design of new UHPFRC structures. Provisions on UHPFRC strengthening technologies are only available in the Swiss norm (SIA 2016) [12] but still limited. Therefore, more research in terms of the strengthening of existing reinforced concrete structures with UHPFRC is still in need.

For UHPFRC strengthening application, the majority of research to date has focused on flexural performances of reinforced concrete (RC) beams retrofitted by UHPFRC, which is summarized in [20]. This research has clarified that UHPFRC can significantly increase the flexural resistance (both load and deformation capacity) of RC beams, showing its potential as a strengthening material in flexure. However, there are still knowledge gaps with respect to shear performance of RC beams strengthened with UHPFRC, which is critical, especially considering its brittle nature. Only few studies are devoted to the analysis of shear response of UHPFRC-RC composite members [21–27], and most of them are limited to the influence of parameters including UHPFRC thickness, strengthening configurations, the volume of steel fibers in UHPFRC, strengthening length of UHPFRC

1.1 Research background

layers, reinforcement ratio in UHPFRC. However, knowledge is still lacking on the overlooked role of the interface between UHPFRC and existing concrete in the shear performance of such composite structures.

It is generally acknowledged that in order to guarantee the composite action and shear strengthening efficiency with UHPFRC, delamination at the interface between UHPFRC and existing concrete should be avoided [28]. Extensive studies are devoted to investigating interface performance between UHPFRC and concrete. Various factors, such as strength of concrete substrate or UHPFRC [29,30], roughness and moisture state of concrete substrate [31,32], as well as the bonding methods [33,34], are investigated. However, most of the studies are focused on the characterization of interface strength through small-scaled bond strength tests, which are rarely linked with the behavior of the interface in the UHPFRC strengthened RC structures. Knowledge on shear performance of macroscale UHPFRC strengthened concrete structures with a focus on the interface behavior is lacking. Perfect bond is generally assumed between UHPFRC and NC in finite element models, which can lead to incorrect predicted crack pattern [35]. Besides resistance to mechanical loads, the damage in a multi-material composite might be initiated due to restrained deformations caused by temperature and moisture gradients, especially under harsh environmental conditions. This might affect the long-term interface behavior between the UHPFRC and the existing concrete, and durability of strengthened structures. So far, no or very few research has been carried out on combined actions of imposed loads and imposed deformations, which might cause the progressive degradation of the hybrid structure and the interface between UHPFRC and NC. Therefore, analysis of the interface behavior and the shear performance of the repair/strengthening system under combination of (severe) environmental and mechanical loading should be identified.

In order to evaluate the strengthening efficiency with the application of UHPFRC, destructive and non-destructive testing (NDT) can be used. However, commonly used destructive tests, such as bond strength test for interface, and material mechanical test (e.g., compressive, tensile and elastic modulus tests) to determine the UHPFRC properties, are time consuming, expensive and not suitable for in-situ evaluation [36]. Therefore, in this thesis, attention has been paid to the application of NDT methods to examine the quality of the interface and UHPFRC materials. Infrared thermography (IRT) is a relatively new NDT method to inspect and monitor the defects in concrete structures, and can be classified into two kinds: passive and active measurements [37]. Compared to passive IRT involving surface temperature difference detection without external heating source, active IRT can provide quicker and more accurate and quantitative analysis as the heat can penetrate inside objects faster and the heat source can generate a controlled temperature field, which helps to locate and measure the size, shape, and depth of the defects [38].

Not only the interface performance, but also the material properties of UHPFRC play a substantial role in determining the strengthening efficiency of UHPFRC and can be analyzed through NDT methods. In general, UHPFRC is typically assumed to have uniform material properties. However, in reality, the inhomogeneous fiber distribution, and more seriously, even fiber segregation in UHPFRC might exist [39]. As the tensile behavior of UHPFRC is highly dependent on its fiber distribution, a quantitative evaluation of the fiber density and orientation in UHPFRC element is usually performed by X-ray tomography and electromagnetic methods [40]. Due to the limitation of size of tested specimens and high cost, X-ray tomography is generally applied on laboratory scale and rare for practical applications. The electromagnetic method, relying on the principle of magnetic induction and electrical resistivity, is more effective since it is more portable, cost-effective and could provide a quick and real-time assessment from rapid data acquisition [41]. Therefore, the active IRT and the electromagnetic method have been used in this research to characterize the delamination and fiber distribution in UHPFRC, respectively, and further quantify their influence on the shear strengthening efficiency of UHPFRC.

1.2 Research aims and objectives

The aim of this thesis is to investigate shear behavior of reinforced concrete structures strengthened with UHPFRC, focusing on the interface behavior between UHPFRC and NC, and the material properties of UHPFRC, which are governing the performance of the strengthening system. In the combined experimental and numerical study, the effect of hygral and thermal gradients (moisture exchange and freeze-thaw cycles) and fabrication method (casting direction combined with different bonding methods) are studied. Besides, to detect interface delamination and measure fiber distribution, non-destructive testing methods (i.e., infrared thermography and electromagnetic method, respectively) are used. To reach this goal, the following sub-objectives are defined:

- (1) To study the influence of concrete substrate moisture state on shear performance of concrete structures strengthened by UHPFRC: Compared to strengthening applications with conventional concrete (e.g., with a w/c ratio higher than 0.3 [42–44]), due to the low water content in UHPFRC (i.e., w/c ratio less than 0.25), the moisture exchange between substrate and UHPFRC likely has a greater impact on the mechanical and bond properties of UHPFRC.
- (2) To assess the risk of debonding at the interface due to restrained shrinkage: Compared to conventional concrete, UHPFRC has a much higher autogenous shrinkage. Therefore, for cast in-situ strengthening applications, interfacial stresses induced by relatively high restrained shrinkage of UHPFRC poses a threat for debonding/slip at the interface.
- (3) To quantify the influence of UHPFRC casting direction on the shear resistance of UHPFRC-RC structures: Casting direction will affect the fiber distribution of

1.3 Research scope

UHPFRC, and thereafter, has a substantial impact on the material and structural performance of UHPFRC in the strengthening system.

- (4) To investigate the effect of freeze-thaw action combined with three various bonding methods on the behavior of interfaces and strengthening system. A clear understanding of strengthening efficiency of UHPFRC applied by different bonding methods and its interface behavior under combined (severe) environmental and mechanical loads are governing for the long-term performance of the hybrid concrete system.

1.3 Research scope

In this PhD study, the UHPFRC is used as a main strengthening material. The following key factors are considered governing for the UHPFRC material, UHPFRC-concrete interface and the behavior of the strengthening system under shear:

- the influence of different moisture levels of concrete substrate (dry and wet surface dry conditions) is critical for interface quality and microstructure development (such as the degree of hydration) of UHPFRC, and is investigated through the material and structural tests,
- the influence of restrained shrinkage of UHPFRC at the interface is critical for the interface behavior and performance of strengthened concrete structure and is, therefore, investigated in material and structural levels,
- the influence of casting direction (horizontal or vertical casting) is critical for the mechanical properties of UHPFRC and performance of strengthened concrete structure and therefore, is studied in structural tests,
- the influence of freeze-thaw action (30, 75 and 120 cycles) combined with different bonding methods (cast in-situ, epoxy-bonded and mechanical anchorage) is critical for the durability of strengthening system and therefore, is studied through material and structural tests.

Besides, the delamination in UHPFRC-RC composites and fiber distribution in UHPFRC are examined by two NDT methods: infrared thermography and electromagnetic methods, respectively. Finally, based on experimental observations in structural tests and assessment from NDT methods, numerical models with a focus on interface behavior and material properties of UHPFRC are made.

The current research involves the use of two distinct UHPFRC mixtures developed at TU Delft, differing solely in the type of steel fibers employed. The initially used steel fibers are no longer produced, and have been consequently replaced in the mid stage of this project by alternative steel fibers, having slightly different diameter.

1.4 Research methodology

In order to realize the objectives of this thesis, the research framework is mainly divided into three parts, as shown in Figure 1.2. Firstly, the material properties of UHPFRC and shear behavior of strengthened beams are quantified considering the governing parameters (e.g., moisture exchange, shrinkage, casting direction, bonding technique) under combined mechanical and environmental (freeze-thaw action) loads. Secondly, the bond quality and fiber distribution of UHPFRC in hybrid beams are assessed by two NDT methods, i.e., active infrared thermography and electromagnetic method, respectively. Standard procedures for the application of NDT methods are developed. The last part discusses the development of a numerical model to enable optimized design of UHPFRC strengthened structure, based on ATENA software [45].

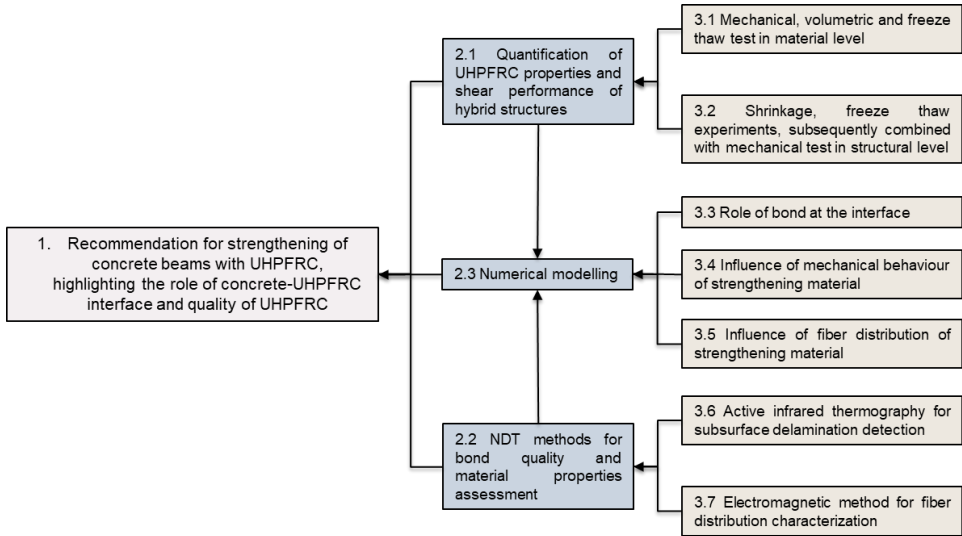


Figure 1.2 Research framework.

1.5 Thesis outline

As shown in Figure 1.3, the outline of the thesis is divided into five parts. Details of the chapters are described as follows:

The first part starts with the research background, research objectives and outline of the thesis (Chapter 1). Chapter 2 presents a critical review on the shear performance of hybrid concrete structures at the macro-scale and the UHPFRC-concrete interface behavior which is governing the response of the hybrid beams at the meso- and micro-scale. In addition, an overview of promising non-destructive methods focusing on assessment of strengthening efficiency with UHPFRC and other strengthening materials (e.g., FRP) in general is provided.

1.5 Thesis outline

In the second part, the properties of used materials and shear performance of concrete beams strengthened with UHPFRC are experimentally characterized. In Chapter 3, material and structural level test methods are described. Material tests to measure the workability of UHPFRC and determine the volume stability and mechanical properties of NC and UHPFRC, are conducted. Structural testing involves the three-point bending test. Varied parameters include moisture exchange between UHPFRC and existing concrete, restrained shrinkage at the interface, casting direction, and bonding techniques combined with freeze-thaw cycles. Chapter 4 presents the material and structural test results. The slump flow, final setting time and (free and restrained) shrinkage of UHPFRC, as well as the compressive, tensile and elastic modulus with and without FT cycles for NC and UHPFRC, are presented. For structural tests, shear behavior is analyzed by comparing the load-deflection response, fracture pattern, crack development, as well as interface behavior (opening and sliding).

In the third part, to evaluate the strengthening efficiency with the application of UHPFRC, the bonding properties and casting quality of UHPFRC are assessed by NDT methods. Chapter 5 focuses on the applicability of active infrared thermography to detect the delamination in the UHPFRC-NC composites. Combining experimental and numerical investigation of small-scaled composite specimens, a standardized testing method and post-processing analytical model for delamination detection are developed. For upscaling application, hybrid UHPFRC-NC beams are tested, and the accuracy is examined by both the visual inspection and x-ray computed tomography (CT scanning). Chapter 6 presents an electromagnetic method to determine the fiber distribution in UHPFRC elements. Small-scale UHPFRC plates with different fiber contents are cast to calibrate the magnetic sensors. Factors influencing the fiber distribution of UHPFRC, including fiber content, casting orientation and vibration time, are experimentally studied and the CT scanning is performed to validate the measured test results. After calibration and validation, the developed electromagnetic method is employed on large UHPFRC-RC beams to quantify the fiber content and orientation on UHPFRC laminates. Finally, based on the fiber content measurements from electromagnetic method, analytical methods are proposed to predict the mechanical properties of UHPFRC.

In the fourth part, shear performance of concrete structures strengthened with UHPFRC is numerically investigated. In Chapter 7, the numerical model is validated with the respective experimental results. Parametric study focusing on the role of governing parameters, i.e., UHPFRC material behavior in tension, interface properties and non-uniform fiber distribution in UHPFRC, on the shear strengthening efficiency is conducted.

In the fifth part, the conclusions drawn from this PhD work and recommendation for the optimization of UHPFRC strengthened concrete structures are given (Chapter 8).

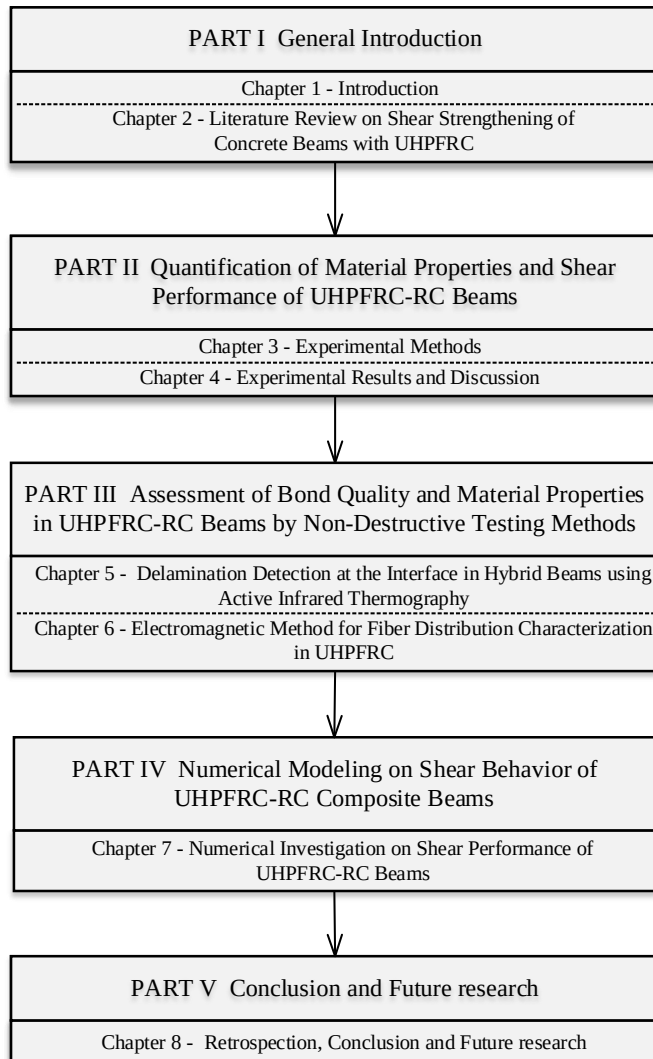


Figure 1.3 Outline of the thesis.

Chapter 2 Literature Review on shear strengthening of concrete beams with UHPFRC

2.1 Introduction

This chapter explores the strengthening applications of UHPFRC in shear, with the focus on the UHPFRC-concrete interface behavior and material performance of UHPFRC which are crucial for the response of hybrid beams. Additionally, non-destructive methods to study concrete-UHPFRC interface performance, and fiber distribution within UHPFRC are critically reviewed.

Section 2.2 discusses the mechanisms involved in shear strengthening and provides examples of various strengthening applications. Section 2.3 reviews and analyzes current analytical and numerical methods for predicting the shear capacity of RC structures strengthened with UHPFRC. Note that both Sections 2.2 and 2.3, not only address strengthening applications, but also explore UHPFRC used in new hybrid structures, where shear capacity improvement is evaluated. In Section 2.4, the focus shifts to the concrete-UHPFRC interface at the meso- and micro-scale, examining factors such as bonding technique, moisture exchange and differential shrinkage between two materials, and coupled action of environmental and mechanical loads. Finally, Section 2.5 provides an overview of current non-destructive testing methods to evaluate the UHPFRC strengthening efficiency, with a particular emphasis on concrete-UHPFRC interface performance and quality of UHPFRC.

2.2 Shear strengthening efficiency of UHPFRC – experimental studies

In a reinforced concrete beam, different failure modes may occur when the beam collapses. For example, as shown in Figure 2.1 (a), under three-point bending, a dominant bending crack is formed vertically at the mid-span of the beam (also the region of maximum bending moment), where the flexural tensile stress exceeds the tensile strength of concrete. As the dominant bending crack propagates until the compression zone of the beam, the beam fails, which is called “flexural failure”. On the other hand, when the failure is induced by cracks outside the central section of the beam (where the bending moment is greatest), the cause of failure could not be attributed to the bending moment (Figure 2.1 (b)). The only noticeable difference is that there is a shear force in the end section of the beam where the crack appeared, which caused the failure. Thus, the term “shear failure” was chosen in literature, but only much later it was recognized that shear stress at failure is far from being constant [46]. As “shear failure” appears to be an unsuitable term for a failure in which shear has little or no influence and because a diagonal crack is the visible feature, other authors classify this failure as “diagonal failure”. Although flexural and “diagonal” or “shear cracks” behave in different orientation, both are governed by the principal tensile stresses and represent the tension failure (mode I failure). As the term “shear failure” is mostly used in literature, this is also the term used in this thesis. Shear failure is generally an undesirable failure mode for RC beams as it is brittle without

prior warning. Therefore, strengthening measures are essential for shear-deficient RC beams.

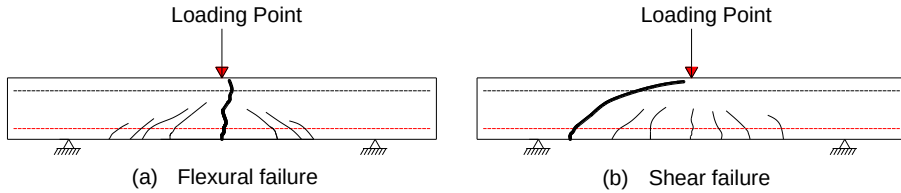


Figure 2.1 Failure modes in RC beams: (a) Flexural failure; (b) Shear failure.

Conventionally, a reinforced concrete jacket has been applied to increase the cross-sectional area and improve the shear resistance of a concrete beam [47]. To realize the same improvement while reducing the additional self-weight load, the use of a UHPFRC jacket can be an effective strengthening method due to its high tensile strength.

Only a few experimental studies have investigated the shear performance of beams strengthened with UHPFRC. These studies investigated factors such as UHPFRC thickness, the amount of rebars, and strengthening configurations. With respect to the role of UHPFRC thickness, results have shown that while an increased thickness can theoretically lead to a proportional increase in shear resistance, the actual improvement is often affected by the interface condition between UHPFRC and concrete and the governing failure mode. With shear failure, thickness may enhance peak load but the debonding at the interface can limit shear enhancement. With flexural failure, the effectiveness of increased thickness is limited, as the load capacity is primarily governed by the reinforcement rather than the additional UHPFRC layer [48].

An overview of typical shear strengthening configurations in experimental studies on the shear capacity of RC structures strengthened with UHPFRC are given in Figure 2.2 and Table 2.1 [21,23–25,27], respectively. Note that although the configuration shown in Figure 2.2 (a) is usually used for flexural strengthening, researchers applied it to study the shear capacity improvement and is therefore included.

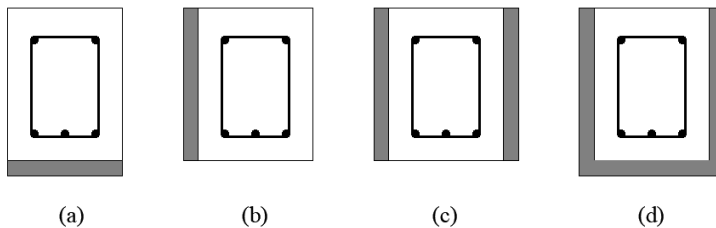


Figure 2.2 Typical shear strengthening configurations with UHPFRC (a) T-sided (Strengthening on tension side); (b) L-sided (Strengthening on one lateral side); (c) 2-sided (Strengthening on two lateral sides); (d) 3-sided beam (Strengthening on three sides).

Table 2.1 Overview of the test results for beams strengthened in shear.

Reference	Specimen name	Dimension $b \times h \times L$ (mm)	Fibers Vol.-%	Strengthening technique	Strengthening pattern		a/d ratio	Cracking load (kN)	Failure load (kN)	Deflection at failure load (mm)	Failure type
					Configuration	UHPFRC thickness (mm)					
Hussein et al. [25]	NS1	150×300×1584	-	-	-	-	3	165	250.84	4.63	Shear
	UNS2-1	150×300×1584	1	Cast in-situ	T-sided	150	3	295	436.24	6.15	Shear
	UNS2-1D	150×300×1584	1	Dowel connection	T-sided	150	3	300	402.25	5.55	Shear
	UNS2-1S	150×300×1584	1	Shear stud connection	T-sided	150	3	300	430.24	6.46	Shear
	UNS2-1.5	150×300×1584	1.5	Cast in-situ	T-sided	150	3	310	439.15	7.66	Shear
	UNS2-1.5D	150×300×1584	1.5	Dowel connection	T-sided	150	3	280	503.56	6.41	Shear
	UNS2-2	150×300×1584	2	Cast in-situ	T-sided	150	3	350	521.56	6.63	Shear
	UNS2-2D	150×300×1584	2	Dowel connection	T-sided	150	3	310	502.45	7.24	Shear
	HS1	150×300×1584	-	-	-	-	3	160	256.61	4.62	Shear
	UHS2-1	150×300×1584	1	Cast in-situ	T-sided	150	3	280	528.41	8.57	Shear
	UHS2-1D	150×300×1584	1	Dowel connection	T-sided	150	3	280	439.22	9.52	Shear
	UHS2-1S	150×300×1584	1	Shear stud connection	T-sided	150	3	290	433.02	6.19	Shear
	UHS2-1.5	150×300×1584	1.5	Cast in-situ	T-sided	150	3	245	403.59	5.15	Shear
	UHS2-1.5D	150×300×1584	1.5	Dowel connection	T-sided	150	3	290	465.79	6.52	Shear
Bahraq et al. [27]	CT-1.0	140×230×1120	-	-	-	-	1	-	383	2.17	Shear
	SB-2SJ-1.0	200×230×1120	2	Cast in-situ	2-sided	30	1	-	567	3.47	Flexure-shear
	SB-3SJ-1.0	200×260×1120	2	Cast in-situ	3-sided	30	1	-	628	3.10	Flexure
	CT-1.5	140×230×1120	-	-	-	-	1.5	-	286	4.40	Shear
	SB-2SJ-1.5	200×230×1120	2	Cast in-situ	2-sided	30	1.5	-	402	5.20	Flexure-shear
	SB-3SJ-1.5	200×260×1120	2	Cast in-situ	3-sided	30	1.5	-	482	4.10	Flexure
	CT-2.0	140×230×1120	-	-	-	-	2	-	276	7.00	Shear
	SB-2SJ-2.0	200×230×1120	2	Cast in-situ	2-sided	30	2	-	346	7.50	Flexure-shear
Garg et al. [21]	SB-3SJ-2.0	200×260×1120	2	Cast in-situ	3-sided	30	2	-	353	4.14	Flexure
	CB-U/R	150×150×700	-	-	-	-	1.6	32	123	3.422	Shear
	UT-U/R	150×150×700	2	Cast in-situ	3-sided	25	1.6	38.25	157.9	8.424	Flexure
	EJ-U/R	200×150×700	2	Cast in-situ	2-sided	25	1.6	39.25	115.75	1.83	Debonding
	CB-O/R	150×150×700	-	-	-	-	1.6	48	140	4.525	Shear
	UT-O/R	150×150×700	2	Cast in-situ	3-sided	25	1.6	56.6	174	8.273	Flexure
A. Sakr et al. [23]	EJ-O/R	200×150×700	2	Cast in-situ	2-sided	25	1.6	55.2	131.375	2.689	Debonding
	C-S	150×300×2000	-	-	-	-	2	48	115	5.35	Shear
	C-F	150×300×2000	-	-	-	-	2	70	253	15.77	Flexure
	C-S-210	210×300×2000	-	-	-	-	2	58	211	7.11	Shear
	ST-2S	210×300×2000	2	EB	2-sided	30	2	93	281	8.23	Flexure

2.2 Shear strengthening efficiency of UHPFRC – experimental studies

Ji et al. [24]	ST-1S	210×300×2000	2	EB	L-sided	60	2	60	153	7.25	Shear
	ST-2S-R	210×300×2000	2	EB and AC	2-sided	30	2	110	331	10.31	Flexure
	ST-1S-R	210×300×2000	2	EB and AC	L-sided	60	2	64	252	6.79	Shear
	RC-2.7	200×500×2700	-	-	-	-	2.7	-	655	-	Shear
	RC-U-2.7	200×550×2700	2.5	-	T-sided	50	2.7	-	717	-	Shear
	RC-RU-2.7	200×550×2700	2.5	-	T-sided	50	2.6	-	922	-	Shear
	RC-RU-2.4	200×550×2400	2.5	EB	T-sided	50	2.2	-	955	-	Shear
	RC-RU-3.1	200×550×3100	2.5	EB	T-sided	50	3.2	-	761	-	Shear

Noted: a/d ratio represents the shear-span ratio; EB denotes epoxy bonded and AC denotes anchorage connection.

Based on the experimental database (Table 2.1), the effects of several parameters on beam shear capacity have been analyzed: of (a) concrete substrate strength, (b) steel fiber content in UHPFRC, (c) reinforcement in UHPFRC, (d) strengthening configuration, (e) connection method, and (f) shear-span ratio ((Figure 2.3 (a) to (f), respectively). The effects of these parameters on the shear capacity of composite beams and the effectiveness of the strengthening have been evaluated using the following equation (2.1):

$$\text{Increase in shear resistance (\%)} = \left(\frac{V}{V_{con}} - 1 \right) \times 100 \quad (2.1)$$

where V represents the shear capacity of the strengthened beam, V_{con} represents the shear capacity of the control beam. Note that *not* all strengthened beams exhibited shear failure (as indicated in Table 2.1). Therefore, in some cases this shear resistance increase presents the lower bound estimate, as further discussed.

First, the effect of compressive strength of the parent concrete on the shear improvement is analyzed when UHPFRC is added. Normal concrete (NC) and high strength concrete (HSC) control beams are compared with hybrid beams where part of the parent concrete is replaced by UHPFRC. Hybrid beams exhibited a 60-110% enhancement of shear capacity (Figure 2.3 (a)) compared to control beams, regardless of the parent concrete's compressive strength.

The relationship between steel fiber content and the increase of the shear resistance of concrete beams is shown in Figure 2.3 (b) [25]. Theoretically, the shear capacity of a beam with fiber reinforcement [49] increases as more fibers bridge the shear crack. However, in [25] it is indicated that the effect of the steel fiber content on the shear capacity of composite beams is unclear (Figure 2.3 (b)), which aligns with the findings of Jongvivatsakul et al. [50]. Still Mohammed et al. [25] reported that an increased fiber content significantly restricts crack width growth.

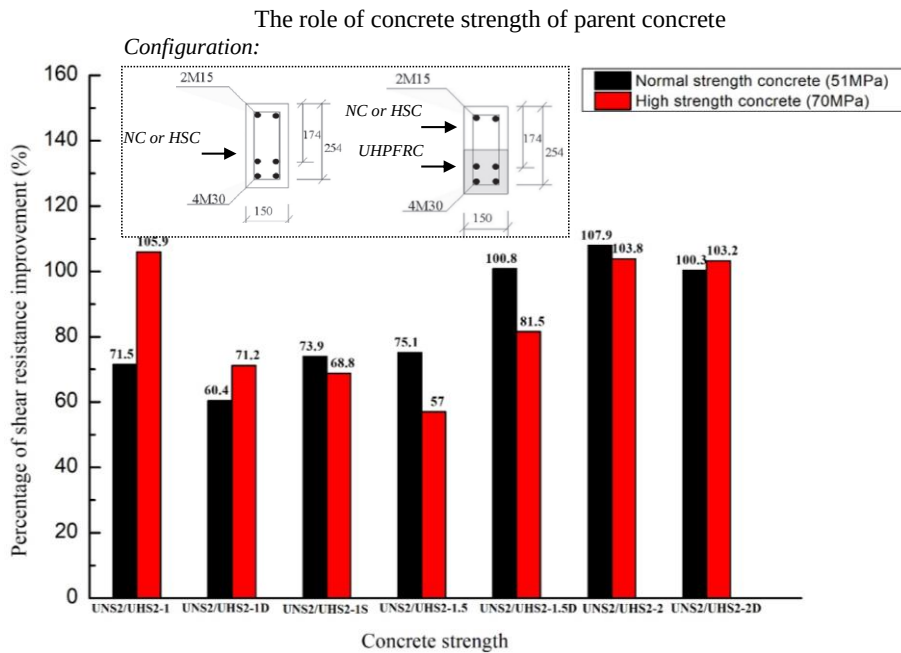
The reinforcement in UHPFRC and the strengthening configuration also influence the shear resistance of the strengthened beams (Figure 2.3 (c) and (d)). Longitudinal rebars increase shear resistance through dowel action (Figure 2.3 (c)). In terms of strengthening configuration, the 2-sided strengthening resulted in a greater shear capacity increase compared to L-sided strengthening with the same thickness. As expected, the shear resistance only slightly increases from 2-sided to 3-sided strengthening (Figure 2.3 (d)). However, Table 2.1 indicates that different strengthening schemes led to shifts in failure modes from shear to flexure [27], meaning that the actual increase in shear resistance is higher than the capacity shown in Figure 2.3 (d). Assessing shear capacity increase when the type of failure is changing (e.g., to flexural failure) is inaccurate and as it only provides a lower bound value.

2.2 Shear strengthening efficiency of UHPFRC – experimental studies

The strengthening effect of different bonding techniques (cast in-situ and two forms of mechanical anchorage) is compared in Figure 2.3 (e). The interface produced by these techniques does not affect the shear capacity of the strengthened beam as long as the interface strength is sufficient to prevent debonding. However, Table 2.1 shows that some beams failed due to debonding. Characterizing the interface performance, especially partial delamination, and evaluating its effect on the shear capacity remained unexplored.

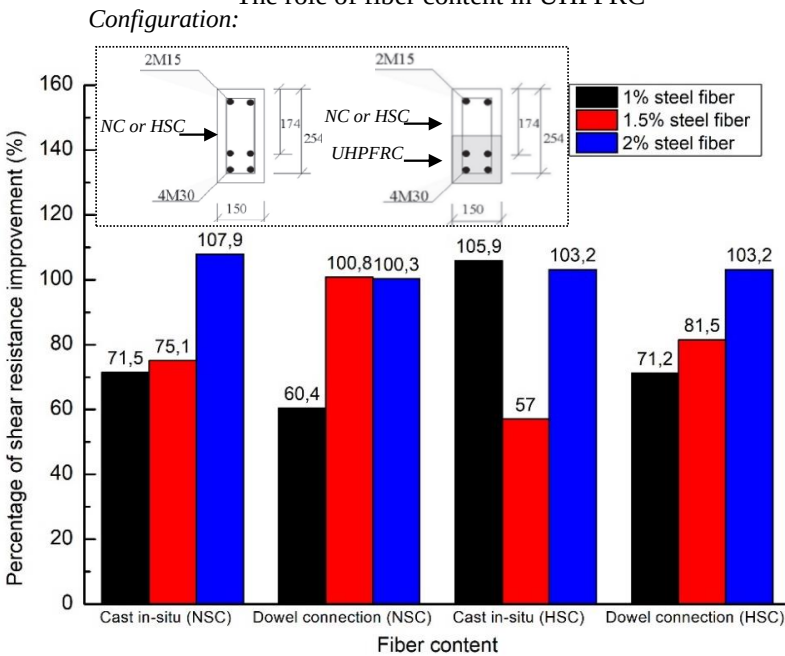
Finally, the influence of the shear-span (a/d) ratio is investigated (Figure 2.3 (f)). The capacity improvement reduces as the shear-span (a/d) ratio increases from around 1 to 3, with failure patterns shifting from shear to flexural failure for 3-sided strengthening. T-sided and 2-sided strengthening, however, still show shear failure.

In conclusion, there is still insufficient knowledge about underlying mechanisms, and the role of interface, type of the bond, and fiber contribution is unknown. More accurate methods for characterization and evaluation of the shear resistance improvement still need to be developed.



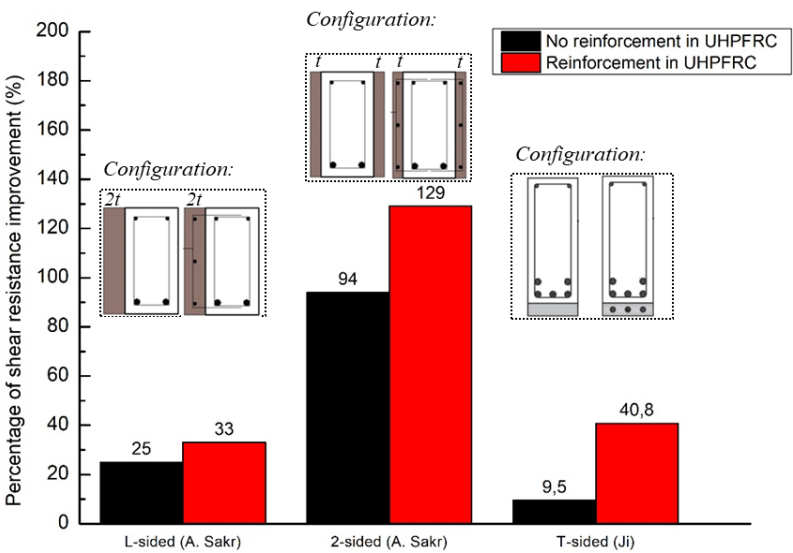
(a)

The role of fiber content in UHPFRC



(b)

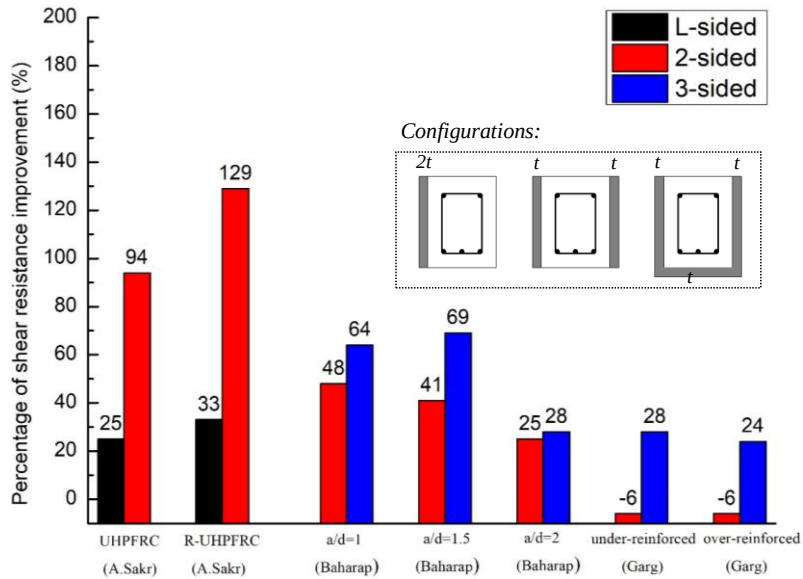
The role of reinforcement in UHPFRC for different configurations



(c)

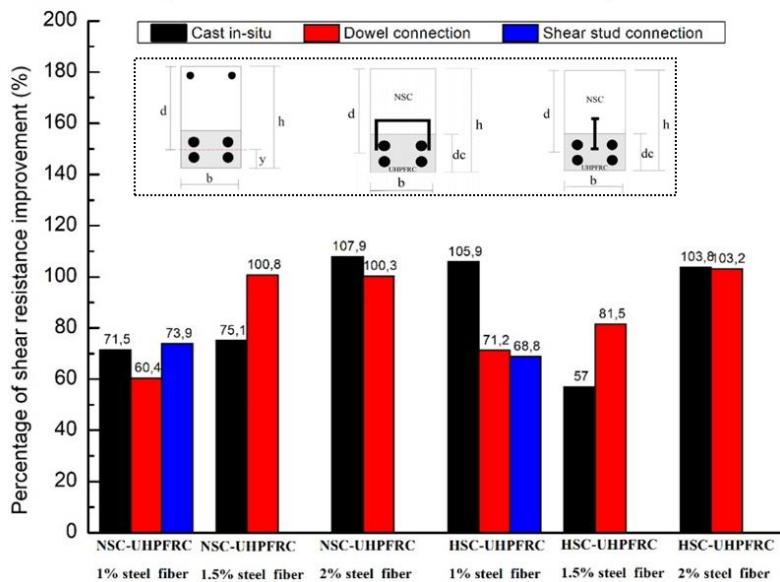
2.2 Shear strengthening efficiency of UHPFRC – experimental studies

The role of strengthening configuration



(d)

The role of connection method



(e)

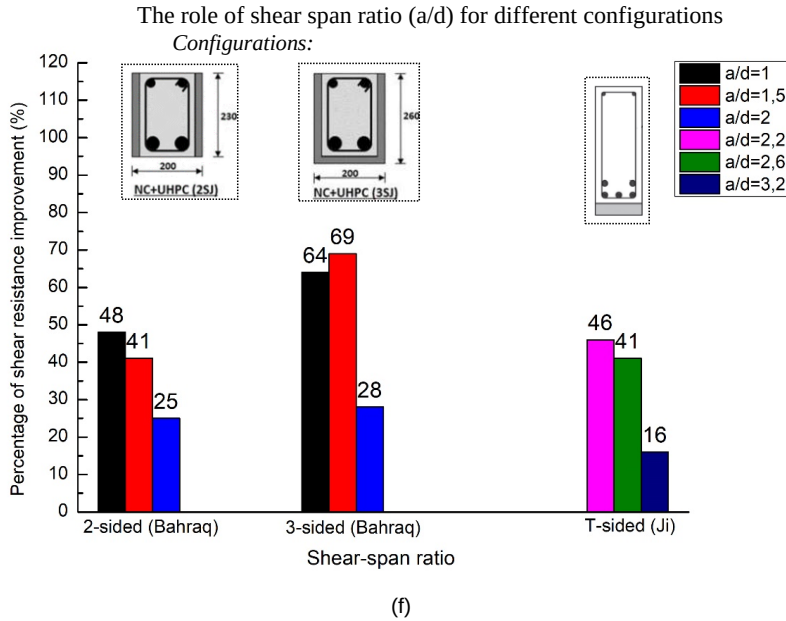


Figure 2.3 Parametric analysis of shear resistance improvement with UHPFRC (in percentage) with changing: (a) concrete strength of the parent concrete [25]; (b) steel fiber content in UHPFRC [25]; (c) reinforcement in UHPFRC [23,24]; (d) strengthening configuration [21,23,27]; (e) connection method [25]; (f) shear-span ratio (a/d) [24,27]. In Figure 2.3 (a), the first and second letters indicate two types of concrete to form hybrid specimen followed by the specimen configuration number. The last number and the last letter indicate the volume of fiber in UHPFRC and the connection method, respectively. Note that the failure pattern may change after strengthening with UHPFRC, thus the improvement ratio indicated in the figure means the minimum improvement in shear resistance; the negative number shown in (d) reflects the reduction in resistance for 2-sided strengthening configuration due to the debonding failure.

2.3 Modeling of shear strengthening efficiency of UHPFRC

Current modeling methods for predicting the shear capacity of UHPFRC-NC structures are reviewed. Table 2.2 provides an overview of the shear resistance predictions from various analytical and/or numerical methods (denoted as $V_{u,ana}$ and $V_{u,FE}$, respectively) compared to experimental results ($V_{u,exp}$). Note that Yin et al. [51] applied six different analytical methods (based on existing standards) to calculate shear capacity. However, here only the results using JSCE-2007 standard (2010) [52] are presented, as it provides the most accurate predicted results among the six methods. As shown in Table 2.2, numerical predictions of shear resistance and fracture patterns generally align well with experimental results. However, there is significant deviation in shear capacities predicted by different analytical models compared to experimental values. More detailed analysis of available analytical and numerical models is provided in the following sections.

2.3 Modeling of shear strengthening efficiency of UHPFRC

Table 2.2 Comparison between analytical or numerical and experimental results.

Reference	Specimen name	Analytical models	FE studies
		$V_{u,ana} / V_{u,exp}$	$V_{u,FE} / V_{u,exp}$
Hussein et al. [25]	NS1	0.45	-
	UNS3-1	0.67	-
	UNS3-1.5	0.82	-
	UNS3-2	0.86	-
	HS1	0.46	-
	UHS3-1	0.70	-
	UHS3-1.5	0.91	-
	UHS3-2	0.85	-
Yin et al. [51,53]	RE-0	0.65	1.05
	OV-25	0.85	0.97
	OV-25a	0.85	1.05
	OV-50	0.94	1.06
	OV-50a	0.80	1.00
	RE-20	0.92	1.05
	RE-32	1.14	0.98
	RE-50	0.8	0.94
A. Sakr et al. [23]	RE-100	1.64	0.97
	C-S	-	0.97
	C-F	-	1.00
	C-S-210	-	1.07
	ST-2S	-	0.96
	ST-1S	-	0.99
	ST-2S-R	-	0.98
Ji et al. [24]	ST-1S-R	-	0.98
	RC-2.7	0.93	-
	RC-U-2.7	0.94	-
	RC-RU-2.7	0.95	-
	RC-RU-2.4	0.96	-
	RC-RU-3.1	0.96	-
Bahraq et al. [27]	CT-1.0	-	0.97
	SB-2SJ-1.0	-	0.96
	SB-3SJ-1.0	-	0.97
	CT-1.5	-	1.03
	SB-2SJ-1.5	-	1.01
	SB-3SJ-1.5	-	1.01
	CT-2.0	-	0.98
	SB-2SJ-2.0	-	1.02
	SB-3SJ-2.0	-	0.97

2.3.1 Analytical modeling of shear resistance

From the available analytical models, two concepts for evaluating shear strength can be distinguished [51]. *The first concept* points that the shear strength of composite structures is the sum of independent contributions from the RC part and the UHPFRC layer:

$$V_R = V_{UHPFRC} + V_{RC} \quad (2.2)$$

where V_R , V_{UHPFRC} , V_{RC} represent the shear resistance of the composite structure, the UHPFRC layer and the RC part, respectively. This concept is based on two assumptions:

(1) The RC component and the UHPFRC layer fail simultaneously; (2) A perfect bond is assumed between concrete and UHPFRC. For the RC part, the shear load is mainly carried by the concrete in the compression zone and the stirrups, with current design codes (e.g., ACI 318 (2008) [54], EN 1992-1-1 (2004) [55], and JSCE-2007 (2010) [52]) providing corresponding equations for shear resistance calculations. For the UHPFRC component, two methods are used:

Combined effect of fibers and matrix: As outlined in ACI 544 (1988) [54] and MC 2010 (2010) [56], this method addresses the shear contribution of UHPFRC without separating effect of fibers and matrix. Similarly, Noshiravani and Brühwiler [57,58] and Masse and Brühwiler [26] proposed a composite hinge model for flexural shear resistance of T-sided reinforced concrete beams with reinforced UHPFRC. This model considers nonlinear interfacial behavior between UHPFRC and RC, and the shear resistance of the R-UHPFRC element in double bending is determined through hinge rotation. Ji and Liu [24] extended this method to simply supported UHPFRC-NC composite beams with stirrups considering the size effect.

Separate contribution from fibers and matrix: As described by Japan Society of Civil Engineers (JSCE) [19], French Society of Civil Engineers (AFGC codes [13]) and the new version of Eurocode 2 [59], the shear resistance of UHPFRC is obtained by separating the shear contribution from cement matrix and fibers:

$$V_{UHPFRC} = V_c + V_f \quad (2.3)$$

where V_c and V_f represent the shear resistance contributions from the matrix and fibers in UHPFRC layer, respectively. Hussein and Amleh [13] proposed an analytical model suggesting that shear resistance of a UHPFRC-NSC/HSC beam without shear reinforcement equals the sum of shear resistance from concrete in the compression zone and fibers. This model was validated by experimental results, as indicated in Table 2.2.

The second concept considers the shear contribution from steel fibers as the equivalent longitudinal rebar ratio:

$$\rho = \rho_s + \rho_{eq,F} \quad (2.4)$$

where ρ_s is the rebar ratio in RC part. $\rho_{eq,F}$ is the equivalent longitudinal rebar ratio and can be calculated as:

$$\rho_{eq,F} = Vol. - \% \left(\frac{f_{ct}}{f_y} \right) \left(\frac{A_{UHPFRC}}{A_{RC}} \right) \quad (2.5)$$

where f_{ct} is the tensile strength of UHPFRC; f_y is the yield strength of the longitudinal rebar; A_{UHPFRC} and A_{RC} are the areas of the UHPFRC and RC parts, respectively, and $Vol. - \%$ is the volume ratio of steel fibers.

2.3.2 Numerical modeling of shear resistance

Only few experimental studies focused on the shear performance of RC structures strengthened with UHPFRC, and there is a lack of reliable numerical methods in this respect. The complexity of shear resistance mechanism itself [60], the variability of UHPFRC constitutive models affected by fiber contents, geometry and orientation [61], and the unclear interaction between the UHPFRC and concrete, make it challenging to predict shear strength improvements in composite members [62].

The “Concrete Damage Plasticity model” (CDP) is typically used to model the nonlinear constitutive behavior of concrete. In CDP, failure modes include concrete tensile cracking and compressive crushing. For UHPFRC, the CDP model includes both strain hardening and softening stages, as shown in Figure 2.4 [63]. Accurate prediction of the shear response in UHPFRC-NC structures, requires not only material modeling for concrete, reinforcement, and UHPFRC, but also careful modeling of the UHPFRC-concrete interface. So far, usually a perfect bond has been assumed [27,33,64,65], based on experimental observation from bond strength and structural tests. However, recent research [23,63,66] has highlighted the need to consider interface modeling to account for possible debonding failures.

The cohesive zone model, which includes a linear elastic traction-separation part, a damage initiation point, and a damage evolution zone, describes the interface behavior (slip and debonding) between concrete and UHPFRC (Figure 2.5) [23,63]. This model has effectively captured debonding failure modes in simulated hybrid structures, provided that input parameters such as interface shear strength, fracture energy, and elastic shear stiffness are accurately defined. Additionally, Yin and Shirai [66] proposed the equivalent beam elements method to simulate interface behavior. This method showed better correlation with experimental results compared to simulations using perfect bond or fully unbonded interfaces.

From Table 2.2, it is evident that current numerical approaches can predict the shear resistance of UHPFRC-NC hybrid structure. Still, due to limited data, it remains unclear if these models can be generically applied to different types of composite structures or boundary conditions.

In strengthening application with UHPFRC, one key aspect of quality control is ensuring that the interface bond strength exceeds the tensile strength of the parent concrete. The weakest zone in the strengthened structural system with UHPFRC was reported to be the near interface concrete (NIC) [58,67]. Therefore, the bilinear traction-separation law shown in Figure 2.5 does not apply for the interface between UHPFRC and concrete, but for the concrete fracture in the NIC zone that is critical. A similar law is used to determine the bond strength of carbon fiber lamellas on concrete substrates where debonding also

occurs due to concrete fracture in the zone near the interface. In this thesis, both NIC and the actual UHPFRC-concrete interface, are referred to as “interface behavior”.

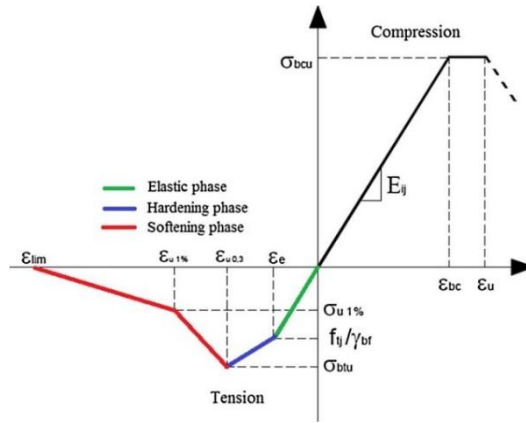


Figure 2.4 The uniaxial stress-strain curve for UHPFRC [63].

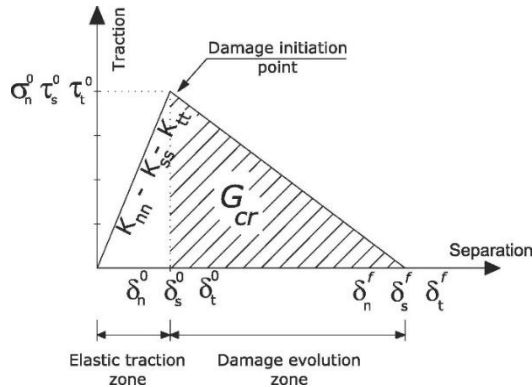


Figure 2.5 Bilinear traction-separation law at interface between UHPFRC and NC [63].

2.4 Behavior of UHPFRC-concrete interface and critical parameters

Research [68,69] highlights that the interface behavior in repaired structures is critical for both structural safety and durability. Generally, there are two types of concrete-to-concrete interfaces – unreinforced and reinforced interface [70]. In some applications (e.g., strengthening), having unreinforced interface can ease the execution. For unreinforced interface, interfacial bond strength depends on the cohesion (combination of adhesion and aggregate interlock) and friction, which is related to the surface roughness and normal force at the interface. For reinforced interface, apart from the abovementioned two parameters, the friction due to clamping effect and dowel action from reinforcement crossing the interface will also contribute to the interface resistance.

2.4 Behavior of UHPFRC-concrete interface and critical parameters

Environmental and/or mechanical loads cause stresses at the interface. Compatibility between two materials should be considered because a mismatch between material properties (e.g., shrinkage, stiffness) might lead to stress concentrations and interfacial failure [71]. Due to different age, it is not possible to have full compatibility between the two concretes in strengthening and repair [72]. High tensile strength (and strain hardening capacity) of UHPFRC, along with low drying shrinkage, are reported to minimize the risks of debonding and excessive cracking [10]. Therefore, it is important to understand systematically the role of time-dependent and coupled mechanical/thermal/hygral processes in hybrid concrete structures.

In this section, interface behavior and the role of governing parameters (type of bond, moisture exchange between the hardening UHPFRC and hardened concrete, etc.) are addressed. Interface behavior is considered through different bond strength tests, but also through structural tests (considering both flexural and shear strengthening applications, especially when delamination is explicitly considered).

2.4.1 Bond strength tests for investigating UHPFRC-concrete interface

Usually, the interface is exposed to (a combination of) tensile and shear stresses. Tensile stress tends to open the interface crack and shear stress causes sliding along the interface. The interface resistance is governed by, for example, the properties of the substrate (surface preparation, porosity, moisture conditions, strength, etc.), the properties of the strengthening material, the use of bonding agent [73–75], and it may change due to time-dependent effects [69]. In order to assess resistance of interface under stresses induced by environmental effect and/or mechanical loading, various test methods have been put forward. They can mainly be divided into four categories, depending on the stress condition: tension, pure shear, combination of shear and compressive or tensile stress, and bending test [76]. It is recommended that the bond test should be selected based on the state of stress that the structure is subjected to in the field [69]. Various researchers [43,69] tried to determine ratios between different bond strength tests. Note that in most of these test methods, usually the failure occurs in the concrete substrate or the strengthening material rather than along the bond plane. Under this condition, the measured value presents only the lower bound limit and not the real bond strength [77].

Tension tests

Tensile bond strength tests can be divided into direct and indirect tensile tests [78–82]. In direct tensile test, avoiding eccentricity is essential because it will generate additional bending moment and make the test results unreliable [69,83,84]. Compared to direct tensile testing, indirect tension tests such as the splitting test are easier to conduct but are suitable only when the interface is smooth since otherwise, there is non-uniform stress distribution along the interface. In addition, it was reported that the direct tension test with UHPFRC-concrete enables a higher possibility for interfacial failure to happen [82].

In order to trigger interfacial failure, Valipour et al. [85] modified the pull-off test (Figure 2.6 (a)) and proposed a debonding test method (Figure 2.6 (b)). Both methods aimed at reducing the contact area between the UHPFRC and the concrete substrate, ensuring that the interface is subjected to high tensile stresses to trigger failure.

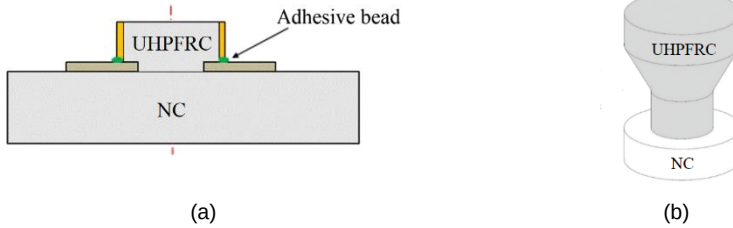


Figure 2.6 Test set-up for proposed UHPC debonding test (a) Modified pull-off test (b) Proposed debonding test method [85].

Shear tests

Current shear test methods induce a bending moment due to the eccentricity of the shear force to the interface [86]. Due to this, test methods have been put forward to neutralize the moment that the shear force causes, thereby failing to represent the real condition and being hard to conduct [87]. Due to the convenience and test stability, bi-surface shear and double-sided shear test methods shown in Figure 2.7 have been adopted to investigate the UHPFRC-NC interfacial shear behavior [31,88]. Based on the test results, pure interfacial failure rarely occurs, and the main failure is a mixed mode failure and concrete substrate failure. Therefore, new test methods are needed to evaluate the shear bond strength between UHPFRC and the concrete.

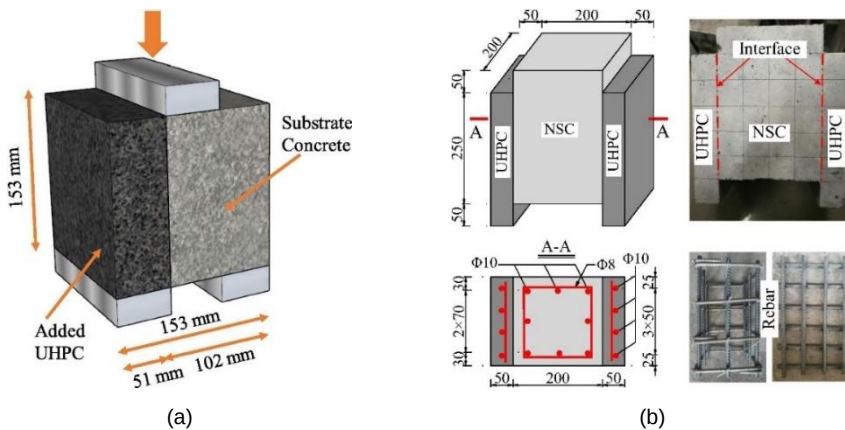


Figure 2.7 Shear test: (a) Bi-surface shear [31]; (b) Double-sided shear [88].

Shear and tension/compression test

Slant shear test with a combination of shear and compressive stresses is usually reported to reflect the realistic stress state in composite structures. Tests have been developed to induce this combination of stresses at the interface [51,89] and is widely used to investigate UHPFRC-NC interface (Figure 2.8) [78,79,81,82,90]. In the common slant shear compressive test, the interface is inclined and interfacial failure happens due to the shear crack along the inclined plane. As the angle between the failure plane and the horizontal direction is theoretically between 50° ~ 70° , the interface with an angle of 60° is commonly selected.

However, this test method has some shortcomings. First, the standard angle does not always lead to the most critical interfacial stress state since critical bond angles are also related to surface roughness. The rougher interface has a lower critical bond angle [75]. Second, test results are affected by stress concentrations due to elastic modulus differences between the two materials [75]. Finally, in this test, the pure interfacial failure occurs only under poor interface preparation condition. As such, these tests in most cases are not suitable to determine the real bond strength between UHPFRC and concrete.



Figure 2.8 Slant shear test set-up for UHPFRC-NC specimen [78].

Bending tests

Bending tests have also been used for bond characterization between cement-based layers [91,92]. Although the bending test is reported to represent the actual stress conditions in structures, it is less efficient as the area of the bonded surface subjected to loading is small compared to the specimen volume [69]. Experimental studies using this method [81,90] showed that typical failure happens either at the interface or in the concrete substrate, with the fully interfacial failure occurring only when the interface is smooth.

2.4.2 Influence of bonding method and roughness on interface behavior

Currently, there are three main bonding techniques to strengthen damaged concrete structure with UHPFRC: (i) cast in-situ, (ii) gluing with a bonding agent and (iii) mechanical anchoring. The role of the applied bonding technique in UHPFRC-NC composites on measured bond strength was investigated in [33]. Cast-in situ specimens with smooth, sandblasted and smooth interface coated by epoxy adhesives are compared. Cast in-situ specimens with smooth surface had the lowest bond strength based on splitting tensile and slant shear tests (Figure 2.9). Cast in-situ specimens with sandblasted interface resulted in the highest bond strength in slant shear test. On the other hand, bonding UHPFRC to the smooth surface of concrete substrate with epoxy had the highest bond strength in the splitting tensile test. In splitting tensile tests, the tensile stress is primarily governing, and thus adhesion between the concrete substrate and UHPFRC plays a dominant role on the interfacial bond strength.

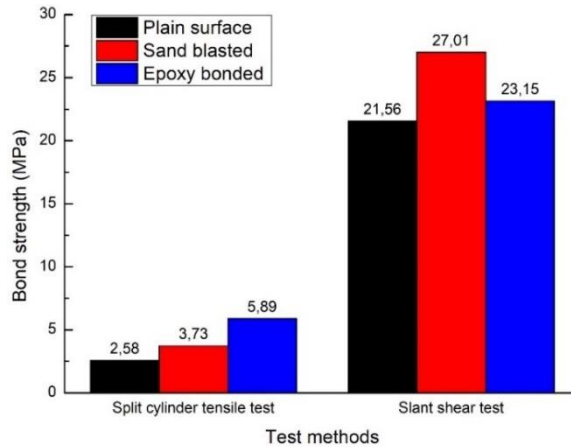


Figure 2.9 Bond strength test results [33].

The comparison of different bonding techniques (cast in-situ and mechanical anchorage through dowel and shear stud connection) on structural shear capacity is shown in Figure 2.3 (e). The results show that the effect of mechanical anchorage is negligible provided that there is no interface failure in reference samples.

Studies [33,93–96] discussed the effect of different bonding techniques on flexural performance of hybrid UHPFRC structures. In-situ application of UHPFRC, gluing with UHPFRC laminate and the use of mechanical anchorage, were reviewed. An overview of the test results and the corresponding strengthening configuration [33,96] is presented in Table 2.3 and Figure 2.10.

2.4 Behavior of UHPFRC-concrete interface and critical parameters

Table 2.3 Overview of beam test results under flexure [33,96].

Reference	Specimen name	Dimension $b \times h \times L$ (mm)	Strengthening technique	Strengthening pattern		Cracking load (kN)	Failure load (kN)	Deflection at failure load (mm)	Failure type
				Configuration	UHPFRC Thickness				
Al-Osta et al. [33]	RC-control	140×230×1600	-	-	-	16	70	19.10	Flexure
	RC-SB-BOTSJ	140×260×1600	Cast in-situ	T-sided	30 mm	33	81	15.31	Flexure
	RC-EP-BOTSJ	140×260×1600	Gluing by epoxy	T-sided	30 mm	47	75	12.13	Flexure
	RC-SB-2 SJ	200×230×1600	Cast in-situ	2-sided	30 mm	41	102	13.38	Flexure
	RC-EP-2 SJ	200×230×1600	Gluing by epoxy	2-sided	30 mm	44	95	15.7	Flexure
	RC-SB-3 SJ	200×260×1600	Cast in-situ	3-sided	30 mm	90	132	4.55	Flexure
	RC-EP-3 SJ	200×260×1600	Gluing by epoxy	3-sided	30 mm	95	85	4.35	Flexure
Tanarslan et al. [96]	Beam-1	120×160×3200	-	-	-	10.52	41.96	75.53	Flexure
	Beam-2	120×190×3200	Gluing by epoxy	T-sided	30 mm	23.05	48.52	65.22	Flexure
	Beam-3	120×190×3200	Mechanical anchorage	T-sided	30 mm	14.32	46.42	54.16	Flexure
	Beam-4	120×190×3200	Gluing by epoxy	T-sided	30 mm with reinforcement	41.32	94.27	24.66	Concrete cover tearing
	Beam-5	120×190×3200	Mechanical anchorage	T-sided	30 mm with reinforcement	29.94	72.63	71.63	Flexure

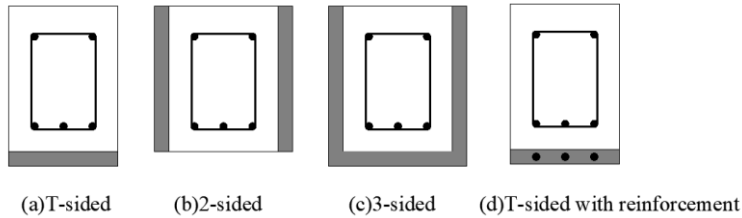


Figure 2.10 Strengthening configuration [33,96].

In general, UHPFRC strengthening enhances the bearing capacity and reduces the deflections of specimens, regardless of the bonding technique used (see Figure 2.11 and Figure 2.12). For all studied bonding techniques there is no significant difference in loading and deflection capacity and no interfacial failure is observed. Beams strengthened by different techniques fail in the same way. It is worth noting that the aforementioned conclusions are drawn based on results of strengthened beams without reinforcement in UHPFRC. When rebars are added in UHPFRC, epoxy bonding results in a larger capacity increase compared to mechanical anchorage, but the deflection ability of these beams is largely reduced and results in brittle failure. However, more tests are required to confirm this trend as it may vary with the changes in the dimension of concrete and/or UHPFRC. In addition, all bonding techniques greatly enhance the cracking resistance, with the highest efficiency in epoxy bonding and lowest in bonding with mechanical anchorage, because the cross-section of UHPFRC and concrete is weakened (see Figure 2.13).

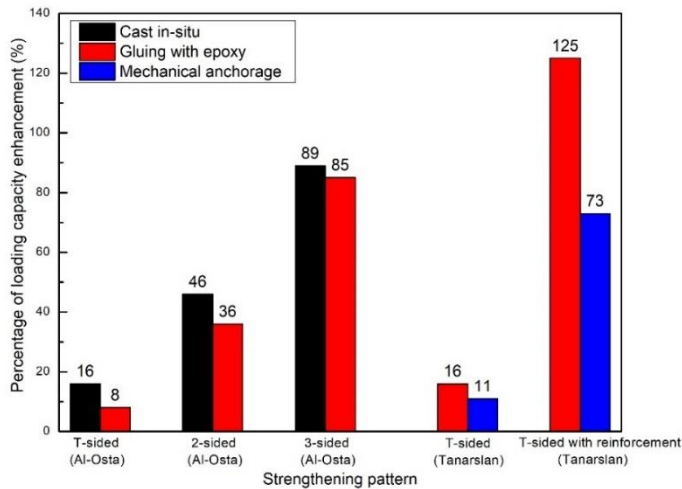


Figure 2.11 Loading capacity enhancement with different bonding techniques (calculated as $F_{u,s}/(F_{u,c}-1) \times 100\%$, $F_{u,s}$ is peak load of strengthened beam, $F_{u,c}$ is peak load of control beam) [33,96].

2.4 Behavior of UHPFRC-concrete interface and critical parameters

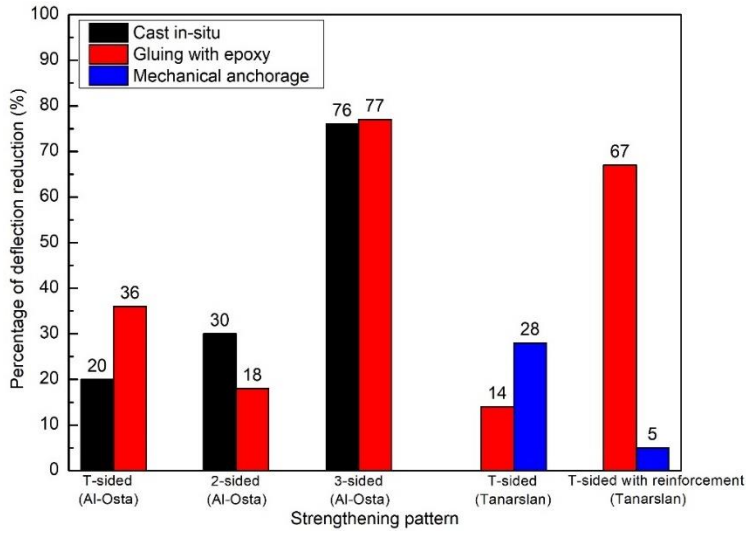


Figure 2.12 Deflection reduction with different bonding techniques (calculated as $(1 - \Delta s / \Delta c) \times 100\%$, Δs is displacement at peak load of strengthened beam, Δc is displacement at peak load of control beam) [33,96].

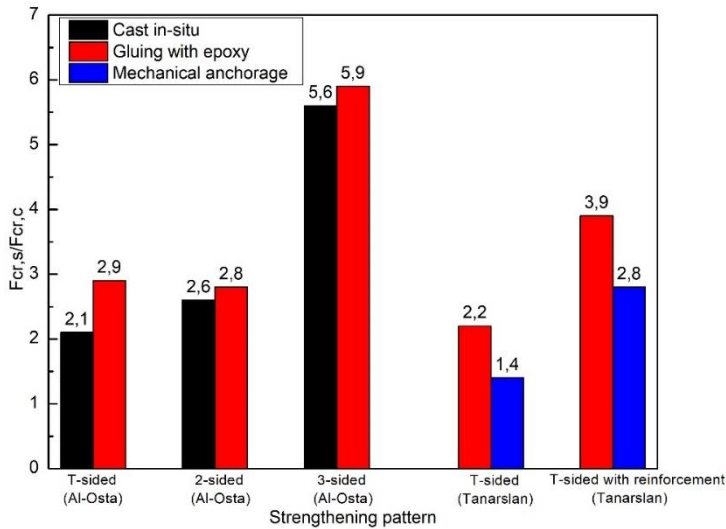


Figure 2.13 Cracking resistance improvement with different bonding techniques (denoted as $F_{cr,s} / F_{cr,c}$, $F_{cr,s}$ is initial cracking load of strengthened beam, $F_{cr,c}$ is initial cracking load of control beam) [33,96].

Studies on shear strengthening with UHPFRC by using different bonding techniques are lacking. Beside mechanical performance, type of bond also affects the durability. Although experimental results have shown that the impermeability of strengthened concrete beams with cast in situ UHPFRC is superior and the ingress of air and water is largely

reduced [79], there has been no comparative study assessing durability aspects for epoxy or anchoring bonding methods.

2.4.3 Influence of moisture exchange on interface behavior

Moisture exchange is relevant only for cast in situ applications. Restrained shrinkage stress and possible cracking/delamination are closely related to microclimatic state of the substrate (hygral and thermal conditions), hydration kinetics of freshly cast material and on-site environmental conditions [97]. The moisture exchange between the two materials affects the cement hydration and therefore determines the interface quality and the interface bond strength [98–100]. Although moisture exchange is reported to increase the air content at the vicinity of the interface [72] and slow down the hydration rate [42,44], it is also reported to improve mechanical anchorage between the two materials [43]. In addition, moisture exchange causes hygral gradients resulting in differential shrinkage and stress concentrations at the interface.

Unlike traditional concrete, UHPFRC has a very low content of water with respect to the cement dosage (w/c ratio). Compared to the strengthening material with a w/c ratio higher than 0.3 [42–44], the moisture state of substrate and the environmental conditions play a greater role for UHPFRC (i.e., w/c ratio less than 0.25). Given its low water content, self-desiccation due to hydration and fast water loss to surrounding environment (or concrete substrate), curing methods have a great impact on the quality of UHPFRC [101]. Although the rate of moisture exchange from UHPFRC to the substrate is lower compared to that of traditional concrete [102], in first hours the water from freshly cast cement (irrespectively of its w/c ratio) is absorbed by the old concrete at the same speed as free water [103]. Therefore, the moisture exchange might be detrimental, especially when a thin layer of UHPFRC is cast. Therefore, as a rule of thumb to have a high bond strength at the interface, for on-site applications with fresh UHPFRC, the existing concrete substrate should be saturated surface dry (SSD) prior to casting fresh UHPFRC. Figure 2.14 shows that in both tension and shear bond strength tests, for the three tested moisture conditions of the substrate, SSD is the optimal condition to obtain a higher bond strength for UHPFRC [104], which is contradictory to the finding for shear bond strength obtained for normal strength concrete in [43]. A systematic approach to investigate the role of moisture exchange in UHPFRC-NC composite is needed.

2.4 Behavior of UHPFRC-concrete interface and critical parameters

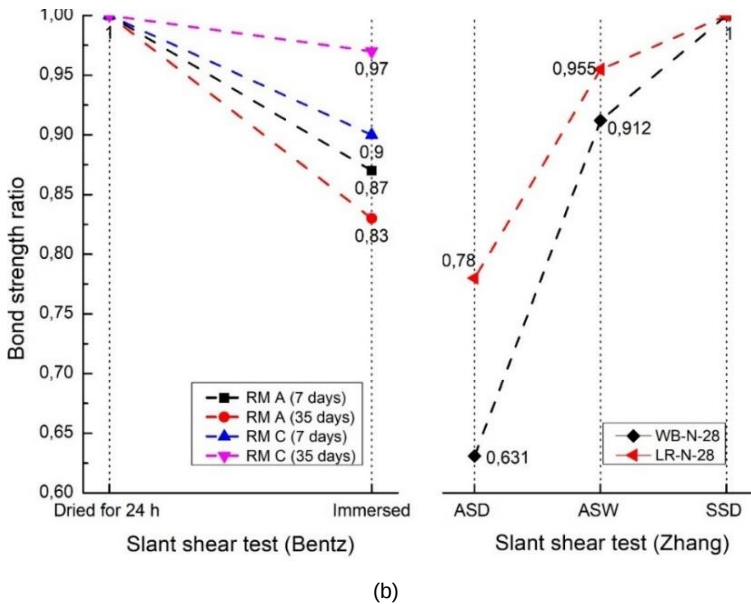
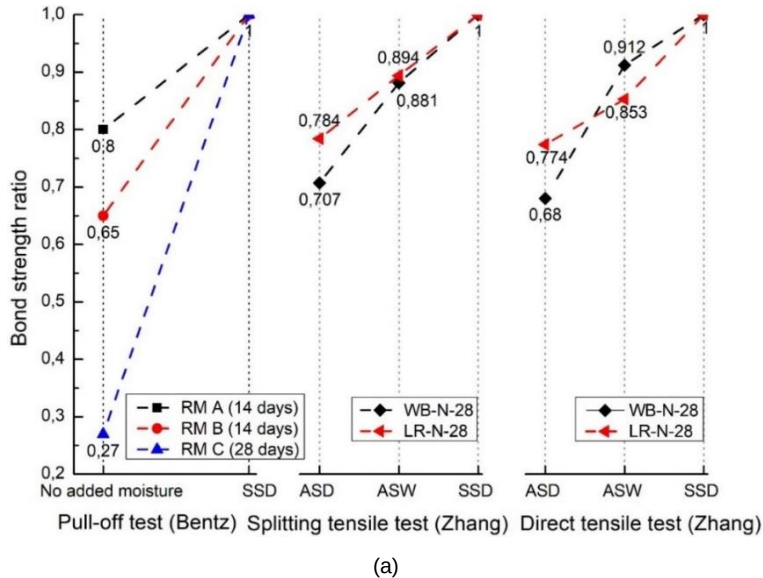


Figure 2.14 Effects of moisture state of the substrate on relative bond strength for different repair materials (named RM A, RM B and RM C, WB-N-28 and LR-N-28) and different moisture state of the substrate (SSD, ASD and ASW stands for saturated surface dry condition, air surface dry condition and air surface wet condition, respectively.): (a) Effects of moisture state of substrate on tensile bond strength for normal strength concrete (RM A, RM B and RM C) and UHPFRC (WB-N-28 and LR-N-28); (b) Effects of moisture state of substrate on shear bond strength for normal strength concrete (RM A, RM B and RM C) and UHPFRC (WB-N-28 and LR-N-28) [43,104].

2.4.4 Influence of shrinkage on interface behavior

When the hybrid structure is subjected to hygral/thermal gradients, there are stress concentrations possibly causing interface failure [105]. In this context, shrinkage is one of the most important parameters influencing the performance of a strengthened concrete structure (Figure 2.15).

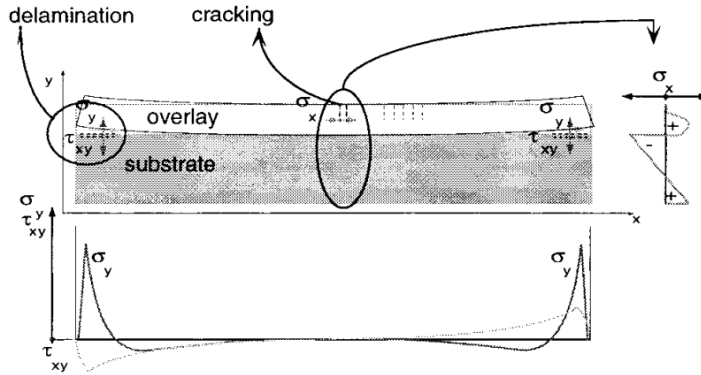


Figure 2.15 Damage mechanisms in strengthening systems [72,105].

In UHPFRC, the drying shrinkage is relatively low (less than $200 \mu\text{m/m}$) but autogenous shrinkage is more significant (greater than $400 \mu\text{m/m}$) [97,106–110]. For in-situ cast UHPFRC used in strengthening, it is important to account for internal stresses caused by restrained autogenous/drying shrinkage. Experimental studies on restrained shrinkage in UHPFRC-strengthened concrete structures [111,112] have shown that autogenous shrinkage is critical during the first 90 days after casting the UHPFRC layers and the shrinkage-induced tensile stress increases with a decrease in fiber content and thickness of the UHPFRC layer (Figure 2.16).

Due to its high autogenous shrinkage, UHPFRC is prone to early age cracking and a higher restrained degree will increase its cracking risk [113]. Tests on free shrinkage strain of reinforced UHPFRC and restrained shrinkage strain of reinforced UHPFRC-RC composites under different curing conditions (normal curing and steam curing) indicated that, the curing condition does not affect the ultimate measured shrinkage at 150 days (Figure 2.17 (a) and (b)). On the other hand, reinforcement inside the UHPFRC and the concrete substrate reduce the measured shrinkage [114]. It was not reported if the cracking pattern and cracks widths were affected by reinforcement.

Due to the high tensile strength and strain hardening property of UHPFRC, it is reported [115] that debonding is unlikely to occur at the UHPFRC-NC interface. Research has also focused on developing low-shrinkage UHPFRC to minimize the risk of delamination for concrete deck repair [116]. This low-shrinkage UHPFRC has been shown to effectively

2.4 Behavior of UHPFRC-concrete interface and critical parameters

mitigate shrinkage effects, as validated through ring test and UHPFRC-concrete composite slab tests [117]. Still, in certain conditions, the performance of UHPFRC may be compromised. For example, poor substrate preparation, harsh site climates, or inadequate connections between the UHPFRC and the parent concrete can reduce the tensile strength and deformability of UHPFRC [97]. Additionally, early exposure to drying condition (e.g., after one day) can increase the risk of curling or delamination when UHPFRC is applied as a repaired overlay [117]. Under such severe conditions and poor curing, combined with external loading, the debonding stresses could exceed the bond strength, potentially leading to delamination.

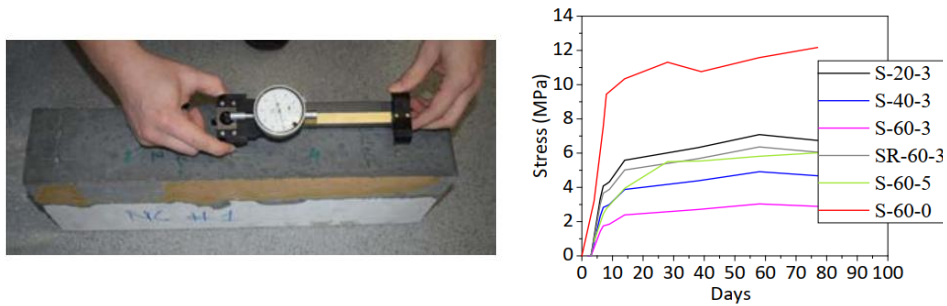


Figure 2.16 Restrained concrete shrinkage stresses (S and SR, respectively denote strengthened UHPFRC with or without rebars inserted in UHPFRC layer, 20, 40, 60 denotes the thickness of UHPFRC layer in mm and 0, 3, 5 denote the volume percentage content of steel fiber.) [112].

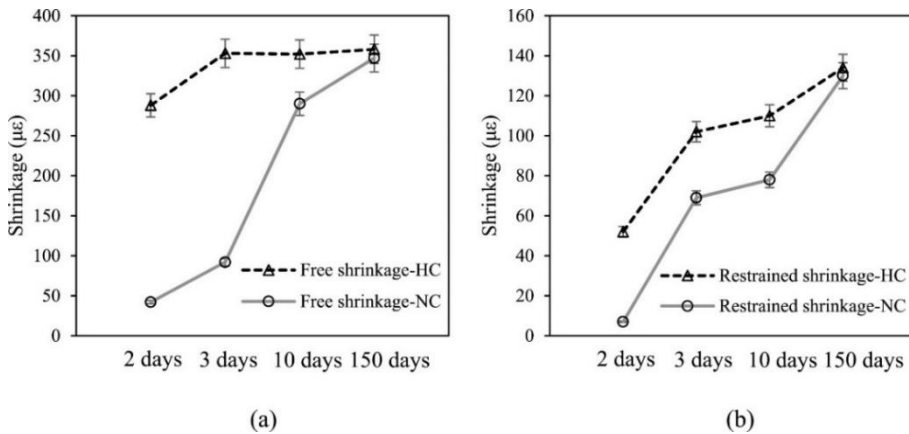


Figure 2.17 Free shrinkage in (a) reinforced UHPFRC and restrained shrinkage in (b) reinforced UHPFRC-concrete composite under different curing conditions (NC and HC respectively denote normal curing and steam curing conditions) [114].

2.4.5 Durability of concrete structures strengthened with UHPFRC

Durability and the life-cycle cost are two main factors to determine the quality of the strengthened structure [18,118]. The impermeability of UHPFRC is particularly critical for repair and strengthening in severe environmental conditions. Various experiments,

including water capillary absorption test [119,120], air/gas permeability test [121] and durability tests (including freeze-thaw, chloride penetration, etc.) [122], have confirmed the exceptional protective properties of UHPFRC. Its excellent durability, makes it an ideal protective layer, preventing the detrimental effect of aggressive environment. Guingot et al. [123] rehabilitated structures such as dams and bridge channels, using UHPFRC to protect against water impact, abrasion, and extreme environmental conditions. Denarié et al. [124] proposed a method to apply in-situ cast UHPFRC to protect a lighthouse in an aggressive marine environment.

Tayeh et al. [79] evaluated the permeability of the interface between the NC substrate and UHPFRC by rapid chloride permeability, gas and water permeability tests, concluding that it effectively resists the penetration of chlorides, and significantly reduces the gas and water permeation. Microscopic observations confirmed the quality at the interfacial transition zone. Muñoz [115] assessed UHPFRC-NC bond performance under freeze-thaw cycles and found out that splitting tensile strength increased with more cycles due to hydration of cement particles at the interface in presence of water (Figure 2.18). Similarly, Feng et al. [125] compared the durability performance of NC and UHPFRC as repair materials under wet-dry and freeze-thaw cycles. In terms of bond permeability, after exposure to 10 wet-dry cycles and 18 freeze-thaw cycles, both UHPFRC and NC show increased penetration rates. However, NC exhibits a penetration rate nearly five times higher than that of UHPFRC. Regarding bond strength, NC experiences a significant decrease (30.8%) after 10 wet-dry cycles, compared to a 9.5% decrease for UHPFRC. Similarly, after 18 freeze-thaw cycles, the bond strength of NC drops by 20.5%, while that of UHPFRC decreases by only 6.9%. These findings underscore the superior resistance of UHPFRC interface to moisture penetration and degradation compared to normal concrete interface, particularly under harsh environmental conditions.

To make it more close to the in-situ conditions, Tan et al. [126] also combined the freeze-thaw cycles with fatigue loads to evaluate the bond performance at concrete-UHPFRC interface. Results confirm that UHPFRC shows satisfactory freeze-thaw resistance and the bond strength remains unaffected even with the addition of fatigue loads. Additionally, Baloch et al. [127], examined the bond behavior of wet-on-wet connected self-consolidating concrete (SCC) and UHPFRC when exposed to high temperatures. They concluded that bond resistance was improved due to the physical anchorage provided by the steel fibers. However, as the fibers are unable to penetrate the interface and reach the concrete substrate, the observed increase in bond resistance might be also due to the restraining effect of the steel fibers, as observed by Guo et al. [128].

2.4 Behavior of UHPFRC-concrete interface and critical parameters

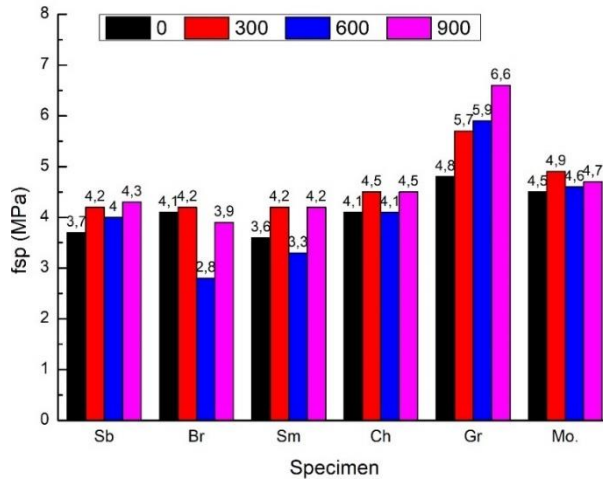


Figure 2.18 Effect of freeze-thaw cycles on splitting tensile strength of composite specimen (f_{sp} = splitting tensile stress, Sb = sandblasted; Br = brushed; Sm = smooth; Ch = chipped; Gr = grooved; Mo = monolithic (only concrete)) [115].

Although the superior durability of UHPFRC-NC composites has mostly been validated through small-scale bond strength tests, the deeper multiscale (covering macro-, meso- and microstructural) analysis of the interfacial zone and interface behavior under (severe) environmental loads is still lacking.

2.4.6 Interface performance (debonding and slip) in structural tests

Experimental investigations into the flexural behavior of composite UHPFRC-NC elements have demonstrated that no debonding is observed in the interface zone, and the beams exhibit monolithic behavior before reaching maximum resistance [25,64,94,129,130]. Bond strength tests, including the pull-out, indirect tensile and slant shear tests, have shown that the bond strength between UHPFRC and concrete is greater than the tensile strength of concrete [79,115]. Consequently, the interfacial slip is negligible and not considered in the analysis. Hence, both numerical modeling and theoretical calculations [33,58,131,132] typically assume a perfect bond. However, while these models accurately predict load-deflection response, they often fail to predict crack pattern [35]. Moreover, it cannot be generalized that RC structures strengthened with UHPFRC are unlikely to fail due to interfacial debonding. As shown in Figure 2.19, the interfacial debonding may still happen [23,96]. In both [23] and [96], the epoxy-bonded technique is used to bond the prefabricated UHPFRC laminate and concrete substrate.



Figure 2.19 Debonding at the interface [23,96].

It is reported [93] that compared to the interface slip between two conventional concretes, the slip between UHPFRC and conventional concrete interface is significantly lower, because UHPFRC with steel fibers has a higher bond strength. In [77], for fiber reinforced concrete with poly-vinyl-alcohol (PVA) fibers, it was explained that fibers prevent the bond failure as they reduce the damage due to shrinkage and thereby increase the cohesion at the interface, leading to a higher bond strength value. Another potential reason is the reduction of bleeding at the interface due to fiber reinforcement. Still, slip can remain large, such as with different strengthening configurations [133]. Moreover, design code “Structural Connections for Precast Concrete Buildings” [134] sets strict slip allowance values, with maximum permissible slips of 0.2 mm at service and 2 mm at the ultimate state. Measurements by Paschalis [59, 103] [93,135] have recorded interfacial slips exceeding 0.3 mm, although effective bonding was still achieved due to dowel connections reinforcing the interface. Tsioulou [92] found that interface preparation and the type of strengthening, including tensile strengthening with UHPFRC along (1) the whole beam length and (2) limited length, and (3) compressive strengthening along the whole beam length affects the strengthening efficiency. For rough interface, the tensile strengthening greatly improved the loading capacity, while tensile strengthening with limited length showed the worst performance due to the debonding between UHPFRC and the concrete layers. Even more, this beam showed a maximum slip value (almost 9 mm) resulting from mechanical loading.

Restrained shrinkage also affects slip or debonding, as discussed in Section 2.4.4. FEM analyses show that specimens strengthened with three-sided jacketing exhibit the lowest slip at ultimate resistance, whereas tensile-side strengthening results in the highest slip (Figure 2.20) [133]. Recent numerical studies are starting to incorporate interface models. For example, based on the push-off bond strength test results, Mohr-Coulomb equation from Eurocode 2 [55] is used to describe interface behavior, showing good agreement

2.4 Behavior of UHPFRC-concrete interface and critical parameters

with experimental results for UHPFRC-strengthened beams [93]. Zhu et al. [136] compared two different interface modeling techniques, finding that adhesion and friction based on AASHTO [137] interface concept was more effective than the friction based on ACI [54] to simulate load-displacement response of concrete slab strengthened with UHPFRC. Accurate interface modeling in analytical and FEM models is crucial for understanding composite structural performance, evaluating debonding risks, and optimizing UHPFRC strengthening efficiency.

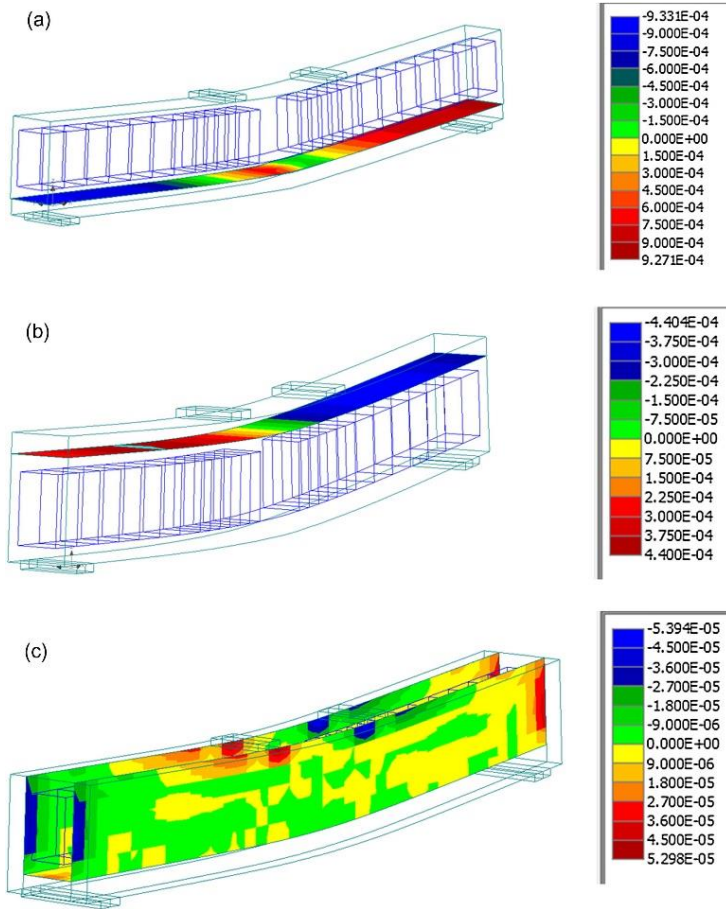


Figure 2.20 Interface slip (m) for strengthened beams at ultimate resistance with different UHPFRC configuration: (a) the tensile side, (b) the compressive side, and (c) three sides jacket [133].

Finally, most studies on UHPFRC-NC composite members have focused on only mechanical loading, such as flexural, shear, punching shear, and torsional loading, under controlled laboratory conditions [33,58,67,96,129,130,132,138–140]. There is a need for research on the combined effects of environmental conditions and mechanical loads, as

commonly observed in practice, as these can create more severe gradients and stress concentrations at the interface, potentially leading to premature failure.

2.5 Non-destructive methods to assess UHPFRC strengthening efficiency

For concrete structures strengthened with UHPFRC, the bond quality at the interface and mechanical properties of UHPFRC (especially its tensile performance) are two governing parameters that influence the strengthening efficiency. Generally, in order to determine the mechanical properties of materials and their bonding properties, the destructive test methods such as commonly used material test (e.g., compressive, tensile, and bending tests) and bond strength test are commonly used. Non-destructive testing (NDT) methods attract more attention since they enable a quick and cost-effective health assessment of strengthened structures without causing damage and can provide valuable on-site performance evaluation. This section provides a brief state-of-the-art about the current status, challenges and prospects of NDT methods employed in quality control and assessment of UHPFRC structures, focusing on the evaluation of UHPFRC material properties and bond performance for UHPFRC-RC composite structures.

2.5.1 Material properties evaluation of UHPFRC

There have been several commonly used NDT methods to evaluate UHPFRC properties. Regarding the elastic properties of UHPFRC, as a pilot study, Washer et al. [141] investigated the applicability of the ultrasonic pulse wave (UPV) to estimate the elastic properties of UHPFRC, so called reactive powder concrete (RPC). It is found that although the presence of steel fibers lowers the wave velocities of the ultrasonic wave, similar to conventional concrete, the RPC can be regarded as an isotropic material within elastic limits. This enables quantification of elastic modulus, based on the relationship between the elastic modulus and ultrasonic wave velocity. Hassan and Jones [142] used UPV and resonant frequency methods to determine the compressive elastic modulus and the Poisson's ratio of UHPFRC with 2% steel fibers. Results showed that although both methods can provide acceptable results, UPV is more promising since it is easy and cost effective. The resonant frequency method can only be used on the laboratory scale, and the size and shape of tested objects are limited. To improve the reliability of NDT results, rather than depending on a single method, Tsioulou et al. [143] first combined different NDT methods (that is, the combination of UPV and Rebound Hammer, termed 'SonReb') to evaluate the mechanical properties of UHPC and UHPFRC. SonReb method provided the most accurate results in terms of compressive strength and modulus of elasticity compared to the individual results from UPV and Rebound Hammer. Furthermore, since material composition will influence the correlation between compressive properties and NDT signals, Najm et al. [144] applied rebound hammer and UPV methods, and varied the fiber length and volume in UHPFRC. The study revealed that the rebound hammer test can provide more reliable results on matured fiber reinforced concrete than young fiber reinforced

concrete. For the UPV method, the fiber length affects the measurement more than the fiber volume as the recorded velocity is higher with longer steel fibers. Al-Huri et al [145] also used rebound hammer, UPV and their combination for the estimation of compressive elastic properties by varying the water/binder ratio, cement and silica fume contents in UHPFRC and curing time. The proposed model taking into account the duration of curing can improve the accuracy of the predicted compressive strength.

Not only can the compressive performance of UHPFRC be evaluated, but its tensile properties, which are governing its repair/strengthening performance in existing concrete structures, can also be assessed by NDT methods. The tensile performance of UHPFRC is primarily determined by the fiber dispersion and orientation in UHPFRC, which is easily affected by many factors, such as the workability of UHPFRC, and casting method and procedure [146,147]. For example, Zhao et al. [148] investigated the effect of vibration time on steel fiber distribution and the flexural performance of steel fiber reinforced concrete (SFRC). Their findings revealed that prolonged vibration caused fiber settlement, leading to segregation in highly flowable SFRC and a reduction in flexural strength, toughness, and fracture energy. Wang et al. [149] examined the effects of vehicle-bridge coupling vibration on UHPFRC, showing that increased vibration amplitude and frequency improved compressive strength and drying shrinkage but weakened flexural and tensile strength due to changes in fiber distribution. Similarly, Leng et al. [150] studied the influence of different vibration parameters (i.e., amplitudes, frequencies, and phases) on the macroscopic, nano-mechanical, and microstructural properties of unhardened UHPFRC. They found that low-energy vibration (before initial setting and with low amplitude) refined the pore structure, optimized fiber orientation, and enhanced both compressive and flexural strengths. However, high-energy vibration (after initial setting and with high amplitude) coarsened the pore structure, disrupted aggregate-matrix interactions, and introduced microcracks in the interfacial transition zone, ultimately weakening the performance of UHPFRC. Therefore, measurement of the actual fiber distribution in UHPFRC elements by NDT methods can help to evaluate the structural performance of UHPFRC members.

CT scanning, unlike image analysis that requires cutting the specimen, can quantify the content and orientation of fibers by creating cross-sectional images without damaging the sample. The 2D and 3D results of the fiber distribution can be obtained. However, the CT scanning technique is generally expensive and limited to small-scale samples, making it difficult for in-situ examination of UHPFRC structures. Therefore, other advanced technologies, such as the electricity method [151] and the magnetic method [152] are proposed for the indirect evaluation of fiber alignment. The electric method relies on the effect of steel fibers on the conductivity of UHPFRC and the magnetic method is based on the ferromagnetic properties of the steel fibers. Barnett et al. [151] evaluated the fiber orientation in UHPFRC panels using the electricity method, and the effectiveness was

confirmed by CT scanning. Nunes et al. [152] developed the magnetic method to measure fiber content and orientation in UHPFRC and later [41] investigated the effect of key factors such as geometry and surface roughness of the elements, and fibers segregation on the NDT results. Based on the measured results in UHPFRC, the tensile strength of UHPFRC can be estimated [153] and the effect of fiber orientation on the tensile response of UHPFRC is evaluated [154]. Despite the proved high efficiency of electricity and magnetic methods, it should be noted that both methods do not allow direct observation of fibers and are easily influenced by the surrounding environmental conditions. In addition, the magnetic method requires standard correlation between magnetic permeability and specific fiber contents prior to measurement.

To monitor the quality of UHPFRC during the manufacturing and curing process (e.g., to quantify the initial defect and damage of UHPFRC), the acoustic emission (AE) technique has also potential [155].

2.5.2 Delamination detection at the interface

It is well known that improper strengthening strategies will lead to the delamination of the UHPFRC-NC composite structures and greatly weaken the strengthening efficiency [23,96,156]. Therefore, the bonding performance to existing structures is of great concern after strengthening application of UHPFRC. In reality, interfaces are always located inside reinforced structures and hard to observe by visual inspection. Therefore, it is of a great interest to evaluate quality after retrofitting with UHPFRC with the help of the NDT technique [157,158].

Several NDT techniques, including ultrasound [159], radiography [160], radar [161], impact echo [162], and infrared thermography (IRT) [38], have been developed to inspect and monitor defects in concrete structures efficiently. Ultrasound and radiography could identify internal flaws and defects quickly and accurately, but the equipment is expensive, and the results are also easily affected by the surrounding environment due to noise. Furthermore, ultrasound can only detect the flaws within local structures, and radiography may arise safety issue [158]. In terms of radar and impact echo, they can provide immediate and efficient detection of internal damage; moreover, radar can even detect flaws several meters underneath the surface of the specimen, and impact echo can work regardless of surface profile of the structures. Radar might provide false results especially in reinforcement congested area, while impact echo has limited detection depth and resolution compared to radiography and radar [163]. IRT is a relatively new NDT method and can be classified into two kinds: passive and active measurements [37]. Passive IRT means that the object itself is regarded as a heat source, and the heat emitted by the surfaces of the objects is recorded by the infrared sensors without additional heat excitation. However, the results of passive IRT are easily influenced by the environmental condition. For active IRT measurement, external heating is necessary to heat the objects' surface

2.6 Conclusion

evenly firstly. Then the heating source is removed, and during the cooling process of the heated objects, the defects could be detected by analyzing the temperature gradient along the surface of the objects. Compared to passive IRT, active IRT can provide quicker and more accurate analysis, as the heat can penetrate inside objects faster and the heat source can generate a controlled temperature field, which helps locate and measure the size, shape and depth of the defects [38].

Previous research [164–169] has shown the advantages of IRT to detect debonding/delamination of composite concrete structures. In order to examine the internal delamination of the reinforced concrete slab, Raja et al. [164] investigated the effect of the ambient environmental condition (i.e., wind velocity and solar radiation) on the accuracy of the IRT measurement and determined the optimal time for data collection from infrared sensors. The results showed that both wind and heat flux will significantly influence absolute thermal contrast on the monitored surface and absolute thermal contrast decreases if a higher wind speed or lower heat flux and heating time is provided. Gu et al. [165] applied active IRT to detect the delamination of concrete members reinforced with CFRP jacketing, and parameters including the interface opening (delamination thickness) and size of the deboned zone, as well as water content and the existence of covered mortar in the delaminated area were analyzed. Active IRT was found to be able to detect the delamination size and the interface opening, and the detectability will increase as the opening at the interface (delamination) and delamination size are larger but decrease in the presence of water and additional layer of covered mortar. Subsequently, Gu et al. [166] also evaluated the effectiveness of passive IRT to detect delamination of concrete-CFRP composite members under different weather conditions. They observed that the accuracy of the results was greatly influenced by the duration of the testing and data collection frequency while barely affected by the ambient temperature. Although IRT is promising in providing reliable information for the FRP wrapped concrete structure, concrete has a much lower thermal conductivity compared to FRP, which will in turn reduce the thermal contrast between the defected and the sound areas, and decrease the detectability [170]. Considering that concrete jacketing [171] is also commonly used for repair/strengthening applications, the potential of IRT for the interface quality assessment between concrete and UHPFRC is unknown.

2.6 Conclusion

This chapter reviews the state-of-the-art on the performance of concrete structures strengthened with UHPFRC, focusing on shear strengthening. Many aspects were discussed, starting from structural behavior at the macroscale, available numerical and analytical methods, through the interface behavior at the mesoscale and its governing parameters. Finally, non-destructive testing methods for material and interface quality evaluation are reviewed. The following conclusions can be drawn:

(1) The shear improvement of existing RC structures reinforced with UHPFRC has been increasingly studied in experimental and numerical studies. However, the structural behavior of shear-deficient RC structures strengthened with UHPFRC under combined mechanical and environmental loads (e.g., wetting-drying, or freeze-thaw cycles), which reflects practical conditions, is still unknown.

(2) Based on the experimental observations (in controlled lab conditions) that a strengthened structure behaves almost monolithically, a perfect bond is generally assumed at the interface. New finite element approaches should be developed to better understand the influence of UHPFRC-concrete interface on the shear resistance of the strengthening system. Furthermore, the mechanical properties (especially the tensile response) of UHPFRC also govern the shear resistance of strengthened RC beams. However, how the tensile properties of UHPFRC contribute to the shear resistance remains unclear. More specifically, it is not clear whether the high materials strength of UHPFRC or its ductility play a more critical role in enhancing capacity of strengthened structures. A better understanding of this could help in optimizing the UHPFRC mixture.

(3) For the UHPFRC shear strengthening applications, the 2-sided strengthening configuration, although not optimal for durability reasons, appears to be the optimal method for enhancing the resistance under mechanical loads. The effect of bonding techniques, moisture exchange between concrete substrate and UHPFRC, restrained shrinkage, and combination of mechanical and environmental loads is still unclear in view of shear capacity of composite beams.

(4) The tensile properties of UHPFRC, which are important for strengthening efficiency with UHPFRC, are highly dependent on its fiber distribution and orientation. The electromagnetic method, based on the ferromagnetic properties of the steel fibers, is promising to evaluate the fiber dispersion for UHPFRC elements in both laboratory studies and in-situ application.

(5) The subsurface interface quality, which governs the integrity of the composite UHPFRC-NC structures, is difficult to examine by direct observation. Active infrared thermography, relying on the thermal contrast between sound and delaminated regions, has great potential in determining the location, size and depth of the possible debonding.

Chapter 3 Experimental methods

3.1 Introduction

In this chapter, the materials and methods used in the experimental program are introduced. In Section 3.2, the raw materials and mixture proportion are presented. In Section 3.3, the experimental procedure to measure the mechanical properties (compressive, tensile strength and elastic modulus) of UHPFRC and NC, as well as workability and shrinkage (free and restrained) of UHPFRC are outlined. Finally, in Section 3.4, a structural test designed to evaluate the shear performance of reinforced concrete beams strengthened with UHPFRC is presented. Details of the specimen design, experimental setup and measurement system are given. Parameters including surface saturation of existing concrete, casting direction of UHPFRC, differential shrinkage and different bonding techniques between UHPFRC and existing concrete under combined environmental (freeze-thaw) and mechanical loads are investigated.

3.2 Materials

3.2.1 Raw materials and proportion of NC and UHPFRC

Concrete strength class C20/25 (w/c ratio 0.60) is used for the fabrication of reinforced concrete beams. The concrete consists of CEMI 42.5N cement, aggregates with a maximum aggregate size of 16 mm, water, and superplasticizer (SP). For the UHPFRC mixture, CEMI 52.5R cement, blast furnace slag (BFS), silica fume (SF) and fine sands with a maximum aggregate size of 1 mm are used. A low water-cement ratio (0.25) and fine particle size distribution are required in UHPFRC to achieve a dense matrix, with low porosity and thereby high compressive strength. Due to the low water-cement ratio, a significant amount of superplasticizer is used to achieve good workability. Two UHPFRC mixtures with steel fibers having the length of 13 mm and varying diameters (i.e., 0.16 mm and 0.2 mm) have been developed. The mixture designs are given in Table 3.1. The properties of the two fiber types are listed in Table 3.2.

Table 3.1 Mix design proportions (in kg/m³).

Mixture	Cement	BFS	SF	Sand	Gravel	Water	SP (Master Glenium 51)	Steel fiber 1	Steel fiber 2
Concrete	260	-	-	847	1123	156	0.26	-	-
UHPFRC1	870	104.4	43.8	1061.1	-	219.9	26.6	125	-
UHPFRC2	858.9	103.1	43.2	1047.5	-	217.1	26.3	-	156.3

Table 3.2 Physical and mechanical properties of different fibers.

Type of fibers	Length (mm)	Diameter (μ m)	Density (kg/m ³)	Nominal tensile strength (MPa)	Young's modulus (GPa)
Steel fiber 1	13	160	7850	2750	200
Steel fiber 2	13	200	7850	2750	200

3.2.2 UHPFRC mixing procedure

For mixing, an ordinary drum mixer with casting volume of 35L is used. The UHPFRC mixing procedure is optimized to realize the acceptable flowability. The mixing process requires 30 min and consists of the following steps (Figure 3.1):

- (1) Dry materials, except the sand with the largest aggregate size (0.5-1.0 mm), are mixed for 5 minutes.
- (2) 90% water is added and mixed for 2 minutes.
- (3) The remaining water with superplasticizer is added and mixed for approximately 10 minutes to achieve a flowable mixture.
- (4) Remaining sand with the largest aggregate size is added and mixed for 3 minutes to achieve a flowable mixture.
- (5) The steel fibers are added in five batches sequentially, with each batch being mixed for 2 min.

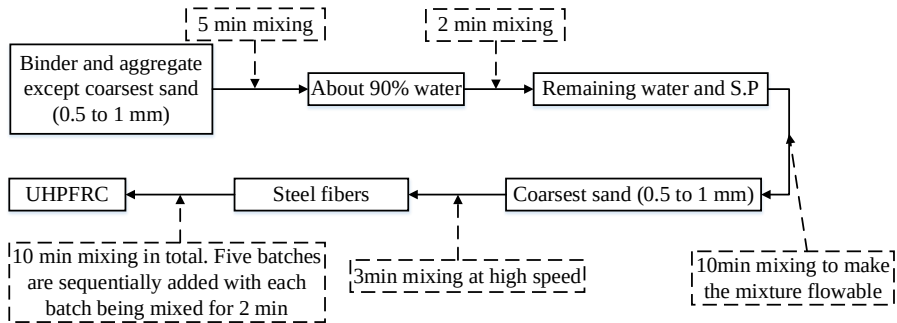


Figure 3.1 Detailed mixing procedure for UHPFRC production in the drum mixer.

3.2.3 Specimen preparation and curing regime of NC and UHPFRC

For compressive strength tests, 100 mm cube specimens are prepared for NC, while for UHPFRC, $40 \times 40 \times 160 \text{ mm}^3$ prismatic samples are cast and cut into 40 mm cubes. For elastic modulus measurements of both NC and UHPFRC, specimens are cast using prismatic molds of $100 \times 100 \times 400 \text{ mm}^3$. Regarding the tension test, 100 mm cubes are cast to measure the splitting tensile strength of NC, and dumbbell samples, recommended by JSCE [19] (Figure 3.2) are fabricated to measure the response of UHPFRC subjected to uniaxial tension.

3.3 Material tests

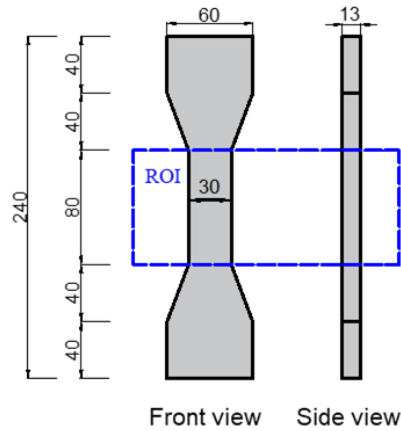


Figure 3.2 Dimensions of the dumbbell specimens (unit: mm).

After casting, all specimens are covered with plastic sheets to prevent moisture loss and subsequently demolded after one day. After demolding, all samples are wrapped with the plastic film and sealed with tape to avoid moisture loss until testing.

3.3 Material tests

3.3.1 Slump flow and final setting time

The flowability of the UHPFRC mixtures is evaluated based on the procedures of EN 1015-3 [172] and the final setting time of UHPFRC is measured using a Vicat setup [173].

3.3.2 Free and restrained shrinkage test

Stress induced by restrained shrinkage cannot be neglected when fresh UHPFRC is cast on the hardened concrete due to the shrinkage difference [174–176]. In this study, we particularly focus on understanding of the role of shrinkage at the interface of strengthened beams. To gain insight, both autogenous and total shrinkage of UHPFRC are measured in material tests.

Two UHPFRC mixtures are used in the current study. The UHPFRC1 mixture is mainly used to study in detail the shrinkage effect in beam tests. The total shrinkage measurement starts at the age of seven days to simulate the conditions of the structural test, as the beam specimens are exposed to laboratory conditions from the age of seven days. In the first seven days, the beams are sealed and there is only moisture transfer between the two materials (the concrete and UHPFRC) [102] (and not the environment). Therefore, three methods are used to measure autogenous shrinkage: (i) the corrugated tube method and (ii) Autogenous Deformation Testing Machine (ADTM) method, both measuring shrinkage immediately after casting, and (iii) prism method measuring shrinkage after hardening (e.g., after 1 day in this study). In addition to free shrinkage tests, the stress development induced by restrained autogenous shrinkage and cracking potential are studied

through Temperature Stress Testing Machine (TSTM) test. For the UHPFRC2 mixture, as only the type of fibers differs, the total and autogenous shrinkage are measured only on hardened prisms from the age of 1 day. This allows for a comparison of the shrinkage between the two mixtures.

For shrinkage measurements on hardened UHPFRC, UHPFRC samples with the dimension of $40 \times 40 \times 160 \text{ mm}^3$ are cast. At the predefined age, the specimens are demolded. The autogenous shrinkage measurement is performed on three specimens wrapped with aluminum foil and sealed with tape to avoid moisture loss (shown in Figure 3.3 (a)). The other three samples, on which total shrinkage measurement is performed, are left unsealed (shown in Figure 3.3 (b)). All samples are placed in a room with a constant temperature of $20 \pm 2 \text{ }^\circ\text{C}$ and a relative humidity of $50 \pm 5\%$. Free deformation measurement is performed using a comparator (accuracy of 0.001 mm) according to the ASTM C157 [177].



Figure 3.3 Testing set-up for autogenous and total shrinkage measurements on hardened UHPFRC: (a) autogenous shrinkage measurement; (b) total shrinkage measurement.

For UHPFRC, the significant part of autogenous shrinkage occurs at a very early age (within the first 24 h) [178], which is not measured in the test described above. Therefore, the corrugated tube method (shown in Figure 3.4) is used to measure the autogenous shrinkage directly after casting. But it should be noted that some inaccuracies might be introduced since the rib of the tube may influence the fiber distribution of UHPFRC. Under gentle vibration, fresh UHPFRC is cast into two corrugated tubes with a length of 425 mm and diameter of 28.5 mm and then closed by plugs. Then tubes are placed to the calibrated dilatometer. The deformation is measured every 5 min for 7 days by linear variable differential transformers (LVDTs) with an accuracy of 0.001 mm. The final setting time is defined as starting point of autogenous shrinkage measurement [110]. It should be noted that compared to ultra-high performance concrete (UHPC), the inclusion of steel fibers in UHPFRC can reduce autogenous shrinkage to some extent [179]. This

3.3 Material tests

reduction is primarily due to the mechanical restraint provided by the fibers, which are bonded to the matrix and resist deformation [180].



Figure 3.4 Corrugated tube method for autogenous shrinkage measurement of UHPFRC.

The autogenous deformations and eigenstresses induced by restrained autogenous shrinkage of UHPFRC are also investigated. The test setup is composed of two parts: (a) Autogenous Deformation Testing Machine (ADTM) and (b) Temperature Stress Testing Machine (TSTM). The free autogenous deformation is measured by the ADTM. As shown in Figure 3.5, prisms with a dimension of $1000 \times 150 \times 100 \text{ mm}^3$ are cast into the wooden mold with low thermal conductivity. To avoid moisture loss and drying shrinkage, the prismatic specimen is fully covered by plastic film. Furthermore, the temperature of the specimen is controlled at 20°C by the circulation water tubes around the specimens to avoid thermal expansion, and three thermocouples are embedded in the middle and two near the end of the specimens to monitor temperature development. In the ADTM setup, the specimen is allowed to deform freely, and steel bars are embedded in the specimen to measure its deformation. LVDTs with a resolution of 0.001 mm are attached at both ends of steel bars to record the deformation of the specimens and the distance between two embedded steel bars on one side is 750 mm .

In the TSMT setup (shown in Figure 3.6), a dog-bone sample is cast that has the same region of interest ($1000 \times 150 \times 100 \text{ mm}^3$) as the ADTM, but is fully restrained at the ends of the dog-bone sample by keeping the distance between two embedded steel bars constant (750 mm). The temperature is also controlled at 20°C and thermocouples are embedded to monitor it. One end of the specimen is fixed, while the other end can move with the hydraulic actuator. Therefore, the actuator can restrain the deformation measured by four LVDTs while the load applied by the actuator (equals to the internal load resulting from autogenous shrinkage) can be recorded. Different from the corrugated tube setup, the end of autogenous expansion is considered as ‘time zero’ for ADTM and TSTM tests as it is the starting time that tensile stress initiates. Stress is recorded every minute until the age of 28 days.

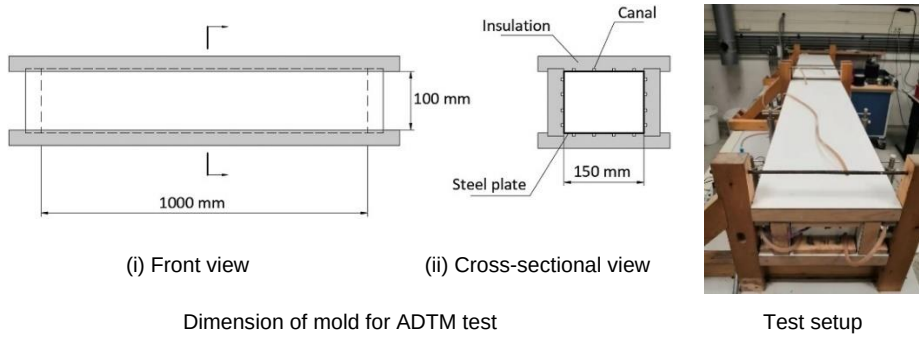


Figure 3.5 ADTM used for free autogenous strain measurements of UHPFRC [181].

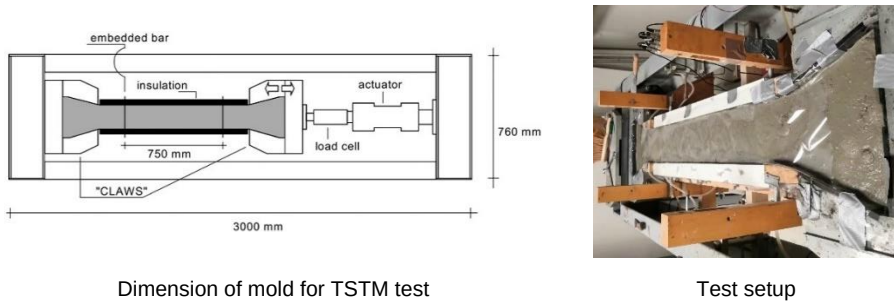


Figure 3.6 TSTM for shrinkage-induced stress measurement of UHPFRC [181].

3.3.3 Compressive test

The compressive strengths of NC and UHPFRC are determined according to NEN-EN 12390 [182], using cubical samples with dimensions of 100 mm and 40 mm, respectively. Three cubes are tested to obtain the representative compressive strength.

3.3.4 Elastic modulus test

To measure the elastic modulus of NC and UHPFRC, two prisms ($100 \times 100 \times 400 \text{ mm}^3$) are tested by a Tonibank machine. Linear variable differential transducers (LVDTs) are vertically attached on the four sides of the specimens to measure the deformation over a gauge length of 135 mm under compressive load. The loading and unloading rates are 1 kN/s and each sample goes through three cycles with the stress ranging from 0.5 MPa until a compressive stress corresponding to 30% of its compressive strength.

3.3.5 Tensile test

The splitting tensile test is conducted to obtain the tensile strength of concrete. For UHPFRC, dog-bone specimens (recommended by JSCE [19]) are cast and tested under the direct tension (Figure 3.7). Note that the thickness of the dog-bone specimen (13 mm) is somewhat larger than that of the strengthening laminates applied in the subsequent beam specimens (10 mm). Therefore, due to preferential fiber orientation in thinner samples,

3.3 Material tests

laminates are expected to have better tensile properties compared to dog-bone specimens [183]. The test is conducted at a constant loading rate of 0.003 mm/s, and the deformation of the specimens over a gauge length of 80 mm is recorded by two LVDTs attached on the specimen.

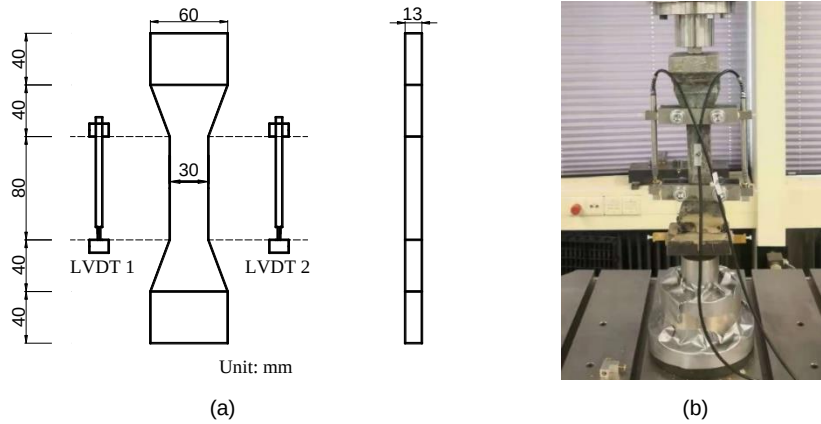


Figure 3.7 Direct tensile tests (a) geometry (in mm) and (b) experimental setup.

3.3.6 Freeze-thaw exposure

The number of freeze-thaw (FT) cycles of the specimens are chosen as 30, 75 and 120 FT cycles to simulate a duration of minimum four years of environmental exposure in the Netherlands. As estimated in [184], the average number of FT cycles in the Netherlands is around 30 cycles per year. This is based on the definition where one FT cycle is defined as the maximum temperature above 0 °C and the minimum temperature below 0 °C during the day. If one FT cycle is defined as the maximum temperature of at least 0 °C and the minimum temperature of -2.2 °C or lower, the average number of FT cycles in the Netherlands is around 30 cycles per two years. In this study, the experiment is designed to investigate the performance of UHPFRC strengthened concrete structures during the initial service period (at least the first four years), as it might be the most susceptible period for the composite system [185] and the bond between the strengthening material and the existing concrete [186–188]. The freezing process is carried out in a cooling environment with an average relative humidity (RH) of 70%, and the thawing process is done in a climate condition with a relative humidity of around 95% to accelerate the deterioration process. The moisture content in the concrete governs the freeze-thaw resistance since fatigue stress induced by volumetric change during the alternative freezing and thawing of pore water will lead to cracking [189]. One FT cycle lasts for about 24 h at the ambient temperature ranging from -20 °C to 20 °C and it consists of two stages, as shown in Figure 3.9. In stage I, the specimens are placed at sustained room temperature (20 °C) for 8 h. Stage II consists of a rapid cooling process in freezing chamber, where

the specimens are exposed to a constant temperature at $-20\text{ }^{\circ}\text{C}$ for 16 h. It should be mentioned that, before the FT tests, a temperature sensor (shown in Figure 3.8) is embedded in the core (center in length, width and height) of a hybrid beam specimen in order to measure the internal temperature evolution during the freezing and thawing processes (as presented by two blue lines in Figure 3.9). The internal temperature is only recorded when the specimens are in thawing condition as in freezing conditions the monitoring system does not work. It is found that during the thawing phase (8 h), the internal temperature of the specimens raises to more than $18\text{ }^{\circ}\text{C}$. To investigate the influence of FT cycles on the material deterioration of NC and UHPFRC at an age of more than 28 days, apart from beam specimens, samples to determine the material properties are prepared. The appearance and mass change of the specimens are inspected at an interval of 10 or 15 cycles, and the compressive strength, elastic modulus, and tensile properties of concrete and UHPFRC after 30, 75 and 120 cycles are measured.



Figure 3.8 Temperature sensor to detect internal temperature variation at the core of the beam specimen during the FT test.

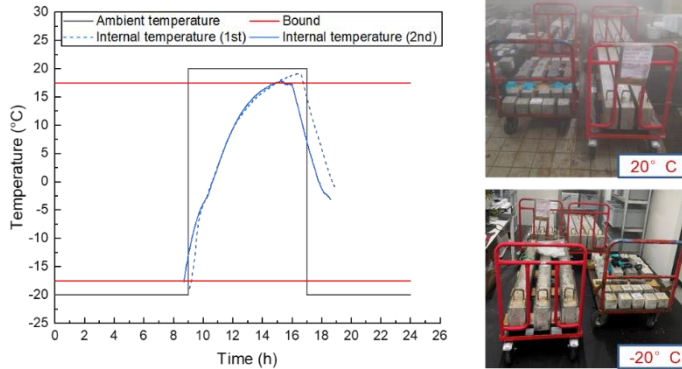


Figure 3.9 Freeze-thaw protocol on specimens.

3.4 Structural tests

Structural tests are performed to assess the shear behavior of RC reference beams and UHPFRC-RC composite beams.

3.4.1 Test parameters

To evaluate the strengthening efficiency of UHPFRC, critical parameters considered in this study include moisture exchange between UHPFRC and RC, the restrained shrinkage at the interface, casting direction, and bonding method. Some specimens are exposed to severe exposure conditions (freeze-thaw cycles) before mechanical testing. The investigated parameters are given in Table 3.3. Prior to the application of UHPFRC, concrete substrates are prepared under two different moisture conditions: saturated surface dry (SSD) and air surface dry (ASD). Under the ASD condition, concrete substrate is dry as the beam is continuously stored under laboratory conditions until strengthening. The potential early-age cracking or delamination of the RC beams strengthened by UHPFRC under restrained shrinkage is monitored by digital image correlation (DIC), as detailed in Section 3.4.3. Furthermore, the casting direction of UHPFRC, which is one of the main factors influencing the fiber distribution in UHPFRC and may therefore affect its strengthening efficiency, is also studied. Two UHPFRC casting directions are investigated: (a) vertical and (b) horizontal casting. The effect of bonding method is also studied, and three different strengthening techniques are used: cast in-situ, epoxy-bonded, and mechanically anchored. The in-situ casting method consists of casting fresh UHPFRC at sides of concrete beams, while the epoxy-bonded and mechanical anchorage technique consists of bonding prefabricated UHPFRC laminates to the concrete beams. Finally, the beams are exposed to freeze-thaw to investigate the effect on severe environmental exposure on mechanical resistance. The specimens are tested after 30, 75 and 120 freeze-thaw cycles.

Table 3.3 Overview of the investigated parameters.

Parameter	Variation
Moisture degree of concrete substrate/restrained shrinkage	Air surface dry (ASD) Saturated surface dry (SSD)
Casting direction	Vertical casting Horizontal casting
Bonding techniques	Cast in-situ, Epoxy-bonding Mechanical anchorage
Freeze-thaw action	30, 75, and 120 cycles

Table 3.4 lists the details of all beam specimens, while the geometry and preparation process of the beams are presented in Section 3.4.2. Five groups are divided, including one control group and four strengthening groups. The aim of the strengthening groups is to study (i) the effect of moisture degree of concrete substrate, (ii) bonding method, (iii) casting direction and (iv) freeze-thaw action. Two mixtures (UHPFRC1 and UHPFRC2) are used: UHPFRC1 is adopted to study the effect of moisture degree and restrained shrinkage, and UHPFRC2 is applied to investigate remaining factors.

Table 3.4 Test beam details.

Group	Beam ID	Dimension $L \times h \times b$ (mm^3)	Strengthening material	Bonding method	Casting direction	FT cycles
Control group	RB	1400×100×200	-	-	-	0
	RB (120)	1400×120×200	-	-	-	0
	RB-30	1400×100×200	-	-	-	30
	RB-75	1400×100×200	-	-	-	75
	RB-120	1400×100×200	-	-	-	120
Strengthening group 1	ST-M1-CIS-ASD	1400×120×200	UHPFRC1	Cast in-situ (ASD)	Vertical	0
	ST-M1-CIS-SSD	1400×120×200	UHPFRC1	Cast in-situ (SSD)	Vertical	0
	ST-M1-EB	1400×120×200	UHPFRC1	Epoxy resin	Vertical	0
Strengthening group 2	ST-M2-CIS-ASD	1400×120×200	UHPFRC2	Cast in-situ (ASD)	Vertical	0
	ST-M2-EB	1400×120×200	UHPFRC2	Epoxy resin	Vertical	0
	ST-M2-MA	1400×120×200	UHPFRC2	Mechanical anchorage	Vertical	0
Strengthening group 3	ST-M2-EB-HC	1400×120×200	UHPFRC2	Epoxy resin	Horizontal	0
Strengthening group 4	ST-M2-CIS-30	1400×120×200	UHPFRC2	Cast in-situ (ASD)	Vertical	30
	ST-M2-EB-30	1400×120×200	UHPFRC2	Epoxy resin	Vertical	30
	ST-M2-CIS-75	1400×120×200	UHPFRC2	Cast in-situ (ASD)	Vertical	75
	ST-M2-EB-75	1400×120×200	UHPFRC2	Epoxy resin	Vertical	75
	ST-M2-CIS-120	1400×120×200	UHPFRC2	Cast in-situ (ASD)	Vertical	120
	ST-M2-EB-120	1400×120×200	UHPFRC2	Epoxy resin	Vertical	120
	ST-M2-MA-120	1400×120×200	UHPFRC2	Mechanical anchorage	Vertical	120

Note: Strengthened beams with in-situ casting UHPFRC using UHPFRC2 mixture are all made under ASD condition.

3.4.2 Specimen preparation

Prior to strengthening, nineteen reinforced concrete (RC) specimens are fabricated. All beam specimens have the same reinforcement configuration, with two $\phi 16mm$ rebars at the bottom (tensile zone) and another two $\phi 12mm$ rebars at the top (compressive zone). Ribbed B500 steel reinforcement bars with a yield strength of about 550 MPa and an ultimate strength of 650 MPa are used. Concrete cover over both the top and bottom reinforcement is 25 mm. No stirrup is placed in the region of the effective span. Two stirrups are placed outside of the supports to lift the beams. The geometry of the beams and the reinforcement details are presented in Figure 3.10.

As shown in Figure 3.10 (b), five RC beams are used as reference beams (RB): four RB specimens without strengthening and another reference beam with the same width as strengthened beams, referred to as RB (120). Among the four RB specimens, three of

3.4 Structural tests

these beams are exposed to 30, 75 and 120 FT cycles, respectively (referred to as RB-30, RB-75 and RB-120), according to the freeze-thaw protocol described in Section 3.3.6.

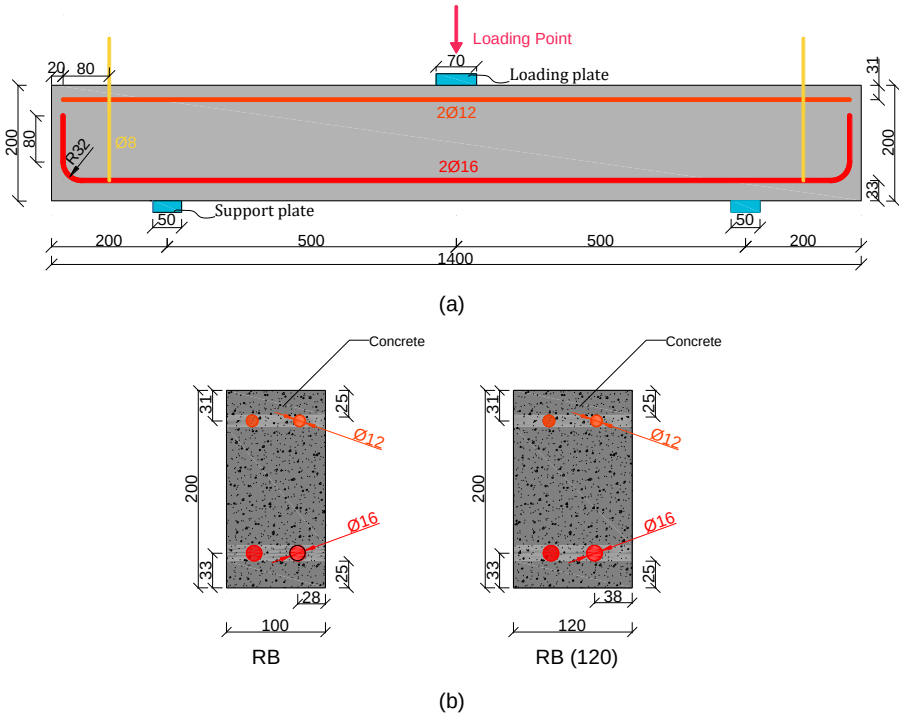
14 beams are strengthened with 10 mm thick UHPFRC on lateral sides (Figure 3.10). For the in-situ cast specimen with SSD condition, a concrete beam is stored at a temperature of 20 °C and relative humidity (RH) of 95% for 24 h before UHPFRC casting. The beam is then taken out of the curing room, and excess water and dust from the surface are wiped by a dry cloth. Other beams (ASD) were sealed and constantly stored under laboratory conditions until strengthening.

Figure 3.11 shows the detailed beam fabrication and strengthening. First, reinforced concrete beams are cast, subsequently covered with plastic film, and cured in sealed condition for around ten months. The 10 mm precast UHPFRC laminates are then made and cured in the same sealed condition for more than one month. Prior to application of the three bonding methods, the interface is treated with air jet to remove the dust and is subsequently cleaned with ethanol. The surface of the RC beams is smooth prior to application of UHPFRC to eliminate the influence of surface roughness. For the in-situ casting method, fresh UHPFRC is cast laterally on RC, as shown in Figure 3.11 (c). The epoxy-bonded and the mechanical anchorage connection methods consist of bonding two pre-fabricated UHPFRC laminates to concrete beams using a two-component epoxy adhesive and bolts, respectively. These types of connection (cold-to-cold) can eliminate the effects of early-age restrained shrinkage since UHPFRC laminates are precast and cured for more than 28 days before being bonded to concrete. Around 1 mm thick two-component epoxy adhesive Sikadur®-30 is used for bonding. Sikadur®-30 is a commercial epoxy adhesive commonly used for strengthening of concrete structures, consisting of a resin and a hardener with a mixing ratio of 3 by weight. Both components are mixed in a bucket using a power drill for at least 3 minutes. The properties of Sikadur®-30, as specified by producers, are listed in Table 3.5. After applying the epoxy, clamps are used to keep UHPFRC laminate tight with concrete until the epoxy resin is hardened to provide sufficient bond (Figure 3.11 (d)).

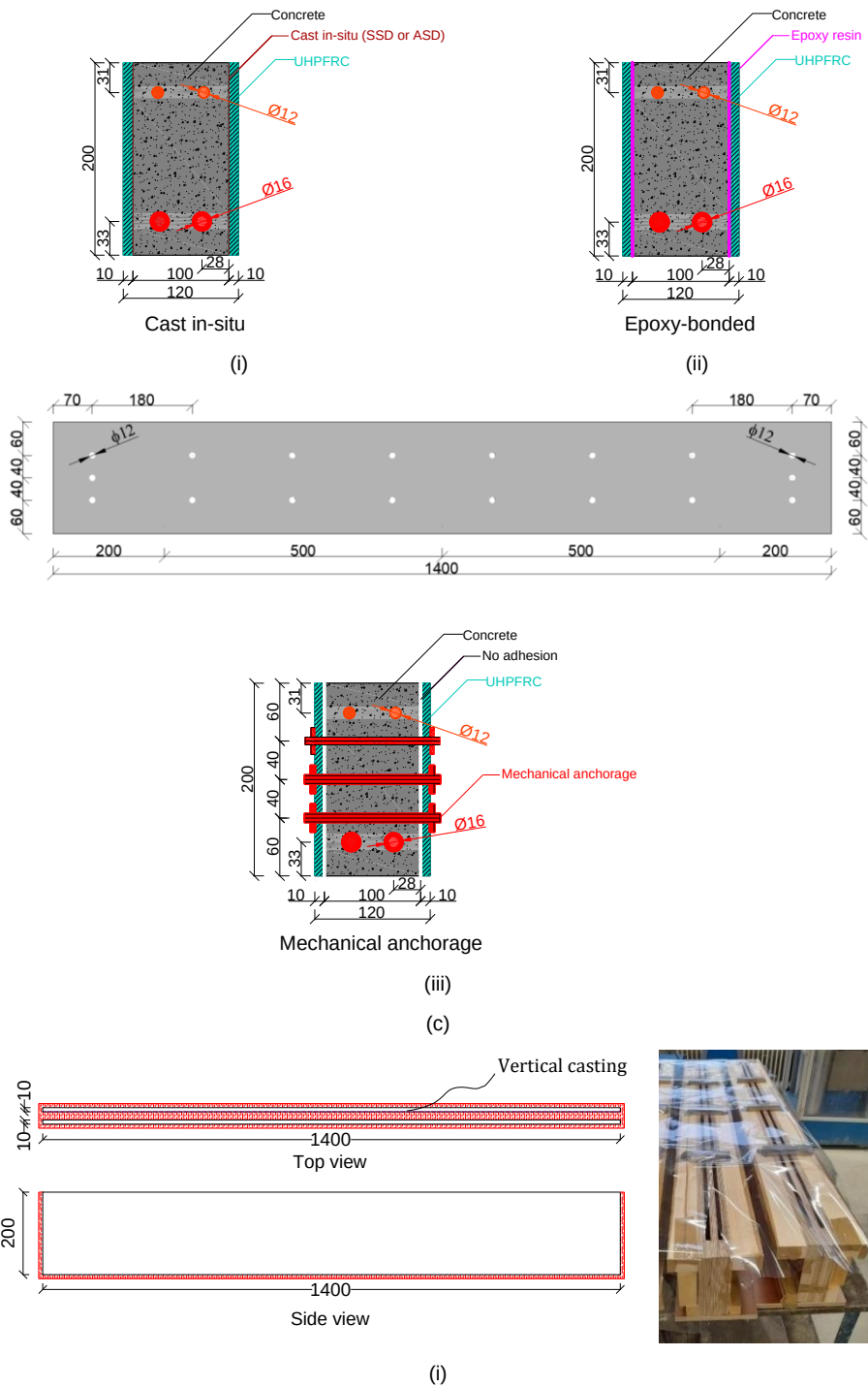
The mechanical anchorage is applied as follows: The RB beam and two precast UHPFRC laminates are placed and clamped on the wooden pallet to avoid sliding during the drilling process. Based on the predetermined hole positions, as sketched in Figure 3.10 (c), holes with a diameter of 12 mm (2 mm larger than the diameter of the bolt) are drilled through the laminates and the whole width of the RB beams by using a diamond tip drill. Subsequently, $\phi 10\text{mm}$ mechanical anchors are installed to connect the two UHPFRC laminates and the RB beam by applying a torque of 15 N·m to tighten the nuts on both sides of the beams. It is noted that unlike previous research that combines cast in-situ or epoxy-bonded with bolt connection techniques [48,96,190], the current study investigates their separate contribution to bonding performance in hybrid structures.

As discussed in Chapter 2, the casting direction of UHPFRC may play a role in the fiber distribution in UHPFRC, and then consequently influence the mechanical performance (especially the tensile behavior) of UHPFRC and its strengthening efficiency. Therefore, to simulate the casting direction of UHPFRC, lamellas are cast horizontally and vertically. Two types of molds are fabricated (Figure 3.10 (d)). For horizontal casting, careful leveling is required to control the thickness of UHPFRC given its small thickness (i.e., 10 mm). With vertically cast plates it is easier to get well controlled thickness. Note that all hybrid beams but one are strengthened by vertically cast plates.

Finally, three groups of beam specimens are exposed to freeze-thaw cycles: cast in-situ, epoxy-bonded and bolt-connected beams. Test protocol is described in Section 3.3.6. The interface condition of the hybrid beams is examined using the infrared thermography before exposure, and after exposure to 30, 75 and 120 freeze-thaw cycles (Chapter 5).



3.4 Structural tests



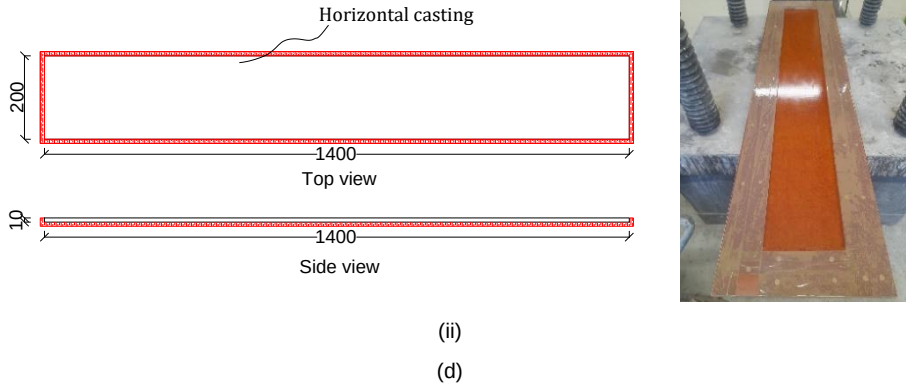


Figure 3.10 Experimental details: (a) Test set-up and geometry; (b) reinforcement configuration for two reference beams; (c) different bonding methods; (d) molds used for different casting directions of precast UHPFRC (unit: mm).

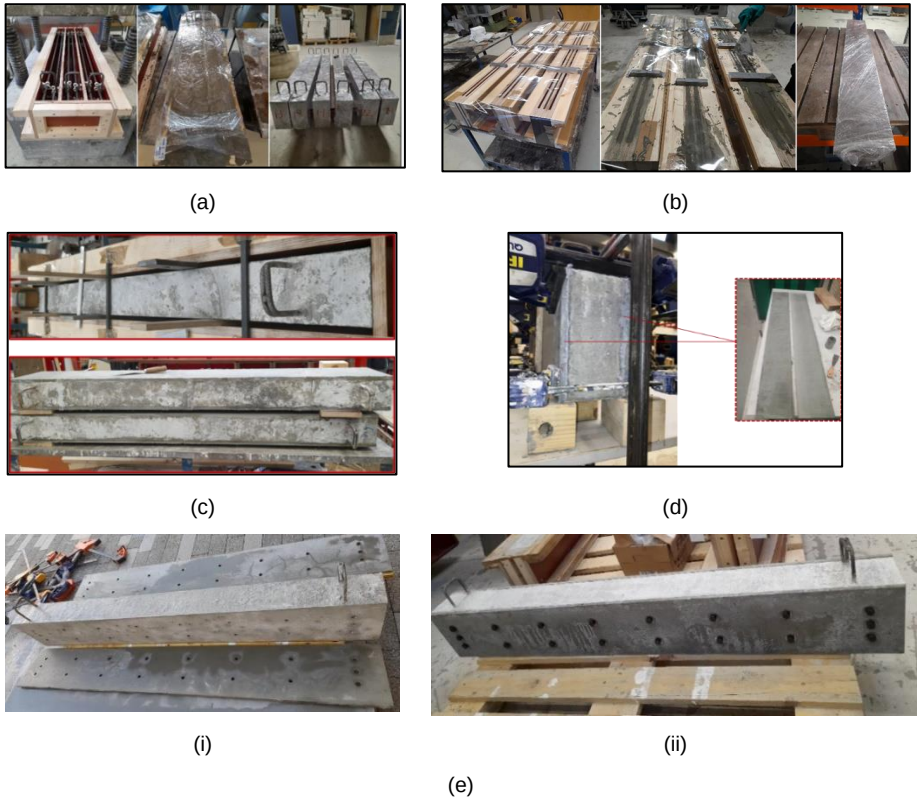


Figure 3.11 Beam fabrication process: (a) Reinforced concrete beam fabrication and curing in sealed condition; (b) 10 mm UHPFRC laminate fabrication and curing in sealed condition; (c) cast-in situ strengthening; (d) epoxy bonded strengthening; (e) bolted connection.

3.4 Structural tests

Table 3.5 The main properties of Sikadur®-30 as provided by manufacturer.

Mechanical properties	-
Compressive strength (after 1 day and cured at a temperature between 10 and 35°C)	55~90 MPa
Tensile strength (after 1 day and cured at a temperature between 15 and 35°C)	20~26 MPa
Tensile bond strength (to concrete) after 7 days and cured under 23°C	> 4 MPa
Shear strength (after 1 day and cured at a temperature between 15 and 35°C)	4~17 MPa
E-modulus in compression cured under 23°C	9.6 GPa
E-modulus in tension cured under 23°C	11.2 GPa
Shrinkage	0.04%
Coefficient of thermal expansion	$2.5 \times 10^{-5} / ^\circ\text{C}$

3.4.3 Measurement system

Shrinkage-induced deformation measurement using 2D-DIC

Digital image correlation (DIC) is used to detect possible crack or debonding due to the stress induced by restrained shrinkage at the interface. DIC is a non-contact image-based measurement technique that is used to capture the deformation field and fracture development in the beams. DIC preparation included: (i) painting white the region of interest of the beams, (ii) making a speckle pattern by air-gun, and (iii) finally capturing reference images to check the quality of the pattern and the noise level. If satisfactory, they are used as reference image for post processing. During testing, a series of images are captured in the measuring zone (Figure 3.12). In addition, one temperature and humidity sensor is placed to monitor the air temperature and relative humidity during the entire testing process. The measurement begins when the specimen is at the age of seven days. And the entire recording duration lasts for more than 90 days to capture the influence of the majority of UHPFRC shrinkage. A 51 Megapixel camera with 8688 by 5792 pixels is used to capture images with a resolution of 0.125 mm/pixel, and GOM Correlate software [191] is used for analysis. A subset size of 19 pixels and a step size of 16 pixels are chosen for the correlation. Before the DIC analysis, the noise level is evaluated in terms of relative deformation and major strain.

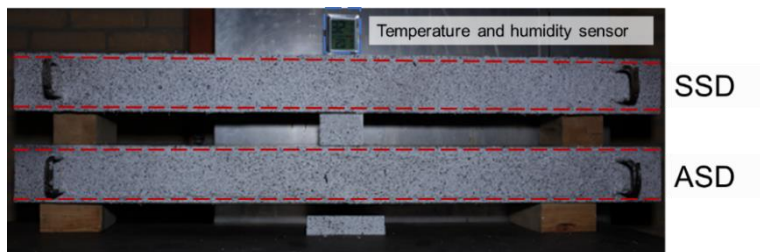


Figure 3.12 Shrinkage-induced deformation measurement using 2D-DIC (the dashed red lines represent interface between RC and UHPFRC).

Mid-span deflection and interface displacement using LVDT

Linear variable displacement transducers (LVDTs) with an accuracy of 0.001mm, are used for measuring deformations (Figure 3.13). For specimens in control groups, displacement at the midspan of the beams is measured. Apart from mid-span deflection measured on the concrete part, for strengthened beams, additional LVDTs are used to measure interface slip and opening near the loading area (Figure 3.13). Based on the findings in [192,193], the maximum slip is expected in the mid-span of the strengthened beam. Therefore, to detect the interface behavior and possible delamination of the critical zone for strengthened beams, four locations on the top side of the specimens, 15 cm from the loading point are selected. LVDT 3, 4, 5 and 6 are used to detect the interface slip, and LVDT 7, 8, 9, and 10 are used to detect the interface opening between UHPFRC and NC. Note that LVDT 2, initially used to record the strain development on the compressive side of the UHPFRC, did not contribute significantly to the analysis and was subsequently removed from the measurement system to avoid obstructing the DIC monitoring area.

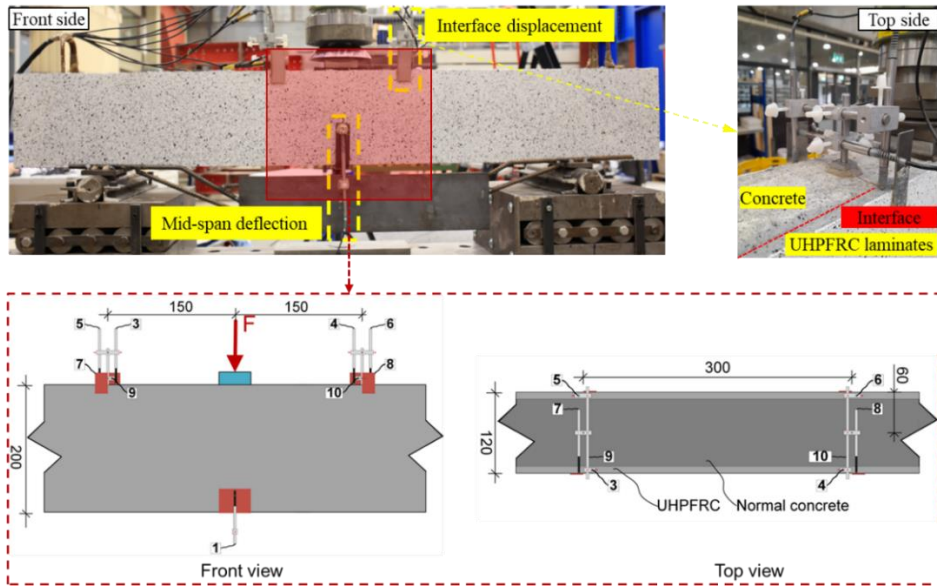


Figure 3.13 LVDTs for mid-span deflection and interface displacement (slip and opening) measurement; the right figure shows a detailed view of the LVDT arrangement, in which the red dashed line denotes the interface between concrete and UHPFRC laminates.

Full-field deformation captured by 2D-DIC

In addition to LVDT measurements, DIC (shown in Figure 3.14) is used on both lateral sides of the strengthened specimens to capture the deformation during the loading process. This is done because the deformations on the front and back sides might differ due to the possible delamination at the interface. GOM correlate software is used for post-processing.

3.5 Summary



Figure 3.14 Digital image correlation on both sides of the specimens for deformation measurement.

3.4.4 Loading scheme

All beams are simply supported and tested until failure under a three-point bending configuration. The test is performed with a displacement-controlled at a rate of 0.01 mm/sec. To simulate realistic boundary conditions in the strengthening application, the applied load and support plates are placed only on the NC part of the strengthened beam.

3.5 Summary

This chapter gives details of experimental procedures in terms of material and structural tests, with a summary provided in Table 3.6. In Chapter 3, the mixture proportion, specimen preparation, and curing method are introduced. Subsequently, the volumetric and mechanical tests for UHPFRC mixtures are described; specifically, freeze-thaw test protocol for both materials and structural tests. Structural tests are conducted to investigate the shear behavior of hybrid beams with UHPFRC. Parameters including moisture condition of concrete substrate, restrained shrinkage at the interface, bonding techniques, UHPFRC casting direction, and freeze-thaw action are considered. Specimens used for materials tests are kept in a same curing condition.

Table 3.6 Summary of material and structural tests.

Type of tests	Properties/Parameters	Test methods	The number of tested specimens	Mixtures	
				UHPFRC1	UHPFRC2
Material tests	Flow	Flow table test (EN 1015-3)	3	Yes	Yes
	Final setting time	Vicat test	2	Yes	No
	Free shrinkage	Shrinkage measurement on hardened UHPFRC (ASTM C157)	3	Yes	Yes
		Corrugated tube method	2	Yes	No
		ADTM	1	Yes	No
	Restrained shrinkage	TSTM	1	Yes	No
	Compressive strength	NEN-EN 12390	3	Yes	Yes
	Elastic modulus	NEN-EN 12390	3	Yes	Yes
	Tensile behavior	Direct tension test (recommended by JSCE)	3	Yes	Yes
	Freeze-thaw resistance	Freeze-thaw test	3	No	Yes

Structural tests	Moisture exchange/ Restrained shrinkage	Shrinkage-induced deformation measurement using 2D-DIC	2	Yes	No
		Three-point bending test	3		
	Bonding techniques	Three-point bending test	3	No	Yes
		Freeze-thaw resistance	10	No	Yes
	Freeze-thaw resistance	Freeze-thaw test	10	No	Yes
		Three-point bending test	10	No	Yes
	Casting direction	Three-point bending test	2	No	Yes

Note: UHPFRC1 is mainly used to investigate the shrinkage of UHPFRC in both material and structural levels; therefore, besides the general shrinkage measurement on hardened specimens, corrugated tube method, ADTM and TSTM are also conducted. Vicat test is also conducted to determine the zero-time of shrinkage measurement for corrugated tube method. For other investigated parameters including bonding techniques, UHPFRC casting direction, and freeze-thaw action, UHPFRC2 mixture is used.

3.5 Summary

Chapter 4 Experimental results and discussion

4.2 Material test results

4.1 Introduction

Chapter 4 provides an overview of structural test results on reference and strengthened beams, along with accompanying material tests on UHPFRC. This chapter provides important insights into the deformational compatibility between UHPFRC and concrete, and shear behavior in composite beams.

In Section 4.2, the fresh properties of UHPFRC, including slump flow and final setting time, defined as “time zero” for autogenous shrinkage, are presented. The volume stability, namely free and restrained autogenous and total shrinkage, and the mechanical properties (i.e., compressive strength, tensile properties and static compressive elastic modulus) are also presented. It should be mentioned that a focus of shrinkage measurement is limited only to UHPFRC1 as this mixture is used to study the role of restrained shrinkage at the interface. In addition to mechanical properties, the surface damage and mass loss of NC and UHPFRC after exposure to freeze-thaw are also inspected.

Structural response results are presented in Section 4.3. Time-dependent deformation due to restrained shrinkage is presented. Behavior of the beams under mechanical loads including load-(mid-span) deflection response, fracture development, cracking kinematics, and the interface displacement are given. Based on the test results, a detailed analysis and additional tests are presented in Section 4.4. The role of different moisture conditions of existing concrete, fiber distribution in UHPFRC and the impact of freeze-thaw is evaluated in detail. Finally, Section 4.5 concludes the main experimental findings.

4.2 Material test results

4.2.1 Slump flow and final setting time

The slump flow of UHPFRC1 and UHPFRC2 is 24.1 ± 0.3 cm and 28 ± 0.5 cm, respectively. These values are within prescribed limits for UHPFRC to allow for sufficient resistance to fiber movement and prevent fiber segregation [194,195]. The target slump flow is between 24 to 30 cm, considering both uniform fiber distribution and casting large quantities of UHPFRC. The final setting time of UHPFRC1 is approximately 10 ± 0.9 h, which is used as the starting time of the autogenous shrinkage measurement for corrugated tube method.

4.2.2 Free and restrained shrinkage

Free shrinkage measurement for UHPFRC1 mixture

Figure 4.1 shows the development of total shrinkage (including both drying and autogenous shrinkage) of the UHPFRC1 mixture after 7 days of sealed curing. The starting time of shrinkage measurement is in line with the demolding time of the beam specimens. The beam specimens are demolded after one week to ensure sufficient strength gain of UHPFRC (more than 90 % of its 28 days' compressive strength). It is found that the specimens

exhibit three stages of shrinkage development: (a) rapid strain increase until around 15 days; (b) steady strain increasing stage with a reduced rate from 15 to 60 days; (c) stable stage without obvious increase in shrinkage after 60 days. The total shrinkage is around $300 \mu\epsilon$.

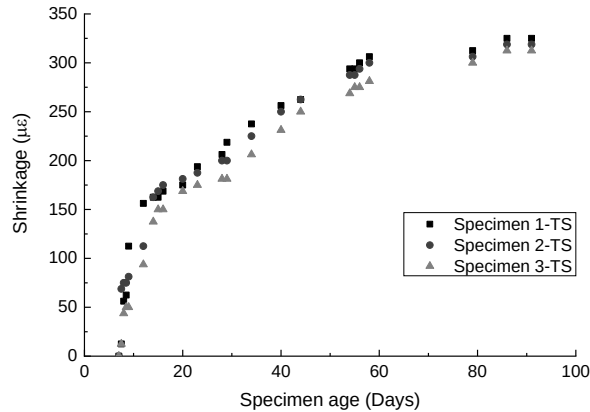


Figure 4.1 Total shrinkage curves for UHPFRC1 measured from the age of seven days, noting that the specimens are in sealed curing condition prior to shrinkage measurement (TS denote the total shrinkage).

Since the autogenous shrinkage of UHPFRC within the first seven days is significant [178], the development of autogenous shrinkage is measured using the corrugated tube method (Figure 4.2). Given that in first hours concrete behaves highly plastically, the strain is recorded immediately after casting (shown in Figure 4.2 (a)) until the final setting time (10 hours) is disregarded from autogenous shrinkage measurement (shown in Figure 4.2 (b)). From Figure 4.2 (b), a significant part of autogenous shrinkage (around $500 \mu\epsilon$) occurs at a very early age (within the first 24 h). After the first week, the autogenous shrinkage reaches more than $600 \mu\epsilon$. By combining early-age autogenous shrinkage, measured using corrugated tubes during the first 7 days, with drying shrinkage, measured on prisms from day 7 onward, a total shrinkage exceeding $900 \mu\epsilon$ can be expected in UHPFRC1. This is much higher than the shrinkage of conventional concrete with a compressive strength lower than 50 MPa, which is in the range of 400 to $600 \mu\epsilon$ [52]. This high shrinkage of UHPFRC might be critical for premature delamination and cracking in the hybrid UHPFRC-NC structure.

4.2 Material test results

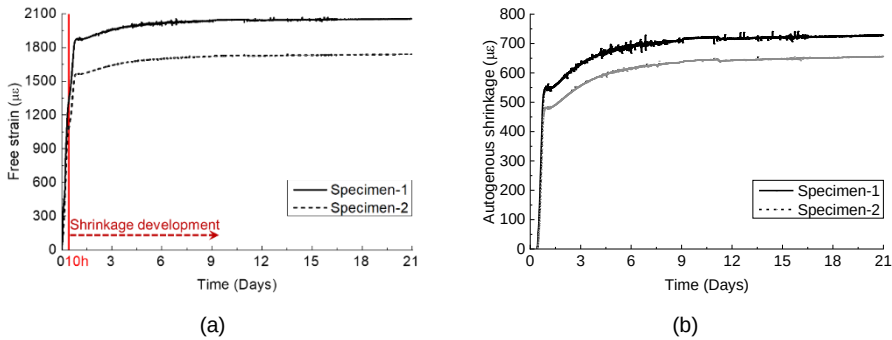


Figure 4.2 Free autogenous shrinkage curves for UHPFRC1 measured from the final setting time: (a) free strain (or deformation) measurement from the beginning after casting; (b) free autogenous shrinkage measured from the final setting time.

Free shrinkage measurement for UHPFRC2 mixture

The autogenous and the total shrinkage (including autogenous and drying) of the UHPFRC2 mixture measured on prism samples, starting from the age of one day are shown in Figure 4.3. Similar to the UHPFRC1 mixture, the development of autogenous and total shrinkage can also be divided into three stages. Based on approximately 150 days of shrinkage measurements, it is observed that both autogenous and total shrinkage strain reach a plateau at around 90 days after casting. Autogenous shrinkage only accounts for roughly a half of the total shrinkage measured on drying prisms, and the total shrinkage of UHPFRC2 is only slightly higher than that of conventional concrete. However, as mentioned before, for UHPFRC, a significant amount of autogenous shrinkage occurs at a very early age (within the first 24 h) [178], which is not measured in this test. Therefore, the measured value is estimated to be only about half of the total shrinkage strain that can form from the onset of autogenous shrinkage (commonly defined as zero time [196]) and can reach more than 1000 $\mu\epsilon$ for comparable UHPFRC mixtures with small aggregate size [197].

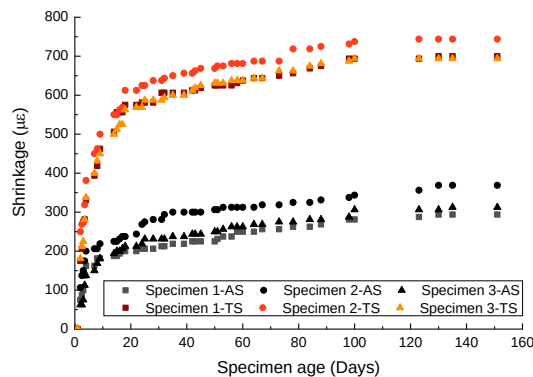


Figure 4.3 Autogenous and total shrinkage for UHPFRC2 measured from the age of one day (AS and TS denote the autogenous and total shrinkage, respectively).

A comparison of the autogenous shrinkage results for both UHPFRC1 and UHPFRC2 mixtures is shown in Figure 4.4. In general, both mixtures exhibit a similar trend: an initial rapid increase until the first four days and followed by a slower, gradual increase until 18 days. However, the autogenous shrinkage of UHPFRC2 is consistently slightly higher than that of UHPFRC1 (within around $50 \mu\epsilon$).

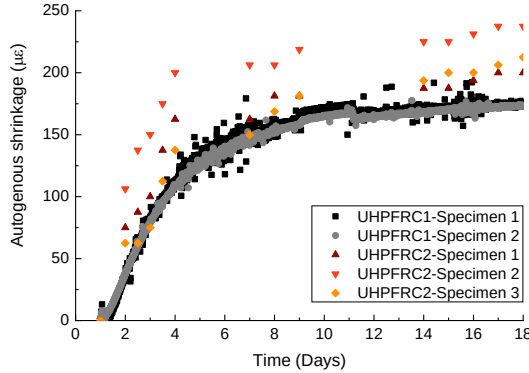


Figure 4.4 Comparison of autogenous shrinkage for both UHPFRC1 and UHPFRC2 measured from the age of one day until 18 days.

Stresses due to restrained autogenous deformation for UHPFRC1

As described in Section 3.3.2, the temperature of the specimen in both ADTM and TSTM setup is kept under 20°C isothermal condition. An example of the controlled temperature history is presented in Figure 4.5. The temperature fluctuation is within one degree, and the initial relatively high temperature variation is due to the heat release from the hydration reaction. The results of free autogenous shrinkage measured by ADTM and eigenstress development of the UHPFRC1 mixtures under fully restrained conditions measured by TSTM are shown in Figure 4.6 (a) and (b), respectively. It should be noted that the zero point on the time axis is the start of mixing. Free autogenous deformation is measured starting from the end of the autogenous expansion (10.68h in this study) and then the subsequent actual self-desiccation shrinkage (i.e., autogenous shrinkage) is plotted. As described in Section 3.3.2, the definition of “time zero” for autogenous shrinkage differs between corrugated tube method and ADTM. Various definitions of “time zero” exist [196] and for corrugated tubes measurement, the final setting time is defined as time zero by standard [198]. While for the ADTM test, “time zero” is defined when the autogenous expansion ends, allowing for more precise measurements as specified in [199]. Despite the differences in definitions, the “time-zero” values (10h for corrugated tube method and 10.68h for ADTM in this study) are generally consistent. In Figure 4.6 (a), it is observed that due to low water/binder ratio of UHPFRC, the autogenous shrinkage for the UHPFRC1 mixture develops very quickly at the early age (more than $300 \mu\epsilon$ until 24 h). Subsequently, there is a decrease in autogenous shrinkage (termed as shoulder effect),

4.2 Material test results

which might be explained by the change of hydration rate due to reaction of silica fume [200,201]. Afterwards, the shrinkage increases at a reduced rate and reaches almost plateau after one week.

Figure 4.6 (b) depicts the development of eigenstresses in UHPFRC1 under fully restrained conditions. Due to the early age expansion, first compressive stresses up to 0.5 MPa develop, which is beneficial to offset the tensile stresses due to autogenous shrinkage. Similar to the trend observed for free autogenous shrinkage, the tensile stress starts developing, reaching plateau after around one week (3.1 MPa). After 25 days, the direct tensile test is carried out under deformation control. A tensile strength of 6.5 MPa is obtained. Therefore, stresses induced by restrained autogenous shrinkage is around 50% of its tensile strength measured at 25 days, which is consistent with the finding in [200]. It indicates that the (autogenous) shrinkage of UHPFRC, which might lead to early-age cracking or delamination of composite structures, should not be neglected for its structural application.

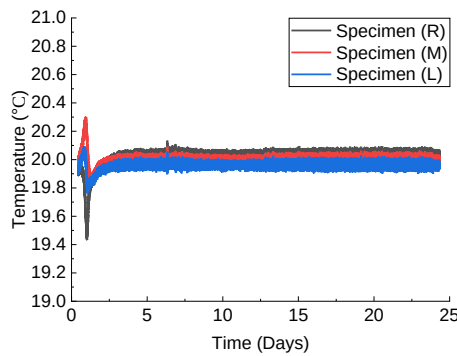


Figure 4.5 Temperature history of the ADTM test set-up.

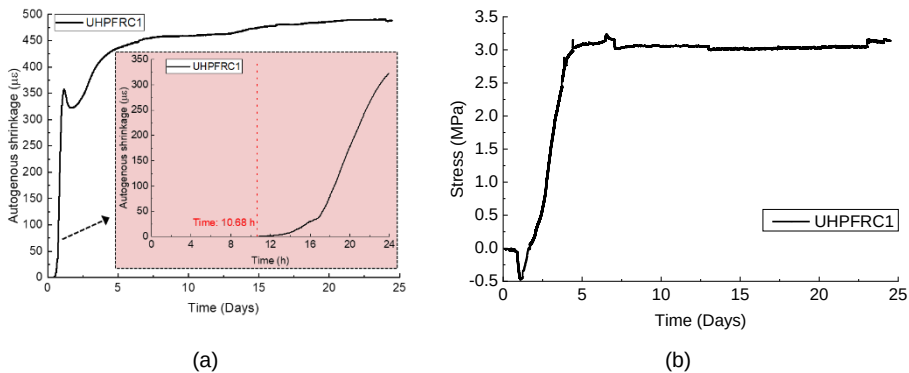


Figure 4.6 Evolution of free autogenous shrinkage and eigenstresses development of UHPFRC1 under fully restrained condition: (a) Free autogenous shrinkage of UHPFRC obtained from ADTM measurements starting at the end of swelling (pink region denotes zoomed view of early age shrinkage development until 24 h); (b) stress obtained from TSTM measurement.

4.2.3 Mechanical properties of UHPFRC and concrete

The material test results, including compressive strength, tensile strength and E-modulus for UHPFRC and NC, at an age when structural tests were conducted are listed in Table 4.1. In Figure 4.7, the stress-displacement relationship under tension and the cracking pattern of two UHPFRC mixtures are shown. From Table 4.1, the average compressive strength for UHPFRC1 and UHPFRC2 are 122.5 and 124.4 MPa, respectively, which meets the minimum strength requirement for UHPFRC [202]. The tensile behavior of UHPFRC is usually divided into three stages (Figure 4.7): linear elastic, strain hardening, and strain softening stages. In the initial stage, all specimens exhibit linear elastic behavior until the cracking tensile strength. Then most of the specimens enter the strain hardening stage, when the stress continues to increase until the peak tensile strength of the specimen is reached. Finally, after the peak load, the dominant macrocrack starts to occur, and tensile stress continuously decreases until the failure of the specimens. For both UHPFRC1 and UHPFRC2 mixture, the deformation is localized in a single crack within the gauge length and the tensile strength and strain capacity varies considerably among the tested specimens, which aligns with prior studies on UHPFRC [64,193,203]. The large variation in the tensile response of the dog-bone specimens is primarily governed by the distribution and orientation of fibers. Moreover, both mixtures have nearly the same compressive strength and elastic modulus.

Table 4.1 Mechanical properties (average $\pm$ standard deviation) of cement-based materials.

Mixture	Compressive strength (MPa)	Elastic modulus (GPa)	Splitting tensile strength (MPa)	First cracking strength (MPa)	Cracking strain (%)	Peak tensile strength (MPa)	Tensile strain capacity (%)
NC	39.0 $\pm$ 3.1	37.7 $\pm$ 1.9	3.4 $\pm$ 0.7	-	-	-	-
UHPFRC1	122.5 $\pm$ 7.9	44.9 $\pm$ 1.6	-	6.2 $\pm$ 1.2	0.017 $\pm$ 0.005	9.3 $\pm$ 1.0	0.33 $\pm$ 0.11
UHPFRC2	124.4 $\pm$ 6.2	46.8 $\pm$ 0.7	-	8.0 $\pm$ 0.5	0.018 $\pm$ 0.005	8.1 $\pm$ 0.5	0.47 $\pm$ 0.44

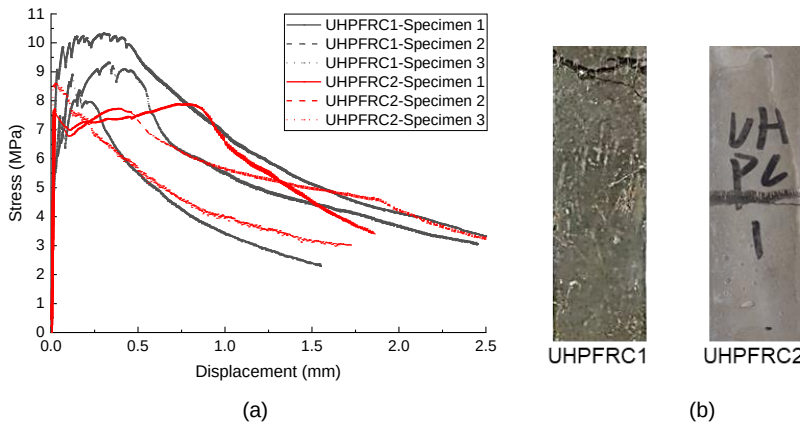


Figure 4.7 Tensile properties of UHPFRC: (a) Stress-displacement behavior of UHPFRC, noting that the LVDT displacement is obtained over the 80 mm measurement length; (b) Cracking pattern.

4.2.4 Freeze-thaw test results

Physical appearance

UHPRFC prisms (40 mm × 40 mm × 160 mm) and 100 mm concrete cubes are prepared for surface profile observation after freeze-thaw cycles (30, 75 and 120 cycles). For UHPRFC2 specimens, there is nearly no change at the surface (Figure 4.8). Note that the corrosion of the surface steel fibers occurs before FT tests due to exposure to the water. For NC specimens, at 75 FT cycles, only a small amount of cement pastes peels off the specimens' surface. The surface spalling becomes more obvious after 120 FT cycles.

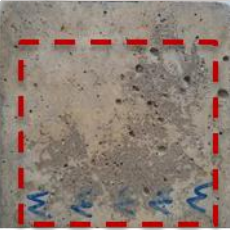
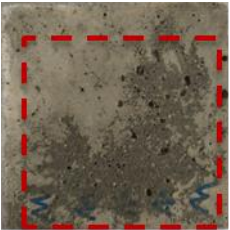
Number of FT cycles	UHPRFC2	NC
0		
30		
75		
120		

Figure 4.8 Surface profile of UHPRFC2 and NC subjected to different freeze-thaw cycles.

Mass changes of NC and UHPFRC2

The same specimens used for surface observation are also used to measure the mass change after freeze-thaw. The measurement is conducted at the end of the thawing process and the surface water on specimens is wiped out by towel before the weight measurement. The results indicate that there is almost no mass change (less than 0.1%) for both UHPFRC2 and normal concrete. The mass of normal concrete is even increased, which could be owing to water absorption of the concrete. UHPFRC has a relatively dense microstructures and impedes the ingress of water. In general, no obvious mass reduction due to the scaling of the specimen is observed for both NC and UHPFRC specimens after freeze-thaw. The maximal mass change is around 0.1%, indicating that both NC and UHPFRC have good resistance to freeze-thaw. Note that specimens are only exposed to moist conditions (RH = 95%) rather than full immersion in water during the freezing process, as specified in existing codes such as ASTM C666/C666M [204], GB/T 50082-2009 [205] or CEN/TR 15177 [206].

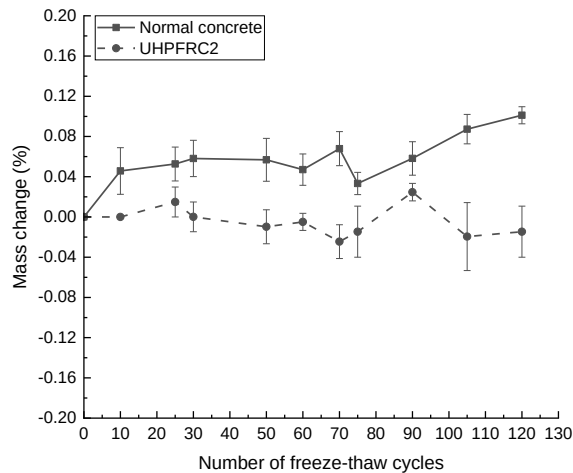


Figure 4.9 Mass loss of UHPFRC2 and NC. The positive/negative value in y axis denotes the increase/decrease percentage of mass.

Mechanical properties of UHPFRC2 and concrete

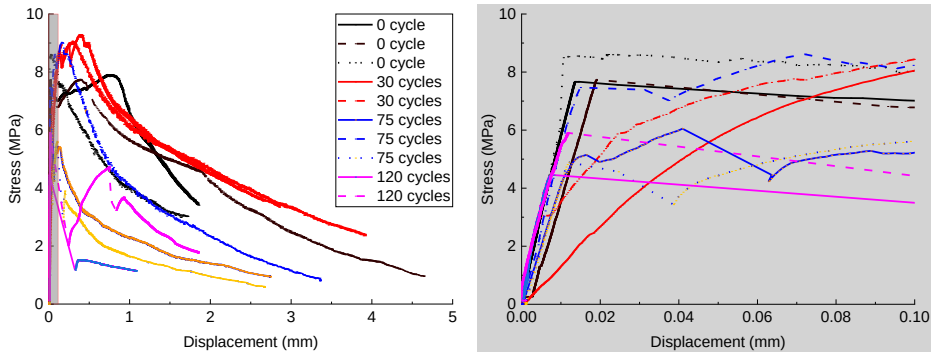
The compressive and tensile strengths, and elastic modulus are measured after predetermined number of cycles (0, 30, 75 and 120) and the results are listed in Table 4.2. After exposure to 120 FT cycles, the compressive strength and elastic modulus of both UHPFRC and NC do not change, i.e., the variation is within their standard deviation. The splitting tensile strength of NC is not affected by freeze-thaw. However, the tensile strength of UHPFRC is affected and a significant reduction after 120 cycles is observed [207]. A possible reason for slight strength increase after 30 cycles might be continued hydration of unhydrated cement particles in UHPFRC when in contact with water at the

4.2 Material test results

start of the freeze-thaw test (10% increase in cracking and 13.6% in peak tensile strength) [208]. After 75 and 120 FT cycles, the cracking (peak) tensile strength is decreased by 27.5% (14.8%) and 35% (36%), respectively. As found in [207,209], considering the differential thermal deformation between the matrix and steel fibers due to their different thermal expansion coefficient, the bond strength at the matrix-fiber interface will be damaged and therefore its tensile strength will be reduced. The tensile stress-displacement relation together with the cracking pattern for UHPFRC under different number of FT cycles is presented in Figure 4.10. It is found that not only the tensile strength but also the strain capacity decreases substantially due to the FT action after being subjected to more than 30 FT cycles. Therefore, the fracture energy is significantly reduced, which agrees with the literature [207,210]. Additionally, the FT action does not change the cracking pattern of the specimens and the deformation is localized in a single crack within gauge length.

Table 4.2 Mechanical properties (average $\pm$ standard deviation) of cement-based materials.

FTCs	Compressive strength (MPa)		Tensile strength (MPa)			Elastic modulus (GPa)	
	Concrete	UHPFRC	Concrete f_{sp}	UHPFRC f_{ct} f_{pt}		Concrete	UHPFRC
0	39.0 $\pm$ 3.1	144.0 $\pm$ 9.5	3.4 $\pm$ 0.7	8.0 $\pm$ 0.5	8.1 $\pm$ 0.5	37.7 $\pm$ 1.9	46.8 $\pm$ 0.7
30	44.3 $\pm$ 4.3	149.9 $\pm$ 12.0	3.8 $\pm$ 0.3	8.8 $\pm$ 0.2	9.2 $\pm$ 0.2	-	-
75	36.5 $\pm$ 1.9	142.8 $\pm$ 13.7	3.7 $\pm$ 0.3	5.8 $\pm$ 1.4	6.9 $\pm$ 1.9	35.0 $\pm$ 2.1	47.4 $\pm$ 1.2
120	38.0 $\pm$ 2.1	152.4 $\pm$ 6.1	4.1 $\pm$ 0.2	5.2 $\pm$ 1.1	5.2 $\pm$ 1.1	37.8 $\pm$ 1.7	47.0 $\pm$ 1.4



(a)

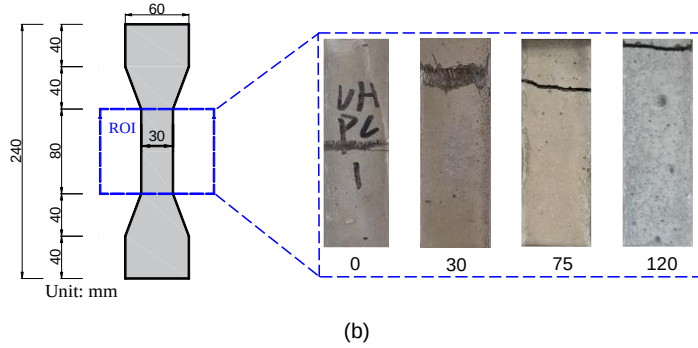


Figure 4.10 Tensile behavior of UHPFRC2 after experiencing different number of FT cycles: (a) stress-displacement relation with zoomed view of initial response; (b) cracking pattern.

4.3 Structural test results

4.3.1 Cracking/debonding analysis of hybrid UHPFRC-RC beams under restrained shrinkage

Deformation over 90 days in cast in-situ strengthened beams with saturated and dry substrate (ST-M1-CIS-SSD and ST-M1-CIS-ASD specimens, respectively) exposed to drying is captured by DIC. During the DIC measurement, the air temperature and relative humidity (RH) are recorded and shown in Figure 4.11. An average air temperature of around 20 °C is maintained, while the RH is in the range of 25% to 55%. The relatively low RH induces high drying shrinkage for concrete [55], which might pose a cracking/debonding risk at the concrete-UHPFRC interface.

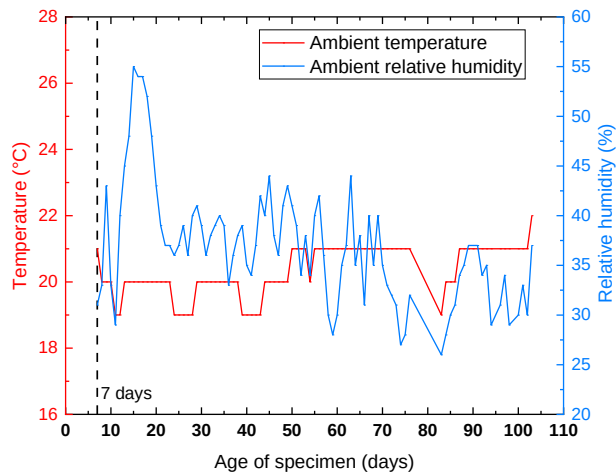


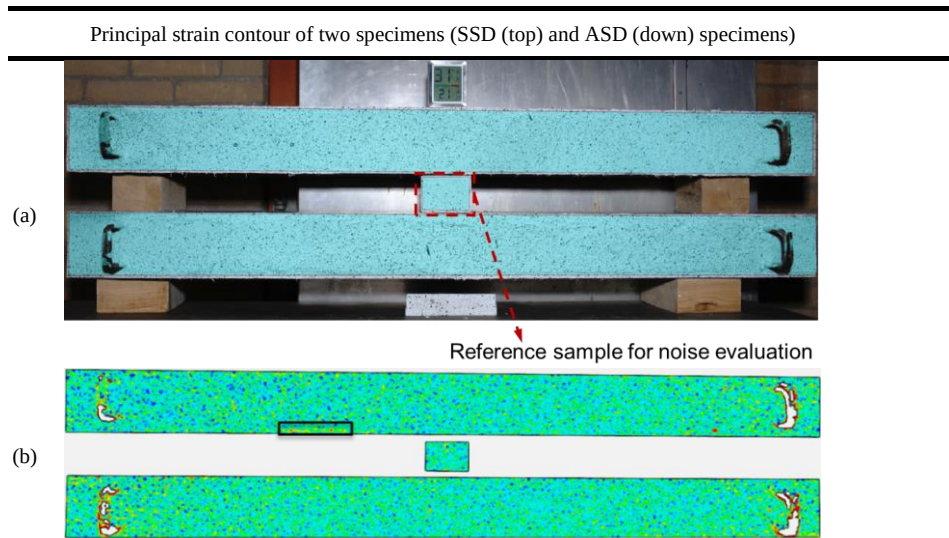
Figure 4.11 Temperature and Relative humidity variation versus age of the tested specimen.

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Figure 4.12 shows the principal strain contour map of two beam specimens (ST-M1-CIS-SSD specimen (top side) and ST-M1-CIS-ASD specimen (bottom side)) at several characteristic time points: (b) 8 days, (c) 10 days, (d) 12 days, (e) 21 days, (f) 28 days, (g) 60 days, (h) 90 days of the age of the specimens. The strain scale is kept the same for all images, and the positive and negative strains represent the tensile and compressive strain, respectively.

Prior to measurement, the noise level of the DIC system is determined by capturing several reference images and comparing the deformation and principal strain variation between the reference images. The deformation and principal strain (ε_x) resolution are evaluated as 0.009 mm and 0.201% (0.122% for ε_y), respectively. Strain induced by shrinkage is relatively low and falls within these limits. Therefore, a single measurement point with high strain is regarded as noise whereas a continuous zone with high strain level is regarded as a crack or delamination.

As shown in Figure 4.12 (b), at the age of 8 days, the strain localization appears at the interface between UHPFRC and NC of specimen with saturated concrete (represented by the black rectangle) due to the differential shrinkage between newly cast UHPFRC and mature concrete. Over time delamination increases. Although in general debonding starts at the free edges, this is not the case in the current study, where delamination probably starts from the weakest zone at the interface.



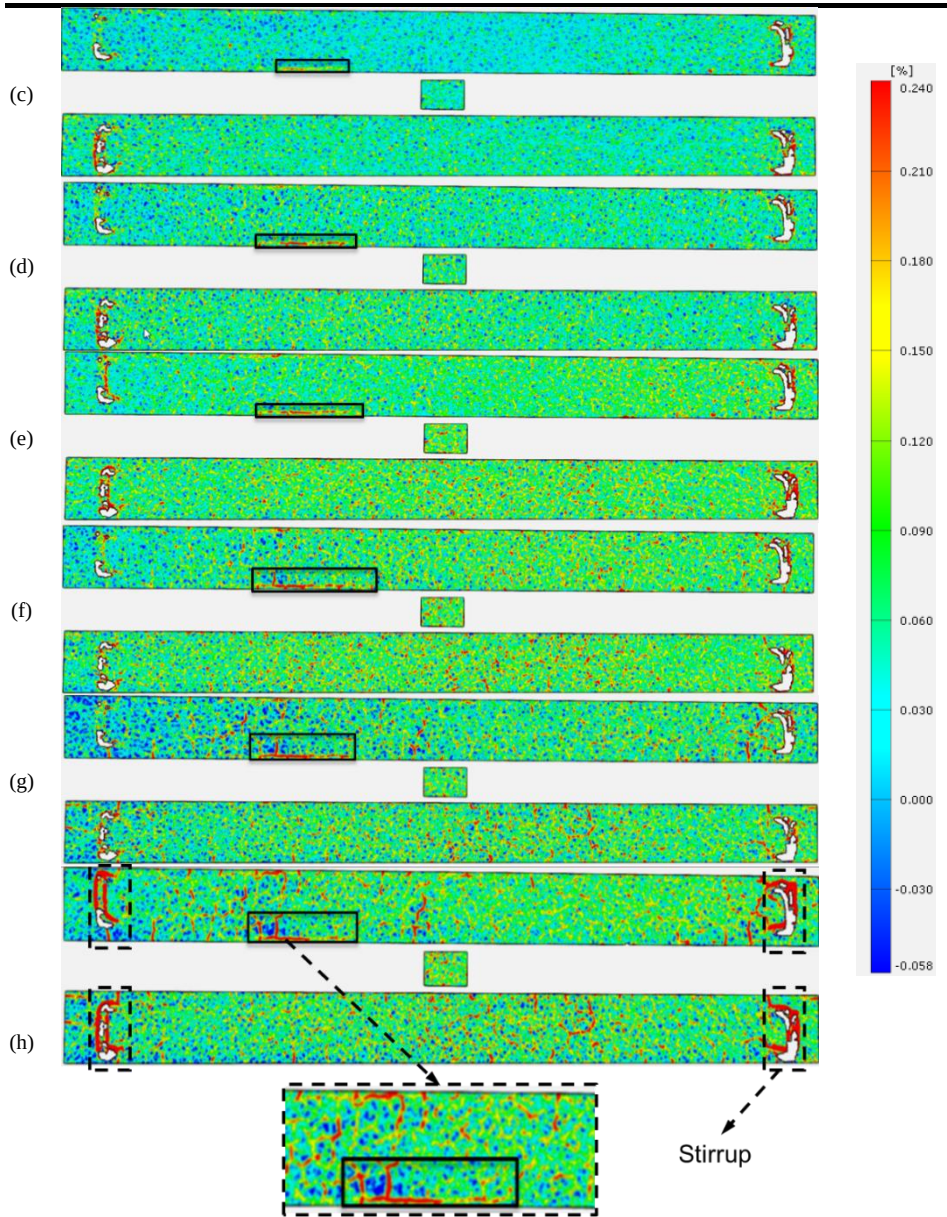


Figure 4.12 Principal strain contour measured at top of strengthened beams with initially saturated (SSD) and dry (ASD) concrete substrate at 8 day (b), 10 days (c), 12 days (d), 21 days (e), 28 days (f), 60 days (g) and 90 days (h) of the age of the specimens with zoomed in final damage; note that (a) is the schematic diagram showing the position of beams and reference sample, and the strain concentration area (the rectangular region enclosed by the dashed black border) at two sides of each beam represents the stirrup.

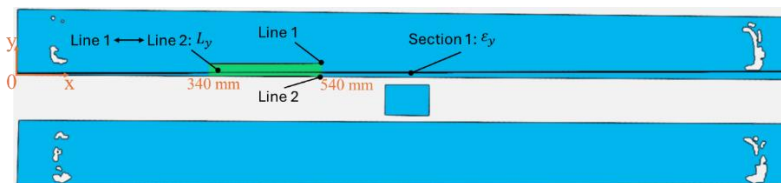
The interface opening along the delaminated zone can be calculated as the deformation change (that is, difference in distance between two lines on either side of the interface) in

4.3 Structural test results

y-direction (Figure 4.13 (a)). In addition, ε_y , the strain along y-direction, is also obtained as a function of time to determine the development of the delamination length.

With the bottom-left corner of the beam taken as the origin, the delamination region extends approximately between 340 and 540 mm along the beam length (Figure 4.13 (a)). The ε_y analysis is focused on this section of the beam, with the noise level for ε_y of 0.1%, while the strain above this value is recognized as the delamination. The ε_y as a function of time is presented in Figure 4.13 (b), revealing that the delamination is not continuous. This observation aligns with the localized delamination shown in Figure 4.12, where the strain concentration areas, represented by the red line, appear as discrete regions. Using ε_y at 90 days as an example, the total delamination length is the sum of several continuous delamination area (AB+CD+EF+GH) while the region such as BC or DE where the ε_y lower than 0.1% is still regarded as sound area. Thereafter, the interface opening along the delamination area is extracted, and the maximum and average interface opening can be calculated. Figure 4.13 (c) shows the development of the delamination length and (average and maximum) interface opening over time. The debonding length increases rapidly in the first 17 days, with the delamination length reaching around 160 mm and remains almost constant afterwards. The fluctuations of the measurement after 17 days are due to the noise.

With respect to the interface opening, values exceeding the deformation noise threshold (0.009 mm) are identified. The interface opening is first detected at the age of 9 days and then stabilizes. The maximum interface opening does not exceed 0.02 mm, and its average value is around 0.01 mm. However, a substantial variation is shown in the interface opening after 9 days due to noise. In general, both the length and opening increase abruptly because of the debonding stresses induced by shrinkage, and then remain stable as the rate of shrinkage decreases. It should be mentioned that the delamination measured by the DIC could not be observed by visual inspection.



(a)

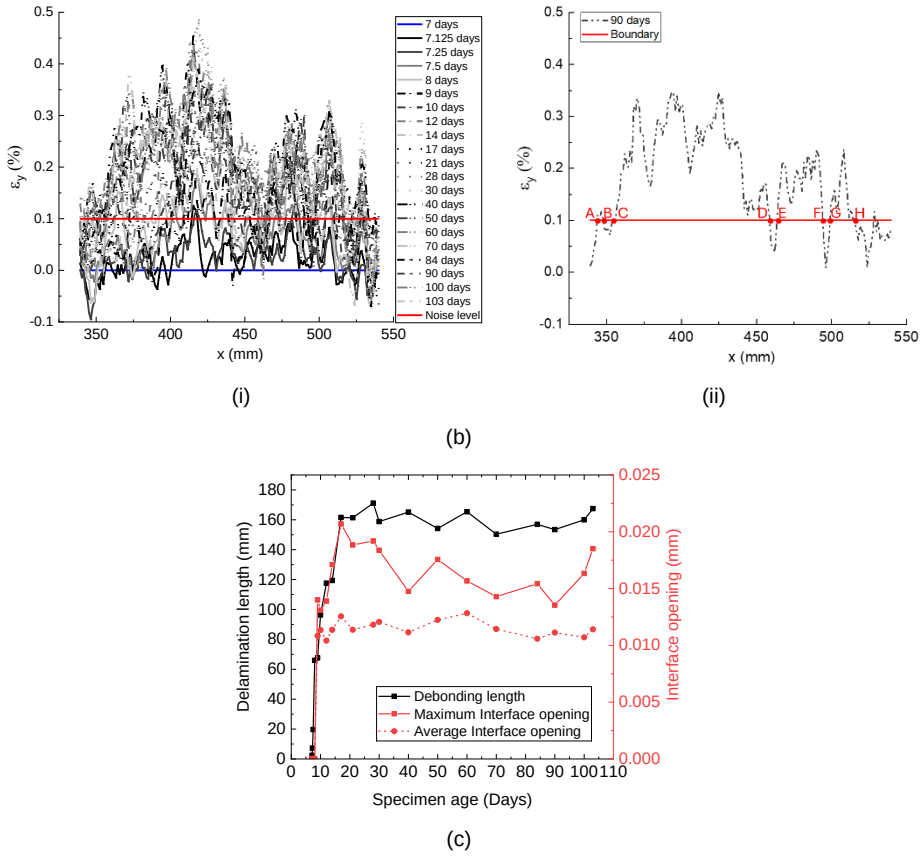


Figure 4.13 Development of the debonding length and (maximum and average) interface opening in the localized delaminated zone of the ST-M1-CIS-SSD specimen throughout the experiment: (a) schematic diagram showing the processing method; (b) results of ε_y at different measuring time (i) together with an example of ε_y at 90 days of the specimens' age (ii); (c) results of delamination length and interface opening development.

4.3.2 Influence of moisture condition of concrete substrate

Shear resistance of the reference and strengthened beams

The shear load-displacement relationship and the schematic view of the energy absorption for the reference and strengthened beams are shown in Figure 4.14 (a). To account for the differences in cross-sectional dimensions between RB and the other tested beams, the shear load is normalized to shear stress by dividing it by the beam width and effective depth of the beams, and the shear stress- (mid-span) deflection responses are shown in Figure 4.14 (b). The concrete substrate in the strengthened beam is initially pre-saturated (ST-M1-CIS-SSD) and dry (ST-M1-CIS-ASD). In addition, also specimen with epoxy bonded prefabricated laminate where moisture exchange does not play a significant role

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is shown (ST-M1-EB). Due to the growth of the dominant shear crack, both the reference and the strengthened beams show brittle failure after reaching maximum bearing capacity. As shown in Figure 4.14, the load-displacement curves for all tested specimens are similar, demonstrating a three-stage damage process: (i) linear elastic stage, (ii) cracking stage, and finally (iii) brittle failure stage. At the initial stage, all beams show linear elastic behavior. As the load continues to increase and enters the cracking stage, the stiffness of all strengthened beams reduces at a lower rate compared to the reference beams. ST-M1-CIS-SSD and ST-M1-EB show almost the same stiffness while ST-CIS-ASD specimen shows lower stiffness. This is due to the moisture exchange between concrete and UHP-FRC. When fresh UHPFRC is cast against a dry concrete substrate, water migrates from UHPFRC to the concrete layer due to a hygral gradient [211]. Moisture loss from UHPFRC affects the hydration, causing lower stiffness of UHPFRC. This will be further verified by comparison of hydration degree in discussion (Section 4.4.1).

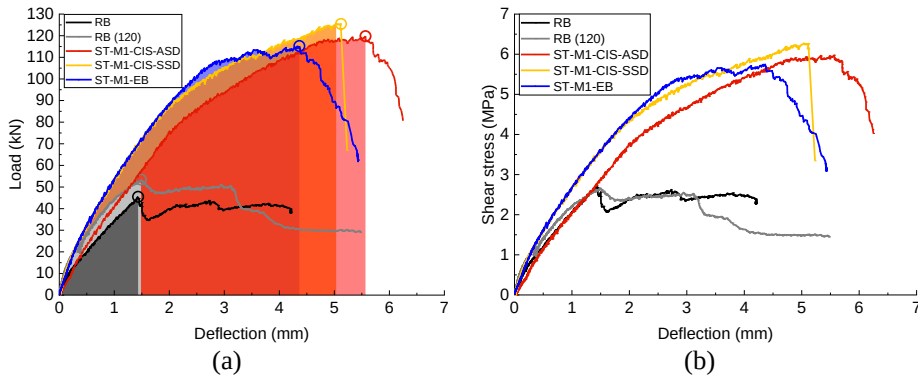


Figure 4.14 Shear response of the tested specimens: (a) Load- (mid-span) deflection relationship; (b) Shear stress- (mid-span) deflection relationship; RB and RB (120) are reference concrete beams with 100mm and 120mm width, whereas ST are strengthened beams.

The test results are summarized in Table 4.3. Compared to the RB specimen, only 18% and 3% increase in load and deformation capacities, respectively, are found in the 20% wider RB (120) specimen. For dimensionless comparison, the peak shear stress is compared, revealing only 2% difference is found between RB and RB (120) specimens. However, a significant enhancement in both peak load and ductility is observed for all strengthened beams with UHPFRC. The maximum loads of the ST-M1-CIS-ASD, ST-M1-CIS-SSD and ST-M1-EB specimens are 163%, 176%, and 153% more than that of RB specimen. In terms of ductility, the corresponding deflections at the peak load of ST-M1-CIS-ASD, ST-M1-CIS-SSD and ST-M1-EB specimens are 5.6, 5.0 and 4.4 mm, which is 290%, 252% and 205% higher than that of the RB specimen, respectively. The results described above validate the effectiveness of using UHPFRC as a strengthening material for shear-critical concrete structures. In addition, the energy absorption, defined as the area under the load-deflection curve until the peak load, is also calculated (Table

4.3). The 20% increase in beam width provides an increase in the energy absorption by 40%. On the other hand, strengthened beams with UHPFRC exhibit around 9-11 times higher energy absorption than that of RB beam. Note that ST-M1-CIS-SSD beam exhibits the highest peak load but on one side of this beam delamination is observed as a failure mode.

Table 4.3 Results in terms of load, deflection, and failure mode.

Specimen ID	F_u (kN)	ΔF_u (%)	τ_u (MPa)	δ_u (mm)	$\Delta \delta_u$ (%)	E_u (kN.mm)	ΔE_u (%)	Failure pattern
RB	45.5	-	2.72	1.4	-	36.9	-	Shear failure
RB (120)	53.5	18	2.67	1.5	3	51.1	38	Shear failure
ST-M1-CIS-ASD	119.8	163	5.98	5.6	290	447.4	1113	Shear failure
ST-M1-CIS-SSD	125.7	176	6.27	5.0	252	434.0	1076	Shear failure + partial delamination
ST-M1-EB	115.2	153	5.75	4.4	205	354.8	861	Shear failure

Note: F_u is the peak load and τ_u is the peak shear stress; δ_u is the mid-span deflection at the peak load; E_u is the energy absorption; ΔF_u , $\Delta \delta_u$ and ΔE_u are the increased percentages of peak load, deflection, and energy absorption of RB (120) and each strengthened specimen to that of RB specimen, respectively.

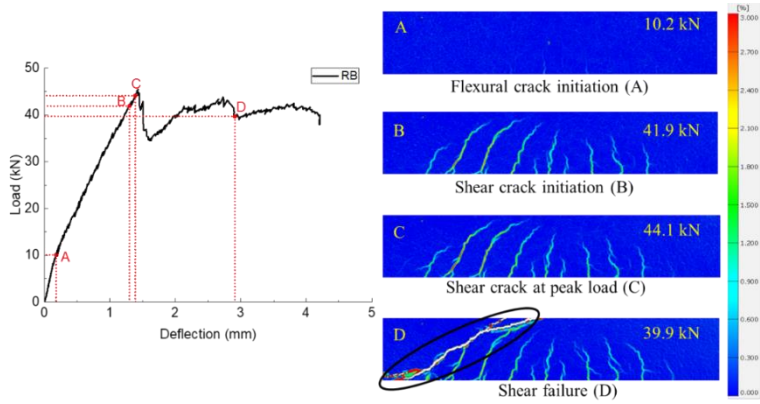
Cracking behaviors and failure modes

All specimens show similar fracture development (Figure 4.15): (i) initial linear elastic stage, (ii) cracking process, from which one dominant inclined shear crack develops; (iii) shear failure. With the help of DIC, the stiffness reduction in cracking process as observed in load-deflection response can be further divided into flexural cracking and shear crack localization. Taking the RB specimen as an example, as shown in Figure 4.15 (a), at the initial linear elastic stage (before reaching point A), the specimen is uncracked. When the load reaches AB region (flexural cracking), several cracks appear at the bottom of the beam, and the number and width of the cracks keep increasing as the load increases. At point B, the dominant shear crack initiates and, shortly after, the peak load (C) is reached and the beam fails. However, for the ST-M1-CIS-SSD specimen, although it fails in a brittle manner, the dominant shear crack does not fully develop on the back side due to the delamination of UHPFRC laminate, unlike the other two strengthened beams (ST-M1-CIS-ASD and ST-M1-EB specimens).

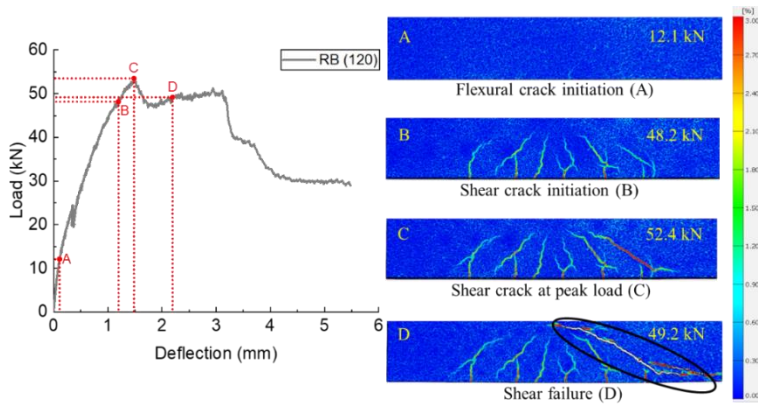
Based on the damage process of each specimen, the distinctive characteristic loads (labelled as A, B and C in Figure 4.15) and corresponding deflections can be extracted, see Table 4.4. The results demonstrate that, compared to the reference specimens, the flexural cracking resistance for strengthened specimens is significantly enhanced due to the excellent tensile properties of UHPFRC. ST-M1-CIS-ASD, ST-M1-CIS-SSD and ST-M1-EB exhibited 2.4, 3.7 and 3.4 times higher flexural cracking load, 2.0, 2.3 and 2.3 times higher shear cracking load, and 2.7, 2.9, 2.6 times higher peak load than those of the RB specimen, respectively. When the three strengthened beams are compared, it is shown

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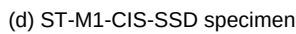
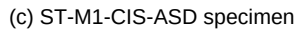
that the earliest initiation of flexural and dominant shear cracks is found in ST-M1-CIS-ASD specimen. This could be attributed to the lower tensile strength in the UHPFRC due to lower degree of hydration and moisture loss.



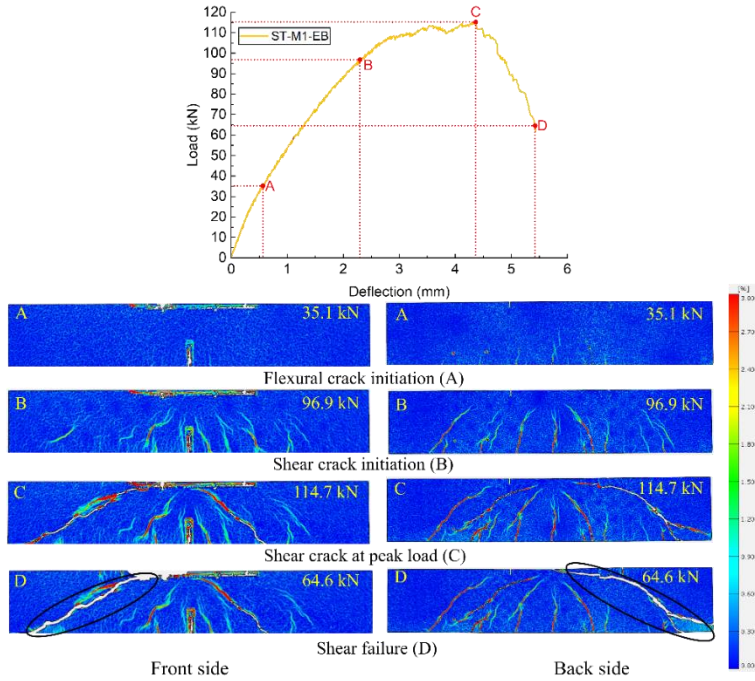
(a) RB specimen



(b) RB (120) specimen



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(e) ST-M1-EB specimen

Figure 4.15 Damage process of the tested specimens.

Table 4.4 Distinctive characteristic loads and corresponding deflections for all tested specimens.

Specimen ID	$F_{cr,f}$ (kN)	$\delta_{cr,f}$ (mm)	$F_{cr,s}$ (kN)	$\delta_{cr,s}$ (mm)	F_u (kN)	δ_u (mm)
RB	10.2	0.2	41.9	1.3	44.1	1.4
RB (120)	12.1	0.1	48.2	1.2	52.4	1.5
ST-M1-CIS-ASD	24.2	0.5	85.0	2.3	119.8	5.6
ST-M1-CIS-SSD	37.5	0.6	95.6	2.4	125.7	5.0
ST-M1-EB	35.1	0.6	96.9	2.3	115.2	4.4

Note: $F_{cr,f}$, $F_{cr,s}$ and F_u is the flexural cracking load, shear cracking load and peak load, respectively; $\delta_{cr,f}$, $\delta_{cr,s}$ and δ_u are their corresponding mid-span deflections.

The failure mode of each specimen is shown in Figure 4.16. All specimens eventually fail in shear, except for the ST-M1-CIS-SSD specimen. As for the ST-M1-CIS-ASD and ST-M1-EB specimens, the failure patterns on both sides (front and back sides) are captured, and they are similar. This indicates that the composite beam behaves monolithically throughout the entire loading history. However, for the ST-M1-CIS-SSD specimen, although having the highest ultimate capacity, the failure patterns on the two sides of the beams are different. Moreover, from the top view of ST-M1-CIS-SSD, clear delamination between the UHPFRC and the NC is found after specimen failure (Figure 4.16). This might have been triggered by delamination that was already observed prior to mechanical

test, due to drying (Figure 4.12). To characterize better the damage mechanisms, further quantitative analysis of shear crack development and interface behavior from LVDT measurement is provided.

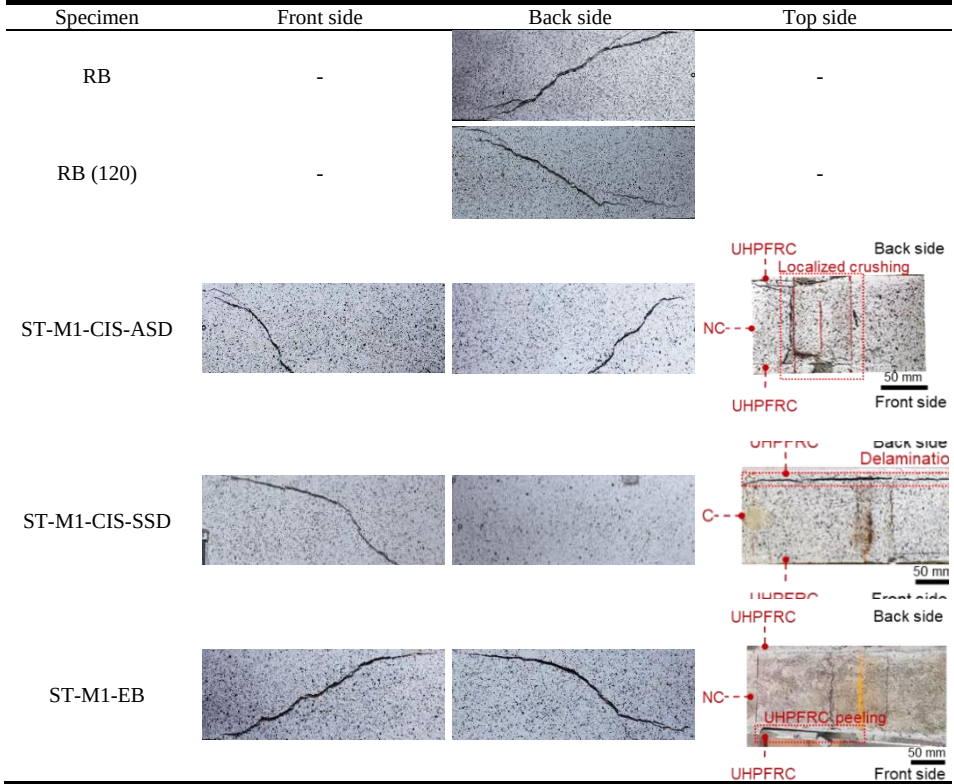


Figure 4.16 Failure modes on both sides for the tested specimens.

Cracking kinematics of tested beams

With 2D-DIC technique, the crack displacement (including flexural crack width, shear crack opening and slip) can be tracked until the failure of the beams. This is vital to understand the damage process of the tested beams and the role of UHPFRC on the fracture behavior of strengthened system. For all tested beams, before DIC analysis, the noise level is analyzed, and in general the accuracy of deformation and the principal strain are 0.005 mm and 0.045%, respectively. The maximum flexural crack width is extracted at the bottom (tension side) of the specimen at each load step. In terms of shear crack kinematics calculation, for 2D-DIC analysis, rigid body motion is eliminated as it will influence displacement measurement (especially crack slip). As shown in Figure 4.17 (a), due to the global rotation of the specimen, the displacement AA' parallel to the crack path will lead to a significant error when quantifying crack slip measurement. Figure 4.17 (b) shows crack displacement calculation which accounts for the effect of rigid body rotation.

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The reference points are grouped into kinematic points (A and B) and rotation points (C and D). Upon loading, the specimen deforms and global rotation occurs, whereby kinematic points (A and B) and rotation points (C and D) are turned into new position, which are named A' and B' , C' and D' . The rigid rotation angle θ is defined as the angle change:

$$\theta = \frac{1}{2}(\theta_1 + \theta_2) = \frac{1}{2}(\arccos \frac{\overline{AC} \bullet \overline{A'C'}}{|\overline{AC}| |\overline{A'C'}|} + \arccos \frac{\overline{BD} \bullet \overline{B'D'}}{|\overline{BD}| |\overline{B'D'}|}) \quad (4.1)$$

Then the angle change due to applied load is:

$$\alpha = \arccos \frac{\overline{AB} \bullet \overline{A'B'}}{|\overline{AB}| |\overline{A'B'}|} - \theta \quad (4.2)$$

Finally, the crack opening and slip are obtained using the following expressions:

$$\text{Crack opening} = |\overline{A'B'}| \cos \alpha - |\overline{AB}| \quad (4.3)$$

$$\text{Crack sliding} = |\overline{A'B'}| \sin \alpha \quad (4.4)$$

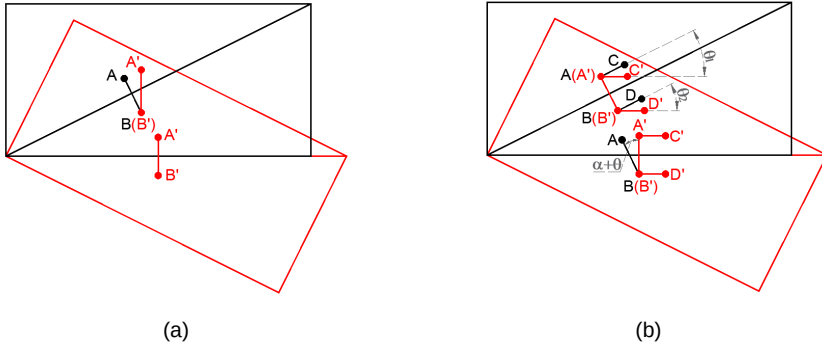


Figure 4.17 Crack displacement calculation: (a) Crack displacement error due to rigid body motion; (b) Crack opening and sliding calculation.

Similar to the crack displacement calculation in aforementioned proposed equation, an open-source software for automated crack detection and crack measurement (ACDM) developed by Gehri et al. [212,213] is used to compute the shear crack kinematic along the whole shear cracking path with the rigid-body motion (rotation) taken into account. Detection of shear crack, crack orientation, and shear crack width and slip can be provided by ACDM procedures.

Figure 4.18 illustrates the deformation - maximum shear crack displacement (opening and slip) for different specimens. For the RB specimen, upon the onset of a major shear crack, the shear crack first opens and slides merely around 0.1 mm at the maximum load, and then the beam fails in a brittle manner. Similar to the RB specimen, the RB (120) also fails abruptly. Until the peak load, the crack opening and slip of RB (120) are larger than

those of the RB beam but still below 0.3 mm. For ST-M1-EB and ST-M1-CIS-ASD specimens, although UHPFRC will mitigate the surface crack width, it will also help the specimen to sustain a higher load and deflection. The maximum shear crack width for the ST-M1-EB and ST-M1-CIS-ASD specimens can exceed 1.2 mm, providing enough warning before failure. In terms of the ST-M1-CIS-SSD specimen, initially as the load increases, the shear crack on both sides of the UHPFRC gradually grows. However, as the load approaches approximately 112 kN, it can be observed that the shear cracking displacement in the front side remains constant, while at back side it starts decreasing. This behavior can be attributed to the gradual delamination occurring between the UHPFRC layer and the RC section at the rear. The UHPFRC laminate functions as a strengthening layer, transferring external loads from the RC component through the interface. As delamination progresses, it results in stress relaxation within the UHPFRC layer and ultimately leads to the closure of the pre-existing shear crack. Shear crack kinematic analysis for ST-M1-CIS-SSD confirms that bond quality is a critical factor governing the failure pattern and shear load capacity of hybrid UHPFRC-NC beams.

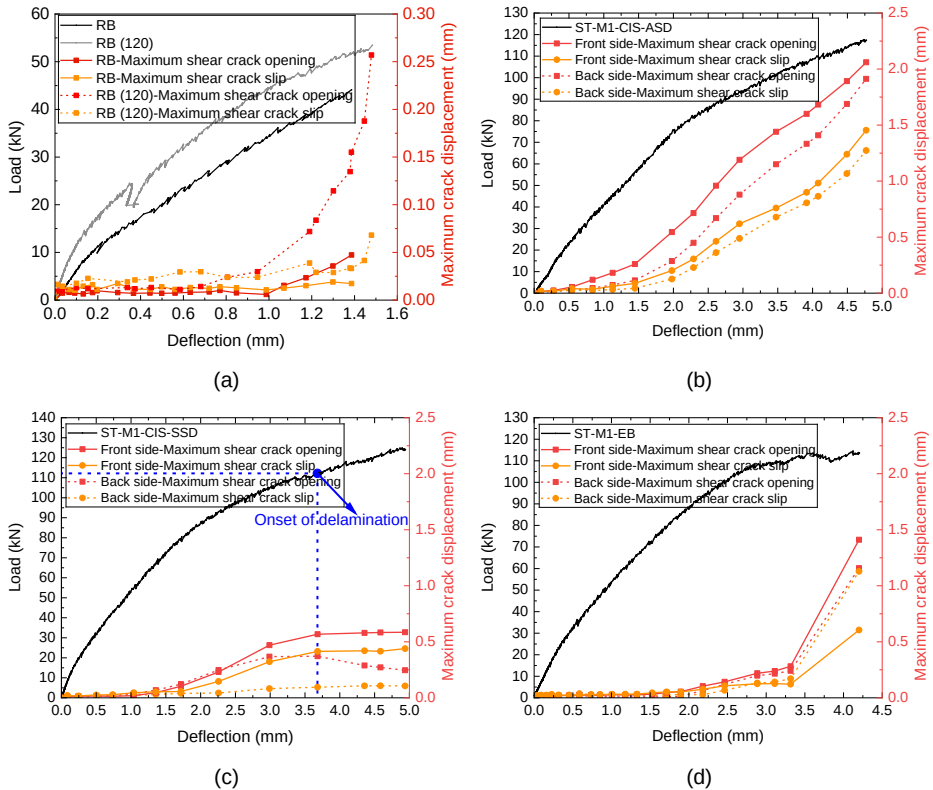


Figure 4.18 Deformation- maximum cracking displacement (opening and slip) of all tested specimens: (a) RB and RB (120); (b) ST-M1-CIS-ASD; (c) ST-M1-CIS-SSD; (d) ST-M1-EB.

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As the failure of the specimens is governed by the major shear crack, a comparison of shear crack displacement (opening and slip) is presented in Figure 4.19. It is found that although the magnitude is lower, the growth of the shear crack slip aligns with its corresponding shear crack opening. The crack opening happens firstly, and later crack slip is activated. Compared to the reference beams, the shear cracks in all strengthened beams develop in a more gradual manner. Both the ST-M1-CIS-ASD and ST-M1-EB specimens still exhibit brittle failure. For the ST-M1-CIS-SSD specimen, there is a gradual increase in crack opening before reaching 110 kN. Due to delamination, the major shear crack on the front side stops growing, and the crack displacement on the back side is reduced compared to the other two strengthened beams. This indicates a more brittle response in the ST-M1-CIS-SSD specimen prior to its ultimate failure.

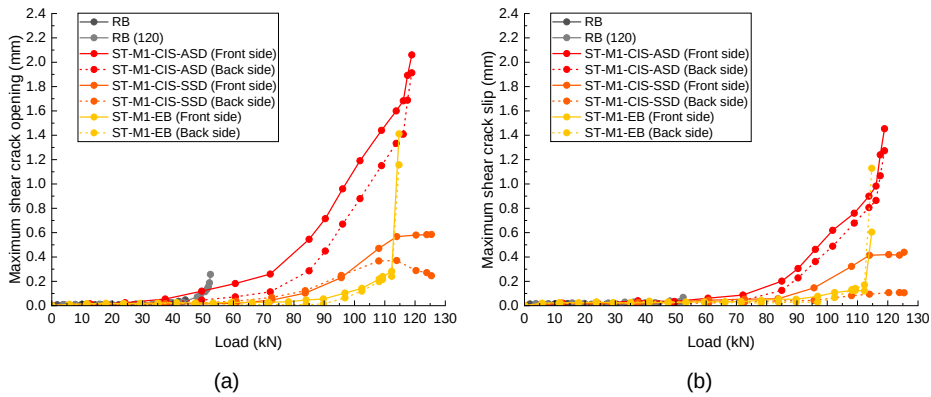


Figure 4.19 Comparison of the maximum shear crack displacement (opening and slip) of all tested specimens: (a) shear crack opening comparison; (b) shear crack slip comparison.

Interface behavior

To investigate the influence of moisture degree of concrete substrate on the bond behavior between concrete and UHPFRC, the load versus interface slip and opening curves are plotted for the ST beams, as shown in Figure 4.20. Although four LVDTs are placed symmetrically at the same distance from the loading point, the interface opening/sliding differs at different measurement points due to asymmetric cracks induced by the applied load.

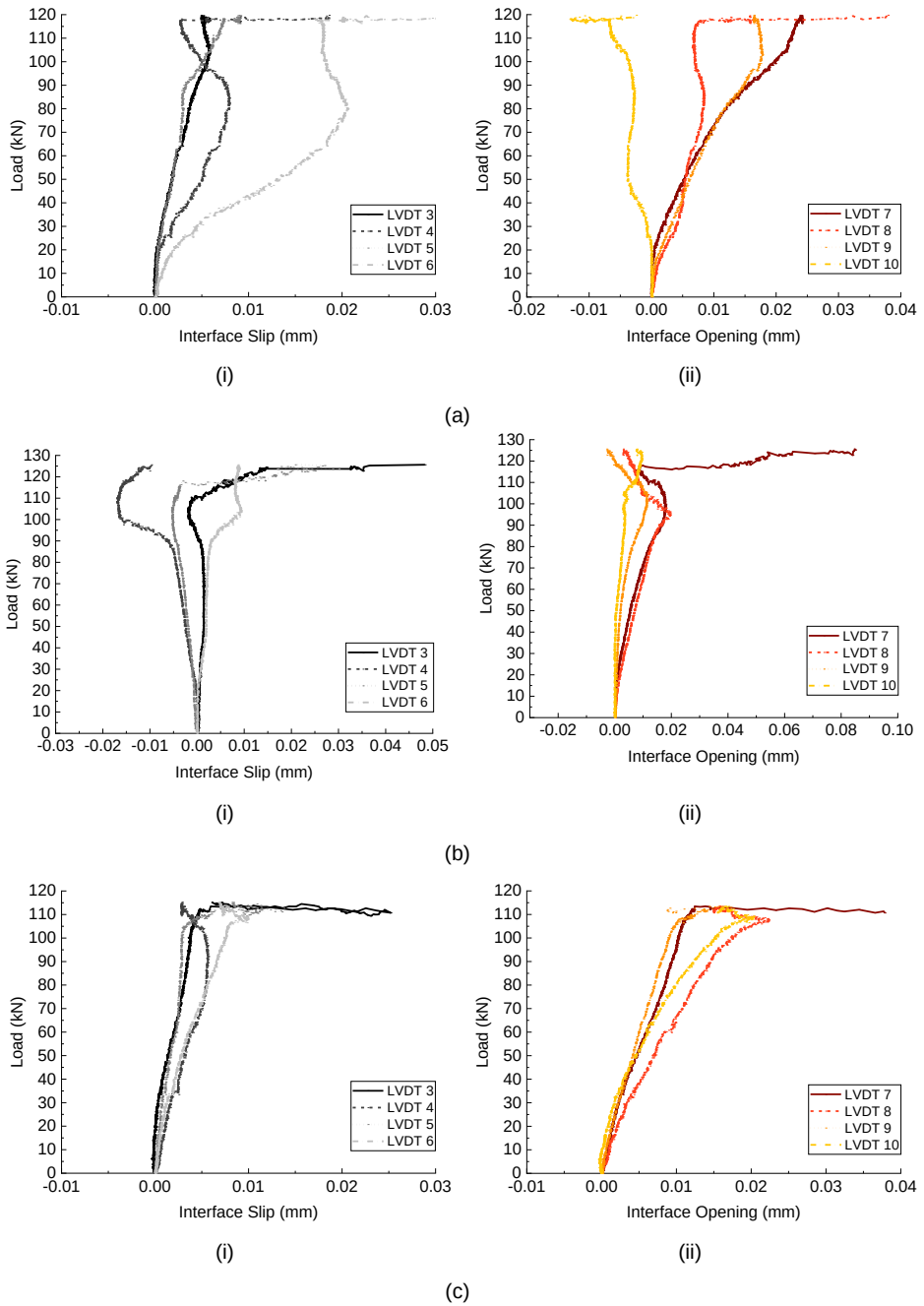
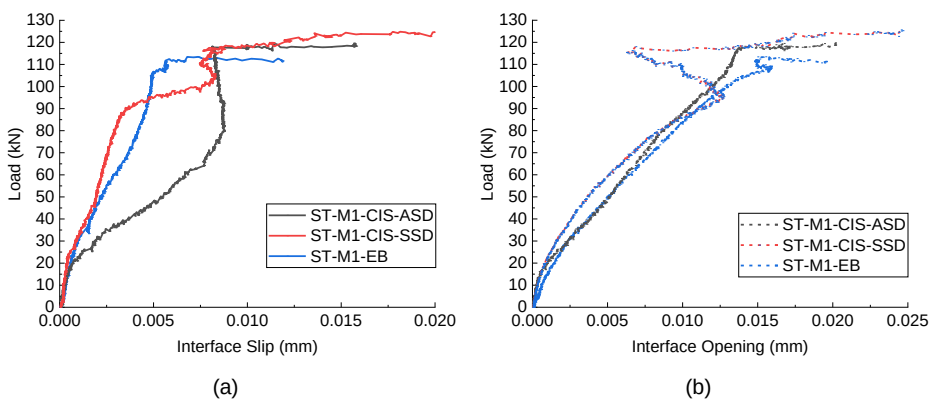


Figure 4.20 Interface displacement (slip and opening) measurement for strengthened beam: (a) ST-M1-CIS-ASD specimen; (b) ST-M1-CIS-SSD specimen; (c) ST-M1-EB specimen.

4.3 Structural test results

Figure 4.21 (a) and (b) illustrate the average curve of the load - interface displacement (slip and opening). In general, when the peak load is reached, the maximum interface displacement (slip and opening) does not exceed 0.02 mm for the ST-M1-EB and ST-M1-CIS-ASD specimens. In the ST-M1-CIS-SSD specimen, although initially it has the lowest interface opening, when the major shear crack propagates (at the load level of 115 until 126 kN), the interface opening develops faster. The increase in interface opening can be explained by the formation and propagation of a major shear crack, which probably imposes strain or propagates through the interface between UHPFRC and NC. Since a similar load-interface slip/opening response is observed in all strengthened beams, a typical load versus interface displacement curve obtained from the ST-M1-CIS-SSD is chosen to provide deeper insight into interface behavior for the UHPFRC-NC composite beam. As presented in Figure 4.21 (c), the curve can be divided into three stages corresponding to the damage process of beams. Before the initiation of a major shear crack (stage I), the interface displacement develops at a low rate as the load increases. Moreover, in stage I, the interface opening increases faster than the corresponding slip, as the interface opening (measured LVDT deformation in the transverse direction) actually represents the transverse deformation of the composite beam. After shear cracking (stage II), the interface slip develops faster, as reflected by a lower slope in stage II than that in stage I, which is attributed to severe bond damage as shear crack propagates to the interfacial zone. For interface opening ST-CIS-SSD shows a “snap-back” behavior as the dominant shear crack grows and the energy releases from initiation and propagation of major shear crack. Furthermore, incompatible displacement occurs between UHPFRC and NC because of higher bond damage. When entering stage III (115.4 kN), the interface displacement increases at the highest rate until the failure of the specimen.



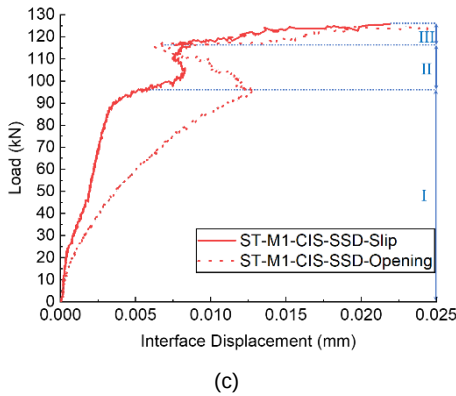


Figure 4.21 Load-interface displacement relationship: (a) Load-interface slip curve for the three tested specimens; (b) Load-interface opening curve for the three tested specimens; (c) Load-interface displacement for ST-CIS-SSD specimen.

4.3.3 Influence of bonding techniques combined with freeze-thaw action

Shear resistance of the reference and strengthened beams

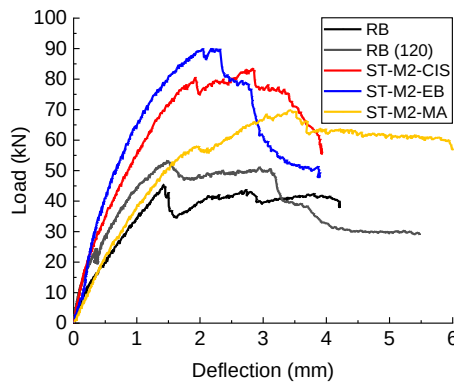
Shear capacity after freeze-thaw exposure was evaluated on hybrid beams strengthened by different bonding techniques. The shear load-displacement relationship for all tested beams is shown in Figure 4.14 (a). It is observed that, the application of UHPFRC is beneficial to increase the load, deformation, and energy absorption capacity regardless of the bonding technique. UHPFRC strengthening improves the load-bearing capacity by 84%, 98% and 54% when cast in-situ (ST-M2-CIS), epoxy-binding (ST-M2-EB), and mechanical anchorage (ST-M2-MA), respectively, are applied. The mechanical anchorage method provides the lowest shear resistance. Holes drilled for mechanical anchorage reduce the volume of concrete and UHPFRC, providing weak zones for fracture initiations, which agrees with the finding in a previous study [96].

Beams exhibiting the highest load capacity show the lowest deflection capacity. Beam strengthened by mechanical anchorage (ST-M2-MA) shows the highest improvement in deflection (141%) while the lowest is observed in ST-M2-EB (58%). Due to its low stiffness, ST-M2-MA, which only relies on bolt connections, shows the highest deflection capacity. This is attributed to the progressive cracking and collapse of the UHPFRC along the localized bolt-connected region. The energy absorption is also calculated and listed in Table 4.5. All three strengthened beams have at least four times higher energy absorption than that of the RB beams and ST-M2-CIS shows the highest energy absorption (170.8 kN.mm) among the three ST beams. In terms of stiffness, the initial stiffness of the beam almost does not change with the application of UHPFRC. As the load increases, the stiffness of the reference beam decreases more compared to strengthened beam. Among the three strengthened beams, the ST-M2-MA specimen exhibits the most rapid stiffness degradation. It should be mentioned that due to the different mixtures of UHPFRC, although

4.3 Structural test results

with the same bonding techniques, there is a substantial difference in terms of load, deflection, and energy absorption between ST-M1-CIS-ASD and ST-M2-CIS, and between ST-M1-EB and ST-M2-EB (Table 4.3 and Table 4.5).

The reliability of the bonding method is also investigated under more severe exposure conditions. It has been shown that the freeze-thaw action causes damage to cementitious materials and consequently leads to performance degradation in concrete structures [185]. Therefore, the beams are tested after exposing them to freeze-thaw cycles (figure 4.14b). In the reference beams, no significant damage is observed after 120 freeze-thaw cycles, which is in line with the concrete compressive strength presented in Table 4.2. The differences between unexposed and exposed RB specimens in peak load are small (12%, 12% and 23% for RB-30, RB-75 and RB-120, respectively) and can be a result of the scatter in material properties. Similar to the reference beams, the shear capacity of beams seems also not to be affected by exposure to freeze-thaw. Although the strengthened specimens after exposure to freeze-thaw show higher loading, deformational and energy absorption capacity, especially for the group of epoxy-bonded specimens, the difference (around 20%) in peak load is small and is likely still within the scatter of the results. To provide a more in-depth analysis of the test results and underlying differences in fracture behavior and capacity of the beams, the fiber distribution for each hybrid beam is measured through the electromagnetic method and its influence on the shear behavior is numerically investigated, which is further discussed in Chapter 6 and Chapter 7, respectively.



(a)

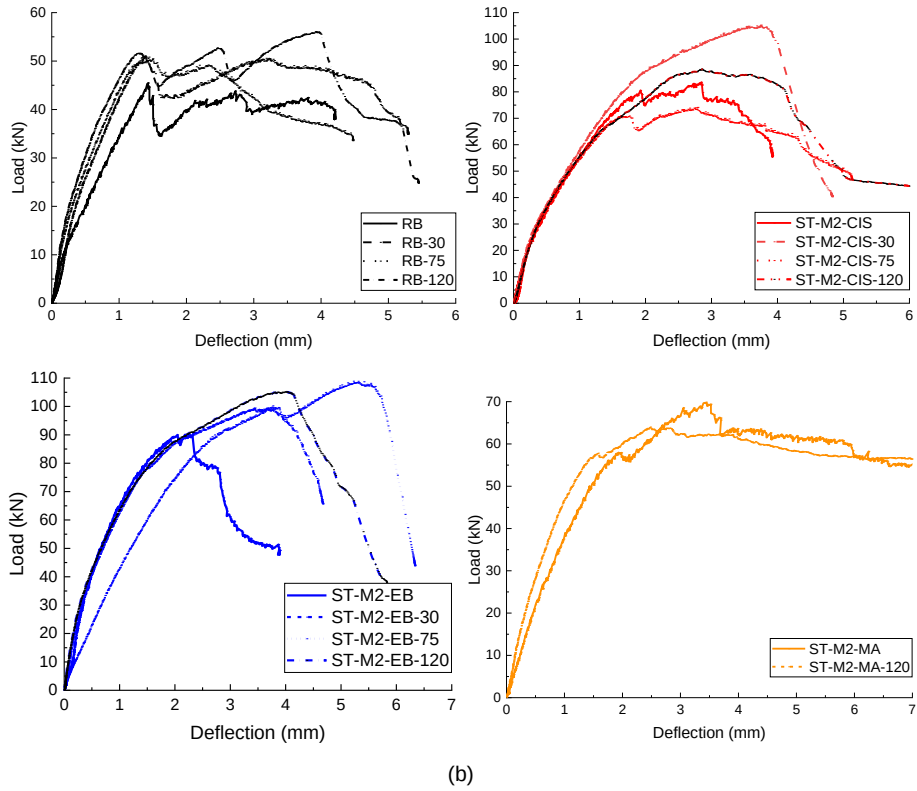


Figure 4.22 Load- (mid-span) deflection response of the tested specimens: (a) different bonding techniques; (b) different freeze-thaw cycles.

Table 4.5 Results in terms of load and deflection (noting that all beams fail in shear).

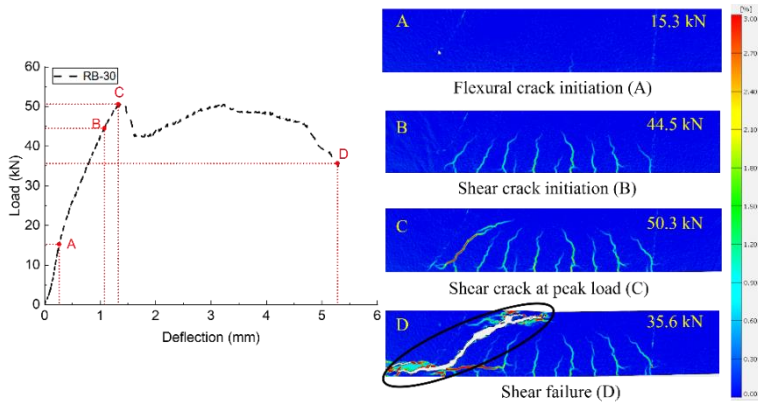
Specimen ID	F_u (kN)	$\Delta F_{u,ST}$ (%)	$\Delta F_{u,FT}$ (%)	δ_u (mm)	$\Delta \delta_{ST}$ (%)	$\Delta \delta_{FT}$ (%)	E_u (kN.mm)	$\Delta E_{u,ST}$ (%)	$\Delta E_{u,FT}$ (%)
RB	45.5	-	-	1.4	-	-	36.9	-	-
RB (120)	53.5	18	-	1.5	3	-	51.1	38	-
RB-30	51.1	-	12	1.4	-	-3	42.2	-	14
RB-75	50.8	-	12	1.4	-	1	47.8	-	29
RB-120	56.1	-	23	3.9	-	174	175.7	-	376
ST-M2-CIS	83.7	84	-	2.9	99	-	170.8	363	-
ST-M2-CIS-30	105.3	-	26	3.8	-	32	281.4	-	64
ST-M2-CIS-75	74.1	-	-11	2.8	-	-2	155.2	-	-9
ST-M2-CIS-120	88.7	-	6	2.9	-	0	173.9	-	2
ST-M2-EB	90.1	98	-	2.3	58	-	140.3	280	-
ST-M2-EB-30	99.5	-	10	3.5	-	53	250.2	-	78
ST-M2-EB-75	108.9	-	21	5.2	-	131	389.9	-	178
ST-M2-EB-120	105.2	-	17	4.0	-	77	313.0	-	123
ST-M2-MA	69.9	54	-	3.5	141	-	162.0	339	-
ST-M2-MA-120	64.0	-	-8	2.5	-	-28	112.1	-	-31

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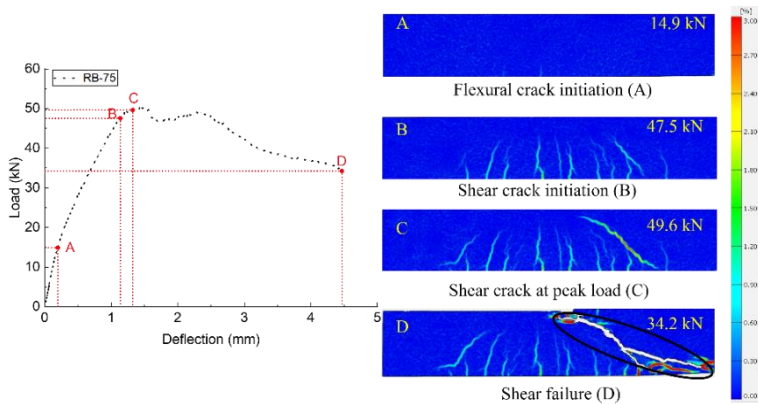
Note: F_u is the peak load; δ_u is the mid-span deflection at the peak load; E_u is the energy absorption; $\Delta F_{u,ST}$, $\Delta \delta_{ST}$ and $\Delta E_{u,ST}$ are the increased percentages of peak load, deflection, and energy absorption of RB-120 and each strengthened specimen to that of RB specimen without FT, respectively; $\Delta F_{u,FT}$, $\Delta \delta_{FT}$ and $\Delta E_{u,FT}$ are the percentages change of peak load, deflection and energy absorption of each specimen after FT to that of corresponding specimen without FT, respectively.

Cracking behavior and failure modes

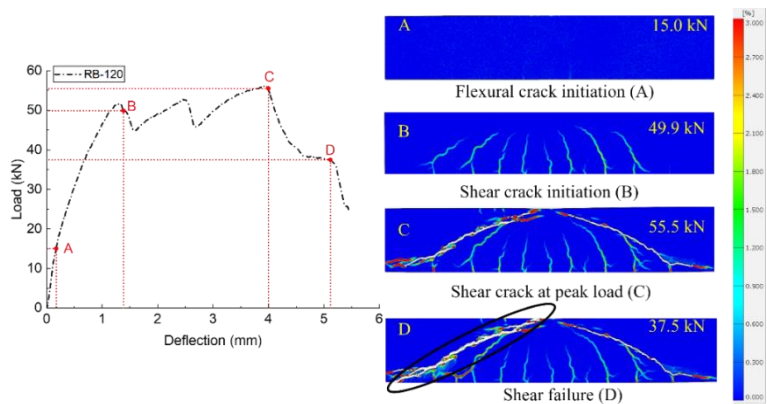
Figure 4.23 presents the damage process of tested specimens (RB, CIS, EB and MA groups) with and without exposure to freeze-thaw cycles obtained by analyzing the principal strain by DIC. The distinctive characteristic loads (labelled as A, B and C points in Figure 4.23) and the corresponding deflections are listed in Table 4.6. The multiple-stage fracture process, as defined earlier, is shown in all specimens except specimens in MA group. In the MA group, despite the brittle failure, the failure patterns in the front and back sides are not similar possibly due to the stress concentrations at the connection with bolt, making the dominant crack propagate along the bolted area.



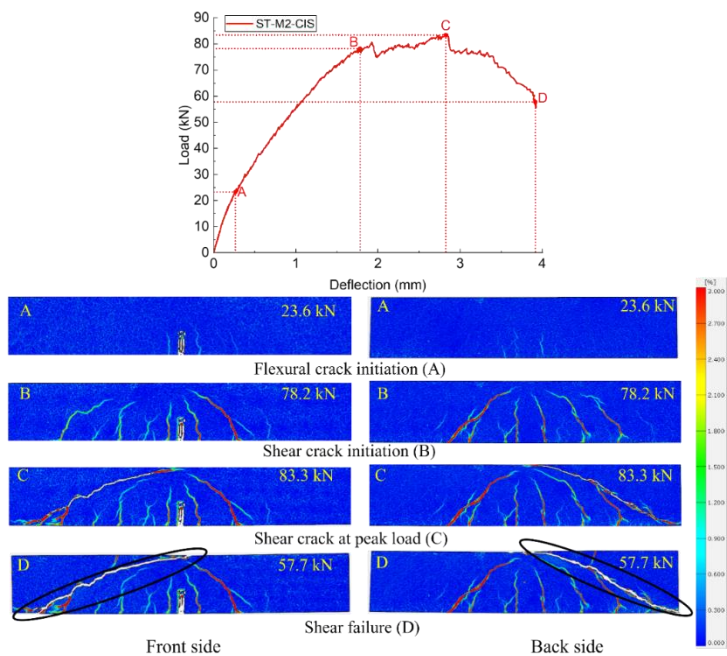
(a) RB-30 specimen



(b) RB-75 specimen

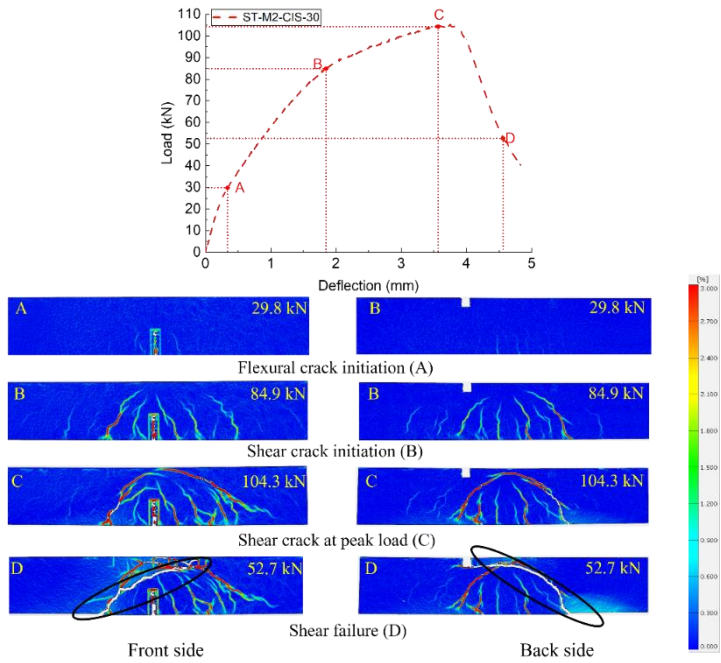


(c) RB-120 specimen

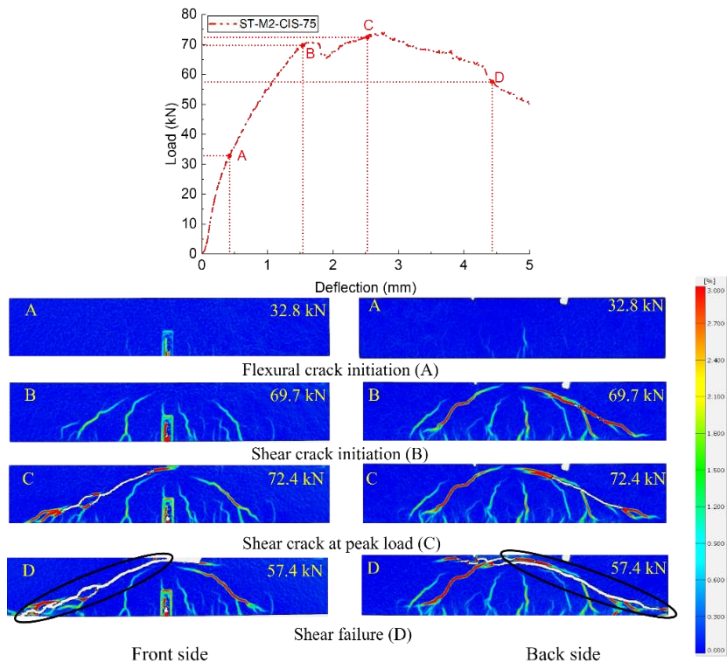


(d) ST-M2-CIS specimen

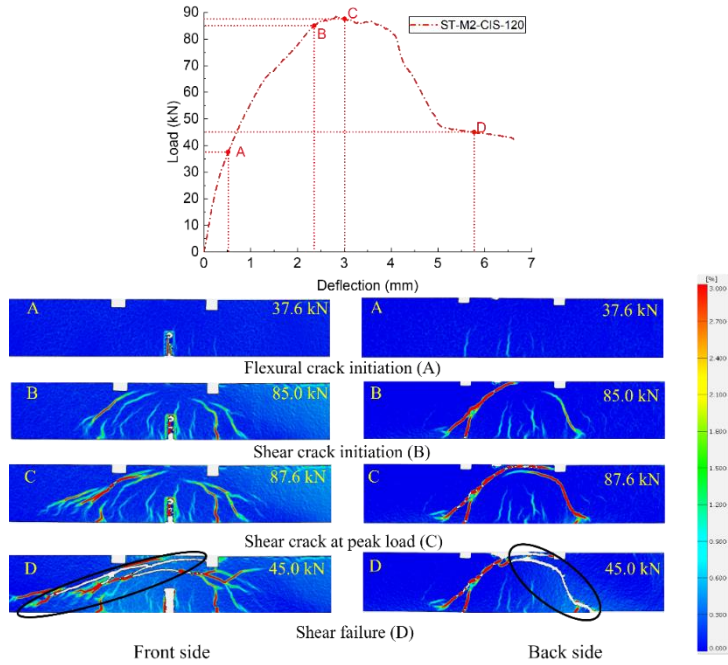
4.3 Structural test results



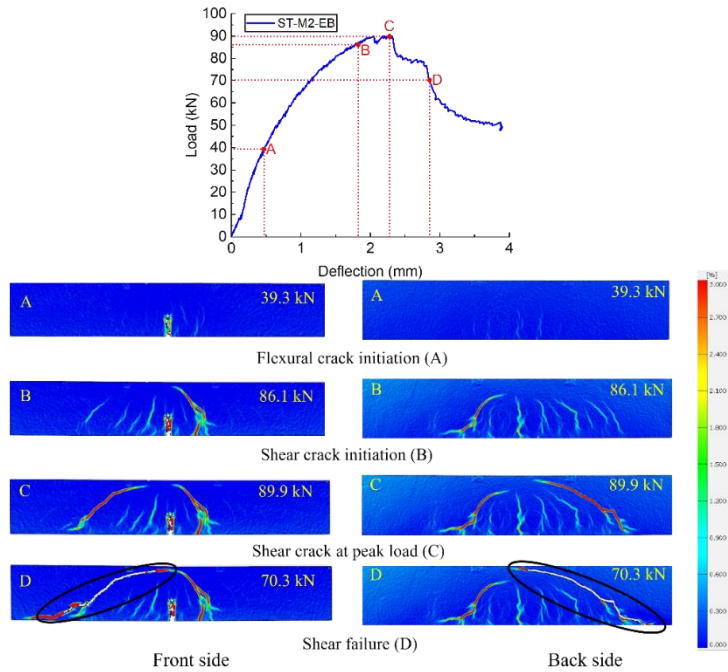
(e) ST-M2-CIS-30 specimen



(f) ST-M2-CIS-75 specimen

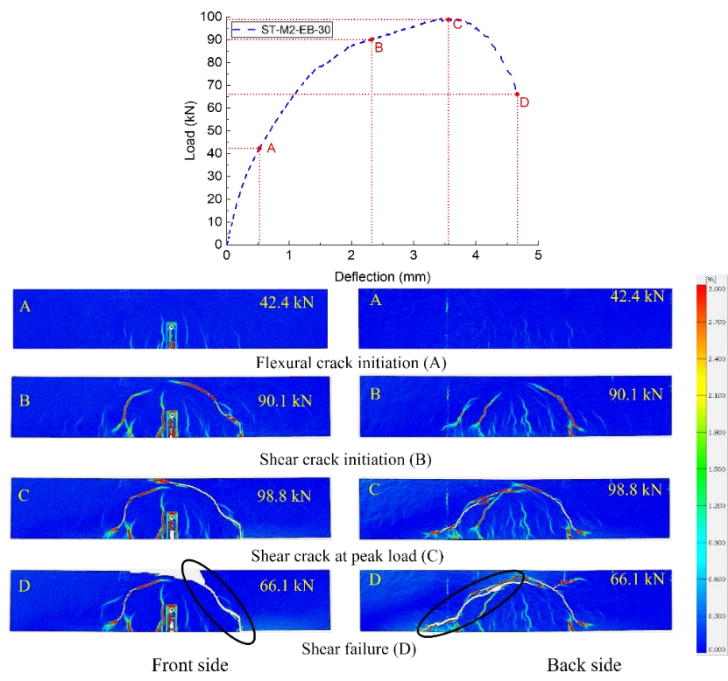


(g) ST-M2-CIS-120 specimen

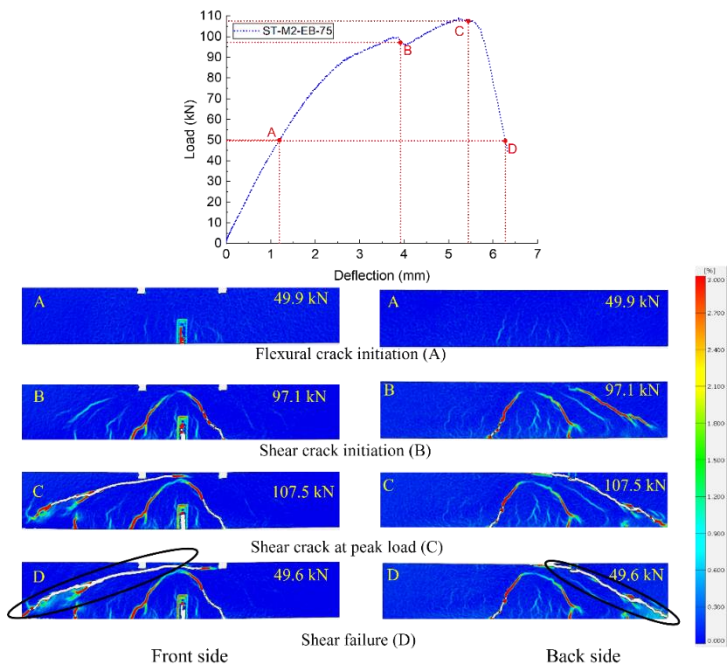


(h) ST-M2-EB specimen

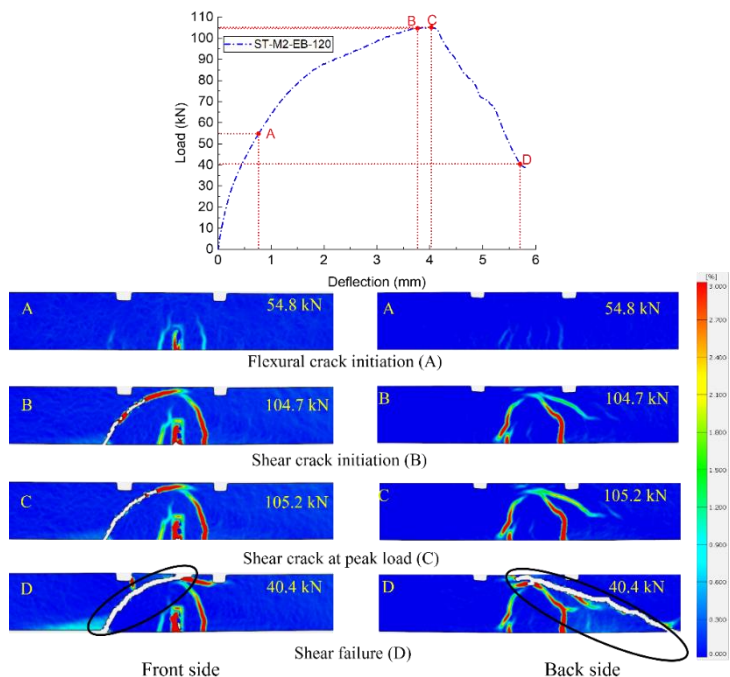
4.3 Structural test results



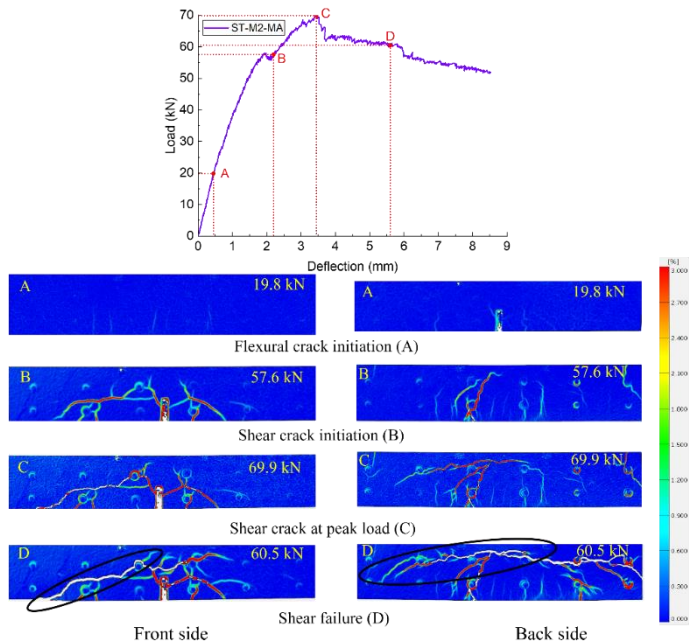
(i) ST-M2-EB-30 specimen



(j) ST-M2-EB-75 specimen



(k) ST-M2-EB-120 specimen



(l) ST-M2-MA specimen

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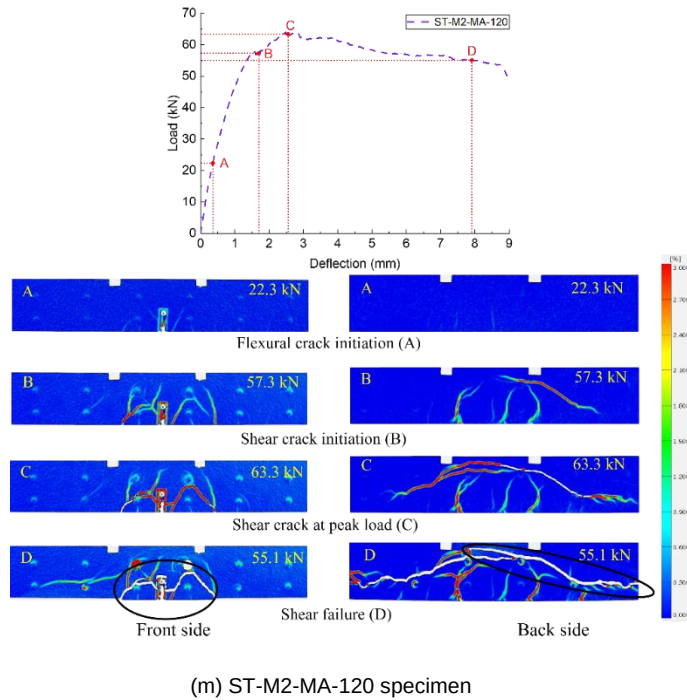


Figure 4.23 Damage process of the tested specimens.

Table 4.6 Distinctive characteristic loads and corresponding deflections for all tested specimens.

Specimen ID	$F_{cr,f}$ (kN)	$\delta_{cr,f}$ (mm)	$F_{cr,s}$ (kN)	$\delta_{cr,s}$ (mm)	F_u (kN)	δ_u (mm)
RB-30	15.3	0.3	44.5	1.1	50.3	1.3
RB-75	14.9	0.2	47.5	1.1	49.6	1.3
RB-120	15.0	0.2	49.9	1.4	55.5	4.0
ST-M2-CIS	23.6	0.3	78.2	1.8	83.3	2.8
ST-M2-CIS-30	29.8	0.3	84.9	1.9	104.3	3.6
ST-M2-CIS-75	32.8	0.4	69.7	1.5	72.4	2.5
ST-M2-CIS-120	37.6	0.5	85.0	2.4	87.6	3.0
ST-M2-EB	39.3	0.5	86.1	1.8	89.9	2.3
ST-M2-EB-30	42.4	0.5	90.1	2.3	98.8	3.6
ST-M2-EB-75	49.9	1.2	97.1	3.9	107.5	5.4
ST-M2-EB-120	54.8	0.8	104.7	3.8	105.2	4.0
ST-M2-MA	19.8	0.5	57.6	2.2	69.9	3.5
ST-M2-MA-120	22.3	0.4	57.3	1.7	63.3	2.6

For the beams in the RB group (without and with exposure to freeze-thaw), all show similar failure mode, as presented earlier for RB and RB (120) in Figure 4.15a and b and Figure 4.23a-c.

The shear cracking resistance of the beams in the CIS group is at least 1.5 times higher than that of the beams in the control group. In all beams except ST-M2-CIS-120, failure

pattern on both sides of the beam is symmetric (Figure 4.23d-f). In ST-M2-CIS-120 at the front side (Figure 4.23g), the inclined crack begins at the web of the beam while at the back side, the shear crack continues from a flexural crack. As the load increases, the shear crack further grows along the diagonal direction to the loading point. At peak load (more than 70 kN), the diagonal crack has grown through the concrete in the compressive zone, causing sudden failure. Beside the loading resistance, the beams in the CIS group have at least 1.3, 1.4 and 1.9 times higher deformation at flexural cracking, shear cracking and peak load, respectively, compared to beams in the RB group. It should be noted that the comparison is not made for the deformation at the peak load of the RB-120 specimen, in which the deformation is much larger due to the random shear crack pattern.

The shear performance is also significantly enhanced in the EB group. The strengthened beams in EB group exhibit around 1.9 times the peak load resistance of reference beams. However, the crack development in each beam is different. In ST-M2-EB and ST-M2-EB-75 specimens (Figure 4.23h and j), the shear crack first initiates at the web of the beam, continues to grow towards the loading point, and leads to beam failure. On the other hand, in ST-M2-EB-30 and ST-M2-EB-120 specimens (Figure 4.23i and k), flexural-shear failure occurs because the dominant shear crack forms from a developed flexural crack. It should be noted that, although the ST-M2-EB-120 specimen exhibits flexural-shear failure from the development of the shear crack, there is a distinction in the failure patterns observed on its front and back sides (Figure 4.23k). The underlying mechanism will be further investigated by the fiber distribution measurements in Chapter 6.

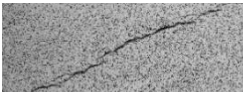
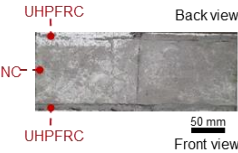
Similar to beams in the CIS and EB groups, the shear performance of beams in the MA group is also improved with the addition of UHPFRC. However, compared to beams in the CIS and EB groups, the improvement is reduced in terms of loading capacity; the maximum improvement for the peak load resistance is 59%. On the other hand, the ST-M2-MA specimen shows better deformational ability: the deformation at flexural cracking load, shear cracking load and peak load are 2.5, 1.7, and 2.5 times higher compared to the reference beams. The difference in loading and deformational capacity improvement in each group originates from different bonding methods. Due to the “rigid bond-slip behavior provided by cast-in situ and epoxy bonded method [70], the UHPFRC and reinforced concrete can work together to resist the force and a higher load resistance is achieved. On the other hand, a higher deformational capacity is shown in “non-rigid” bond-slip behavior with reinforced interface by mechanical anchorage [70]. As there is no adhesion between UHPFRC and concrete, the damage and failure patterns for the beams in the MA groups are different and random. For the ST-M2-MA specimen, a dominant shear crack appears at the front side and propagates along the edge of bolt-connected region (also stress concentration region). On the back side, initiated from one flexural crack, the dominant crack develops horizontally along the edge of the bolt on the top side of the specimen. In ST-M2-MA-120, the shear crack seems to appear only on one side of

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the specimen. At the front side of the specimen, several flexural cracks develop first at the bottom of the mid-span of the specimen and then are gradually horizontally connected along the bolts. Finally, the fully developed and connected flexural cracks lead to the failure of the specimen.

The failure mode of each specimen is shown in Figure 4.24. All specimens eventually fail in shear. In the ST-M2-MA-120 specimen, several flexural cracks develop at the bottom of the middle of the beam and then connect to make the UHPFRC spall. From the top view of beams in the MA group, the separation is seen between UHPFRC and NC as there is no bond at the interface. For beams in the CIS and EB groups, shear failure, characterized by dominant shear crack, happens on their both sides and it is found that due to the randomness of shear failure, the shear crack pattern varies in different specimens.

Observed from the top view of the specimens, three typical damage phenomena can be distinguished: UHPFRC fracture, interfacial delamination, and concrete cracking. Attention is focused on the interface, since in general delamination weakens the strengthening efficiency of UHPFRC. Also, the freeze-thaw cycles may affect the interface between UHPFRC and normal concrete [125]. However, from the final failure mode it is unclear when the delamination occurs. Therefore, a further quantitative analysis of the interface behavior from the LVDT measurement is conducted.

Specimen	Front side	Back side	Top side
RB	-		-
RB-30	-		-
RB-75	-		-
RB-120	-		-
ST-M2-CIS			

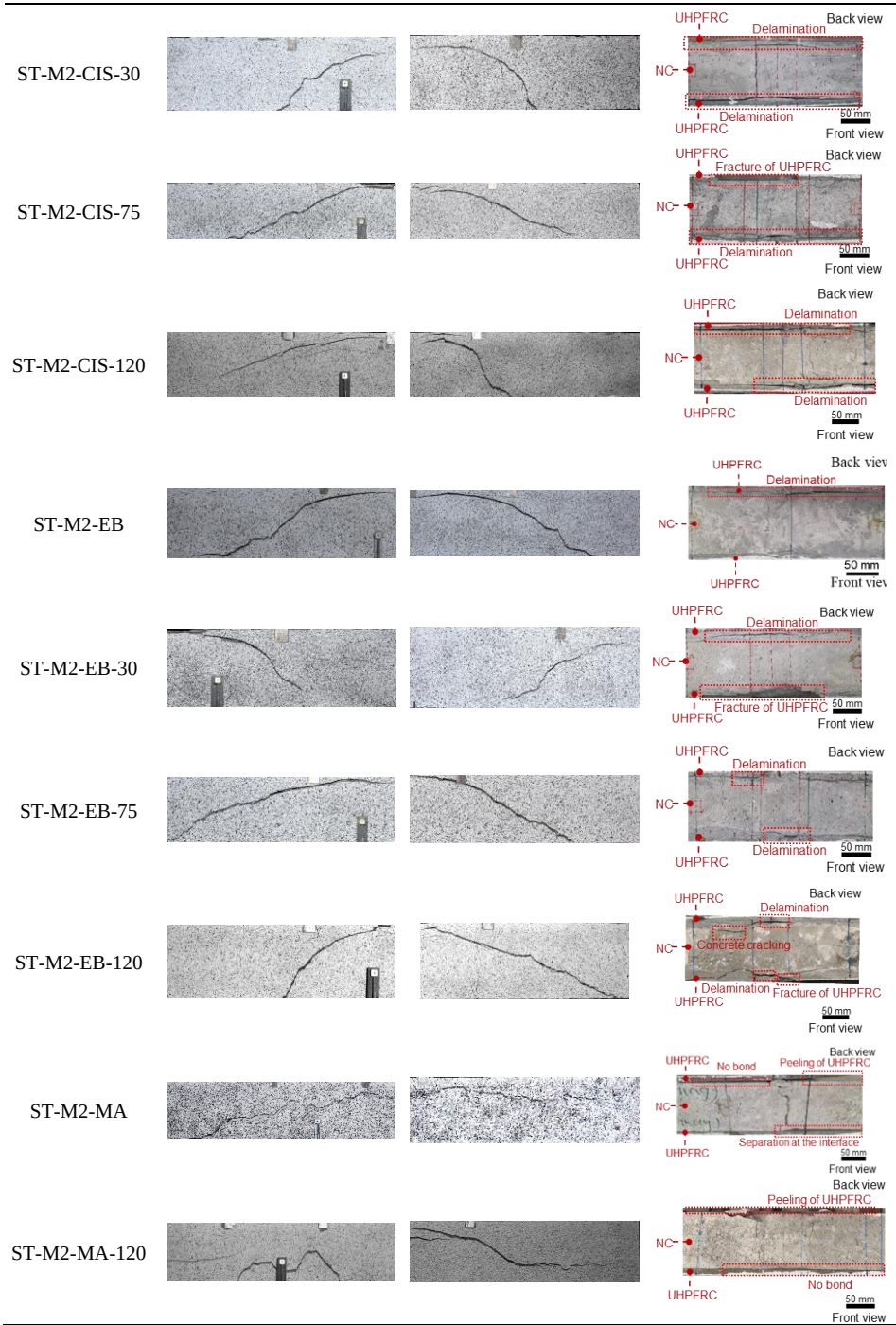
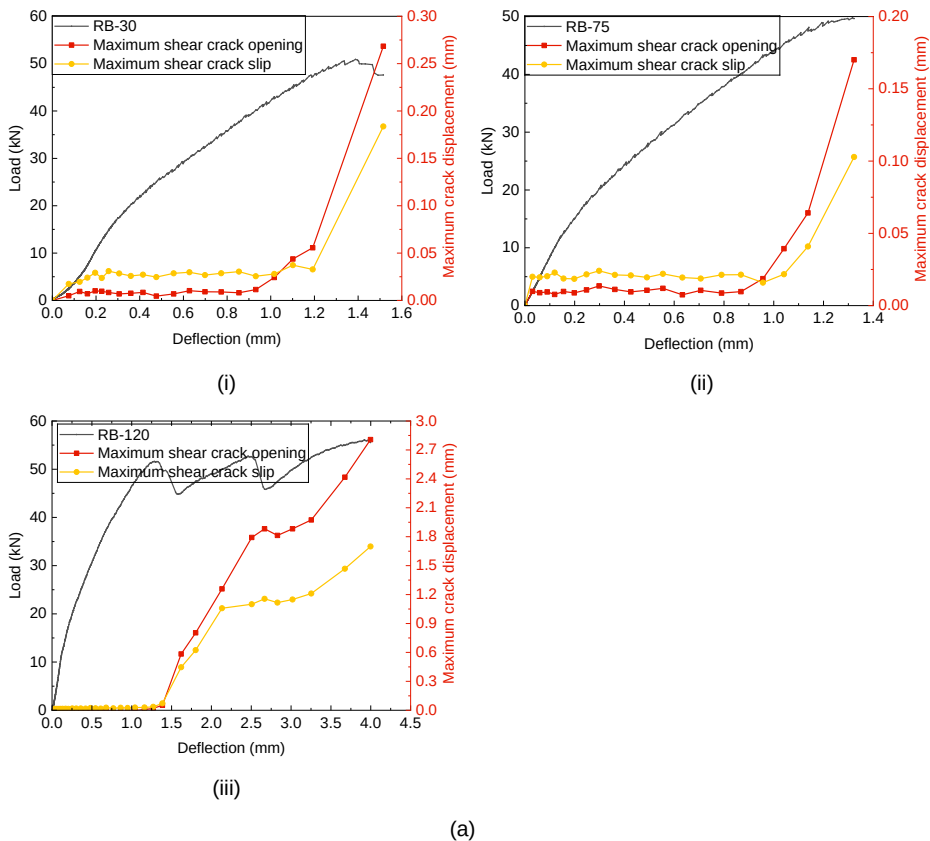
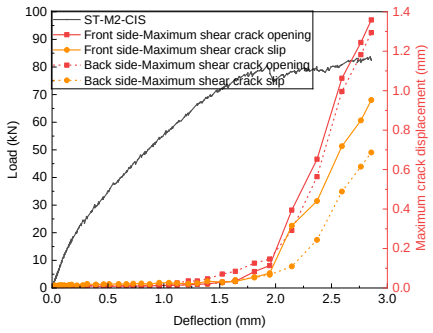


Figure 4.24 Failure modes on both sides for the tested specimens.

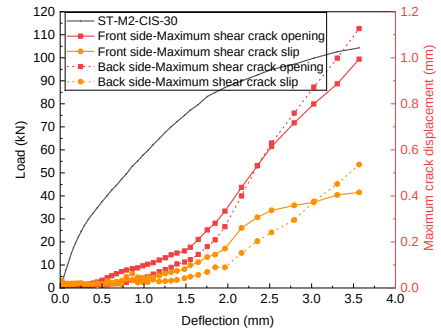
Cracking kinematics of tested beams

Figure 4.25 shows the maximum crack displacement (opening and slip) along the major crack for each specimen. Similarly as observed earlier (Figure 4.19), the opening starts earlier, and in general, the opening of the diagonal crack is higher than its sliding. For RB beams, except the RB-120, the major shear crack opening and sliding at the peak load is below 0.3 mm. Due to the random shear pattern, RB-120 specimen exhibits higher deformation, therefore enabling a larger crack displacement (around 2.8 mm) at the peak load. All strengthened beams, except those bonded by mechanical anchorage, behave in a similar way in terms of crack displacement. Close to the peak load, the critical shear crack occurs and the crack displacement rapidly increases, followed by sudden shear failure. Compared to RB specimens in general, the crack opening and slip are higher and the maximum shear crack width at the peak load exceeds 0.35 mm. For beams in the MA group, the dominant shear crack appears only on one side while the failure is governed by a flexural crack on the other side. It is found that the maximum crack width can be more than 1.0 mm.

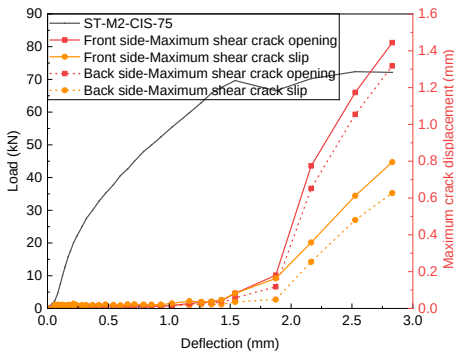




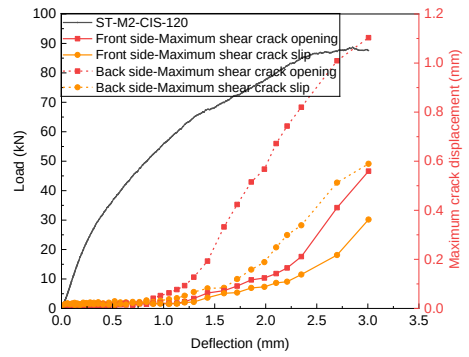
(i)



(ii)

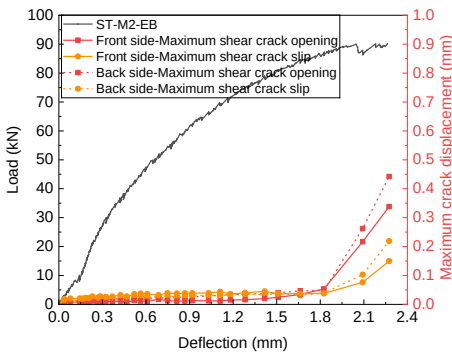


(iii)

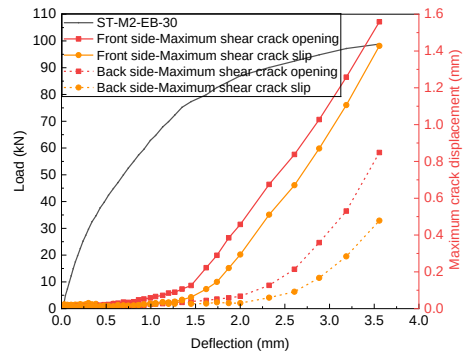


(iv)

(b)



(i)



(ii)

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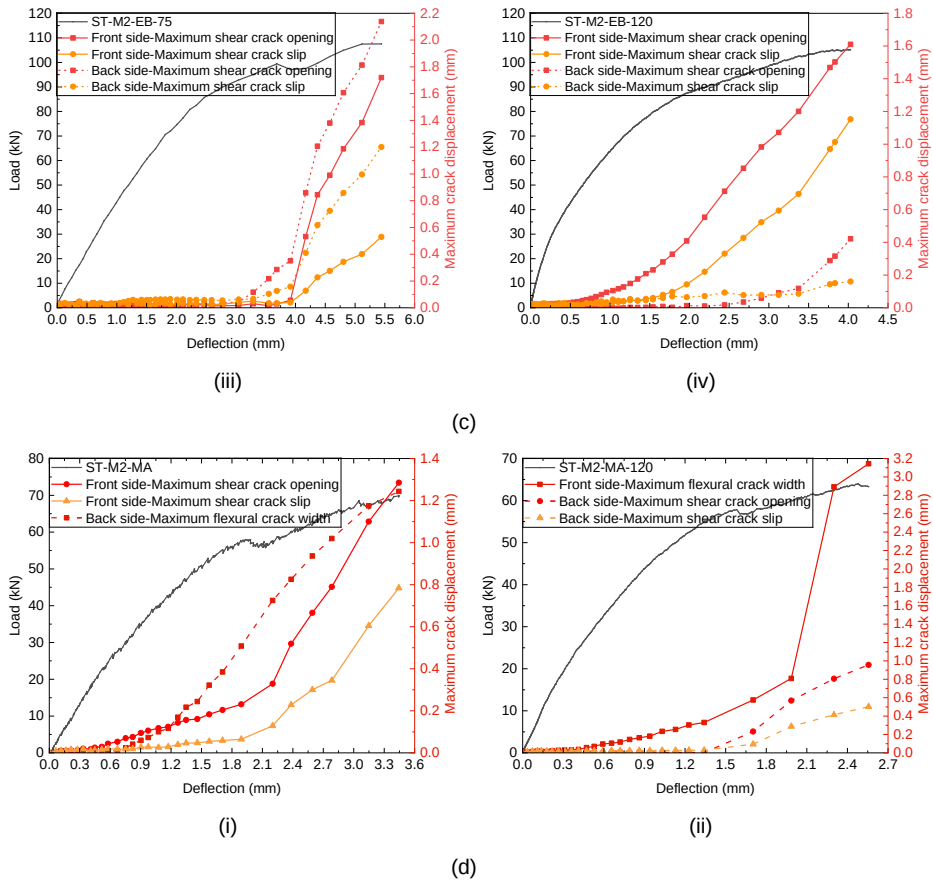


Figure 4.25 Deformation- maximum crack displacement (opening and slip) of all tested specimens: (a) RB group; (b) CIS group; (c) EB group; (d) MA group (noting that for MA group, based on the different type of dominant crack on both sides, the maximum shear crack displacement is presented only for one side and the maximum flexural crack width is shown for another side).

Figure 4.26 shows a comparison of the maximum shear crack opening and slip for every beam in terms of different bonding techniques. In ST-M2-MA the crack opens earlier and faster, and large crack displacement is observed at the peak load. It is interesting to note that, although a higher peak load is seen in the ST-M2-EB specimen, ST-M2-CIS has a significantly larger shear crack displacement at peak load. This is not in line with previous results (Figure 4.14) when the cast-in situ specimens with SSD condition and epoxy bonded had the same stiffness. Further analysis of the possible role of fiber orientation with this UHPFRC mixture is performed in Chapter 6.

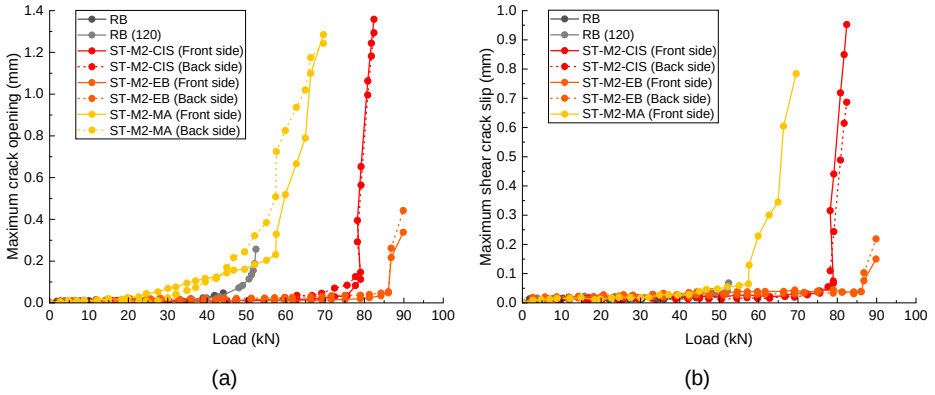
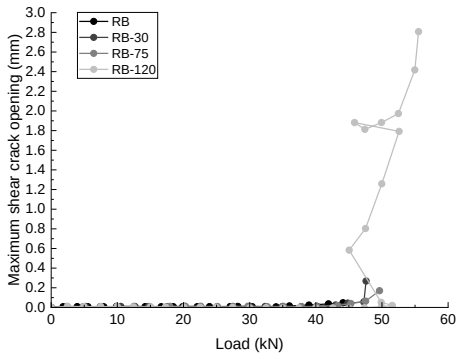


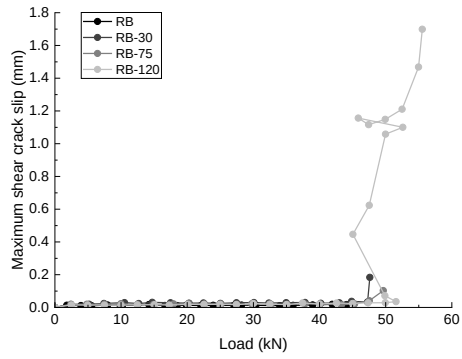
Figure 4.26 Comparison of the maximum crack displacement (opening and slip) of all tested specimens in terms of different bonding techniques: (a) crack opening comparison; (b) shear crack slip comparison.

In Figure 4.27, a comparison of maximum shear crack opening and slip for every tested beam in terms of different number of freeze-thaw cycles is presented. In the RB group, all beams except the RB-120 specimen experience brittle failure when the shear crack is insignificant (crack width is below 0.3 mm). In the CIS group, the shear crack also opens and slides soon after its initiation, except for the ST-M2-CIS-30 specimen which exhibits earlier shear crack initiation and a more gradual increase of maximum crack displacement. Although the ST-M2-CIS-30 specimen has the highest peak load in the CIS group, the maximum crack displacement at the peak load is lowest. In both the ST-M2-CIS-30 and the ST-M2-CIS-120 specimens, from the development of shear crack displacement, it is found that the shear crack does not grow at the same time on both sides: the shear cracks on the front side of ST-M2-CIS-30 and the back side of ST-M2-CIS-120 specimen initiate earlier compared to the other side. This is even more evident for some beams in the EB group, as observed from the shear crack development in ST-M2-EB-30 and ST-M2-EB-120 specimens. Major shear cracks for beams in the EB group exhibit a more gradual increase in crack displacement after the initiation of shear crack compared to CIS group. In the MA group, the dominant cracks on the front and back sides develop non-simultaneously and asymmetrically. Furthermore, compared to the dominant shear crack appearing only on one side, the dominant flexural crack on the other side develops faster and has a larger crack width at peak load.

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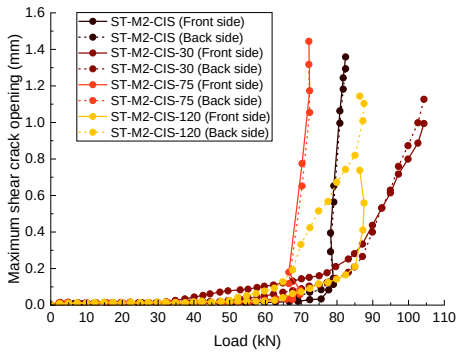


(i)

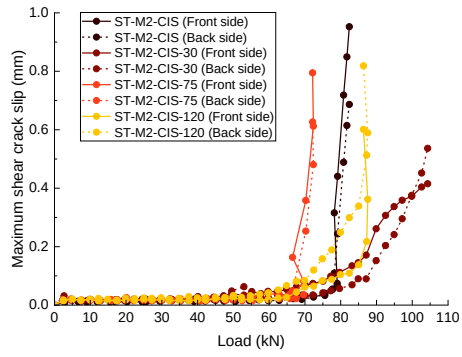


(ii)

(a)

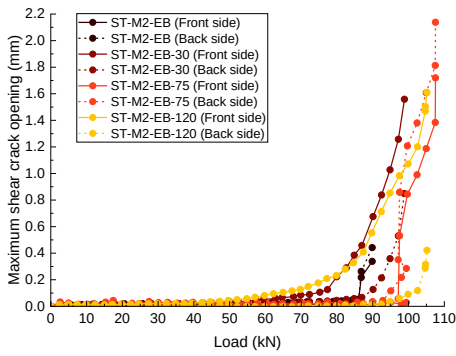


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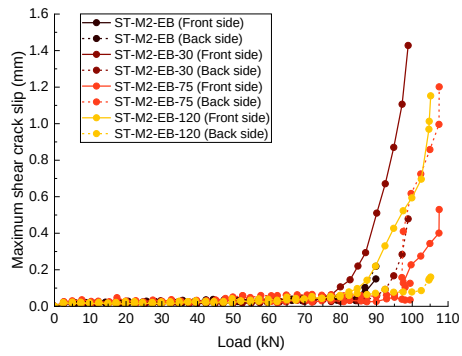


(ii)

(b)



(i)



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(c)

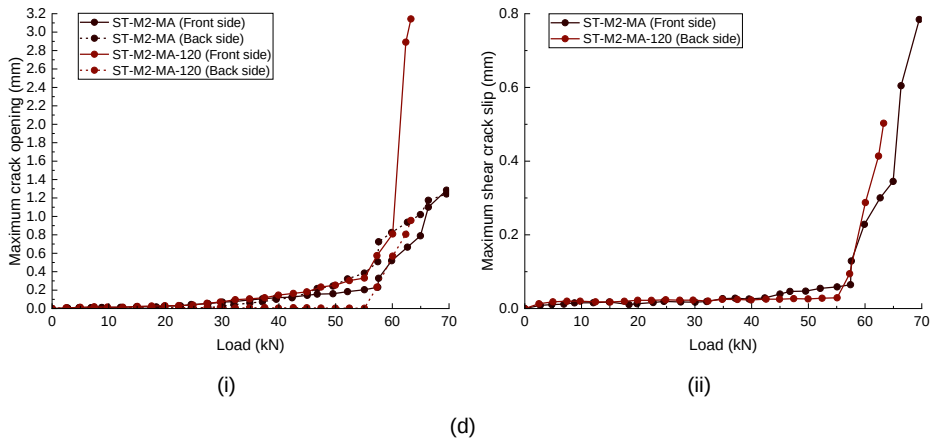


Figure 4.27 Comparison of the maximum crack displacement (opening and slip) of all tested specimens in terms of different freeze-thaw cycles: (a) RB group; (b) CIS group; (c) EB group; (d) MA group (noting that for the MA group, based on the different type of dominant crack on both sides, the maximum shear crack displacement is presented only for one side and the maximum flexural crack width is shown for another side).

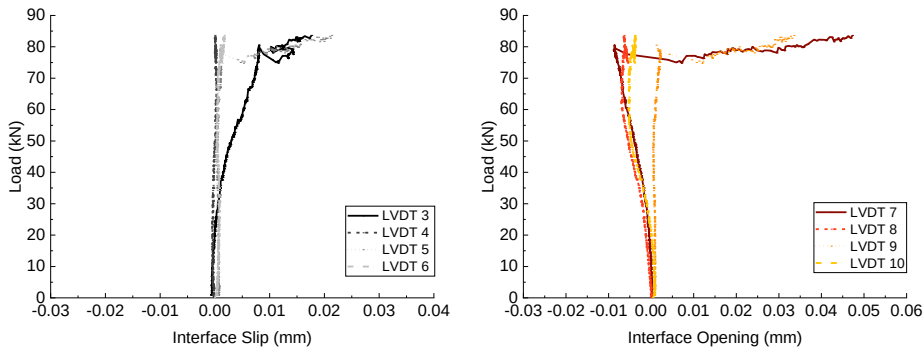
Interface behavior

For ST beams strengthened by different bonding techniques and exposed to different number of freeze-thaw cycles, the results in terms of load versus interface displacement (slip and opening) relationship are presented in Figure 4.28. For beams in the CIS group except the ST-M2-CIS-30 specimen, the maximum slip and opening do not exceed 0.02 mm and 0.05 mm, respectively. For the ST-M2-CIS-30 specimen, large interface opening (0.11 mm) is shown at the measurement point LVDT 7. It seems that severe bond damage occurs in the front side of ST-M2-CIS-30 specimen, which is confirmed by the deformation-maximum crack displacement presented in Figure 4.25 (b (ii)). Although the front UHPFRC laminate initially has a larger crack displacement, when the load is more than 90 kN, the crack displacement (especially the crack slip) develops at a lower speed (even reduced growth rate for the crack slip) than its back side. Similarly as earlier observed, this occurs because of delamination. Note that only partial delamination happens as the crack displacement continues to increase, indicating that the front laminate is still capable of withstanding the load transferred from the RC part. For beams in the EB group, except the ST-M2-EB-75 specimen, the maximum interface displacement is merely 0.02 mm. For the ST-M2-EB-75 specimen, when the ultimate force is nearly reached, the maximum interface opening (measured by LVDT 9 on the back side) experiences sudden increases, which is around 0.6 mm at the peak load. However, no obvious delamination is observed (shown in Figure 4.24) and the main shear cracks on front and back sides are symmetrically developed as presented in Figure 4.25 (c (iii)). The sudden change in displacement recorded by LVDT 9 could be caused by the propagation of a major shear crack in the measuring zone (shown in Figure 4.24), leading to erroneous measurement of interface

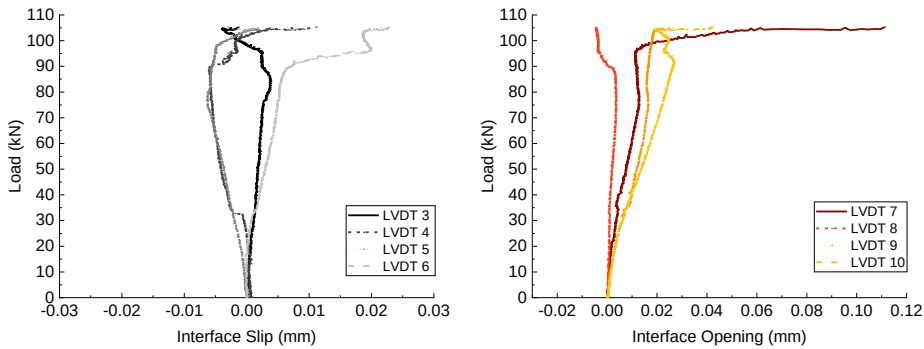
4.3 Structural test results

opening. For beams in MA groups, the absence of adhesion at the interface results in maximum interface displacement among all beams, which can even reach around 0.7 mm.

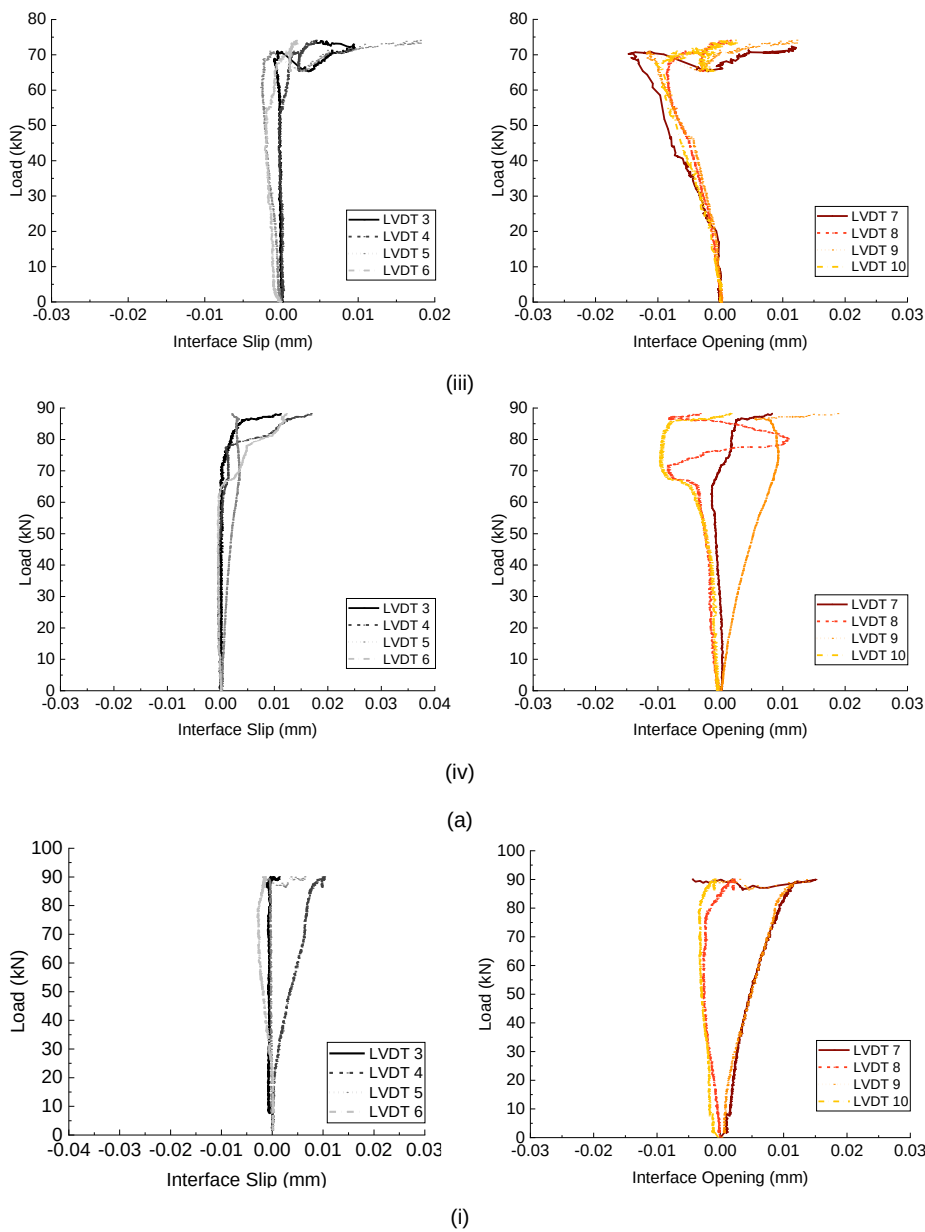
As stated in fib Bulletin 43 [134], the maximum allowed interface slip is 0.2 mm under the serviceability limit state and 2.0 mm under the ultimate limit state. For epoxy-bonded specimens with or without exposure to freeze-thaw cycles, the small slip and opening prove that no interface delamination is observed, and the interface remains intact until the peak load. It is concluded that an effective bonding quality between UHPFRC and NC could be provided with the use of epoxy resin. However, for cast in-situ specimens, without special concrete surface treatments such as high-pressure water jetting or sand blasting, the risk of delamination remains as sufficient bond cannot be guaranteed especially for ST-M2-CIS-30.



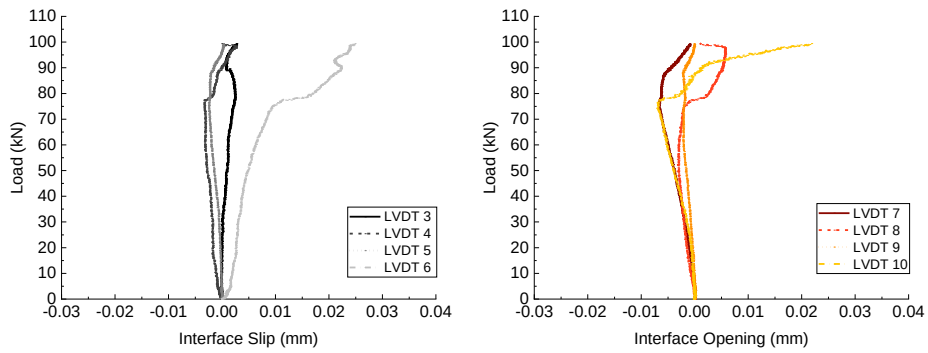
(i)



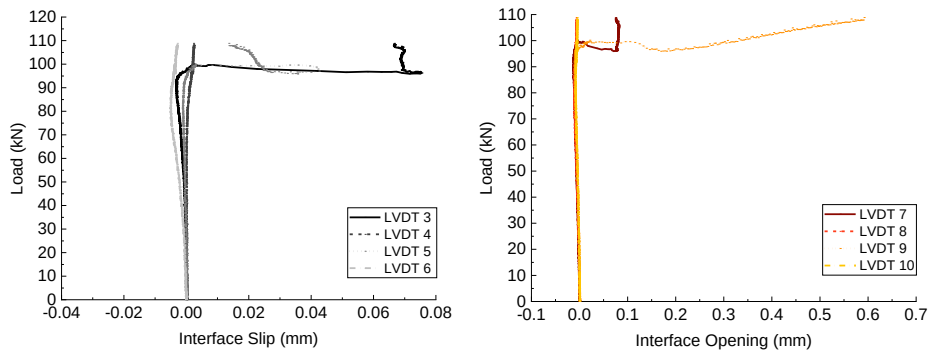
(ii)



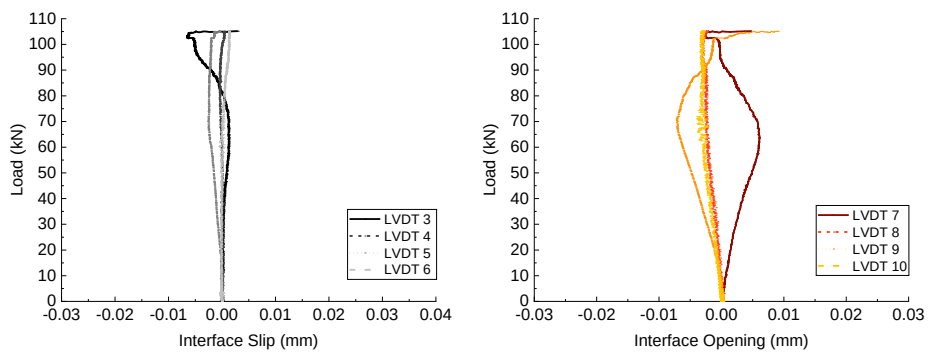
4.3 Structural test results



(ii)



(iii)



(iv)

(b)

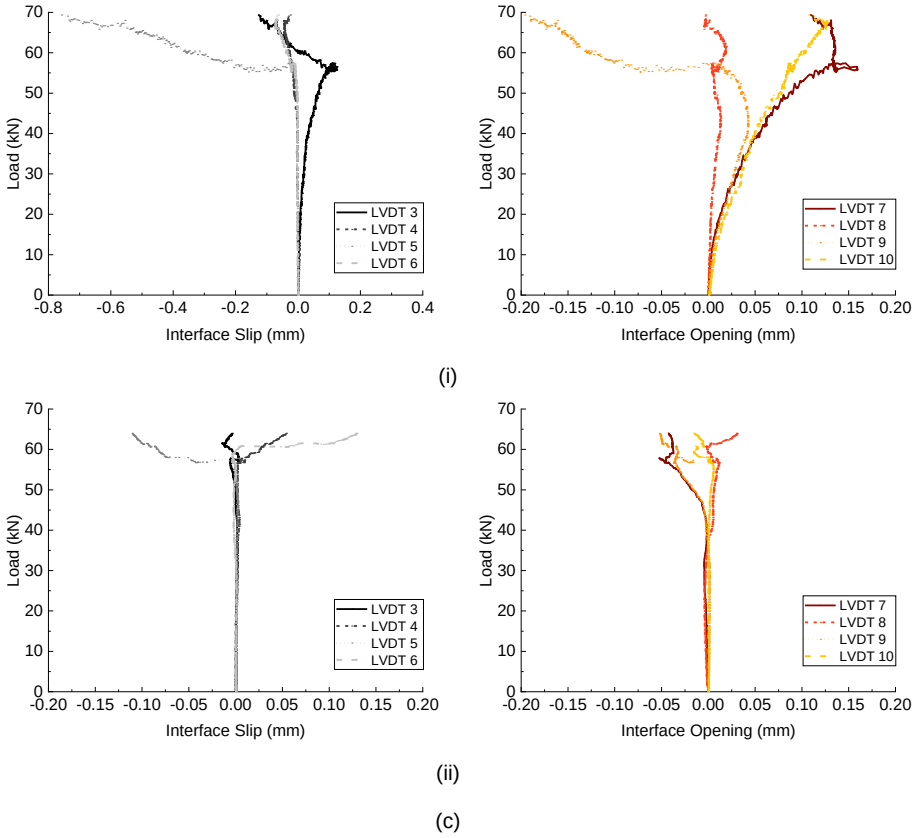


Figure 4.28 Interface displacement (slip and opening) measurement for strengthened beam: (a) CIS group; (b) EB group; (c) MA group.

4.3.4 Influence of casting direction

Shear resistance of the reference and strengthened beams

The fiber distribution of the UHPFRC possibly influences its strengthening efficiency. Two specimens with precast UHPFRC plates that are cast vertically (ST-M2-EB) and horizontally (ST-M2-EB-HC) are tested. Figure 4.29 shows the load- (mid-span) deflection responses of the tested beams and Table 4.7 summarizes the main test results. A substantial difference is found in ultimate strength and deformation at the peak load for hybrid beams with UHPFRC cast in different directions. Compared to the ST-M2-EB specimen, the ST-M2-EB-HC beam shows a 48% increase in ultimate capacity, around 111% higher deflection at peak load and 232% improvement in energy absorption. Therefore, the contribution of UHPFRC to the improvement of shear resistance is significantly influenced by the casting direction of UHPFRC, or more specifically, by the fiber distribution. Figure 4.30 shows the fiber distribution at main shear cracks of ST-M2-EB and ST-M2-EB-HC specimens. Unlike the non-uniform fiber distribution shown in Figure

4.3 Structural test results

4.30 (a) for the ST-M2-EB specimen, the fibers are more evenly dispersed if the UHPFRC is horizontally cast (Figure 4.30 (b)). ST-M2-EB beam exhibits a poorer shear performance in terms of loading, deformational, and energy absorption ability, likely due to difference in fiber distribution which influences crack bridging ability [214,215]. No significant difference in terms of initial stiffness is observed between the specimens. However, as the dominant shear crack propagates to the top layer of the specimen, due to lack of fiber bridging effect, ST-M2-EB exhibits considerably lower peak load and deflection compared to the ST-M2-EB-HC specimen.

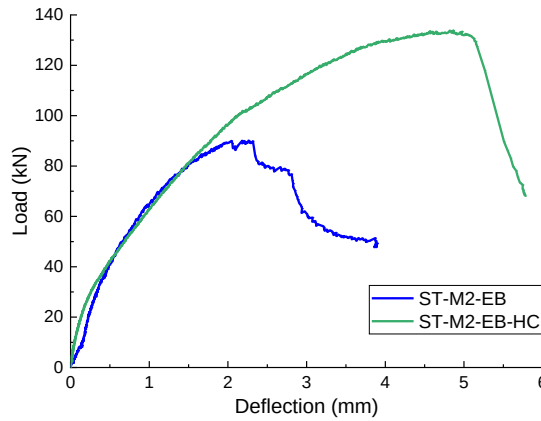


Figure 4.29 Load- (mid-span) deflection response of the epoxy-bonded specimens with different casting directions.

Table 4.7 Results in terms of load, deflection, and failure mode.

Specimen ID	F_u (kN)	ΔF_u (%)	δ_u (mm)	$\Delta \delta_u$ (%)	E_u (kN.mm)	ΔE_u (%)	Failure pattern
ST-M2-EB	90.1	-	2.3	-	140.3	-	Shear failure
ST-M2-EB-HC	133.1	48	4.8	111	465.5	232	Shear failure

Note: F_u is the peak load; δ_u is the mid-span deflection at the peak load; E_u is the energy absorption; ΔF_u , $\Delta \delta_u$ and ΔE_u are the percentages change of peak load, deflection, and energy absorption of ST-M2-EB-HC specimen with horizontal casting method to that of corresponding specimen ST-M2-EB with vertical casting method, respectively.

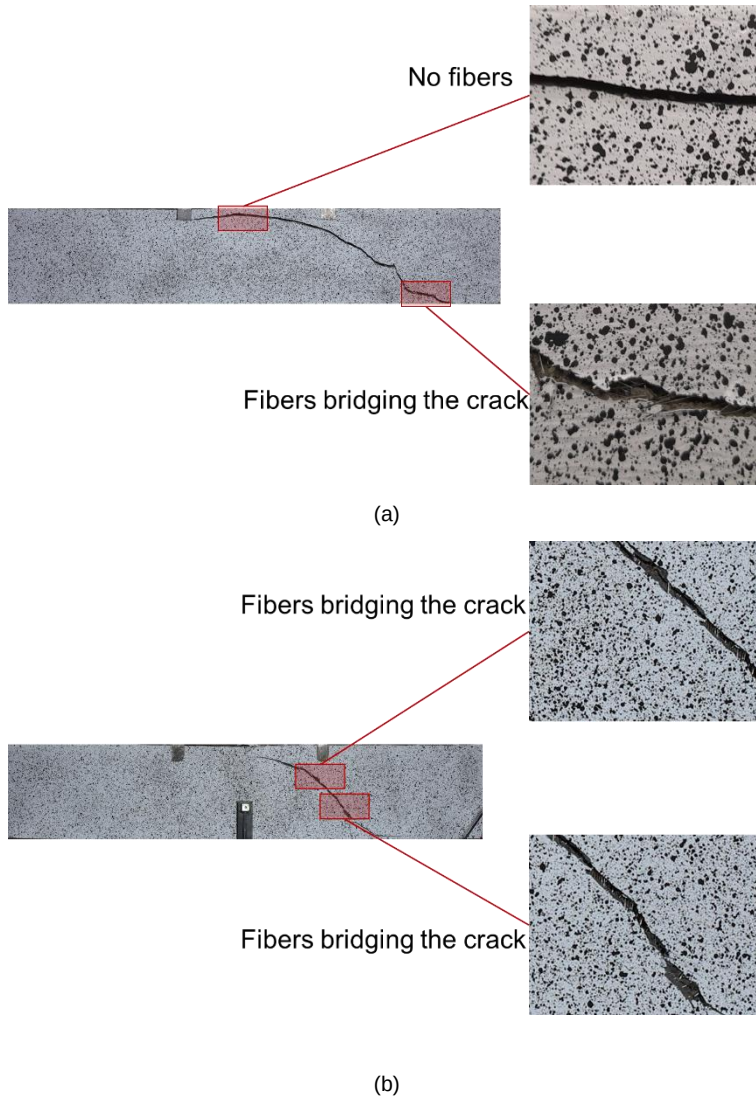


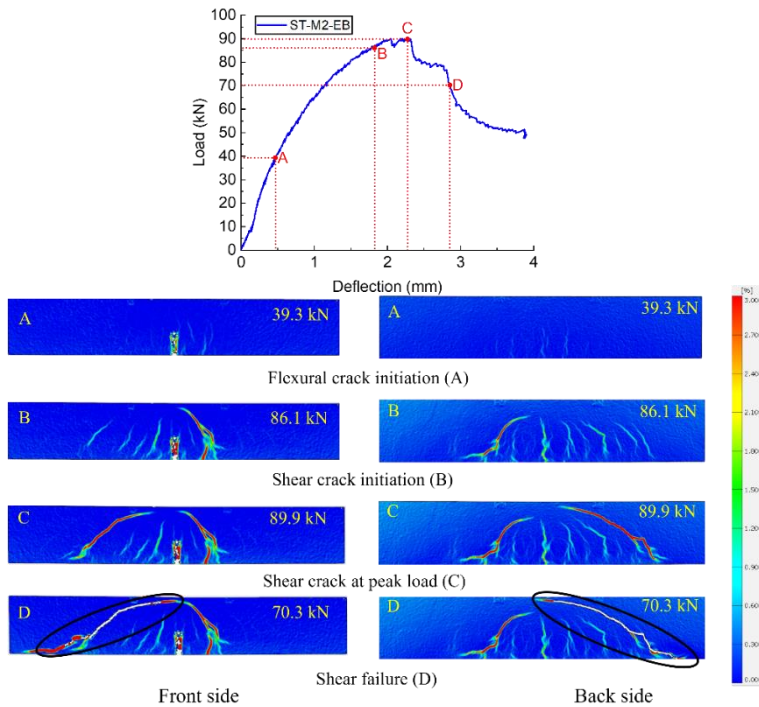
Figure 4.30 Fiber distribution at the main shear crack along the depth of UHPFRC laminates: (a) ST-M2-EB specimen; (b) ST-M2-EB-HC specimen.

Cracking behaviors and failure modes

The damage process of the tested specimens is shown in Figure 4.31 and triple-stage fracture development (that is, the initial linear elastic stage – flexural cracking process followed by shear cracking process - shear failure) is exhibited by both specimens. Despite comparable fracture processes, the damage pattern differs. In ST-M2-EB specimen, a dominant web-shear crack initiates and grows towards the loading and support point, finally leading to the failure of beams. ST-M2-EB-HC is characterized by typical flexural

4.3 Structural test results

shear failure. Furthermore, the loading and deformational ability at distinctive characteristic loads and corresponding deflections (presented in Table 4.8) for the two tested specimens are different. In terms of flexural cracking resistance, compared to ST-M2-EB-HC, the flexural crack initiates at a slightly higher load level and the corresponding deformation (10% and 31% higher, respectively). This is due to the accumulation of steel fibers at the bottom (i.e., tension) side of the vertically cast UHPFRC laminate, the tensile strength of the bottom UHPFRC is improved and delays the onset of the flexural crack. However, the shear cracking and peak load resistance are significantly higher in ST-M2-EB-HC. The shear cracking load and peak load are improved by 20% and 48%, respectively, and their corresponding deflections are increased by 27% and 114%, respectively. Non-uniform fiber distribution of UHPFRC causes the initiation of shear crack and the final shear failure to occur at a lower load level because shear crack occurs earlier and propagates in the weak zone of UHPFRC (i.e., fiber-deficient regions). Therefore, a uniform fiber distribution seems to be more beneficial to improving shear performance.



(i) ST-M2-EB specimen

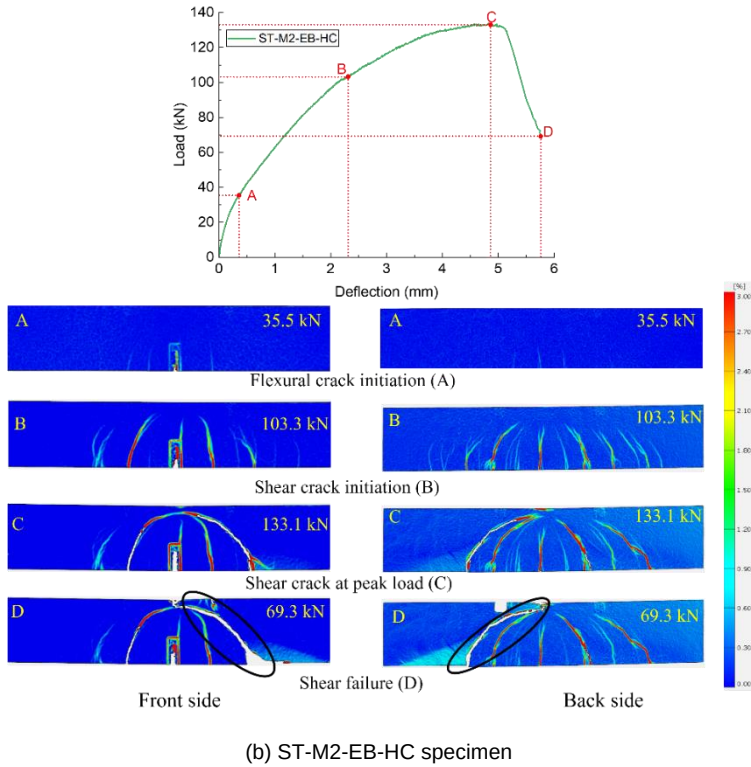


Figure 4.31 Damage process of the tested specimens.

Table 4.8 Distinctive characteristic loads and corresponding deflections for all tested specimens.

Specimen ID	$F_{cr,f}$ (kN)	$\delta_{cr,f}$ (mm)	$F_{cr,s}$ (kN)	$\delta_{cr,s}$ (mm)	F_u (kN)	δ_u (mm)
ST-M2-EB	39.3	0.5	86.1	1.8	89.9	2.3
ST-M2-EB-HC	35.5	0.4	103.3	2.3	133.1	4.9

The failure mode of each specimen is shown in Figure 4.32. Beams eventually fail in shear and the failure patterns on both sides are similar because of sufficient bond strength provided by the epoxy resin. However, the inclination of the shear crack (defined as the angle between the longitudinal axis of the beams and the inclined compressive strut) is different and a larger shear crack angle is seen in the ST-M2-EB-HC specimen. From the top view of the specimens, although there is a delaminated zone for both specimens, the ST-M2-EB specimen exhibits delamination at the interface after its failure. To determine whether the delamination occurs prior to peak load or after reaching it, the interface displacement analysis is performed.

4.3 Structural test results

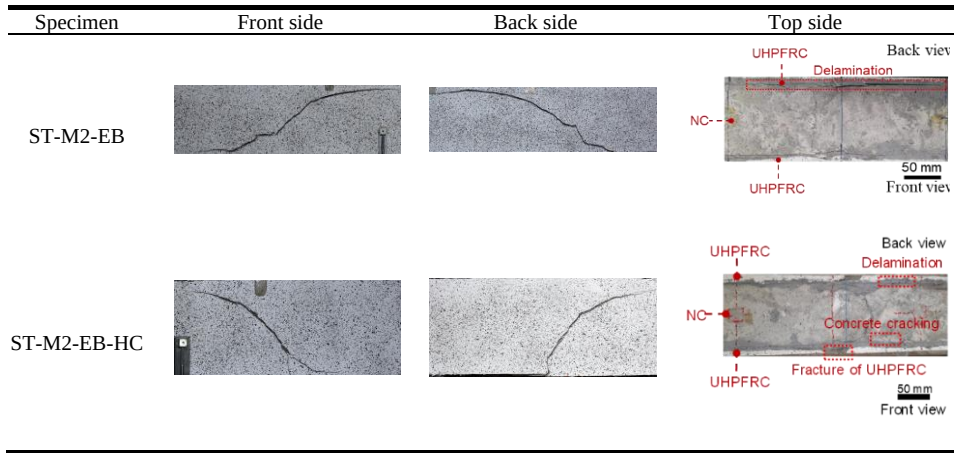


Figure 4.32 Failure modes on both sides for the tested specimens.

Cracking kinematics of tested beams

Figure 4.33 presents the deformation - maximum shear crack displacement (opening and slip) for ST-M2-EB and ST-M2-EB-HC specimens. Upon the initiation of the dominant shear crack, both specimens show a gradual increase in crack width, and crack displacement rapidly increases when the peak load is nearly reached. The crack opening and slip of ST-M2-EB-HC specimen are significantly larger at the peak load compared to those at ST-M2-EB. The uniform fiber distribution in the ST-M2-EB-HC specimen enables a higher peak load. As a result, the maximum shear crack displacement until the peak load can exceed 1.5 mm, which provides enough warning before the final failure.

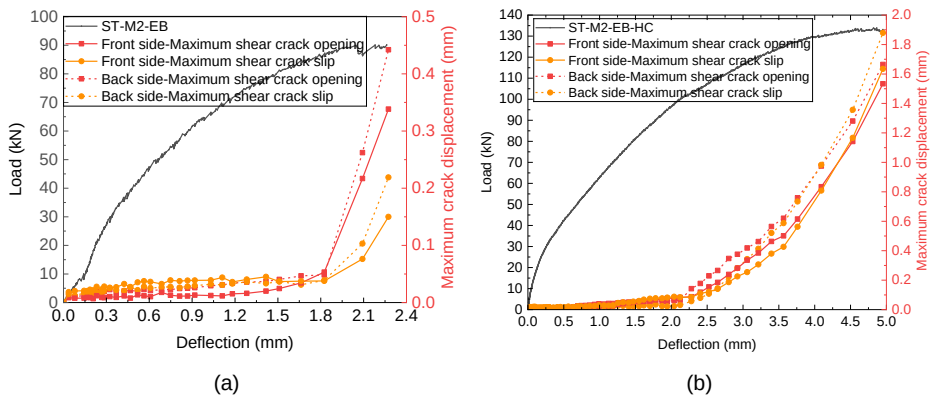


Figure 4.33 Deformation - maximum crack displacement (opening and slip) of all tested specimens: (a) ST-M2-EB; (b) ST-M2-EB-HC.

Figure 4.34 shows a comparison of maximum shear crack opening and slip for both beams. Compared to the horizontally cast laminates, beams strengthened with vertically cast exhibit the major shear crack growing faster at the same load level. Although the loads for

the initiation of the shear crack are similar, at the peak load, ST-M2-EB-HC exhibits four and eight times larger crack opening and sliding, respectively, compared to ST-M2-EB specimen. For ST-M2-EB-HC specimen, when the shear crack occurs, there is still enough remaining bearing capacity (around 30 kN) before reaching the peak load, demonstrating a more ductile response. Therefore, it is concluded that the uniform fiber distribution in the UHPFRC not only increases the load and deformational capacity, but also provides formation of larger shear crack, providing more warning before the final shear failure.

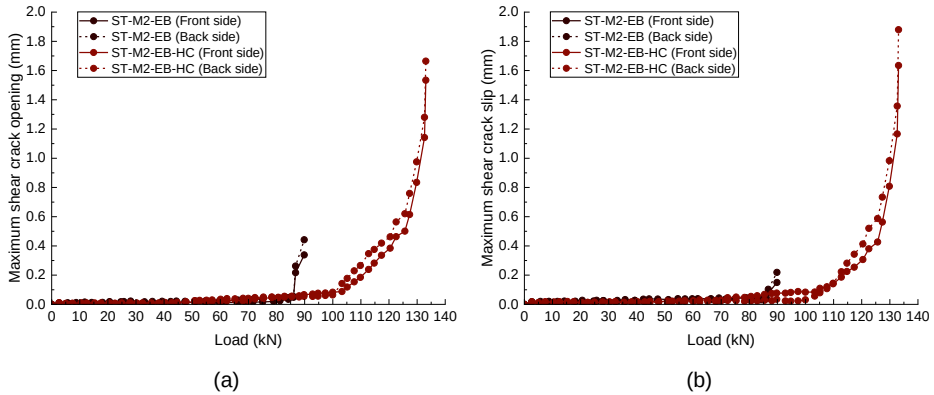


Figure 4.34 Comparison of the maximum shear crack displacement (opening and slip) of all tested specimens: (a) shear crack opening comparison; (b) shear crack slip comparison.

Interface behavior

For epoxy-bonded beams with different casting directions, the results in terms of the load versus interface displacement (slip and opening) relationship are presented in Figure 4.35. As stated above, the use of epoxy resin is effective to provide sufficient bond and keep the composite beam intact until the failure of the beams, which is also confirmed in Figure 4.35. The ST-M2-EB-HC specimen can sustain more than 130 kN, and under such high load level, the maximum slip and opening do not exceed 0.02 and 0.025 mm, respectively. Moreover, the debonding phenomenon for ST-M2-EB (shown in Figure 4.24) occurs after the peak load, which only influences the post-peak structural behavior.

4.4 Discussion

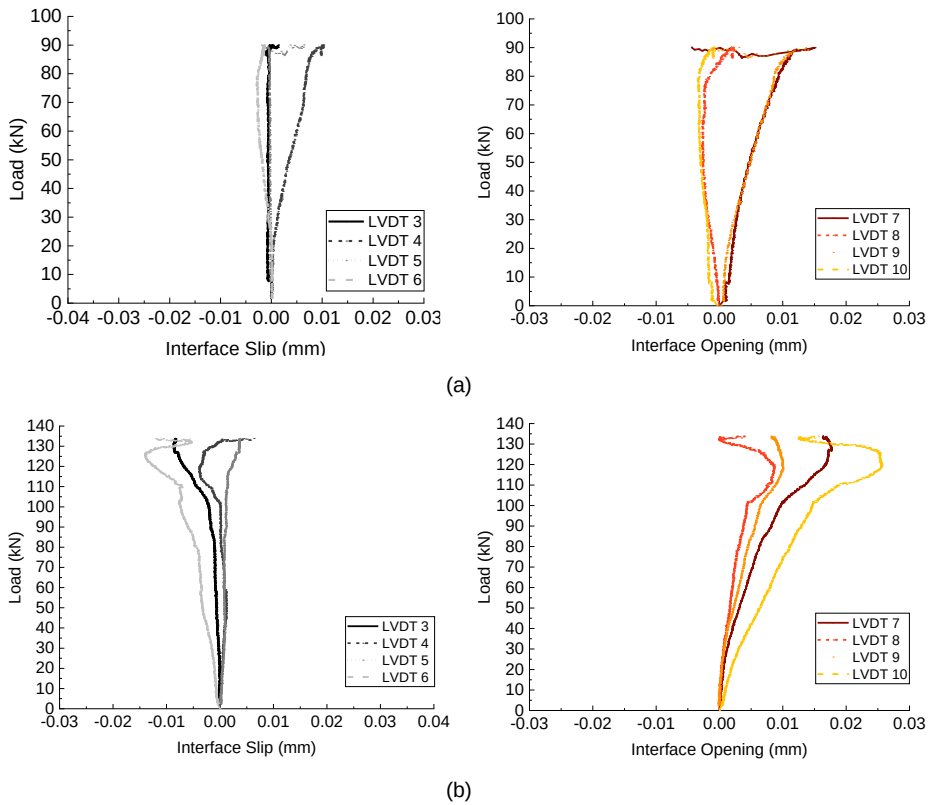


Figure 4.35 Interface displacement (slip and opening) measurement for strengthened beam: (a) ST-M2-EB specimen; (b) ST-M2-EB-HC specimen.

4.4 Discussion

UHPFRC shows great potential to increase the shear capacity of RB beams without transverse reinforcement if good material properties of UHPFRC and the bond quality between UHPFRC and concrete are provided. However, many parameters influence interface and materials performance, and performance of the composite beam including restrained shrinkage, moisture state of concrete substrate, different bonding techniques, freeze-thaw action and casting direction of UHPFRC. To better explain the structural behavior shown in Section 4.3, an in-depth analysis is given in the following section.

4.4.1 Influence of moisture degree of the concrete substrate

Exposure to drying condition for more than 90 days and subsequently under applied external load, delamination occurs in the ST-M1-CIS-SSD specimen while in-situ cast specimen dealt with ASD condition keeps intact. This difference in interface performance is in line with findings of Bentz, et al. [43], revealing a better shear bond strength with dry

substrate due to a more solid surface interlock at the interface between the UHPFRC and the NC.

On the other hand, as described in Section 4.3.2, moisture transport at the interface under ASD conditions reduces the hydration of the strengthening material, therefore lowering the stiffness and tensile properties of the UHPFRC. Consequently, the initial stiffness is lower, and flexural cracks appear earlier in ASD specimen compared to epoxy-bonded and SSD specimens with limited moisture transfer.

To compare the hydration degree of the three specimens, a non-evaporable water test is conducted. The hydration degree of cement paste is proportional to the content of non-evaporable water [44].

After the beam test, the UHPFRC from the three strengthened specimens is split and ground into several pieces. Subsequently, UHPFRC is dried in an oven at 40 °C for 4 h to achieve a constant mass and then further dried at an increased temperature of 105 °C for 24 h to remove evaporable water. The weight of the samples is measured and recorded as W_{105} . Finally, the dried samples are oven-heated at 1050 °C for 6 h to remove the non-evaporable water, and the weight is also measured and recorded as W_{1050} .

It should be noted that UHPFRC instead of only the cement paste is used and only the weight of cement paste is used in the calculation. Therefore, the weight of cement needs to be calculated. Assuming that the total weight has negligible change before and after hydration and based on the weight ratio of each component, W_{cem} can be calculated. Moreover, if the cement loss on ignition is not taken into account, the hydration of 1 g anhydrous cement can produce 0.23 g of non-evaporable water. Therefore, the degree of hydration can be calculated by the following equation:

$$\alpha = \frac{W_{105} - W_{1050}}{0.23 \times W_{cem}} \quad (4.5)$$

Based on the calculation, a comparison of the results of hydration degree and the initial stiffness for three specimens is shown in Figure 4.36. The initial stiffness is calculated as F_i / δ_i , and in current study F_i and δ_i are load level of 10 kN and its corresponding deflection, respectively, at which the specimens are uncracked. It is found that the calculated hydration degree of UHPFRC for all specimens is lower than 35%, which is due to relatively low water-binder ratio and is in line with measured results of degree of hydration for UHPFRC in literature [216]. In Figure 4.36, it is observed that the initial stiffness of the specimens is correlated with the hydration degree of UHPFRC. Higher hydration degree of UHPFRC leads to a higher initial stiffness of the strengthened beam. SSD specimen has a higher degree of hydration than epoxy-bonded specimen possibly due to the additional water supplied by the saturated substrate for further hydration [217].

4.4 Discussion

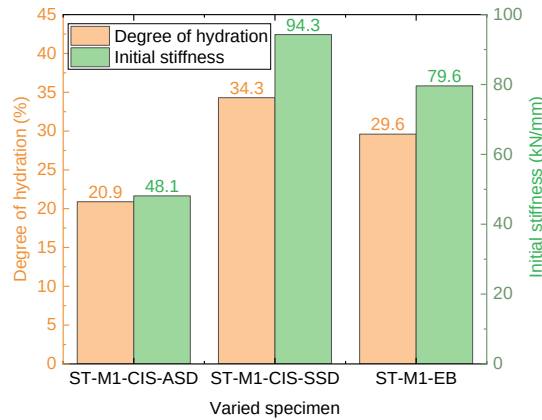


Figure 4.36 Comparison of the hydration degree and the initial stiffness for three specimens.

4.4.2 Fiber distribution from microscopic observations

In the current study, two UHPFRC mixtures with slightly different diameter of steel fibers are used. Figure 4.38 shows that the ST-M1-EB specimen has much higher loading and deformational capacities compared to the ST-M2-EB specimen although the UHPFRC laminates from both specimens are cast vertically. It is assumed that the difference in shear resistance might be caused by varied fiber distribution along the height of UHPFRC rather than the material properties of UHPFRC for both mixtures. ST-M2-EB-HC specimen has the largest shear force and deflection due to its most uniform fiber distribution among the three specimens.

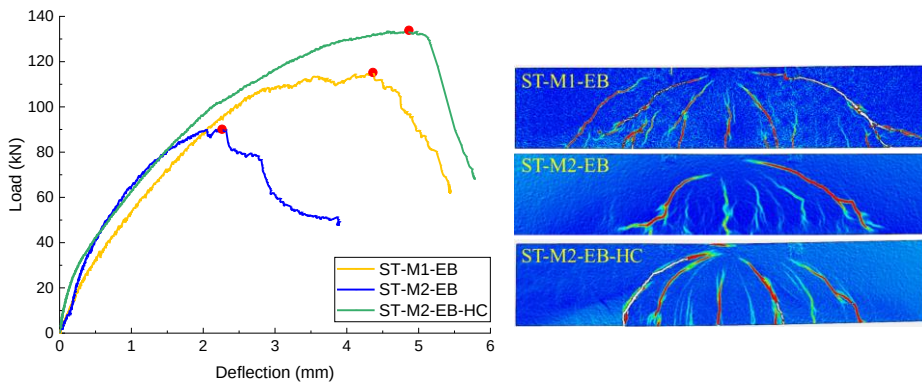


Figure 4.37 Load-deflection response together with crack pattern at the peak load for ST-M1-EB, ST-M2-EB and ST-M2-EB-HC specimens.

To determine the fiber distribution for ST-M1-EB and ST-M2-EB specimens, both beams are cut from the mid-span and thin cross-sectional sample is extracted from the beam. Then the thin sample is polished to create smooth reflecting surfaces. Finally, the image of polished surface is obtained through digital microscope and the fiber distribution can be inspected.

The fiber distribution for ST-M1-EB and ST-M2-EB specimens are shown in Figure 4.38. It is shown that the steel fibers in ST-M1-EB specimen disperse more uniformly than those in ST-M2-EB specimen, in which most of the fibers accumulate at the bottom of the UHPFRC plate.

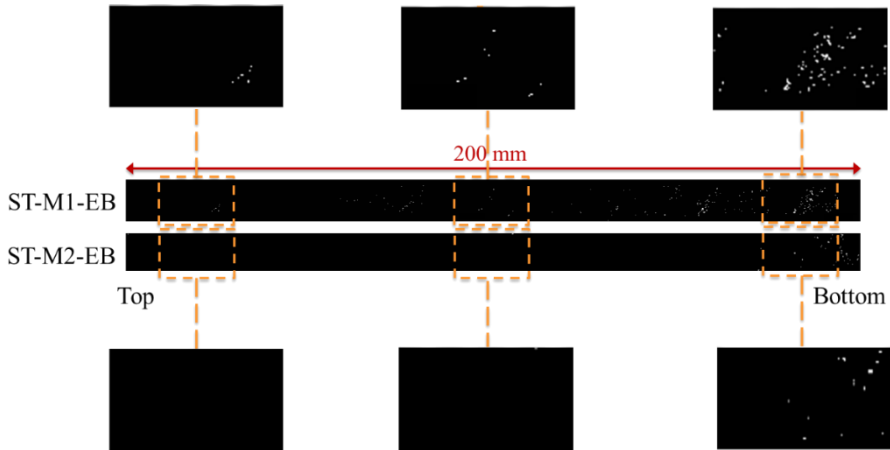


Figure 4.38 Fiber distribution for both specimens from image analysis, noting that the white dots represent the presence of steel fibers.

4.4.3 Influence of freeze-thaw cycles

From the results, it is found that the non-uniform fiber distribution of UHPFRC rather than freeze-thaw action plays a role in shear behavior of strengthened beams. The material properties of UHPFRC and concrete, and bond at the interface do not show obvious damage due to freeze-thaw. Therefore, the number of FT cycles might be insufficient to cause structural degradation. It is difficult to predict the number of FT cycles that would lead to damage of the strengthened beams due to the following reasons. First, although many codes or standards such as ASTM C666/C666M [204], GB/T 50082-2009 [205] or CEN/TR 15177 [206] define test protocols to assess the freeze-thaw resistance of concrete material, the mechanisms and rate of damage might differ in RC structures where steel reinforcement and interfaces are also present. Currently, there is no universal testing method for the structural performance of RC members. Furthermore, due to the differences in specified exposure conditions, the number of FT cycles varies in different standards. For example, 300 cycles in ASTM C666/C666M, minimum 25 cycles in GB/T 50082-2009, and 56 cycles in CEN/TR 15177. Finally, compared to conventional concrete, UHPFRC can sustain more FT cycles [218] due to its denser microstructure. The UHPFRC-concrete bond has also been reported to be stronger than the concrete-concrete bond due to the existence of fibers [156], leading to higher resistance to FT action [126].

4.5 Conclusion and implications

Therefore, more FT cycles and/or harsher conditions such as submersion in water environment or frost salt scaling under FT cycles should be applied in further studies to evaluate the robustness of the UHPFRC strengthening system.

Note that applied LVDT measurements are limited to only several points at the interface. The interaction between concrete and UHPFRC along the entire interface area and possible (local) debonding could therefore not be captured. Therefore, the feasibility of infrared thermography for the quantitative study of interface behavior along the whole interface length will be investigated in Chapter 5.

4.5 Conclusion and implications

In this chapter, the shear performance of reinforced concrete beams strengthened with UHPFRC is analyzed. Many parameters, including the restrained shrinkage in composite elements, the moisture degree of concrete substrate, bonding techniques combined with freeze-thaw cycles, and casting direction of UHPFRC, are discussed. The material properties of UHPFRC and normal concrete, and structural behavior in terms of cracking/debonding risk under restrained shrinkage, load-deflection response under applied load, cracking behavior, cracking kinematics, and interface properties are evaluated. Based on the material and structural test results, the following conclusions can be drawn:

(1) UHPFRC shows significant autogenous shrinkage at early age (within the 24 h), and under sealed and fully restrained conditions, the stress induced by autogenous shrinkage can reach around 50% of the 28 days tensile strength of UHPFRC. Therefore, the (autogenous) shrinkage and possible early-age cracking or delamination risk for concrete structures strengthened with in-situ casting UHPFRC cannot be ignored.

(2) After exposure to 120 freeze-thaw cycles, based on surface profile observation, mass change, and compressive and elastic modulus test results, neither UHPFRC nor NC show damage. However, it seems that the tensile properties, especially the strain softening capacity of UHPFRC are weakened after more than 75 cycles, possibly due to reduced bond strength at the matrix-fiber interface.

(3) Compared to reference beams, with UHPFRC laminates applied on lateral sides of the shear-deficient reinforced concrete beam, significant increases in load, deflection, energy absorption capacity and cracking resistance, are found. This proves that reinforced concrete structures strengthened with UHPFRC have significantly improved shear resistance. Additionally, due to the fiber bridging effect of UHPFRC, the maximum crack widths for all strengthened beams are much larger compared to reference beams, proving that the application of UHPFRC as a shear strengthening material could provide more warning signals before brittle shear failure.

(4) The strengthening efficiency of UHPFRC is influenced by moisture conditions of the concrete substrate. For cast in-situ application with two different concrete moisture

states (initially dry and saturated surface dry substate, ASD and SSD), it is found that localized delamination occurs with SSD condition due to the restrained shrinkage at the interface. A better interface performance under ASD condition might be caused by moisture exchange causing interlock at the interface between the UHPFRC and NC. On the other hand, ASD conditions, due to moisture exchange, reduce the hydration degree of UHPFRC. For strengthening with ASD condition, due to moisture loss and the subsequently lower degree of hydration of UHPFRC, the stiffness and the tensile properties of UHPFRC are reduced, reducing the flexural and shear cracking resistances of the strengthened beam.

(5) The improvement in shear resistance is affected by the applied bonding methods, and a larger load increase is obtained with better bond. Epoxy bonding can provide a strong bond, with negligible interface slip or opening. However, for cast in-situ techniques with smooth interface, the bond strength can be insufficient without proper surface preparation, such as sandblasting or water jetting, leading to larger interface displacements and potential delamination. Mechanical anchorage methods, without adhesion at the interface and therefore lack of monolithic composite action, result in the highest interface displacement, asymmetric shear crack patterns and lowest shear resistance.

(6) For beams subjected to the coupled load and freeze-thaw action, it seems that the freeze-thaw action has no effect on shear performance of both reference and strengthened beams, being in line with material test results (i.e., surface inspection, compressive strength, and elastic modulus measurements). Instead of freeze-thaw action, it seems that the fiber distribution in UHPFRC plays a more crucial role for shear strengthening efficiency, crack development and failure pattern of strengthened concrete beams with UHPFRC.

(7) By comparing the specimens strengthened by UHPFRC with different casting directions, horizontal casting can provide a more uniform fiber distribution of the UHPFRC laminate, allowing for a much higher shear strengthening efficiency. An increase of 48%, 111% and 232% for load, deflection, and energy absorption capacity, respectively, is found when using horizontally cast over vertically cast laminates. Additionally, the fiber distribution due to different casting direction also affects the damage process and failure patterns. In vertically cast specimens, the dominant shear crack tends to initiate earlier in fiber-deficient areas, reducing shear cracking load. However, flexural cracking load is increased as the accumulation of steel fibers at the bottom enhances the tensile strength of UHPFRC.

4.5 Conclusion and implications

Chapter 5 Delamination detection at the interface in hybrid beams using active infrared thermography

5.1 Introduction

As discussed in Chapter 4, interface debonding/sliding can only be examined at several points along the interface by using LVDTs. To examine the interface performance along the entire bonding area, Chapter 5 focuses on the applicability of active infrared thermography (IRT) to detect the delamination in hybrid UHPFRC-NC beams under controlled lab environment through lab-scale experiment and numerical modeling.

Section 5.2 describes the fabrication of small-scaled UHPFRC-NC specimens, arrangement of test set-up, and the testing process. Optimal testing parameters (i.e., heating time, cooling time, distance between the heat source and the object) are determined on model experiments in order to develop a standard testing method. For this, a numerical model is established (Section 5.3) and the heating time is determined through FEM analysis (Section 5.4). Results are compared to experimentally measured key indicators for delamination (i.e., absolute temperature difference and signal to noise ratio (SNR)). In Section 5.5, the SNR ratio calculation method is embedded into the developed analytical model. The model is applied to determine the size of the delaminated area. In Section 5.6, the developed approach is applied to test two reinforced concrete-UHPFRC members and validated through CT-scanning. Thereafter, the interface properties for hybrid beams subjected to FT cycles are examined by IRT and brief conclusions are given in Section 5.7.

5.2 Experimental methods

The principle of active IRT to detect the delamination is explained in Figure 5.1 which shows a composite sample with a delamination at the interface. When the specimen is exposed to external heat waves, its surface temperature can be measured by infrared thermography (IRT). The presence of delamination affects how heat propagates through the material, allowing for its detection.

To understand how heat moves through the material and influences the detection process, it is essential to consider the three primary physical mechanisms of heat transfer: conduction, convection, and radiation [219]. *Conduction* refers to the transfer of thermal energy through molecular interactions, where heat moves from regions of higher to lower temperature, and is characterized by the thermal conductivity of material. *Convection* involves the transport of heat via fluid (liquid or gase) motion; however, in cases such as narrow delamination (with crack opening less than 5 mm), it is negligible due to the lack of fluid flow. *Radiation*, which involves the emission and transfer of energy through electromagnetic waves, depends on the emissivity of materials. In dry air conditions, its effect is also minimal. Thus, conduction is the predominant mechanism relevant to delamination detection.

Since the thermal conductivity of still air is around 0.025 W/mK, and that of concrete is around 1.8 W/mK, heat moves faster through concrete than through the air. As a result,

when a layer of UHPFRC is heated, delamination provides higher resistance to heat flow, i.e., more heat will accumulate above a delaminated part than above a well-bonded part ($T_2 > T_1$ shown in Figure 5.1). In this chapter it is investigated whether infrared thermography (IRT) could be used to detect delamination between a layer of UHPFRC and normal concrete.

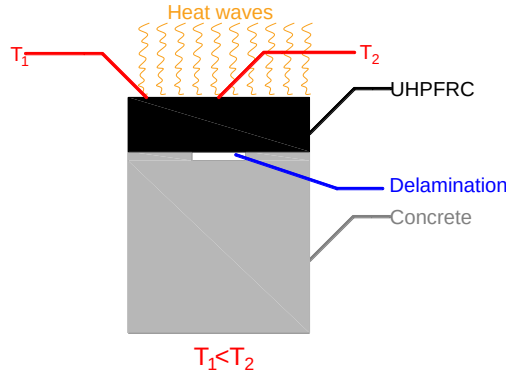


Figure 5.1 Principle of active infrared thermography.

5.2.1 Test specimen preparation

Three types of tests, with increasing level of complexity, are designed. The composite UHPFRC-NC specimens with artificially made delamination are fabricated in the first and second stages “model” experiments. The manufacturing process consists of bonding a 10 mm thick UHPFRC lamella by epoxy to the $100 \times 100 \times 100 \text{ mm}^3$ concrete cubes (Figure 5.2). To imitate delamination, a PIR insulation which has a similar thermal conductivity to air, is used. The PIR layers with a size of $30 \text{ mm} \times 30 \text{ mm} \times 1.3 \text{ mm}$, are placed in the center of the concrete specimen and attached to the concrete block by Vaseline. First stage “model” test (C1, Figure 5.3a) is made with only one delaminated area whereas more delaminated areas are in the second stage tests (S1-S5, Figure 5.3b-f).

As shown in Table 5.1 and Figure 5.3, C1 specimen is made to develop the standard testing process. The delamination locations are varied in test samples S1 and S2. The aim is to investigate if delamination can be detected irrespectively of its location, given that the halogen lamp is used as a heat source which might result in non-uniform heat flux. In S3, the delamination thickness is varied to examine the detectability for the minimum delamination thickness (interface opening). Furthermore, S4 is designed to have varied dimension of the delamination. Finally, S5 is set with random delamination size, thickness and closely spaced location to further examine the effectiveness of the developed testing protocol and post-processing method in “model” samples.

5.2 Experimental methods

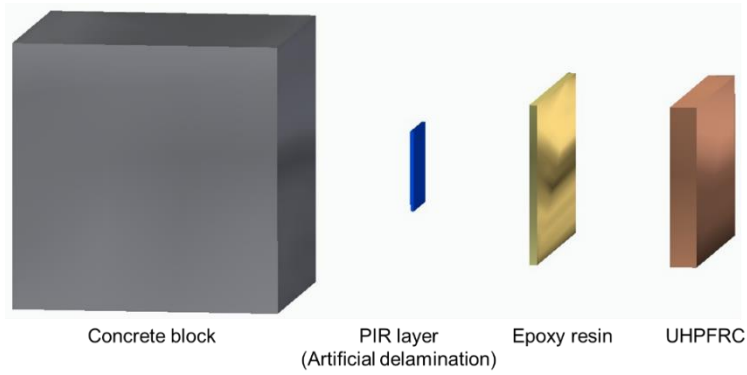


Figure 5.2 Specimen fabrication.

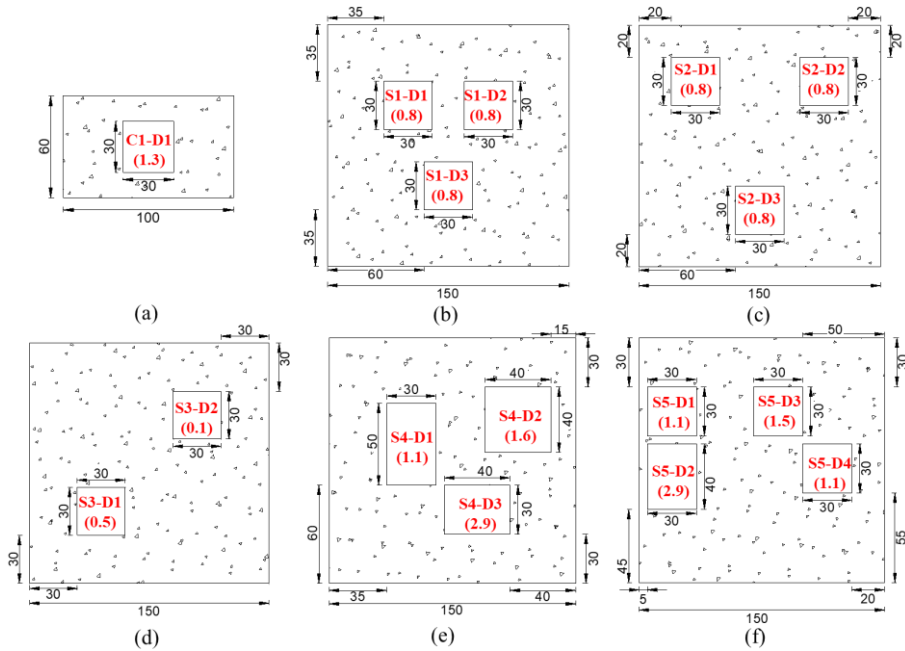


Figure 5.3 Location and size of artificial delamination for different samples: (a) C1; (b) S1; (c) S2; (d) S3; (e) S4; (f) S5 (unit: mm).

Table 5.1 Details of all tested specimens.

Type of test	Sample	Delamination zone	Size	Delamination thickness (mm)
First stage “model” test	C1	C1-D1	30×30	1.3
		S1-D1	30×30	0.8
	S1	S1-D2	30×30	0.8
		S1-D3	30×30	0.8
Second stage “model” test	S2	S2-D1	30×30	0.8
		S2-D2	30×30	0.8
		S2-D3	30×30	0.8
	S3	S3-D1	30×30	0.5
		S3-D2	30×30	0.1
		S3-D3	30×30	0.1

S4	S4-D1	30×50	1.1
	S4-D2	40×40	1.6
	S4-D3	40×30	2.9
S5	S5-D1	30×30	1.1
	S5-D2	30×40	2.9
	S5-D3	30×30	1.5
	S5-D4	30×40	1.1

5.2.2 Test setup and testing process

Test set-up

Figure 5.4 shows the experimental set-up of active IRT with the use of halogen lamp for the detection of the interfacial delamination on UHPFRC retrofitted concrete substrates. The set-up includes heating, sensing and control units. A 1000W halogen lamp, which can generate 22000 lumens, is placed and focused on the central surface of the test sample as a heat source. The infrared camera (FLIR A320 camera) is positioned perpendicular to the plane of the object at a distance which ensures that the entire specimen is within its field of view. The infrared camera has a temperature measurement range from -20 °C to 120 °C, focal length of 18mm, an image resolution of 320 × 240 pixels and frame rate of 5 fps.

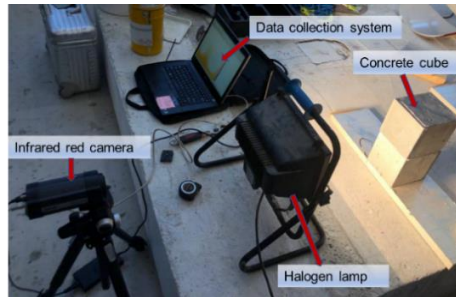


Figure 5.4 Experimental setup.

Testing process

The entire testing process mainly includes two phases: (a) sample heating and (b) the surface temperature recording during the cooling process after removal of the halogen lamp. In order to obtain the highest thermal contrast to detect delaminated area, several parameters need to be taken into account: the distance between the heat source and the test samples, heating time and cooling time.

The heating distance between the lamp and target need to be calibrated to ensure an even heat distribution at the surface of the composite UHPFRC-NC sample. The heat source should not be too close nor too far away from the object, as short distance might induce inhomogeneous heat distribution and long distance might require a prolonged duration of heating time. Multiple trial tests are conducted to determine the distance from the object

5.3 FEM simulation

to the halogen lamp. As shown in Figure 5.4, a concrete block is heated from different distances, and the standard deviation of the surface temperature is calculated. The distance between the test sample and heat source is adjusted until the standard deviation of the surface temperature is minimized to achieve a homogeneous heat distribution on the surface of the concrete block. Finally, the distance is set as 40 cm and a constant heat flux of 1750 W/m^2 is generated at the surface of the test specimens under such condition.

The heating time is important as sufficient heat flux is required to achieve the highest thermal contrast between sound and defect area. The optimal heating time can be determined when the highest relative thermal contrast between sound and defected area, also called running thermal contrast (RTC) [220], is achieved:

$$RTC = \frac{T_d - T_s}{T_s} \quad (5.1)$$

where T_d and T_s are the temperature above a defected area and sound area respectively. It is important to note that in this research, a constant heat flux rather than a constant temperature is used as a heat source. In this study, the heating time is determined through the COMSOL simulation in Section 5.4.

Finally, the cooling time is also crucial as it defines the time when the delaminated areas are best detectable. The signal-noise ratio (SNR) [169] is used to determine the optimal cooling time and can be defined as:

$$SNR(t) = 20 \lg \frac{|T(t) - T_s(t)|}{\sigma_s(t)} \quad (5.2)$$

where t is the cooling time, starting from the removal of the heat source, $T(t)$ is the temperature distribution within the sample and $T_s(t)$ is the average temperature above the sound areas. $\sigma_s(t)$ denotes the standard deviation of the sound areas. The above defined equation for SNR calculation also takes into account the noise from surrounding environmental condition. When the target specimen starts to cool down after heating, the SNR of the delaminated area will increase initially until it reaches the peak value, and afterwards the SNR will decrease with continued cooling process. The time when the delaminated areas are best detectable is the time when the peak value of SNR(t) is achieved.

5.3 FEM simulation

The COMSOL Multiphysics 5.6 software is used for finite element analysis. Since it is not possible to capture the temperature profile in the test specimens during the heating process, the optimal heating time is determined through FEM.

5.3.1 Material properties

To model the heat transfer within the test specimen, material properties of concrete, UHP-FRC and air (or PIR properties) are defined. The properties of each component are listed in Table 5.2. Based on the results of Nagy et al. [221], the volumetric heat capacity ρ_c and the thermal conductivity λ of UHPFRC are determined. As for the emissivity ϵ of UHPFRC, the same value is adopted as for normal concrete, because the emissivity ϵ depends on the color of the material, which is nearly the same for both materials.

Table 5.2 Thermal properties of NC and UHPFRC.

Materials	Density [Kg/m ³]	Volumetric heat capacity ρ_c [J/m ³ K]	Thermal conductivity λ [W/mK]	Emissivity ϵ [-]
Concrete	2400	2.024×10^6	1.8	0.93
UHPFRC	2200	2.14×10^6	2.82	0.93
Air	1.2	1.2×10^3	0.032	-
PIR	32	3.2×10^4	0.022	-

5.3.2 Model validation

C1 specimen from the first stage “model” test (Table 5.1) is simulated using COMSOL Multiphysics 5.6 software. As shown in Figure 5.5 (a), the numerical model is composed of the concrete cube with sides of 10 cm and a 10 mm thick UHPFRC laminate with the debonding defects (30 mm $\times$ 30 mm $\times$ 1.3 mm). In order to simulate the same boundary condition as in the experiment, a constant heat flux (1750 W/m²) is applied on the surface of the C1 specimen (Figure 5.5 (b)).

To determine the optimal heating time, a simulation with a minimal delamination thickness of 0.05 mm is conducted. After the heat source is removed, the temperature data during the cooling process, above sound area and delaminated area, is investigated.

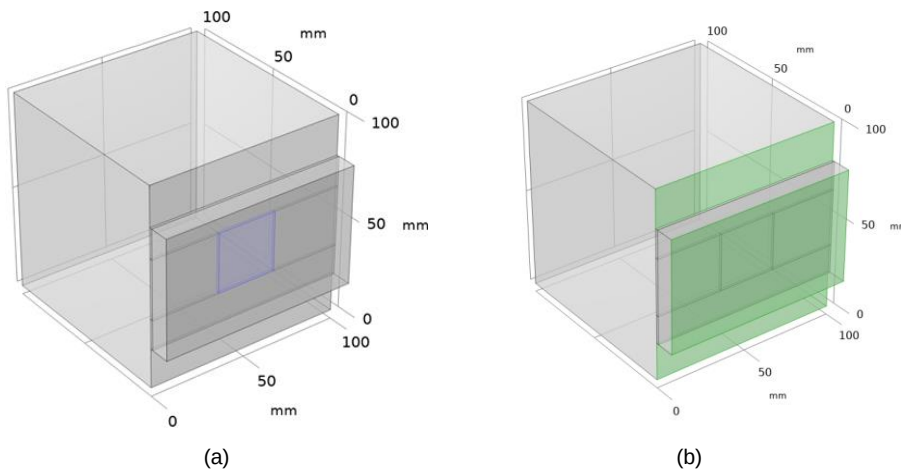


Figure 5.5 Model establishment: (a) C1 specimen embedded with delaminated area (blue zone) in COMSOL; (b) Simulation of the heat flux (area in green) in COMSOL.

5.4 Results and discussion

5.4.1 Heating time

With the aid of COMSOL simulation, the optimal heating time is determined based on the calculated results of thermal contrast and material properties of PIR layer and air which represent delamination. The thermal conductivity of the PIR insulation is 0.022 and the thermal conductivity of air is $0.025 + 5.5d$ where d is the thickness of the delamination in m [222]. To calculate the optimal heating time, a delamination thickness of 0.05 mm is used. As a constant heat flux is applied on the surface of UHPFRC, the surface temperature of the UHPFRC layer continuously increases while remaining uniform over the entire area.

The RTC results during the heating process are presented in Figure 5.6. From Figure 5.6, it is found that the thermal contrast increases initially and reaches its maximum around 1500 seconds (s) of heating time. The point where the secant slope of thermal contrast curve reaches 10 % of the maximal secant slope is defined as the optimal heating time. Although the maximal running thermal contrast for PIR and air layers are different, the optimal heating time is very close (1449 s for air and 1496 s for PIR layer). Therefore, heating time of 1500 s (25 minutes) is used during the testing phase.

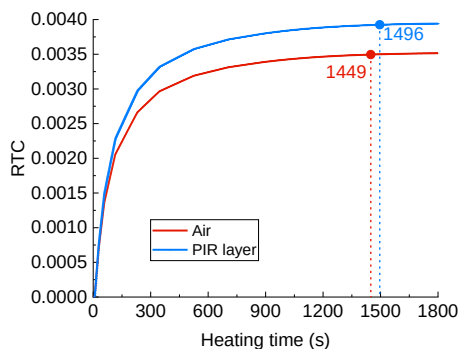


Figure 5.6 Thermal contrast versus heating time at the backside of the layer of UHPFRC due to a delamination of PIR and air, respectively.

5.4.2 Verification against experimental results

Based on the previous results, a uniform heat flux of 1750 W/m^2 is modelled as an external heat source for 25 min, and then the temperature variation is monitored during the subsequent 10 min cooling process. To validate the accuracy of the COMSOL simulation, the SNR and the absolute thermal contrast (ATC) are calculated and compared to experimental results. The validation process is schematized in Figure 5.7. It should be noted that for the numerically calculated SNR ratio, the experimentally obtained standard deviation of the temperature in the sound area is also used since the ideal condition in COMSOL provides uniform temperature in sound area.

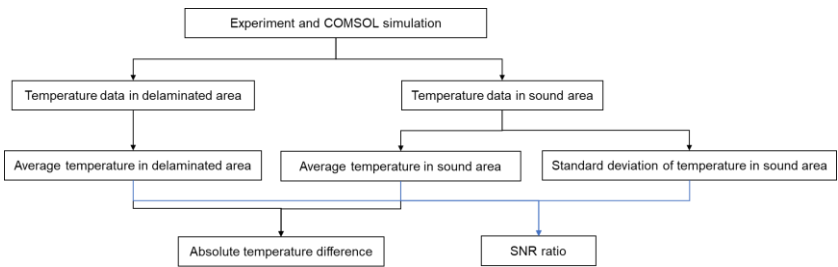


Figure 5.7 Model verification process.

In Figure 5.8, the experimental and numerical results of absolute surface temperature field for the C1 specimen are compared. In general, the experimental results are in good agreement with the simulation results. Compared to the real shape and location of the delamination in the C1 specimen (Figure 5.3), both the simulated and experimental results can reasonably distinguish the edges of the delamination. But it is found that, unlike in the experiment, in FEM there is no noise error.

In Figure 5.9 (a), the experimentally and numerically obtained absolute thermal contrast for C1 specimen is compared. Similarly, in Figure 5.9 (b) the experimental and numerical signal-to-noise ratio (SNR) is compared. As expected, the highest absolute thermal contrast occurs at the start of cooling phases, and then it starts to descend as the cooling time increases. The predicted curve follows the same shape of experimentally measured results though a slightly lower initial thermal contrast is observed. Unlike the temperature difference that consistently reduces with cooling time, SNR first increases (Figure 5.9 (b)). Within 100 s of cooling, the SNR value reaches its maximum, which is the best time for detecting a defect/delamination area. Afterwards, the SNR gradually decreases as the temperature difference between delaminated and sound areas further decreases. In general, the simulated and measured curves show similar trend although the predicted values are slightly underestimated, which might be due to assumed isotropic material properties. The numerical prediction agrees reasonably with the experimental results and 10 min temperature measurement during cooling period is sufficient get the best detectability of the delaminated area.

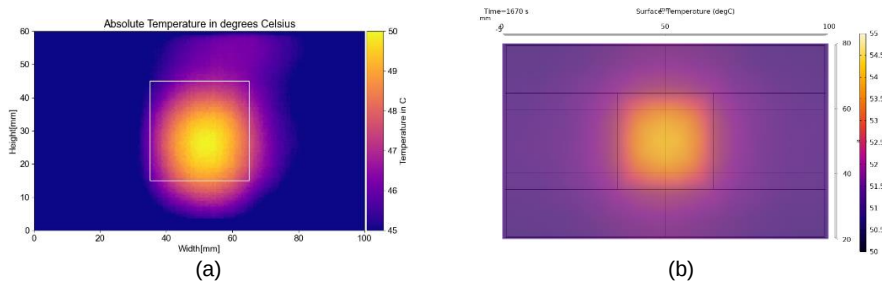


Figure 5.8 Experimental and numerical results of surface temperature field for C1 specimen: (a) experimental results; (b) simulated results.

5.5 Analytical models for data processing

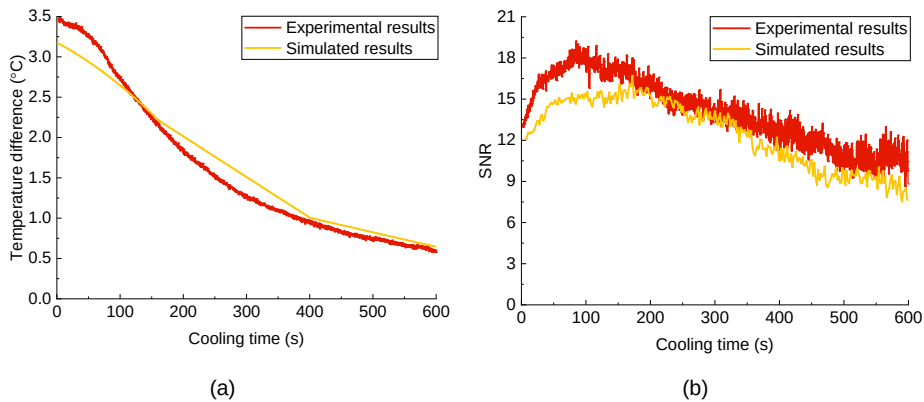


Figure 5.9 Comparison of tested and predicted absolute temperature difference and SNR ratio: (a) absolute temperature difference of sound and delaminated area; (b) SNR ratio comparison.

5.5 Analytical models for data processing

Combining “model” test and numerical simulation, a standard testing procedure, including the distance between the object and halogen lamps, the optimal heating and the cooling time, for best detectability of the delamination has been developed. In Section 5.5, based on SNR method, the analytical model to post process the temperature data gathered from infrared sensor is proposed. Firstly, the frame at the optimal cooling time is extracted, followed by using the SNR method to detect and characterize the delaminated area.

5.5.1 Optimal frame and delamination detection

The optimal frame is chosen based on the SNR method. Firstly, the absolute temperature difference is calculated from the COMSOL model with the minimal delamination thickness of 0.05 mm. Thereafter, the standard deviation (STD) of temperature above sound area as a function of cooling time is calculated. Finally, the optimal cooling time and corresponding frame are defined when the maximum SNR value, based on the temperature difference from COMSOL model and measured STD above sound area, is obtained.

However, the location of the sound area within the inspected specimen is usually unknown. Therefore, two methods are adopted respectively, when the area that is certainly not delaminated (e.g., sound area) within the sample is known and when it is unknown (Figure 5.10 (a) and (b)). If the location of sound area is unknown, the standard deviation (STD) of temperature for the entire specimen is calculated.

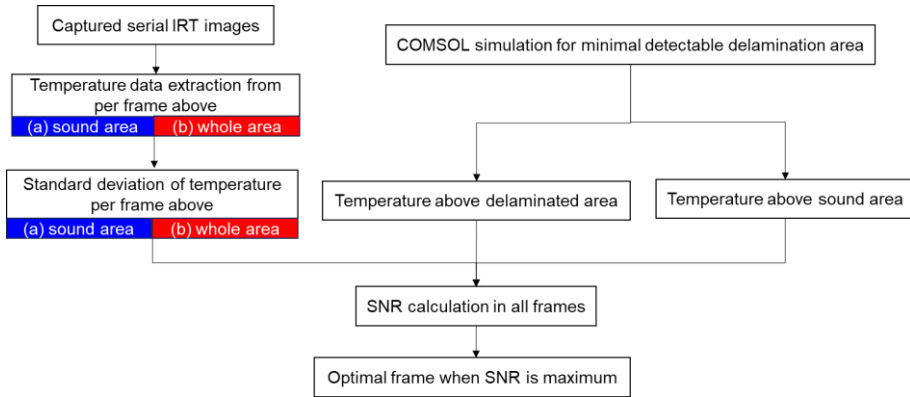


Figure 5.10 Procedures of optimal frame determination for best detectability of delamination: (a) optimal frame determination with provided prior knowledge of sound area; (b) optimal frame determination without provided information of sound area.

As shown in Figure 5.11 (a), when the sound area is known and used as reference area, the average temperature and standard deviation of temperature above the sound area is calculated firstly (Figure 5.10 (a)). Thereafter, the SNR can be calculated in the optimal frame of the entire specimen. If the SNR value of the pixel is positive, the pixel is defined as delamination. While if the SNR value of the pixel is negative, the negative value will be replaced by 0 instead and the pixel is recognized as non-delamination. Finally, all values are plotted in seismic scale in order to clearly visualize the delaminated and sound area of the entire specimen.

When the location of the sound area is unknown, the absolute temperature difference between sound and delaminated areas is calculated from the COMSOL model considering the minimal delamination thickness of 0.05 mm. Based on the temperature data extracted from the experimental optimal frame, the average temperature of the entire optimal frame is calculated and the temperature difference between the average temperature and the temperature of every pixel can be determined. One criterion is defined by comparing the temperature difference from test to that from COMSOL model: the pixel will be recognized as non-delamination if the temperature difference of such pixel is lower than 50% of the temperature difference calculated by COMSOL, otherwise the pixel will be defined as delamination. It should be noted that 50% is only a suggestion in the current study and further studies need to be conducted to determine a more reasonable value, or range of values for valid sound area recognition. Once the (certain) sound area is recognized, the same analytical procedures as with a prior known sound area is followed. The SNR value can be determined. Finally, both the delamination ($\text{SNR} > 0$) and non-delamination ($\text{SNR} = 0$) areas could be recognized.

5.5 Analytical models for data processing

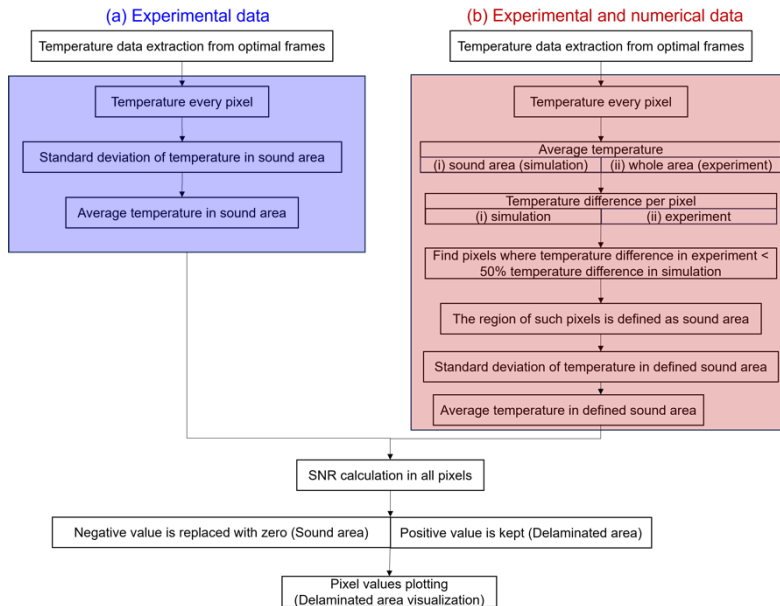


Figure 5.11 Flowchart of delaminated areas determination from the data of the optimal frame: (a) with prior information of sound area; (b) without provided information of sound area.

5.5.2 Results and discussion

With the developed testing protocol and post-processing method, the delamination in all test specimens is evaluated. From the results of the C1 specimen (Figure 5.12 (a)), it is found that the SNR method can well predict the delamination area in the specimen, although the noise around the delaminated area can still be detected. Note that rectangles that are used as references (sound area) are labelled with Bx2 in corresponding raw images. There is nearly no difference in delamination detection with or without using the location of sound area, which confirms the accuracy of the developed analytical method.

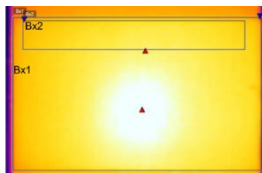
In order to investigate the edge effect, S1 and S2 specimens are compared. As presented in Figure 5.12 (b) and Figure 5.12 (c), both developed procedures (with or without prior knowledge of sound area) can recognize the defected area. However, in the S1 specimen, the defected boundaries of two delamination zones, S1-D1 and S1-D2, are difficult to differentiate. The possible reason is that the delaminated zones are close to each other, and thereby mutually interfere the delamination evaluation. In contrast, if the distance between the defects is large enough (as shown in Figure 5.12 (c), delamination zones are more clearly visualized with almost no overlap.

The results for S2 and S3 specimens in Figure 5.12 (c) and Figure 5.12 (d) show that the delamination thickness will influence the detectability. Under the condition that the size of defects is kept the same and the distance between each defect is large enough to avoid

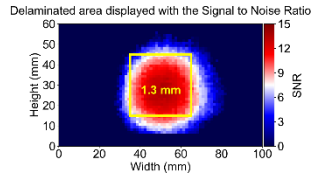
the edge effect, the delamination in S2 is more clearly detected, with a higher SNR value. But it should be mentioned that it is in general impossible to correlate the SNR value with the delamination thickness since not only the delamination thickness, but also other factors such as the delamination size will influence the SNR value [166].

Finally, the size of delamination is also analyzed. From the results of C1, S1, S2 and S3 specimens, it is found that a designed defected region (30 mm × 30 mm) can be well detected. In S4 and S5 specimens, with a larger size of delamination, the detectability is not affected, and the region of delamination can clearly be identified. However, S5-D4 defect has the same delamination size as the defects in S1 and S2 specimens but has a larger delamination thickness, while it shows less detectability.

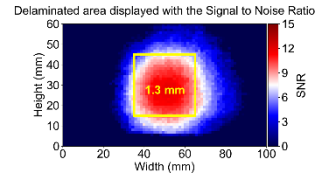
Additionally, from the results of S1 to S5, it is also observed that, with thermal information selected from different sound zones as prior knowledge, though the defects can be detected, the detected SNR values are different. This is possibly due to the different noise level in these regions and non-uniform heat distribution during heating process. The analytical method with no prior knowledge about sound area is more objective and in general could present good detectability in terms of size and location of the delamination.



(i) Raw image

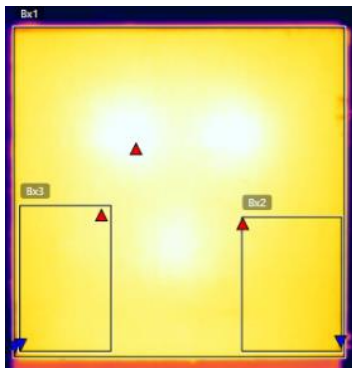


(ii) Result computed with temperature data from sound area Bx2 as labelled in (i) Raw image.

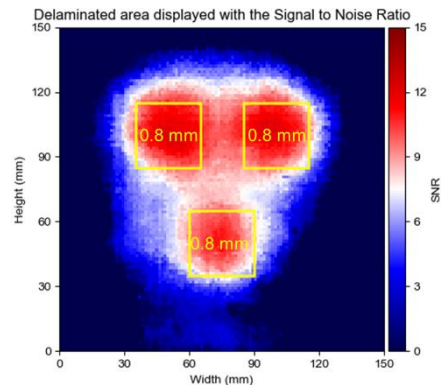


(iii) Result computed without a known sound area (therefore Bx1 or entire specimen data is used).

(a) C1

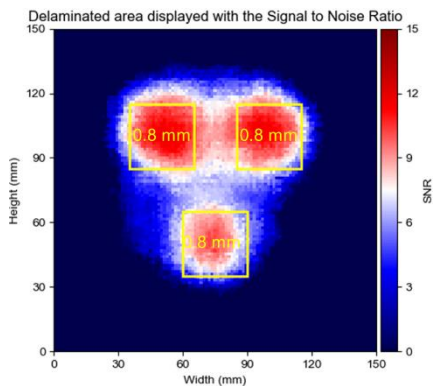


(i) Raw image

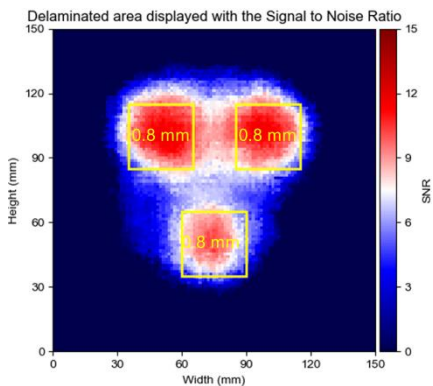


(ii) Result computed with temperature data from sound area Bx2.

5.5 Analytical models for data processing

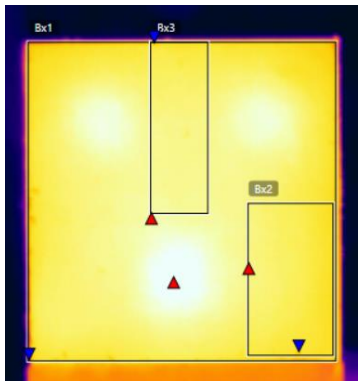


(iii) Result computed with temperature data from sound area Bx3.

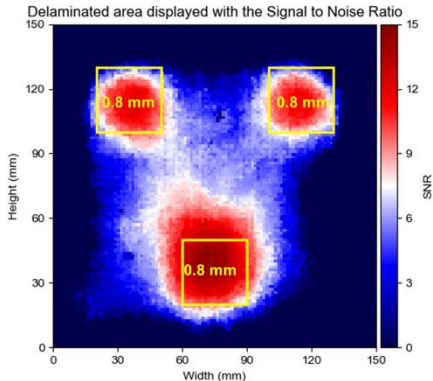


(iv) Result computed without a known sound area.

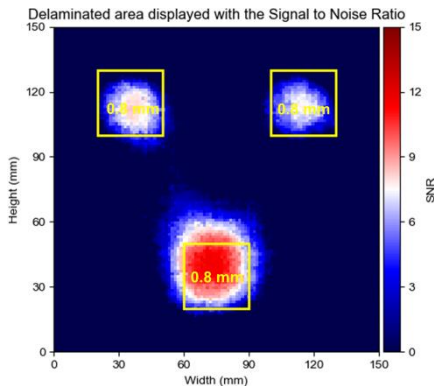
(b) S1



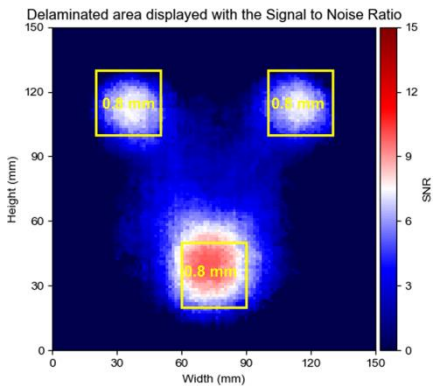
(i) Raw image



(ii) Result computed with temperature data from sound area Bx2.

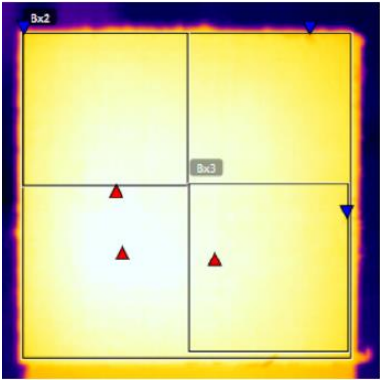


(iii) Result computed with temperature data from sound area Bx3.

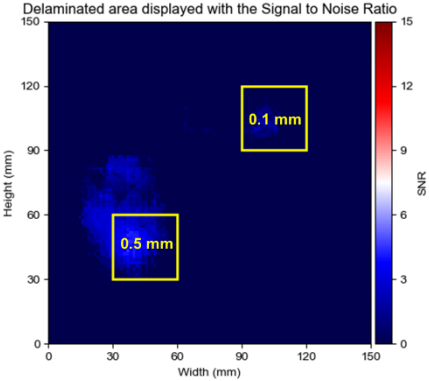


(iv) Result computed without a known sound area.

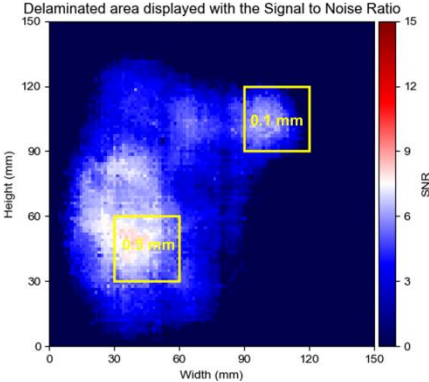
(c) S2



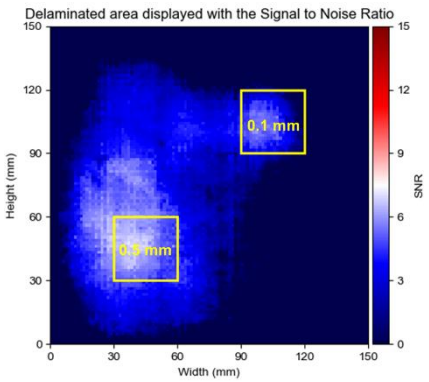
(i) Raw image



(ii) Result computed with temperature data from sound area Bx2.

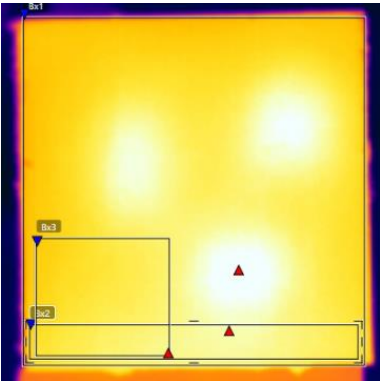


(iii) Result computed with temperature data from sound area Bx3.

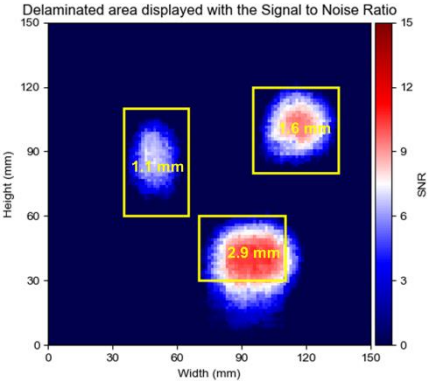


(iv) Result computed without a known sound area.

(d) S3

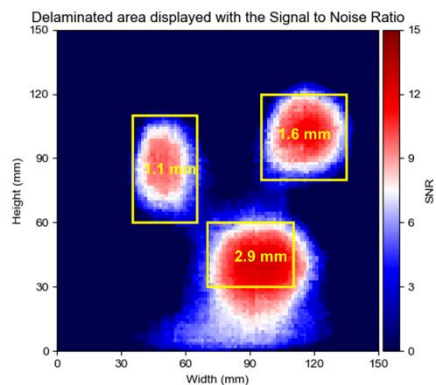


(i) Raw image

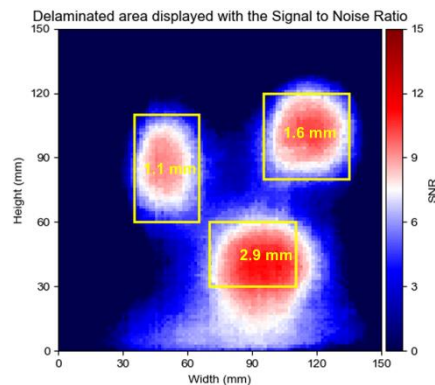


(ii) Result computed with temperature data from sound area Bx2.

5.5 Analytical models for data processing

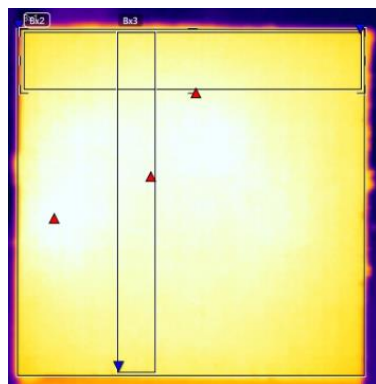


(iii) Result computed with temperature data from sound area Bx3.

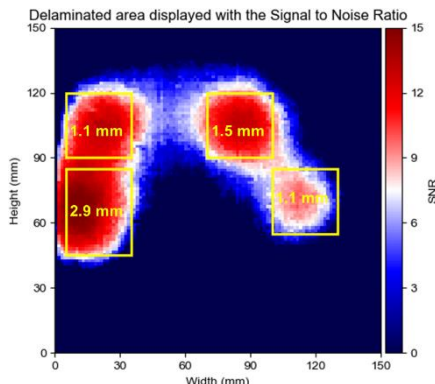


(iv) Result computed without a known sound area.

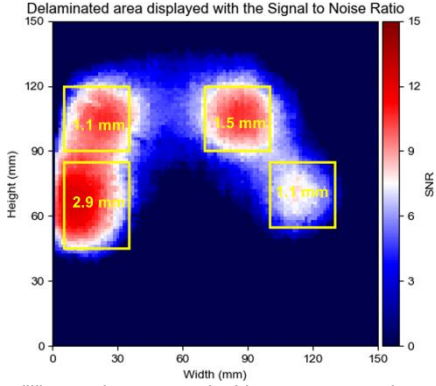
(e) S4



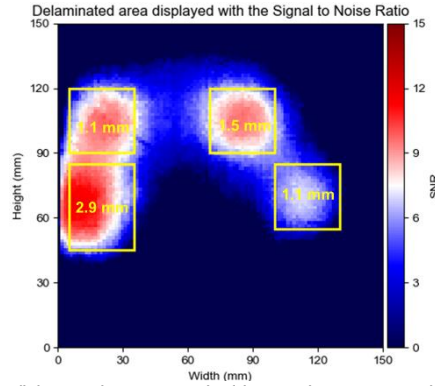
(i) Raw image



(ii) Result computed with temperature data from sound area Bx2.



(iii) Result computed with temperature data from sound area Bx3.



(iv) Result computed without a known sound area.

(f) S5

Figure 5.12 Results of all test specimens with and without prior information of sound area.

The two methods are also evaluated by the performance metrics: (a) Sensitivity or recall (True positive ratio, TPR), (b) Fall out (False positive ratio, FPR), (c) Miss rate (False negative ratio, FNR), (d) Specificity (True negative ratio, TNR), (e) Accuracy, (f) Precision, (g) F1 score. These metrics are calculated using the following equations:

$$TPR = \frac{TP}{TP + FN} \quad (5.3)$$

$$FPR = \frac{FP}{TN + FP} \quad (5.4)$$

$$FNR = \frac{FN}{TP + FN} \quad (5.5)$$

$$TNR = \frac{TN}{TN + FP} \quad (5.6)$$

$$Accuracy = \frac{TP + TN}{TP + FP + TN + FN} \quad (5.7)$$

$$Precision = \frac{TP}{TP + FP} \quad (5.8)$$

$$F1 \text{ score} = \frac{2TP}{2TP + FN + FP} \quad (5.9)$$

where TP, TN, FP and FN denote the number of true positive, true negative, false positive, and false negative respectively. When a pixel is above a delaminated area, it is either true positive (TP) if correctly identified, or false negative (FN) if wrongly labelled as sound. Likewise, when a pixel is above sound area, it is either true negative (TN) if correctly identified as sound or false positive (FP) if wrongly labelled as a defect. The accuracy of the methods represents the ratio of total correct detection (both delaminated and sound) to the total number of detections and is used to quantify the overall correctness of the models. The precision is defined as the ratio of correct positive detection to the total number of positive prediction and evaluates the accuracy of delamination predictions. A high precision value means that the model is able to minimize false defect detection. F1 score is served as a composite metric that balances precision and recall (equals to TPR) and is ranged from 0 to 1. A higher F1 score value indicates better model performance in terms of both precision and accuracy.

Table 5.3 summarizes the values of performance metrics for the test specimens using two analytical models. It is found that generally both methods can provide reasonable results in terms of TPR and accuracy. However, the precision is not high in all cases as an additional major part of pixels are incorrectly identified as delaminated. Furthermore, if the sound area is not carefully chosen, the performance of the model is not good. For instance, in the case of S2 and sound area Bx2, the accuracy is only marginally acceptable, and the precision is low. One possible explanation for this inaccuracy is the random nature of

5.6 Beam test and validation

noise fluctuations, which are challenging to eliminate, and the difficulty in ensuring uniform heating conditions.

Table 5.3 Summary of delamination performance metrics for all test specimens using two analytical models.

Sample	Analytical models	TPR	FPR	FNR	TNR	Accuracy	Precision	F1 score
C1	Sound area Bx2	1.00	0.24	0.00	0.76	0.79	0.41	0.58
	Unknown sound area	1.00	0.42	0.00	0.58	0.64	0.29	0.44
S1	Sound area Bx2	1.00	0.53	0.00	0.47	0.54	0.20	0.33
	Sound area Bx3	1.00	0.34	0.00	0.66	0.70	0.28	0.43
S2	Unknown sound area	1.00	0.39	0.00	0.61	0.65	0.25	0.40
	Sound area Bx2	0.99	0.54	0.01	0.46	0.52	0.19	0.32
S3	Sound area Bx3	0.84	0.16	0.16	0.84	0.84	0.39	0.54
	Unknown sound area	0.95	0.40	0.05	0.60	0.64	0.23	0.37
S4	Sound area Bx2	0.50	0.09	0.50	0.91	0.88	0.31	0.38
	Sound area Bx3	0.92	0.56	0.08	0.44	0.59	0.15	0.25
S5	Unknown sound area	0.95	0.45	0.05	0.55	0.58	0.14	0.25
	Sound area Bx2	0.67	0.13	0.33	0.87	0.83	0.50	0.57
S6	Sound area Bx3	0.96	0.34	0.04	0.66	0.71	0.36	0.52
	Unknown sound area	1.00	0.51	0.00	0.49	0.57	0.28	0.44
S7	Sound area Bx2	0.99	0.36	0.01	0.64	0.70	0.34	0.51
	Sound area Bx3	0.94	0.28	0.06	0.72	0.76	0.39	0.55
S8	Unknown sound area	0.98	0.36	0.02	0.64	0.69	0.34	0.50

5.6 Beam test and validation

After the evaluation of delamination in small-scale “model” tests, two large-scale hybrid beams consisting of one cast in-situ specimen and one epoxy-bonded specimen, are also tested and evaluated through the developed method. The hybrid beam has the same dimension of the tested strengthened beams in Chapter 3 and the details of the hybrid beams can be also found in [223]. In order to validate the reliability of the methods for delamination detection in large-scale hybrid RC beams, CT scanning is also conducted in one of the hybrid beams for verification.

5.6.1 Beam test procedure and results

Severe delamination between UHPFRC layer and concrete part is observed in the beam with in-situ cast UHPFRC (Figure 5.13). The delamination length reaches more than 80% of the interface length along the beam. For another hybrid beam with epoxy-bonded UHPFRC laminates, no visual delamination is observed. However, it should be noted that for both hybrid beams, the internal delamination is difficult to be detected. Therefore, the IRT is applied and beam test results are post-processed through developed methods. Due to a larger size in hybrid beams compared to composite concrete blocks, the testing procedures are also upscaled (shown in Figure 5.14). During the test, three 1000 W halogen lamps, which can produce 16000 lumens, are evenly placed at a distance of 70 cm from the test beam in order to heat the surface of the sample homogenously as much as possible

for 25 min. The distance between lamps and test sample is determined by the measurement of heat flux, which is aimed at 1750 W/m^2 , the same as for the small-scale specimen tests. The infrared camera (FLIR A320 camera) is set such that it captures the entire beam during the cooling process.



Figure 5.13 Interface debonding for one of the hybrid beams.



Figure 5.14 IRT test process for one of the hybrid beams: (a) Uniform heating process using three lamps; (b) Data recording during cooling process after removal of the lamps.

After data gathering, the two analytical methods (with/without prior information of sound area) are adopted to determine the delaminated region in hybrid beams. Figure 5.15 and Figure 5.16 shows the SNR results for both beams. For ST-CIS specimen, since the delamination can clearly be observed from visual inspection, an area indicated as Bx2 and shown in Figure 5.15 (a), is assumed as sound. For ST-EB specimen, no delamination could be observed, thus only the analytical method without information of sound area is employed. The delamination is detected before and after the three-point bending test in order to differentiate the delamination from the applied load and eliminate the influence of premature delamination during drying.

From Figure 5.15, it is found that the SNR results differ when the two methods are applied. With an assigned sound area, the detected debonding area is much larger than with the

5.6 Beam test and validation

unknown sound (reference) area. From visual observation, the delamination length reaches more than 80% of the interface length and in this case the method of assigning a sound area gives a more accurate result than the method without a known sound area. Almost the entire beam is delaminated. The method without known sound area, however, underestimates the region of delamination at the interface between the NC core and the UHPFRC layer. As described in Figure 5.15 (b), for analytical method without information of sound area, the sound area will be recognized if the temperature difference of such zone is lower than 50% of the temperature difference calculated by COMSOL. This implies assumption that there is a significant part of sound area compared to the amount of delaminated area. If the sample is severely delaminated, this method will misidentify part of the delaminated area as sound zone, which leads to a conservative result. Therefore, the accurate estimation of the delaminated area needs to be determined beforehand if there is no certainty about the sound area.

For the ST-EB specimen, both the surface and underneath defects (crack and delamination) are reasonably identified, which can be validated from surface cracks and the delamination profile. As shown in Figure 5.16, the surface defects agree well with the defects detection results in terms of their location and size. However, it is also found that the developed methods cannot distinguish different types of defects, such as the crack and delamination.

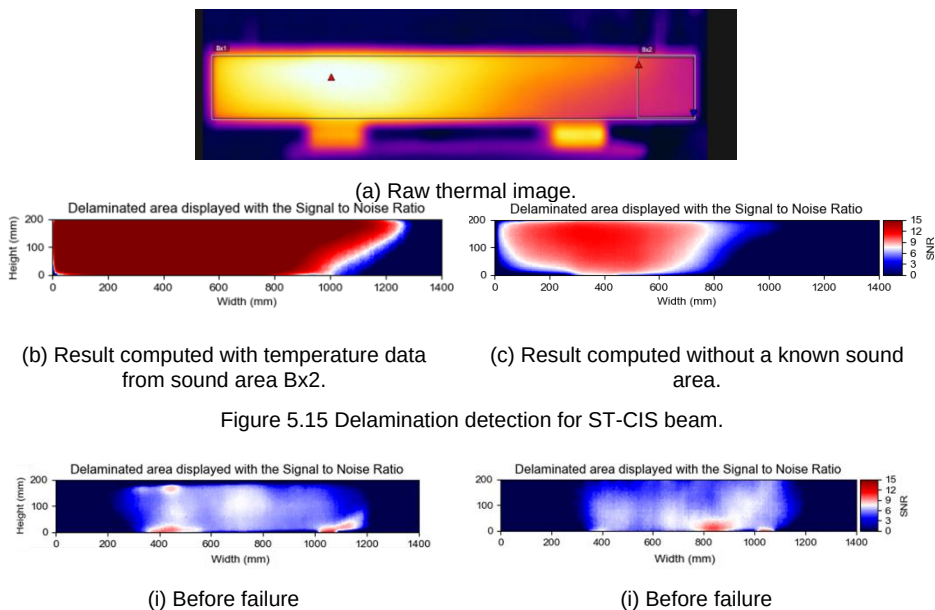


Figure 5.15 Delamination detection for ST-CIS beam.

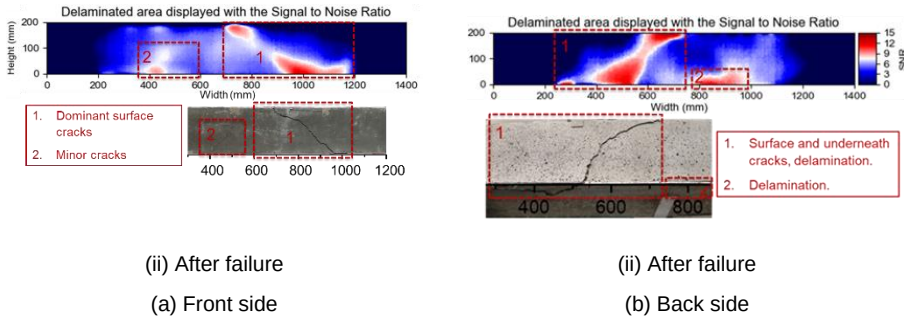


Figure 5.16 Delamination detection for both sides of ST-EB specimen before and after loading test.

5.6.2 Validation from X-ray computed tomography (CT)

In order to verify the accuracy of the developed methods based on IRT technology and to quantify the delamination in hybrid beam, CT for the large beam sample (i.e., ST-EB) is conducted (shown in Figure 5.17). CT technique is an advanced non-destructive testing technique that uses x-rays to create detailed cross-sectional images of the internal structure of a specimen. By capturing 2D images from various angles, these images are then computationally reconstructed to 3D structure, allowing for precise visualization and analysis of internal features [224]. Therefore, CT can help to detect the invisible defects such as cracks and voids. However, it should be noted that due to the relatively large size of the hybrid beam, a resolution of 0.59 mm is obtained.



Figure 5.17 CT scanning of the ST-EB specimen.

Quantitative analysis is carried out and the results are visualized using the image analysis software Avizo. Slices from the surface of UHPFRC layer to the interfacial zone are selected and combined for shallow defect detection. As illustrated in Figure 5.18, the spatial distribution of visible surface cracks is displayed using Avizo software. Thereafter, the spatially distributed cracks are projected onto a 2D plane and a comparison of the results from analytical method based on IRT technology and CT is shown in Figure 5.19. The comparison reveals that the results from CT in general agree well with the results from infrared thermography in terms of the crack visualization. However, the delamination (shown in Figure 5.16 (b)) is not well detected in CT since the resolution of CT is limited

5.6 Beam test and validation

to 0.59 mm (larger than the delamination thickness). Though more vulnerable to the noise error and edge effect, the IRT shows a better delamination detectability given that a delamination thickness of 0.1 mm could be detected in S3-D1.

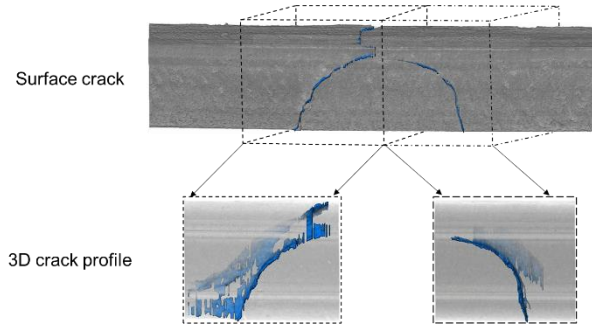


Figure 5.18 Projection of 3D crack profiles to 2D plane for ST-EB specimen.

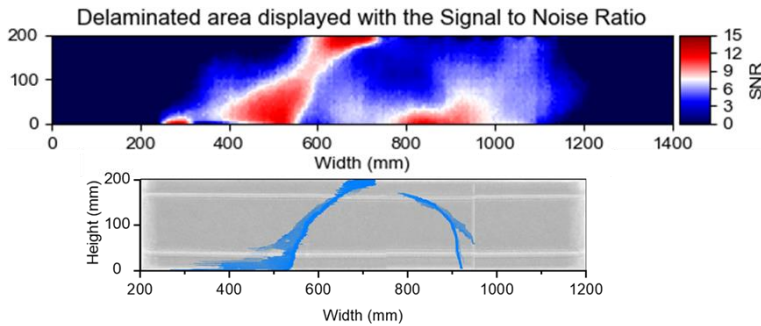


Figure 5.19 Comparison of results between IRT technology and CT scanning for ST-EB specimen.

5.6.3 Evaluation of interface properties in strengthened beams after FT

In order to examine the performance of interface and possibly delamination due to freeze-thaw, among all tested beams, three cast in-situ beams (i.e., ST-M2-CIS-30, ST-M2-CIS-75 and ST-M2-CIS-120 specimens) and one epoxy-bonded beam (ST-M2-EB-120) are selected for infrared thermography test. The purpose to select three beams bonded by cast in-situ technique is because they are more sensitive to shrinkage and freeze-thaw cycles. For epoxy-bonded technique only the beam exposed to final 120 FT cycles is examined.

The infrared thermography test is conducted both before and after the predefined number of freeze-thaw cycles and follows the same testing procedure described in Section 5.6.1. Since the location of sound area is unknown, the analytical method without prior information of sound area is applied to examine the interface performance in hybrid beams. Figure 5.20 and Figure 5.21 show the SNR results for both sides of ST-M2-CIS-30 and ST-M2-CIS-75 specimens, respectively. However, for ST-M2-CIS-120 and ST-M2-EB-120 specimens, the SNR results for only one side of the specimen are presented, as shown in Figure 5.22 and Figure 5.23, respectively.

In terms of CIS bonding techniques, from Figure 5.20, it is found that after 30 FT cycles, the interface properties become better, which is possibly due to the continued hydration of the unhydrated cement particle in UHPFRC at the interface as additional water is provided during FT test [115]. While after more than 75 FT cycles, as shown in Figure 5.21 and Figure 5.22, it seems that freeze-thaw action starts to play a dominant role in weakening the bond. This phenomenon also occurs in epoxy-bonded specimen subjected to 120 FT cycles. In Figure 5.23, when interface properties are compared without and with FT cycles, it seems that the bond becomes worse, and the delaminated area is larger after 120 FT cycles. However, given the observed variability in results discussed in Section 5.5.2, this trend could also be influenced by scatter of the results caused by the inherent weakness of the analytical method without information of the sound area. The average temperature of the entire optimal frame is used for temperature difference calculation in experiment and 50% of the temperature difference calculated by COMSOL is assumed as threshold. Therefore, further tests with a model specimen (that is, non-delaminated small UHPFRC-NC specimen) on top of the beam serve as a reference (i.e., sound area) would be required to confirm this trend and rule out variability.

In addition, when comparing SNR results before and after FT cycles, there is an inconsistency in the identified locations of delamination. Specifically, delamination that is detected in beams prior to FT action is not consistently detected in the same location in beams after FT action - this inconsistency is particularly evident in beams exposed to more than 75 FT cycles. Many factors might lead to such false identification, such as edge effect, non-uniform heat, and ambient environmental condition. Therefore, in order to provide a more quantitative analysis of interface properties due to FT action, future research is needed to pre-process the temperature data (such as noise filtering, temperature calibration and vignetting [225]) and to establish a more effective and objective analytical method to enhance the signal.

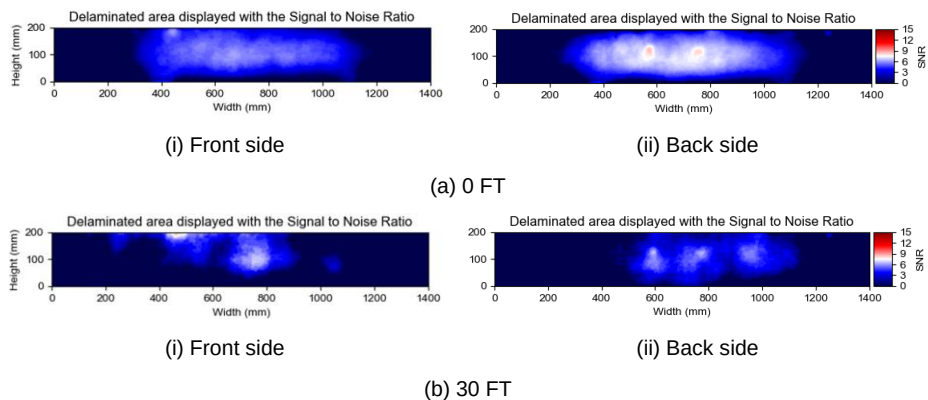


Figure 5.20 Delamination detection for both sides of ST-M2-CIS-30 specimen before and after 30 FT cycles.

5.6 Beam test and validation

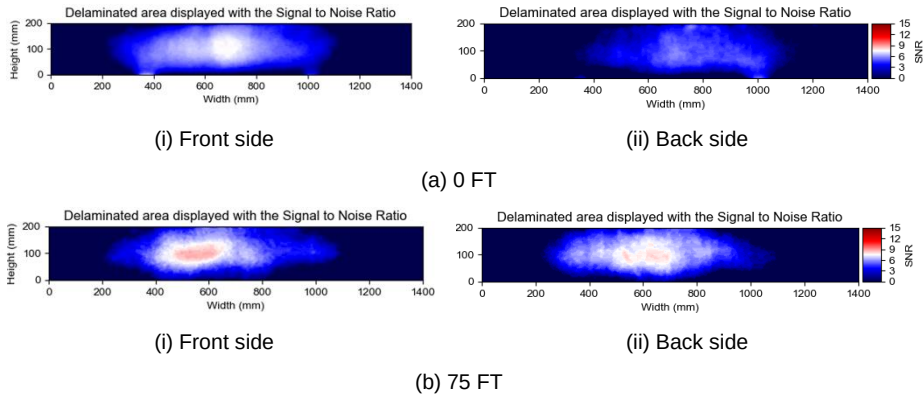


Figure 5.21 Delamination detection for both sides of ST-M2-CIS-75 specimen before and after 75 FT cycles.

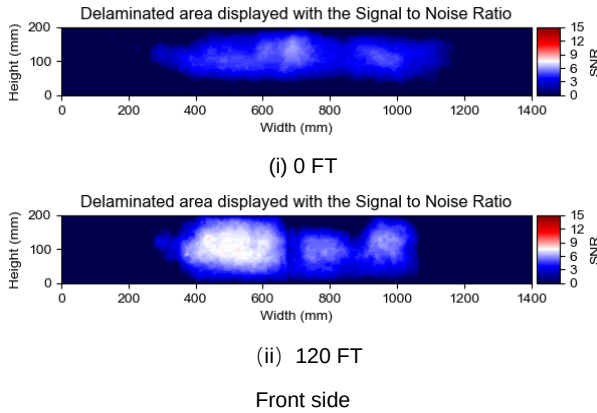


Figure 5.22 Delamination detection for the front side of ST-M2-CIS-120 specimen before and after 120 FT cycles.

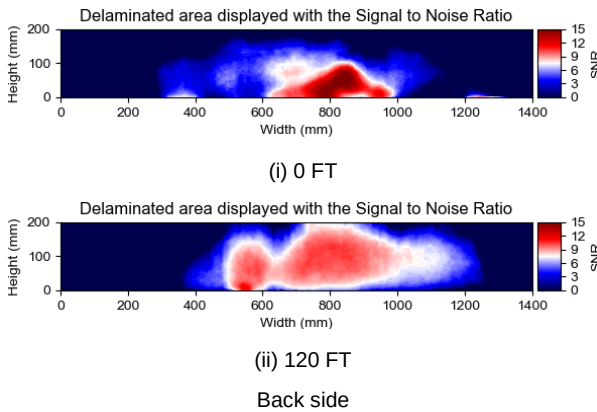


Figure 5.23 Delamination detection for back side of ST-M2-EB-120 specimen before and after 120 FT cycles.

5.7 Conclusion

Subsurface defect (such as delamination or underneath crack) detection using active infrared thermography is promising for the inspection of concrete structures. Although the defects can be visually identified by a temperature profile along the surface, more precise delamination detection and establishing clear and objective criteria for delamination categorization are essential. This chapter develops a standardized testing method and post-processing method for delamination detection in UHPFRC strengthened concrete beam using active IRT. Six small-scaled specimens with predetermined artificial delamination are evaluated. Through combined experimental and numerical study, three critical parameters (i.e., distance between specimen and lamp, optimal heating, and cooling time) are defined. Thereafter, the postprocessing method is developed and its accuracy is evaluated based on SNR results and performance metrics. In addition to the small-scaled composite specimens, for upscaling application, two large composite UHPFRC-NC beams are also tested using the developed testing procedures and analytical methods, and the accuracy for the epoxy-bonded specimen is examined by both the visual inspection and CT scanning. Finally, the interface properties for four hybrid beams after FT cycles are examined, and the following conclusions are drawn:

(1) Based on combined experiments and numerical study, it is found that 25 min heating can provide enough thermal contrast for minimal 0.1 mm delamination thickness detection for UHPFRC-NC composites. Finally, the optimal cooling time is determined when the maximum SNR is reached, and this is defined as an optimal frame for delamination detection.

(2) The SNR method is applied to develop the post-processing method for delamination detection. The analytical models are divided into two parts: optimal frame extraction and delamination area recognition. And two situations are considered: information of sound area is known beforehand or not. Based on the SNR and performance metrics' results, it is found that both analytical methods can reasonably detect the delamination in terms of its location and size, though the results are easily affected by the non-uniform surface heat distribution, environmental noise and edge effects. In addition, for the analytical method with known sound area, the selection of appropriate sound area with small standard deviation of temperature (reference area) is important for the accuracy and precision of defect detection. On the other hand, the analytical method without information of sound area is more objective and generally could also provide reliable results.

(3) Varied parameters including the edge effect, the delamination thickness and the size of the delamination, are investigated. Based on the results, it is observed that there is interference if two delamination are too close to each other. In addition, large delamination thickness will increase its detectability as higher thermal contrast and SNR could be

5.7 Conclusion

obtained. It also should be mentioned that uneven distribution of temperature along the surface might lead to unreliable results.

(4) Two hybrid beams are also examined and analyzed by IRT with the developed testing method. The results demonstrate that compared to analytical models with known sound area, the analytical models without information of the sound area will underestimate the real size of the delamination for severely delaminated specimen due to its assumption that 50% of the temperature difference calculated by COMSOL is used as threshold. However, both methods are effective to detect the defects for hybrid beam without obvious visual imperfection, and its accuracy is both validated by visual surface observation and CT scanning.

(5) The interface properties of four beams (three CIS beams and one epoxy-bonded beam) after freeze-thaw cycles are examined. The results indicate that for the cast in-situ technique, no delamination happens and the interface properties probably improve after initial 30 FT cycles possibly due to continued hydration of the unhydrated cement particle in UHPFRC near the interface. However, after more than 75 freeze-thaw cycles, the interface properties deteriorate due to damage caused by the more dominant freeze-thaw action. A similar interfacial degradation occurs in the epoxy-bonded specimen after 120 freeze-thaw cycles. However, such trend could also result from scatter of the results caused by the inherent weakness of the analytical method without information of sound area. In addition, it should also be noted that some inconsistencies in the observed delamination locations in hybrid beams before and after freeze-thaw cycles highlight the limitations of the current analytical model.

Future research is still necessary to further improve the accuracy and to reduce the noise, edge effect and uneven heat distribution, therefore, higher performance metrics can be achieved. In addition, a minimum delaminated thickness of 0.1 mm is investigated and detected in current study, and it is recommended to continue reducing the dimension of delamination until the minimum detectable delaminated size is determined. Finally, for the analytical method without prior information of sound area, its accuracy is dependent on the reliability of the assumption that 50% of the temperature difference calculated by COMSOL is used as threshold. Therefore, further work is recommended, such as conducting tests with model non-delaminated UHPFRC-NC specimens (as a reference) alongside the test specimens, to validate the accuracy of the analytical method without known sound area.

Chapter 6 Electromagnetic method for fiber distribution characterization in UHPFRC

6.1 Introduction

As indicated in Chapter 4, the non-uniform fiber distribution in UHPFRC may govern the shear response of the hybrid beams. To address this, Chapter 6 introduces the electromagnetic method to measure the fiber dispersion in UHPFRC panels for all tested beams.

In Section 6.2, the non-destructive electromagnetic method, including its principles, and procedures for determining fiber content and orientation, is described. Thereafter, the experimental program is presented in Section 6.3 and the corresponding test results are summarized in Section 6.4. The magnetic probes are calibrated firstly, and the influence of the parameters including the casting directions, and vibration time, are evaluated. Finally, the method's accuracy is validated using x-ray imaging.

After validation, in Section 6.5, this method is applied to measure the fiber distribution for all tested beams. Then the influence of non-uniform fiber distribution on the shear capacity of hybrid beam is evaluated. Section 6.6 presents the analytical method to predict the main mechanical properties (i.e., compressive strength, elastic modulus and tensile response) of UHPFRC based on the fiber contents. Finally, Section 6.7 concludes Chapter 6.

6.2 Description of the electromagnetic method

6.2.1 Principles

The magnetic permeability (μ) of a material is a property that indicates how easily a magnetic field can be created within the material. It is defined as the ratio of the magnetic flux density, generated within the material by a magnetizing field, and the magnetic field strength. To compare the magnetic permeability of a material (μ) with that of non-magnetic materials like vacuum (μ_0), the relative magnetic permeability (μ_r) is used, which is expressed as:

$$\mu_r = \frac{\mu}{\mu_0} \quad (6.1)$$

UHPFRC contains two parts: (1) cementitious matrix and (2) steel fibers. Similar to air or other non-magnetic materials, the relative magnetic permeability of the matrix equals to 1 while the relative magnetic permeability of steel fibers is much higher than 1 due to their ferromagnetic property. Therefore, the magnetic permeability can be used as an indicator of the fiber content within the UHPFRC element.

Based on the previous work by Nunes et al. [41,153], two kinds of inductors (i.e., I-shaped ferrite magnetic probe and U-shaped ferrite magnetic probe) are developed at TU Delft, as shown in Figure 6.1 (a) and (b). The inductor consists of ferrite core with copper wire coil around it. A magnetic field is generated if an electric current is applied and passes

through the copper coil. The inductance of the resulting magnetic circuit L , which is defined as the amount of magnetic flux produced for a given electric current, can be measured by LCR meter, as depicted in Figure 6.1 (c). If the inductor is placed on a UHPFRC member, the measured inductance of the magnetic circuit depends on the relative magnetic permeability of the composites. Nunes et al. [41,153] proposed that within the normal steel fibers content in UHPFRC (i.e., lower than 4% by volume), the relative magnetic permeability of UHPFRC ($\mu_{r,i}$) in a given direction i can be simplified by the following equation:

$$\mu_{r,i} \approx \frac{L_{UHPFRC}}{L_{air}} \quad (6.2)$$

where L_{UHPFRC} is the measured inductance through the UHPFRC material and L_{air} is the measured inductance in the air (also represents the reference value).

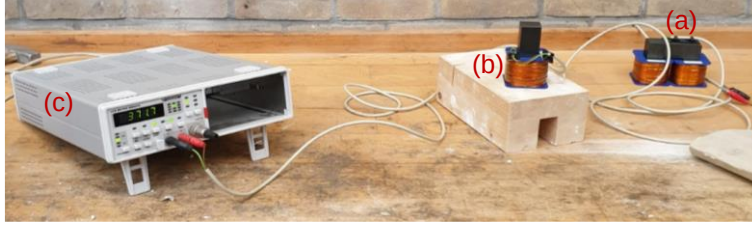


Figure 6.1 Experimental set-up: (a) U-shaped ferrite probe; (b) I-shaped magnetic probe; (c) LCR meter.

6.2.2 Fiber distribution and orientation measurement

For a thin UHPFRC layer, that is, the fiber length is larger than the thickness of UHPFRC, it is assumed that fibers are in 2D plane and there is a negligible interaction of the fibers. Based on the above assumption, Nunes et.al [41,153] proposed a simplified physical model to predict the relative magnetic permeability of thin UHPFRC elements in any direction and found that the relative magnetic permeability of the UHPFRC depends on the fiber distribution, and fiber orientation with respect to the measuring direction.

Fiber content

The mean value of the relative magnetic permeability ($\mu_{r,mean}$) is obtained by averaging the measurements of relative magnetic permeability taken in two orthogonal directions, as expressed by:

$$\mu_{r,mean} = \frac{\mu_{r,i} + \mu_{r,j}}{2} \quad (6.3)$$

where $\mu_{r,i}$ and $\mu_{r,j}$ are the relative magnetic permeability for the orthogonal directions i and j , respectively. And it is found that the $\mu_{r,mean}$ is independent of fiber orientation

6.2 Description of the electromagnetic method

and has a linear relation with fibre content (V_f). Therefore, the $\mu_{r,mean}$ can serve as an indicator of the fiber content, and the relation between $\mu_{r,mean}$ and V_f can be expressed as:

$$V_f = \frac{\mu_{r,mean} - 1}{k_v} \quad (6.4)$$

where k_v is a proportionality constant and needs to be calibrated experimentally.

Fiber orientation

Based on the $\mu_{r,i}$ and $\mu_{r,j}$, as proposed by Nunes et.al [41,153], the preferential fiber orientation ($\rho_{i,j}$) can be identified and expressed as:

$$\rho_{i,j} = \frac{\mu_{r,i} - \mu_{r,j}}{2(\mu_{r,mean} - 1)} \quad (6.5)$$

The fiber orientation indicator ($\rho_{i,j}$) is independent of the fiber content, and a positive or a negative value of $\rho_{i,j}$ indicates that the fibers are more preferably aligned along i or j direction. If $\rho_{i,j} = 0$, a uniform fiber distribution is expected.

The fiber orientation factor ($\alpha_{0,i}$) is an effective parameter to characterize the fiber orientation in the cross-section and can be calculated as follows:

$$\alpha_{0,i} = \frac{N_f \cdot A_f}{V_f \cdot A} \quad (6.6)$$

where N_f is the number of the fibers in the cross-section, A_f is the cross-sectional area of single fiber, V_f is the fiber content and A is the cross-section area of specimen. Nunes et.al [41,153] found that the relation between the fiber orientation indicator ($\rho_{i,j}$) and the fiber orientation factor along i and j direction ($\alpha_{0,i}$, $\alpha_{0,j}$) is approximately linear, and their relations can be expressed as follows:

$$\alpha_{0,i} = k_\alpha \rho_{i,j} + \alpha_{0,iso} \quad (6.7)$$

$$\alpha_{0,j} = -k_\alpha \rho_{i,j} + \alpha_{0,iso} \quad (6.8)$$

where k_α is the slope of the linear expression and can be experimentally determined. $\alpha_{0,iso}$ is the fiber orientation factor when the fiber orientation is uniformly distributed, and is relevant to the ratio between thickness of UHPFRC element (h_u) and the fiber

length (l_f). For fibers in 2D plane (i.e., h_u / l_f close to 0), $\alpha_{0,iso} = 0.64$, while for in 3D (i.e., $h_u / l_f \rightarrow \infty$), $\alpha_{0,iso} = 0.5$ [41].

6.3 Experimental program

6.3.1 Specimen preparation

As described in Section 6.2, in order to quantify the fiber content and orientation, two indicators ($\mu_{r,mean}$ and $\rho_{i,j}$) are proposed and the proportionality constant k_v can be calibrated experimentally. Calibration is performed by testing specimens with different fiber contents.

The UHPFRC2 mixture is used, and a total of 27 UHPFRC plates with a dimension of $200 \times 200 \times 10 \text{ mm}^3$ are fabricated. Additionally, to determine the compressive and tensile properties of UHPFRC with varying fiber contents (0%, 1%, 2%, 3%, and 4%), three cubic samples with dimensions of 40 mm and dog-bone samples are prepared. As shown in Table 6.1, the specimens are categorized into three groups to investigate the effects of casting direction and vibration on fiber distribution and orientation. Group 1 and Group 2 include horizontally and vertically cast specimens without vibration, respectively. While in group 3, specimens are vertically cast with vibration for various time durations. During mixing and casting, care is taken to ensure random fiber orientation within the UHPFRC specimens. However, in Group 3, due to the vibration of the vertically cast UHPFRC specimens, the fibers tend to sink and align horizontally at the bottom of the plates. Inductance measurements are performed immediately after demolding (i.e., at the age of 1 day) as the relative magnetic permeability is independent of the age and composition of the matrix. To determine the fiber content and orientation, the sensitivity of the measurements is assessed by considering factors such as edge effects, surface painting, and surface roughness of the UHPFRC plates. Furthermore, the impact of casting direction and vibration time on fiber distribution and orientation is investigated. Details of the specimens are provided in Table 6.1.

Table 6.1 Details of all tested specimens.

Group	Specimen ID	Dimension ($L \times B \times t \text{ (mm}^3\text{)}$)	Fiber content (%)	Casting direction	Vibration time (min)
Group 1	H1-1	$200 \times 200 \times 10$	1	Horizontal	-
	H1-2		1	Horizontal	-
	H2-1		2	Horizontal	-
	H2-2		2	Horizontal	-
	H2-3		2	Horizontal	-
	H2-4		2	Horizontal	-
	H3-1		3	Horizontal	-
	H3-2		3	Horizontal	-
	H4-1		4	Horizontal	-
	H4-2		4	Horizontal	-
Group 2	V1-1		1	Vertical	-

6.3 Experimental program

	V1-2	1	Vertical	-
	V2-1	2	Vertical	-
	V2-2	2	Vertical	-
	V2-3	2	Vertical	-
	V2-4	2	Vertical	-
	V3-1	3	Vertical	-
	V3-2	3	Vertical	-
	V4-1	4	Vertical	-
	V4-2	4	Vertical	-
	V2-T1-1	2	Vertical	1
	V2-T2-1	2	Vertical	2
	V2-T2-2	2	Vertical	2
Group 3	V2-T3-1	2	Vertical	3
	V2-T3-2	2	Vertical	3
	V2-T5-1	2	Vertical	5
	V2-T5-2	2	Vertical	5

Note: For the specimen ID, the first letter represents the horizontal (H) or vertical (V) casting direction, the first number denotes the fiber volume (%) and Tn (in group 3) represents vibration for n minutes. Finally, the last number denotes the serial number of the plate for the same kind of specimen since at least two plates are tested to ensure reliable test results. Four replicates for 2% UHPFRC are cast, as this mix is used in all beams. However, only one specimen, vibrated for a minute, is tested as others failed during casting.

6.3.2 Testing process

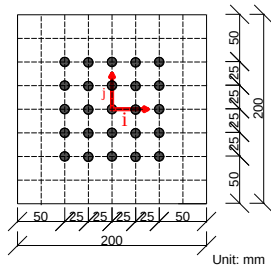
The testing process is divided into three main steps. Firstly, the I-shaped ferrite magnetic probe (shown in Figure 6.1 (b)) is calibrated to measure fiber distribution, and the U-shaped ferrite magnetic probe is calibrated to measure the fiber orientation indicator. Although both probes can detect fiber distribution, only the I-shaped probe is used in this study given its better precision and ability to minimize distortion in measurement that occurs near the edges of specimens (i.e., edge effects [41]). After calibration, in the second step, the effects of fiber content, casting direction, and vibration time are investigated for their impact on fiber distribution and orientation measurements. Finally, some UHPFRC plates are selected for x-ray absorption imaging to assess fiber distribution and validate magnetic probe measurements.

Calibration (fiber distribution and orientation)

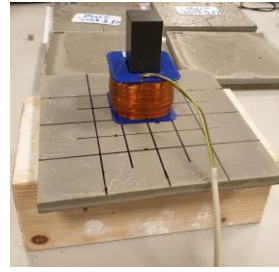
The UHPFRC plates in group 1 and group 2 are selected to calibrate the I-shaped probe. For horizontally cast specimens in group 1, measurements are performed only with smooth (i.e., cast against mold) surface. For vertically cast specimens from group 2, since both sides are cast against the mold and are therefore smooth, the inductance measurement is done on both sides. The detailed measurement and k_v determination procedures are as follows:

- (1) As shown in Figure 6.2, 25 measurement points are arranged in a square matrix. Each pair of adjacent points is 25 mm apart, and the distance from each side measurement point to the edge of the specimen is 50 mm.

- (2) For each measurement point, the inductance is recorded in both the i and j directions to account for fiber orientation. The mean relative magnetic permeability is then calculated. Finally, the mean values of relative magnetic permeability from all 25 measurement points are averaged to determine the overall mean $\mu_{r,\text{mean}}$ for the entire specimen.
- (3) After measuring all UHPFRC plates in Groups 1 and 2, the relationship between $\mu_{r,\text{mean}}$ and fiber content (ranging from 0% to 4%) is plotted. The proportionality k_v can then be determined as detailed in Section 6.2.2.



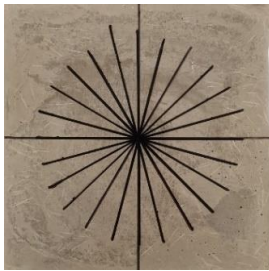
(a)



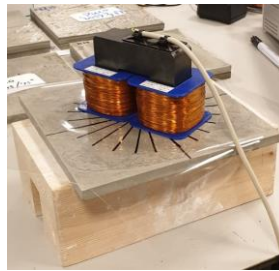
(b)

Figure 6.2 Fiber distribution measurement: (a) schematic view; (b) test image.

For fiber orientation measurements, the U-shaped ferrite magnetic probe is used. As shown in Figure 6.3, the inductance is measured by aligning the probe with the measuring direction starting at 0° (i.e., the i direction) and rotating in 15° increments until a full 360° rotation is completed. This results in a total of 25 measurements per specimen. Theoretically, the inductance value at an angle θ should be the same as at $\theta+180^\circ$. Using equation (6.2), $\mu_{r,i}$ and $\mu_{r,j}$ are calculated, and the fiber orientation indicator $\rho_{i,j}$ is determined using equation (6.5). Additionally, the k_v value for the U-shaped ferrite magnetic probe is also determined when measuring UHPFRC plates with varying fiber contents (0-4%).



(a)



(b)

Figure 6.3 Fiber orientation measurement: (a) measurement arrangement (angles and direction); (b) test image.

Sensitivity analysis of inductance measurement

The edge effect can occur when a measurement point is near the edge of the specimen. In such cases, the magnetic field generated by the probe extends beyond the UHPFRC specimen, which can reduce the inductance value if there is no ferromagnetic material outside the measurement area. To ensure reliable measurements, as recommended by Nunes et al. [41], the minimum edge distance for both probes must be determined. The testing procedures in this study involve the following steps:

- (1) Two plates (V2-2 and V4-2 plates) are used. The I-shaped and U-shaped probes are placed at the center of each plate to measure their initial inductance values.
- (2) Subsequently, as shown in Figure 6.4, 10 mm of UHPFRC is removed from the edge of the specimen in 6 steps, reducing the plate size from 200 mm to 80 mm. Inductance is measured after each removal.
- (3) Once the measured inductance starts to decrease, it indicates the onset of the edge effect and the minimum edge distance can be determined.

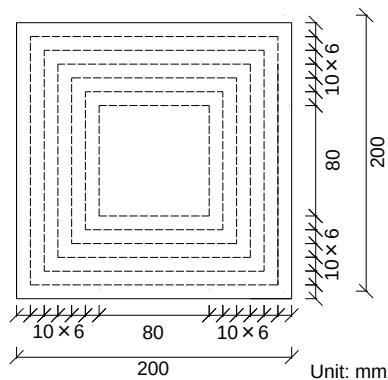


Figure 6.4 Test protocol to study the edge effect and determine minimum edge distance of the probes.

The surface roughness affects the accuracy of the measurement since it reduces the contact area between probes and the UHPFRC plates. A lower inductance value is measured for a rougher surface and such unfavorable effect is confirmed by Nunes, et.al [41]. In the current study, the effect of surface roughness is studied by testing two horizontally cast UHPFRC plates (H1-2 and H3-2 specimens). For horizontally cast UHPFRC specimens, the cast surface is generally rougher than the cast against mold surface. The cast surface is smoothened by a sandpaper. Therefore, for one plate, the inductance measurement is conducted on cast, sanded and smooth (cast against mold) surface.

The white paint (coating) on the UHPFRC surface may alter the magnetic permeability due to its different magnetic properties, potentially affecting the inductance measurement. Four plates with different fiber contents (V1-2, V2-1, V3-1, V4-1) are selected to measure

inductance at 45° intervals. Firstly, inductance is measured for each specimen without paint. Subsequently, as shown in Figure 6.5, a thin layer of white painting is applied on the surface of the specimen and inductance is measured again. Finally, by comparing inductance measurements before and after painting, the effect of white paint is analyzed.



Figure 6.5 Test process to study the effect of white painting: (a) specimens painted in white; (b) inductance measurement.

Parameters influencing fiber distribution and orientation

The casting direction can alter the flow pattern of fresh UHPFRC, causing fibers to align with the flow direction and affect the fiber dispersion and orientation. In this study, as shown in Figure 6.6, two types of molds, are prepared for horizontal and vertical casting of UHPFRC. The I-shaped and U-shaped probes are then used to measure inductance in specimens from Group 1 (horizontally cast) and Group 2 (vertically cast). The fiber distribution and orientation for all specimens are analyzed and compared to assess the impact of the casting direction.

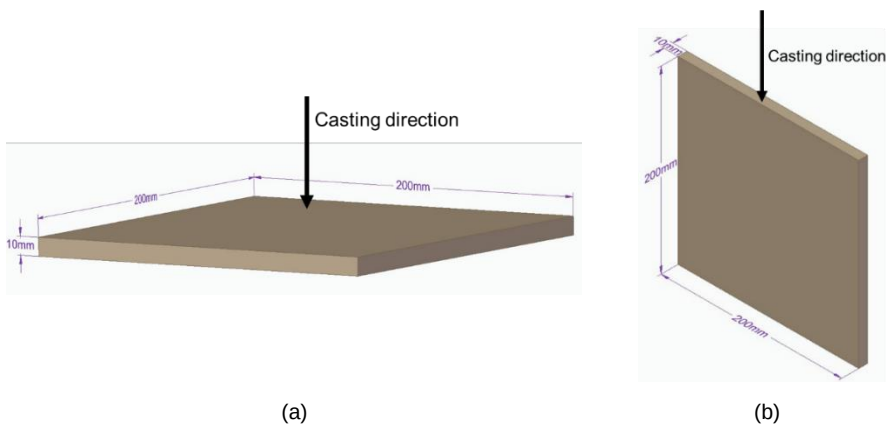


Figure 6.6 UHPFRC casting direction: (a) Horizontal casting; (b) vertical casting.

Vibration during casting can affect the fiber distribution of fresh UHPFRC, causing fibers to concentrate at the bottom of the specimen and potentially leading to segregation.

6.3 Experimental program

To quantify this effect, specimens in Group 3 are cast vertically and vibrated on a vibration table at 100 Hz for various time intervals (1 min, 2 min, 3 min, and 5 min). After hardening and demolding, the same testing procedures are used with the I-shaped and U-shaped probes to measure fiber distribution and orientation. The results are then compared to evaluate the impact of vibration.

Validation process

Since the electromagnetic method does not provide direct observation of steel fibers inside UHPFRC elements, x-ray imaging is performed on 10 specimens with different fiber contents to validate the reliability of the probe measurements. The fiber distribution and orientation from inductance measurements are compared with the fiber distribution observed through x-ray imaging. The V2-T5-1 specimen is also selected to examine fiber segregation due to vibration.

X-ray imaging provides the 2D distribution of fibers in UHPFRC elements, which is acceptable given that the fiber length (13 mm) exceeds the plate thickness (10 mm). However, overlaps of fibers in the 2D images hinder accurate calculation of the actual fiber content. Therefore, this study focuses on a qualitative comparison between probe measurements and x-ray images rather than a quantitative analysis of local fiber contents.

Prior to imaging, the UHPFRC plate is fixed by placing it vertically in a styrofoam holder and wrapping it with a plastic strip (Figure 6.7). The plastic strip is used because its low density allows x-rays to penetrate without creating artifacts. The holder with the sample is then positioned for scanning. A high resolution of 60 μm is used to distinguish the steel fibers, which have a diameter of 200 μm .



Figure 6.7 CT scanning of UHPFRC plate.

6.4 Test results and validation

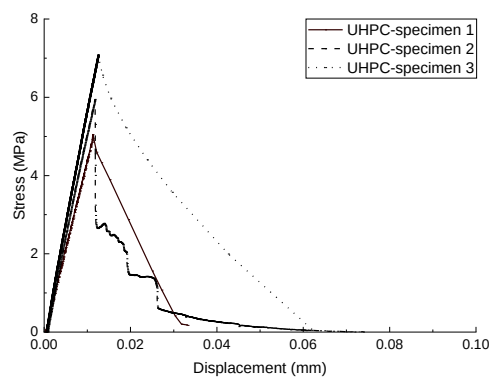
6.4.1 Mechanical properties of UHPFRC with different fiber volume

The material test results for UHPFRC with different amount of fibers, including compressive strength, tensile strength (first cracking f_{ct} and post-crack tensile strength f_{pt}), and cracking strain ε_{ct} and tensile strain capacity ε_{pt}) and E-modulus, at an age of 28 days are listed in Table 6.2. In Figure 6.8, the stress-displacement relationship under tension and the cracking pattern of UHPFRC mixtures with varied fiber contents are shown. When the fiber content is increased from 0 to 2%, the mechanical properties of UHPFRC are improved, in which the compressive strength, elastic modulus, peak tensile strength, and tensile strain capacity are increased by 39%, 23%, 88%, 488%, respectively. As expected, steel fibers have a most substantial influence on the tensile properties of UHPFRC. However, when the fiber content reaches a certain value (2% in current study), adding more steel fibers does not affect significantly the mechanical properties of UHPFRC, possibly due to the reduced workability and fiber agglomeration at high fiber volume. From Figure 6.8, it is also found that generally the tensile behavior of UHPFRC is governed by the amount of steel fibers. When the fiber content is lower than 1%, the tensile behavior of UHPFRC can be divided into two phases: linear elastic and strain softening phases (shown in Figure 6.8 (a) and (b)). In the initial stage, specimens exhibit linear elastic behavior until the cracking tensile strength is reached. Subsequently, UHPC (without steel fibers) experiences a brittle failure. Compared to UHPC, the low amount of steel fibers ($\leq 1\%$) enables UHPFRC a much-prolonged softening branch, therefore the fracture energy is significantly improved. When the fiber content is more than 2%, assuming uniform fiber orientation, after the linear elastic phase, the specimen exhibits a short stage of load increase as the cracks widen, due to the fiber bridging effect prior to the softening stage. The tensile behavior is nearly the same for UHPFRC with fiber contents ranging from 2 to 4%. In addition, for all UHPFRC, irrespective of the fiber content, the crack patterns under tension are the same, with the deformation being localized in a single crack.

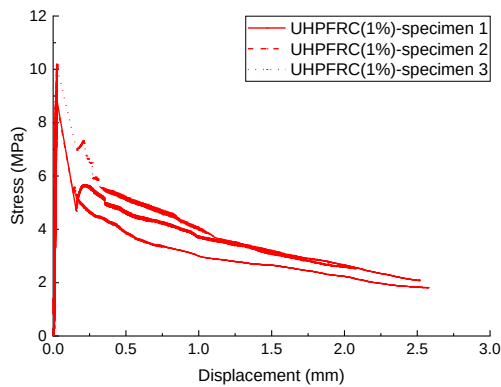
Table 6.2 Mechanical properties (average $\pm$ standard deviation) of UHPFRC with varied fiber content.

Mixture	Compressive strength (MPa)	Elastic modulus (GPa)	First cracking strength (MPa)	Cracking strain (%)	Peak tensile strength (MPa)	Tensile strain capacity (%)
UHPC	107.1 $\pm$ 13.2	35.5 $\pm$ 4.2	6.0 $\pm$ 1.0	0.017 $\pm$ 0.001	6.0 $\pm$ 1.0	0.017 $\pm$ 0.001
UHPFRC (1%)	132.9 $\pm$ 11.9	39.0 $\pm$ 5.2	9.4 $\pm$ 0.7	0.025 $\pm$ 0.004	9.4 $\pm$ 0.7	0.025 $\pm$ 0.004
UHPFRC (2%)	149.2 $\pm$ 5.4	43.7 $\pm$ 4.9	11.1 $\pm$ 0.9	0.026 $\pm$ 0.005	11.3 $\pm$ 0.6	0.10 $\pm$ 0.08
UHPFRC (3%)	155.5 $\pm$ 4.3	45.0 $\pm$ 2.8	9.1 $\pm$ 1.0	0.027 $\pm$ 0.003	12.1 $\pm$ 1.1	0.20 $\pm$ 0.09
UHPFRC (4%)	159.3 $\pm$ 4.6	45.4 $\pm$ 1.3	11.3 $\pm$ 0.5	0.025 $\pm$ 0.001	11.8 $\pm$ 0.7	0.14 $\pm$ 0.05

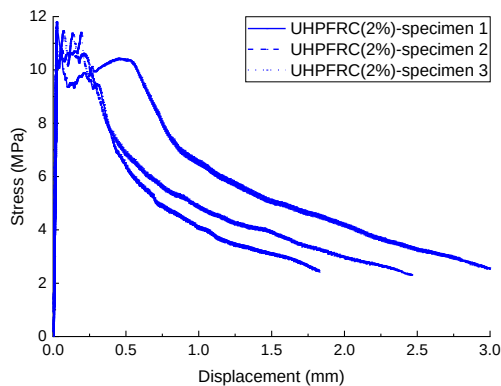
6.4 Test results and validation



(a)



(b)



(c)

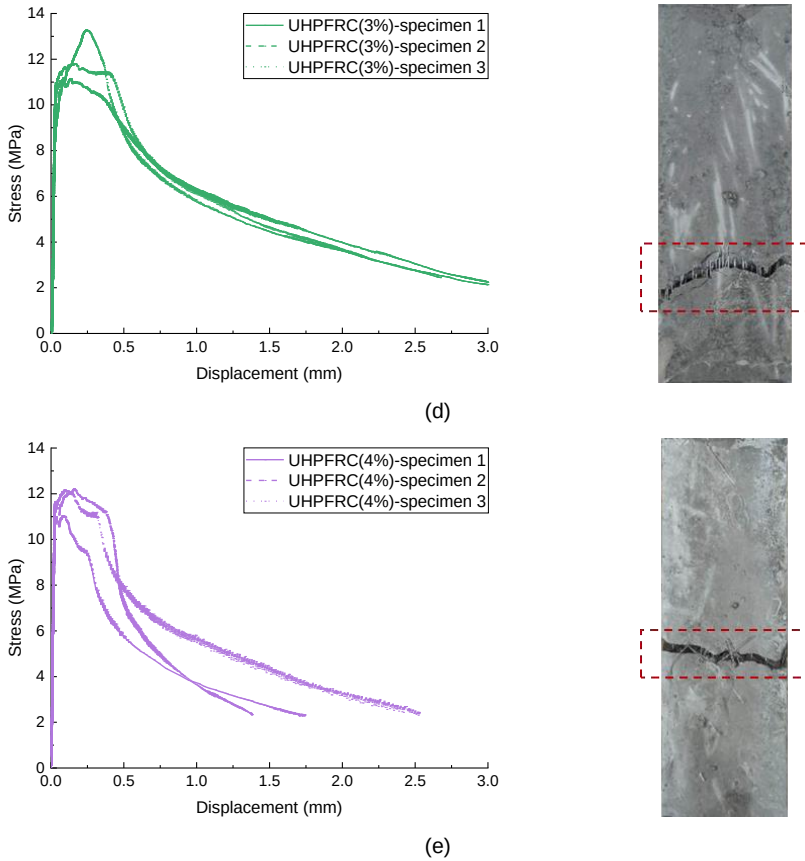


Figure 6.8 Stress-displacement behavior of UHPFRC with varied fiber content (left) with crack patterns (right): (a) UHPC; (b) UHPFRC (1%); (c) UHPFRC (2%); (d) UHPFRC (3%); (e) UHPFRC (4%).

6.4.2 Fiber content and k_v factor calibration

As described in Section 6.2.2 and Section 6.3.2, k_v is a calibration constant and can be determined based on UHPFRC plates with different fiber contents. The results are plotted in Figure 6.9. It is found that a fitted linear equation $\mu_{r,\text{mean}} = 2.6082V_f + 1$ best represents the relationship between the mean relative magnetic permeability ($\mu_{r,\text{mean}}$) and the fiber content V_f (ranging from 0 to 4%). It should be mentioned that the highest variation is found in 2% UHPFRC plates, which might be caused by an uneven fiber distribution during casting due to the relatively high fluidity of the mixture.

6.4 Test results and validation

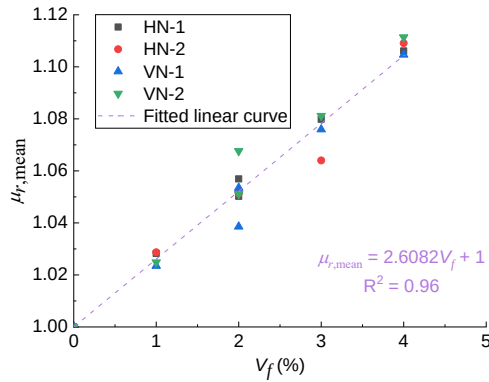
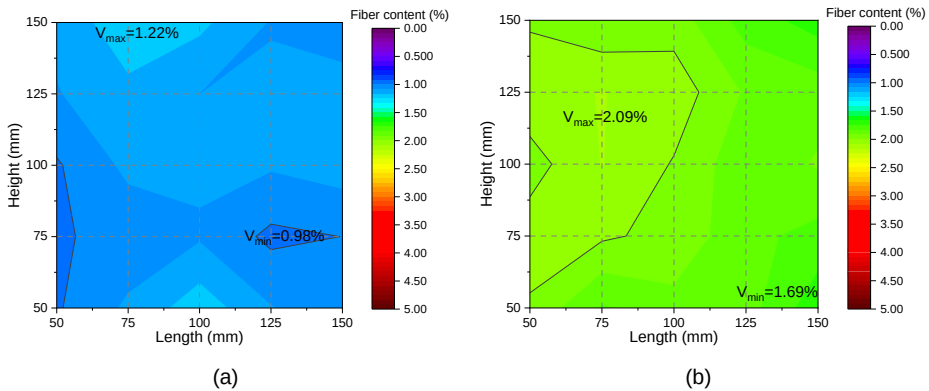


Figure 6.9 k_v factor for the I-shaped probe.

After determining k_v factor, the local fiber contents for each specimen can be calculated. Results for UHPFRC plates of varying fiber contents, cast horizontally, are shown in Figure 6.10. For example, the H1-1 specimen theoretically has 1% fiber content, but the local fiber content varies, with a maximum of 1.22% and a minimum of 0.98% (Figure 6.10 (a)). It is observed that as the fiber content increases, the variation in local fiber content also grows, from 0.24% to 0.93%. This is likely because higher fiber content reduces UHPFRC flowability, increasing the potential for fiber agglomeration.



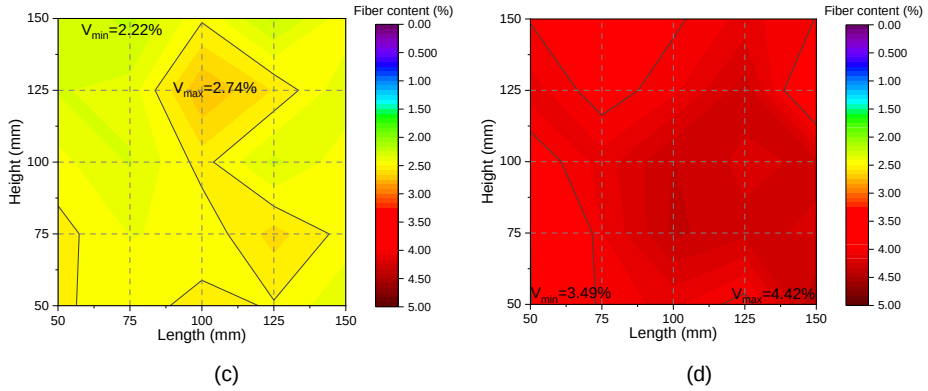


Figure 6.10 Fiber content contour plots for plate with different fiber content: (a) H1-1; (b) H2-1; (c) H3-2; (d) H4-1.

6.4.3 Determination of fiber orientation

As described earlier, $\rho_{i,j}$ is fiber orientation indicator to reflect the preferential fiber orientation. With the use of U-shaped probe and following the measurement procedures from Section 6.3.2, the fiber orientation for each specimen can be determined. Results of $\mu_{r,i}$ and $\rho_{i,j}$ at varied measurement angles are shown in Figure 6.11. Figure 6.11 (a) shows that, regardless of fiber content in UHPFRC plates, the $\mu_{r,i}$ values measured at every angle for the same plate are nearly identical, indicating a random fiber orientation. This is further supported by the $\rho_{i,j}$ results in Figure 6.11 (b), where the $\rho_{i,j}$ values are relatively small (within 12%). As described in Section 6.2.2, a more random fiber orientation corresponds to a $\rho_{i,j}$ value closer to 0, although a slight preferential fiber orientation is noted at 60° for the H3-1 specimen. As the fiber content increases, the measured $\mu_{r,i}$ value also increases, since more fibers lead to a higher inductance.

6.4 Test results and validation

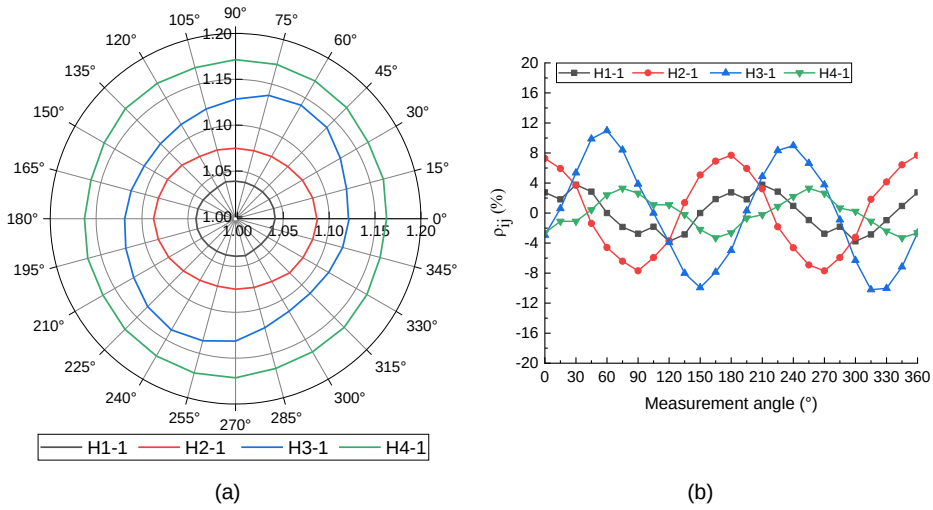


Figure 6.11 Fiber orientation indicators in UHPFRC plates with different fiber contents: (a) $\mu_{r,i}$ and (b) $\rho_{i,j}$ at varied measurement angle.

6.4.4 Sensitivity analysis of inductance measurement

The edge effect

Figure 6.12 shows the changes in $\mu_{r,\text{mean}}$ and $\Delta\mu_{r,\text{mean}} / \Delta l$ with reduced UHPFRC element size for I-shaped and U-shaped probes. As seen in Figure 6.12 (a), the measured $\mu_{r,\text{mean}}$ using the I-shaped probe remains nearly constant (only a 0.3% reduction) when the specimen size decreases from 200 mm to 100 mm, indicating a minimum edge distance of 50 mm. Figure 6.12 (b), the measured $\mu_{r,\text{mean}}$ with the U-shaped probe remains stable (less than a 1% decrease) until the specimen size is reduced to 120 mm, after which $\mu_{r,\text{mean}}$ starts to decrease significantly, reflecting a minimum edge distance of 60 mm. The U-shaped probe requires a larger minimum edge distance compared to the I-shaped probe due to its larger measurement area.

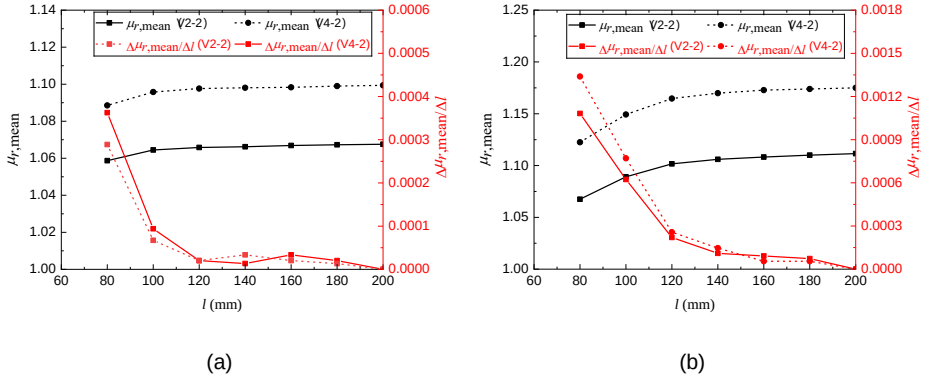


Figure 6.12 The variation of $\mu_{r,mean}$ and $\Delta\mu_{r,mean}/\Delta l$ with the reduced size of UHPFRC element: (a) I-shaped probe; (b) U-shaped probe.

The surface roughness

Inductance measurements for the H1-2 and H3-2 plates are conducted on three surface conditions: cast (rough) surface, sanded surface, and smooth surface. Figure 6.13 (a) reveals that fibers are oriented randomly in all three surface conditions, with the plotted curves being nearly circular. However, the value is significantly lower for the rough surface compared to the smooth surface, due to reduced probe contact and lower magnetic intensity, as well as possible fiber segregation along the thickness of the plates. Manual smoothening by sand paper had minimal effect, unlike by using specialized grinding equipment as performed in [41]. Figure 6.13 (b) shows that fiber contents are significantly lower under the rough condition—by 54% and 31% for H1-2 and H3-2 plates, respectively. This indicates that surface roughness greatly affects fiber content measurements.

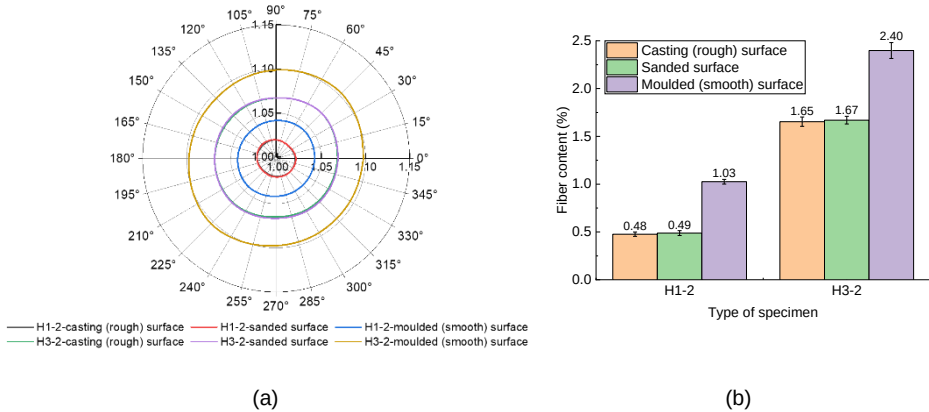


Figure 6.13 The effect of surface roughness: (a) $\mu_{r,i}$ at varied measurement angle, (b) measured fiber contents.

The white paint (coating)

Figure 6.14 shows the measured fiber contents for four UHPFRC plates with and without painting. The average fiber content slightly decreases (by less than 4%) after painting, indicating that the effect of the white paint is minimal. If a thicker non-ferromagnetic layer is present between the UHPFRC plates and the probes, the impact could be more significant, as noted in [226], where a 90% attenuation is observed with a distance greater than 24 mm between the UHPFRC and probe.

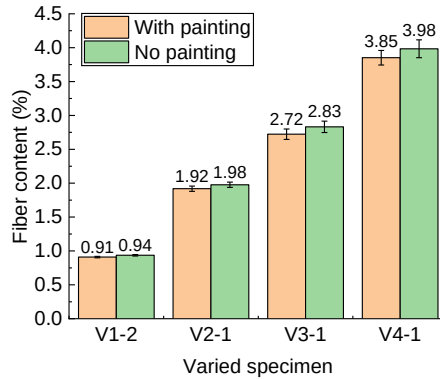


Figure 6.14 The effect of white painting.

6.4.5 Factors affecting fiber distribution and orientation

The effect of casting direction

Figure 6.15 shows the local fiber content for eight UHPFRC plates cast either horizontally or vertically with varying fiber contents. For horizontally cast plates (Figure 6.15 (a), (b), (c), (d)), the fiber content varies randomly across different regions, with no clear trend as the fiber content increases. This randomness is mainly due to the placement position of the fresh UHPFRC during casting.

In vertically cast plates (Figure 6.15 (e), (f), (g), (h)), the fiber content locations are also random when fiber content is below 2%. However, for fiber contents above 2%, fibers tend to accumulate more at the top of the specimens. This accumulation may be due to fibers agglomeration, caused by the plate thickness being smaller than the fiber length. Additionally, for both casting directions, the difference between $V_{\max}$ and $V_{\min}$ becomes more pronounced when the fiber content exceeds 2%, likely due to reduced flowability and fiber segregation.

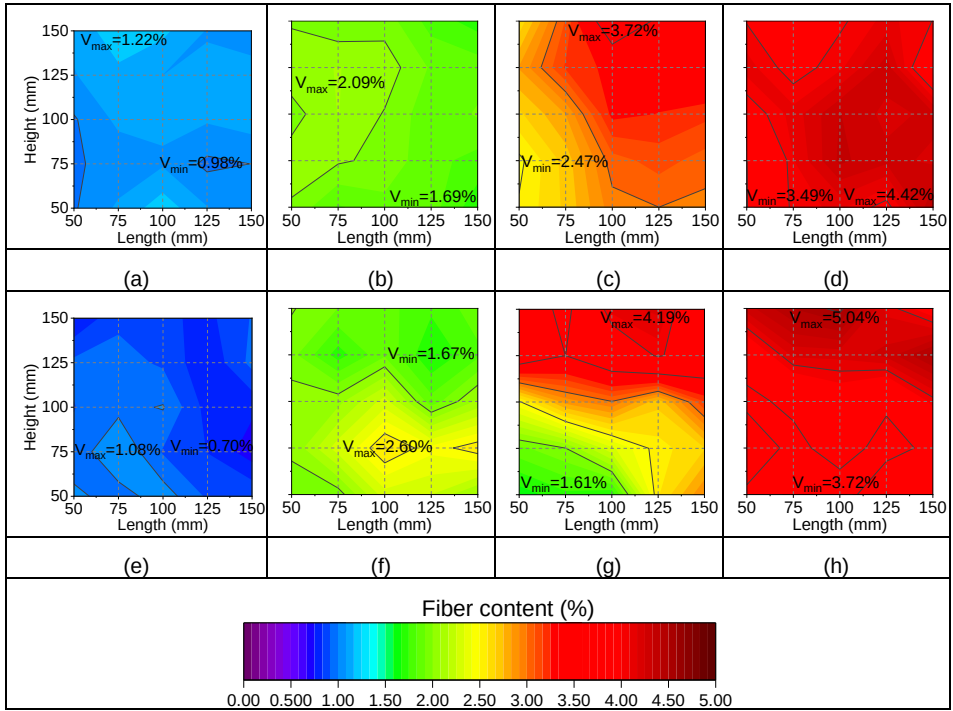


Figure 6.15 Fiber content contour plots for horizontally cast plates (top) and vertically cast plates (bottom) with increasing fiber volume from left to right: (a) H1-1; (b) H2-1; (c) H3-1; (d) H4-1; (e) V1-1; (f) V2-1; (g) V3-1; (h) V4-2.

Figure 6.16 presents the effect of casting direction on fiber orientation for the eight UHP-FRC plates. The results show that, regardless of the casting direction, the value $\mu_{r,i}$ varies slightly at different measurement angles, with the curve resembling a circle. The maximum value $\rho_{i,j}$ is below 10% for all specimens except H3-1, where $\rho_{i,j} \approx 12\%$ and fibers are oriented preferentially at 60° . Thus, the casting direction has a negligible impact on fiber orientation.

6.4 Test results and validation

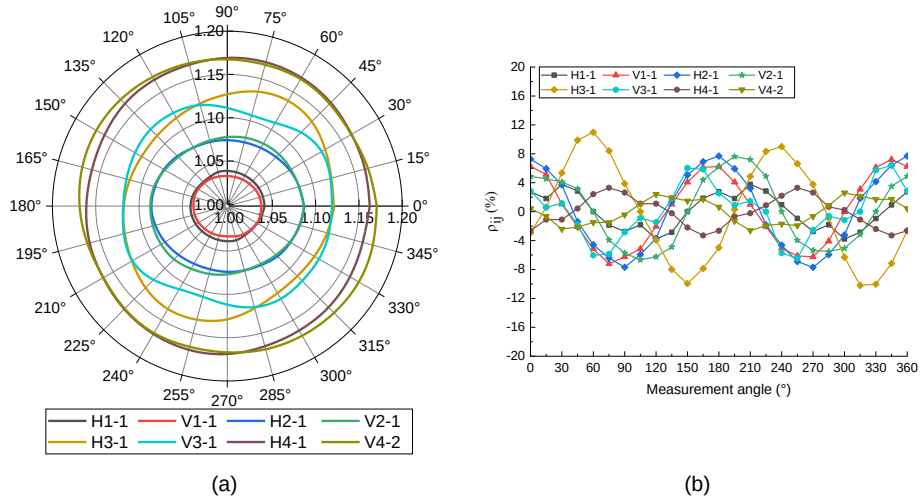
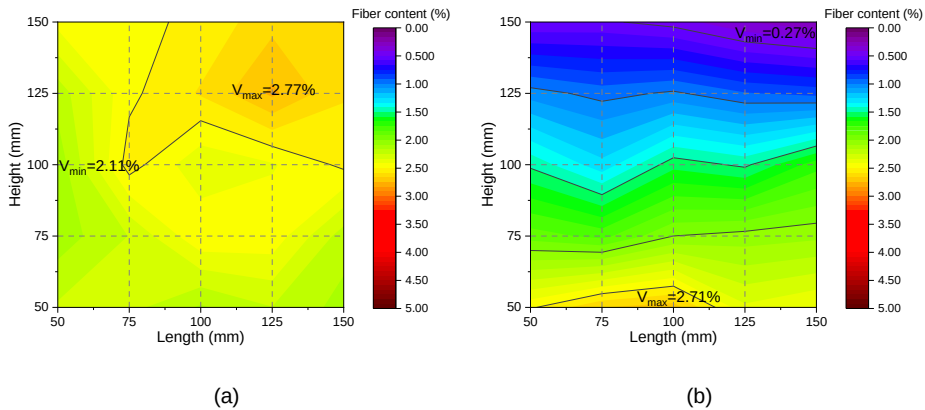


Figure 6.16 Fiber orientation plots for plates with different fiber content and casting direction: (a) $\mu_{r,j}$ and (b) $\rho_{i,j}$ at varied measurement angle.

The effect of vibration

Figure 6.17 shows the fiber content distribution for four vertically cast 2% UHPFRC plates with varying vibration time. Vibration time significantly affects fiber distribution. With one minute of vibration, the fibers remain randomly distributed (Figure 6.17 (a)). However, with vibration time exceeding 2 minutes, noticeable fiber segregation occurs. Most fibers, due to their higher density, sink to the bottom of the plates. As vibration time increases, segregation becomes more pronounced, with fiber content increasing at the bottom and larger areas showing fiber content as low as 0-0.5%.



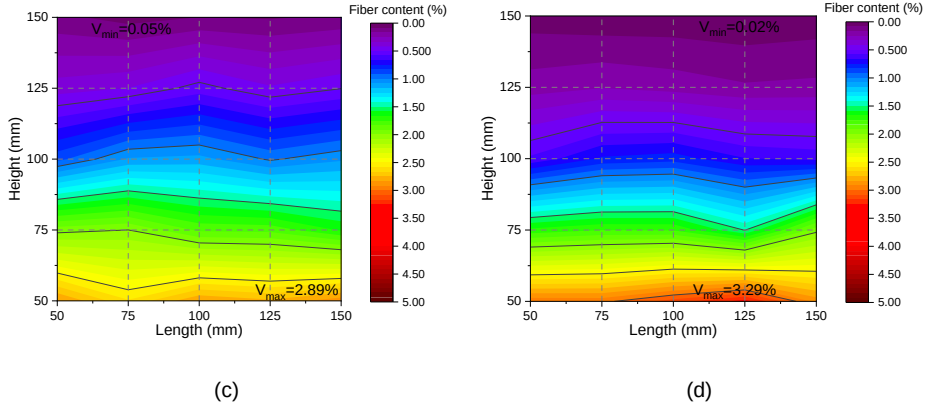


Figure 6.17 Fiber content contour plots for plates with 1min, 2min, 3min and 5min duration of vibration: (a) V2-T1-1; (b) V2-T2-1; (c) V2-T3-1; (d) V2-T5-1.

The fiber orientation results for those four UHPFRC plates with different vibration time are also presented (Figure 6.18). The increase of vibration time greatly decrease the magnitude of $\mu_{r,i}$ at different measurement angle and $\mu_{r,i}$ is highest at 90° (or 270°) and lowest at 0° (or 180°) (Figure 6.18 (a)). This is due to the fiber segregation at the bottom of the elements. As shown in Figure 6.19 (a), at the 90° , the U-shaped probe is closest to the fiber segregation area among all measurement angles, and therefore the inductance intensity is higher and results in a larger $\mu_{r,i}$. On the contrary, the farthest distance to fiber segregation area at 0° (or 180°) leads to the lowest $\mu_{r,i}$ among all measurement angles (Figure 6.19 (b)). This is further confirmed by the $\rho_{i,j}$ results shown in Figure 6.18 (b). $\rho_{i,j}$ becomes larger as the vibration time increases and the highest $\rho_{i,j}$ value is obtained at 90° or 270° . But it should be noticed that the high $\rho_{i,j}$ value is caused by fiber segregation but does not indicate that the fibers have a preferential orientation at 90° or 270° . For vibrated specimens, x-ray imaging will additionally be used to evaluate the fiber orientation.

6.4 Test results and validation

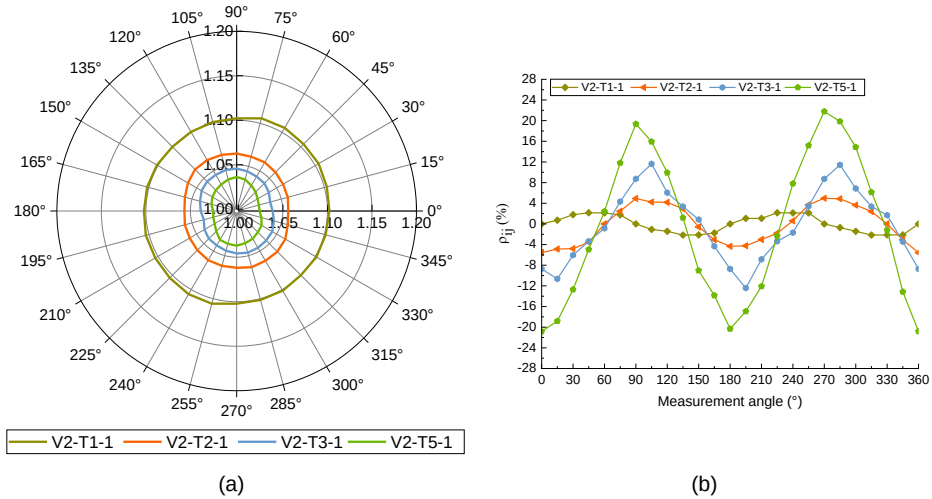


Figure 6.18 Fiber orientation plots for vertically cast 2% UHPFRC plates with different vibration time: (a) $\mu_{r,i}$ and (b) $\rho_{i,j}$ at varied measurement angle.

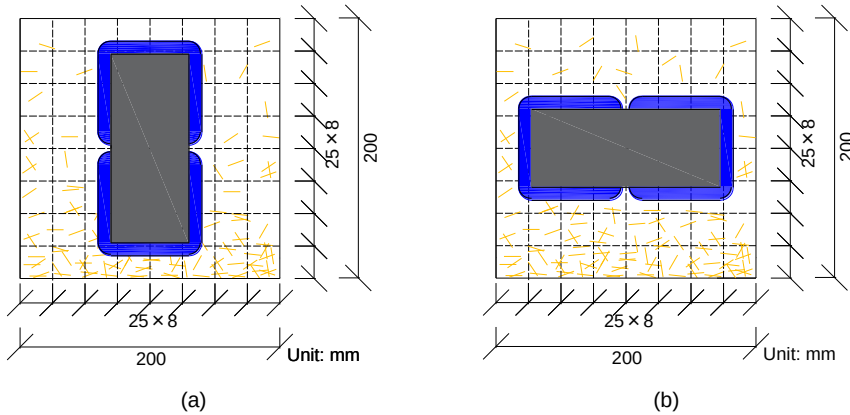
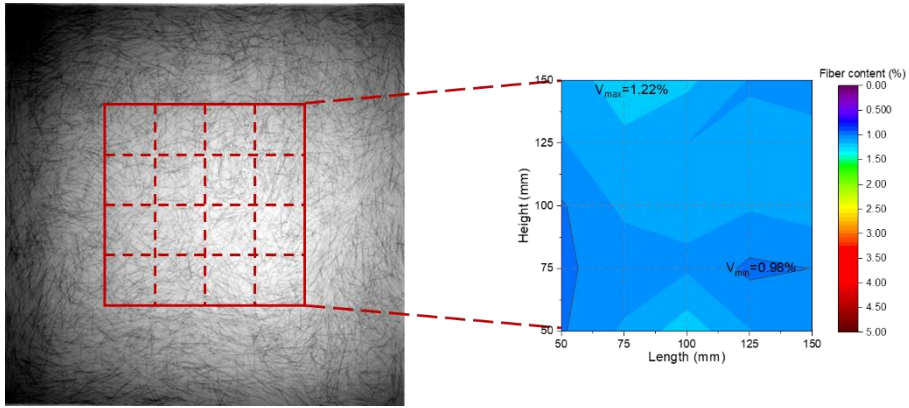


Figure 6.19 $\mu_{r,i}$ for vertically cast 2% UHPFRC plates with fiber segregation: (a) $\mu_{r,i}$ at 90° or 270° (b) $\mu_{r,i}$ at 0° or 180°.

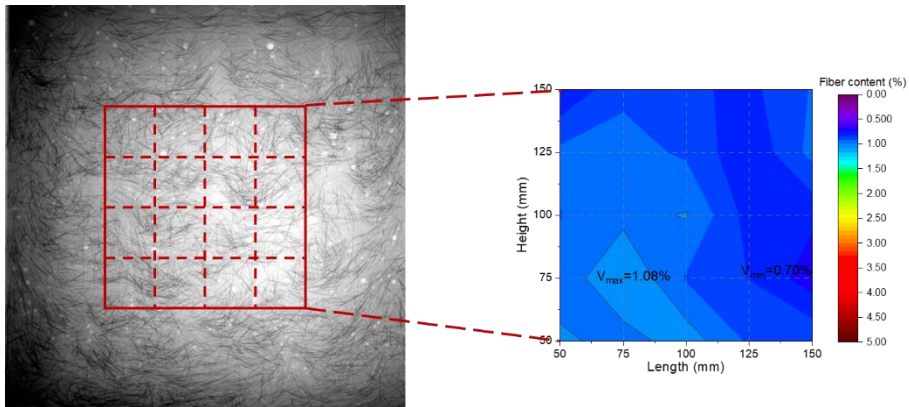
6.4.6 Validation by x-ray imaging

X-ray images and fiber content contour plots from the electromagnetic method for seven plates are compared (Figure 6.20). To avoid edge effects, measurements start 50 mm from the edge, focusing on the central 100 mm × 100 mm area of each plate. Generally, the magnetic probe measurements align well with x-ray imaging results. Fiber content and casting direction do not show significant impact on fiber dispersion.

For plates without vibration, H3-1 and V3-2 (Figure 6.20 (e) and (f)) exhibit significant differences in local fiber contents. Both x-ray images and contour plots based on electromagnetic method indicate that fibers are concentrated at the top, with fewer at the bottom. However, the most significant effect is from vibration. Figure 6.20 (g) shows that after five minutes of vibration, the V2-T5-1 plate exhibits severe fiber segregation, with most fibers settling at the bottom and almost none at the top.

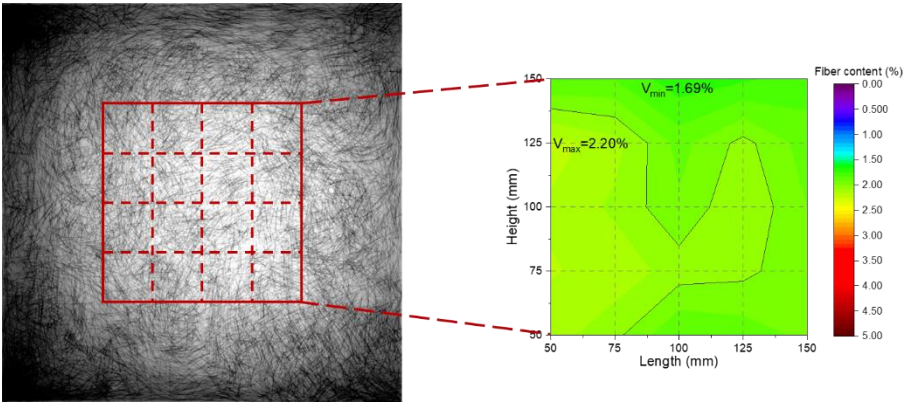


(a)

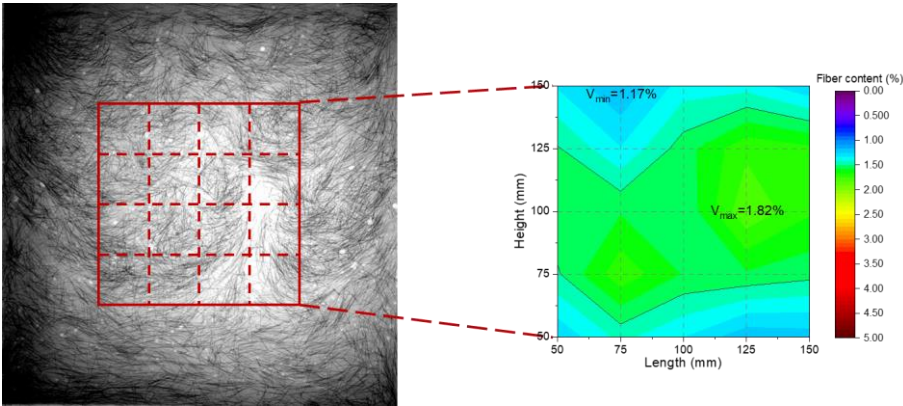


(b)

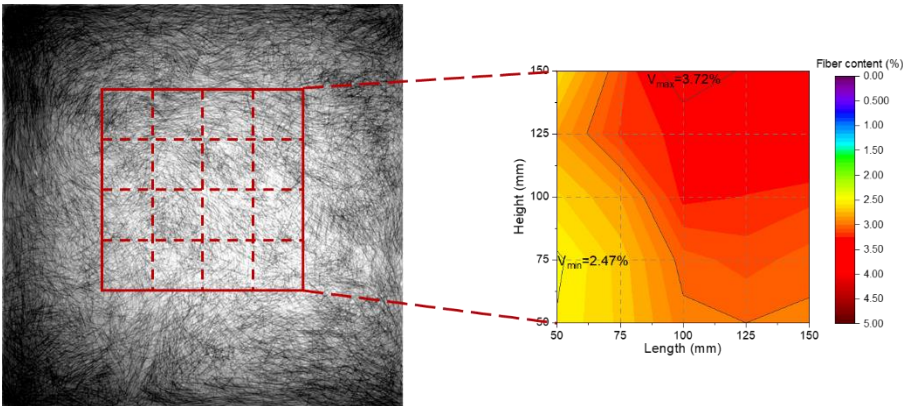
6.4 Test results and validation



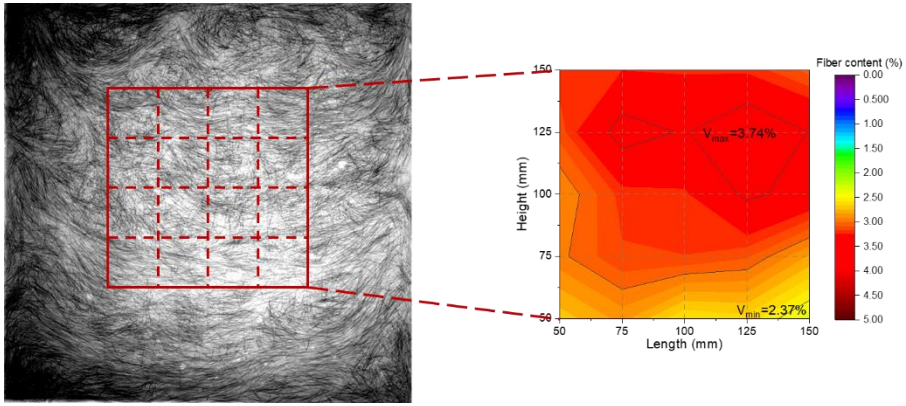
(c)



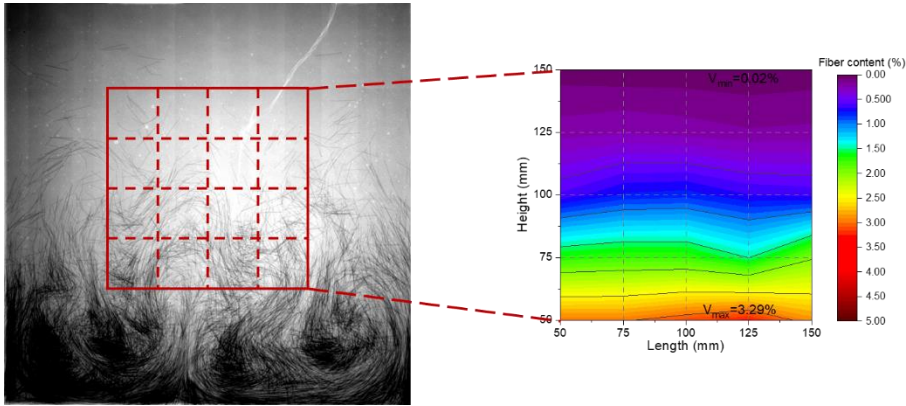
(d)



(e)



(f)



(g)

Figure 6.20 Fiber content contour plots for plates with different casting directions, fiber contents and vibration time: (a) H1-1; (b) V1-1; (c) H2-4; (d) V2-3; (e) H3-1; (f) V3-2; (g) V2-T5-1.

Due to 2D representation, x-ray images do not provide fiber contents. To reflect the local fiber distribution, fiber area percentages are calculated and compared. For example, for the V2-T5-1 specimen (Figure 6.21), the following steps are taken:

- (1) Higher resolution x-ray imaging is performed on the central top, middle, and bottom of the plates.
- (2) ImageJ software is used to enhance the contrast between fibers and the matrix.
- (3) The number of fibers (N) and the area of each fiber (A_f) are counted. Thus, the total area of fibers ($A_t = N \times A_f$) can be obtained. Given the known scanning area A_i , the fiber area percentage ($A_p = A_t / A_i$) can then be calculated for each scanning area.

6.4 Test results and validation

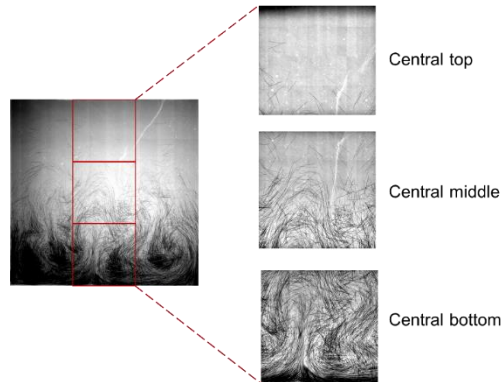


Figure 6.21 Individual CT scan of central top, middle and bottom part of V2-T5-1 plate and contrast enhancement through image J tool.

A comparison between area percentages (A_p) computed from x-ray data and local fiber contents (V_f) obtained from the electromagnetic measurements is shown in Table 6.3. Although x-ray images provide only fiber area percentages rather than exact local fiber contents, there is a correlation between the two. Generally, a higher fiber area percentage in a plate region corresponds to a higher local fiber content. For instance, in the V2-T5-1 specimen, the top region has only 1.5% fiber area, aligning with a fiber content of 0.03%, while the bottom region shows a significant 61% fiber area, corresponding to a fiber content of 3.05%.

Table 6.3 Comparison between area percentages from CT results and local fiber content from electromagnetic method for seven tested specimens.

Specimen name	Central top		Central middle		Central bottom	
	Area percentages (%)	Fiber contents (%)	Area percentages (%)	Fiber contents (%)	Area percentages (%)	Fiber contents (%)
H1-1	33.2	1.13	30.9	1.07	28.6	1.08
V1-1	26.6	0.85	32.0	0.90	31.5	0.97
H2-4	42.8	1.77	41.7	2.04	40.2	1.96
V2-3	36.4	1.30	37.1	1.59	31.4	1.29
H3-1	51.4	3.26	46.1	3.04	47.0	2.83
V3-2	50.1	3.19	48.8	3.26	45.7	2.63
V2-T5-1	1.5	0.03	15.1	0.88	61.0	3.05

Based on the test results in Table 6.3, an exponential equation is proposed to establish the relationship between the A_p and V_f , as shown in Figure 6.22. The proposed equation is described as follows:

$$V_f = 0.27(e^{0.05A_p} - 1) \quad (6.9)$$

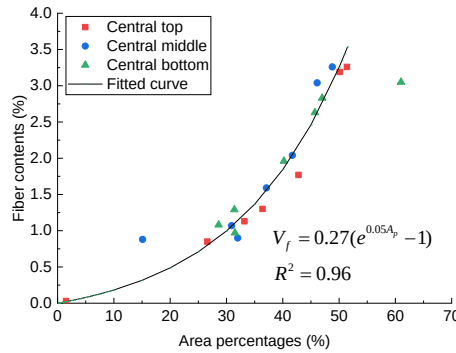


Figure 6.22 Comparison of the test results and predicted results by equation (6.9).

6.5 Beam test and discussion

After calibration of magnetic probes and validation of its effectiveness to measure the fiber dispersion inside the specimens, the magnetic method is applied to measure the fiber distribution in hybrid beams, especially the beams tested after FT cycles. However, it should be noted that longitudinal reinforcement in the strengthened beams may influence the inductance measurements, as fiber content was calibrated on UHPFRC plates. Given that the effective penetration depth of magnetic probes developed in [227,228] was 25 mm whereas the reinforcement is placed at the depth of (30mm) from the surface, probably the influence is negligible. However, this should be investigated in future for the specific probes developed at TU Delft.

6.5.1 Measurement layout for hybrid beam

Figure 6.23 illustrates the layout for measuring fiber distribution in beam specimens. Due to the larger size of the UHPFRC panels ($1400 \times 200 \times 10 \text{ mm}^3$) compared to the smaller UHPFRC plates ($200 \times 200 \times 10 \text{ mm}^3$), measurements are taken 50 mm vertically and 100 mm horizontally for beam specimens. Although a total of 42 measurement points is performed, only the 30 points within the effective span (highlighted by the pink rectangle in Figure 6.23) are critical, as these points affect the shear behavior of the beams. The measurement procedures for these points follow the same methods used for the smaller UHPFRC plates, as described in Section 6.3.2.

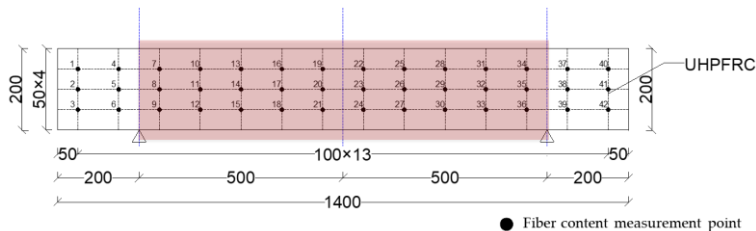
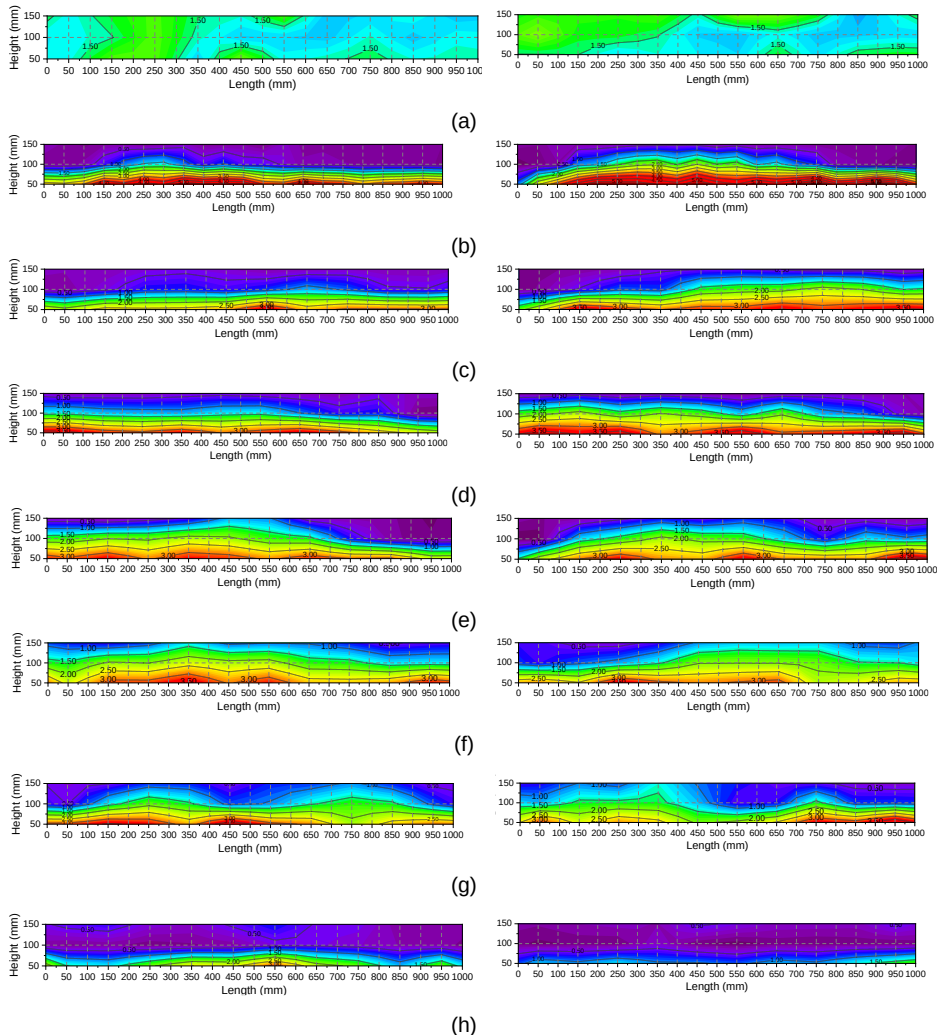


Figure 6.23 Layout of fiber distribution measurement for beam specimens (unit: mm).

6.5.2 Beam test results

Figure 6.24 shows the fiber distribution within the effective span for CIS and EB beams subjected to freeze-thaw action, as well as the horizontally-cast ST-M2-EB-HC beam. For each beam, results are shown for two lateral UHPFRC plates.

In the vertically-cast beam specimens, noticeable fiber segregation is observed due to vibration during casting, with most fibers settling at the bottom. In contrast, the ST-M2-EB-HC beam exhibits a more uniform fiber distribution throughout both the length and height, with a global fiber content of around 1.5% and local fiber content ranging between 1% and 2%. The minimal variation in local fiber content indicates little to no segregation in horizontally cast lamellas in ST-M2-EB-HC specimen.



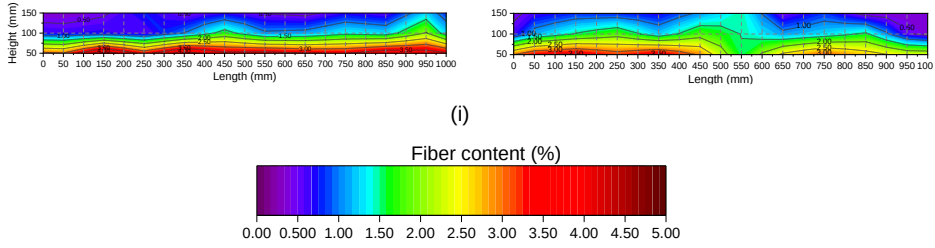


Figure 6.24 Local fiber distribution for different tested specimen: (a) ST-M2-EB-HC; (b) ST-M2-EB; (c) ST-M2-EB-30; (d) ST-M2-EB-75; (e) ST-M2-EB-120; (f) ST-M2-CIS; (g) ST-M2-CIS-30; (h) ST-M2-CIS-75; (i) ST-M2-CIS-120.

6.5.3 Discussion

As discussed in Section 6.5.2, the degree of fiber segregation varies among specimens. To quantify this segregation, the fiber distribution across all specimens is analyzed. The contour plots in Figure 6.24 show that fibers predominantly segregate in the height direction.

To measure this, the local fiber content along the height of cast-in situ and epoxy-bonded strengthened beams is averaged, as detailed in Table 6.4 and Table 6.5, respectively. A method to quantify the fiber segregation degree is used, also referred to as the fiber distribution coefficient [39,229].

$$\varphi(x) = \sqrt{\frac{\sum (x_i / \bar{x} - 1)^2}{n}} \quad (6.10)$$

$$\alpha = e^{-\varphi(x)} \quad (6.11)$$

Where α is the fiber distribution coefficient (segregation degree); $\varphi(x)$ is the coefficient of variation; x_i is the fiber content at each height of the UHPFRC specimen, in %; $\bar{x}$ is the average content of the fiber, in %; n is the total number of measurement points along the height (e.g., $n = 3$ in current research).

The segregation degree for each UHPFRC panel is calculated using Equations (6.10) and (6.11), with the results summarized in Table 6.6 and Table 6.7. Here, the values of α range from 0 to 1, where a lower α indicates more severe fiber segregation.

For the CIS beams, as presented in Figure 6.25 (a), the ST-M2-CIS-75 specimen exhibits the lowest α , indicating the most severe fiber segregation. This is in line with the lowest shear resistance for the strengthened beam (74.1 kN). Conversely, the ST-M2-CIS-30 specimen shows a relatively low α of 0.49 but achieves the highest shear resistance among the CIS beams.

For epoxy-bonded specimens, the shear capacity is primarily influenced by the fiber distribution within the UHPFRC laminates due to the strong bonding quality of the epoxy resin. As shown in Figure 6.25 (b), there is a strong correlation between the α value and the peak load of epoxy-bonded specimens, with a higher α resulting in a higher peak load. The ST-M2-EB-75 and ST-M2-EB-120 specimens have similar α value, resulting in only a slight difference in peak loads (around 4%). Additionally, due to different casting directions, ST-M2-EB shows the lowest α and peak load, while ST-M2-EB-HC shows the highest α and peak load. Thus, a uniform fiber distribution is critical for enhancing the shear resistance of UHPFRC-RC structures, highlighting its importance in strengthening RC structures with UHPFRC.

Table 6.4 Fiber distribution along the height of cast in-situ specimen.

Height	Cast in-situ beam specimens							
	ST-M2-CIS		ST-M2-CIS-30		ST-M2-CIS-75		ST-M2-CIS-120	
	Panel 1	Panel 2	Panel 1	Panel 2	Panel 1	Panel 2	Panel 1	Panel 2
200	-	-	-	-	-	-	-	-
150	0.74%	0.61%	0.59%	0.51%	0.42%	0.31%	0.53%	0.70%
100	1.63%	1.22%	1.23%	0.90%	0.33%	0.14%	0.92%	1.20%
50	2.99%	3.02%	3.36%	3.07%	2.07%	1.15%	3.71%	2.46%
0	-	-	-	-	-	-	-	-

Table 6.5 Fiber distribution along the height of epoxy-bonded specimen.

Height	Epoxy-bonded beam specimens									
	ST-M2-EB		ST-M2-EB-30		ST-M2-EB-75		ST-M2-EB-120		ST-M2-EB-HC	
	Panel 1	Panel 2	Panel 1	Panel 2	Panel 1	Panel 2	Panel 1	Panel 2	Panel 1	Panel 2
200	-	-	-	-	-	-	-	-	-	-
150	0.21%	0.30%	0.25%	0.26%	0.32%	0.21%	0.44%	0.53%	1.45%	1.63%
100	0.34%	1.46%	0.79%	0.82%	0.78%	1.93%	1.22%	0.92%	1.38%	1.43%
50	4.10%	4.46%	3.15%	3.13%	3.03%	3.46%	3.25%	3.71%	1.53%	1.45%
0	-	-	-	-	-	-	-	-	-	-

Table 6.6 Segregation degree for cast in-situ specimens.

Specimen ID	Cast in-situ beam specimens							
	ST-M2-CIS		ST-M2-CIS-30		ST-M2-CIS-75		ST-M2-CIS-120	
	Panel 1	Panel 2	Panel 1	Panel 2	Panel 1	Panel 2	Panel 1	Panel 2
Panel ID								
$\bar{x}$ (%)	1.79	1.62	1.73	1.49	0.94	0.53	1.72	1.45
$\phi(x)$	0.52	0.63	0.69	0.75	0.85	0.83	0.82	0.51
α	0.60	0.53	0.50	0.47	0.43	0.44	0.44	0.60
$\bar{\alpha}$	0.56		0.49		0.43		0.52	
Peak load (kN)	83.7		105.3		74.1		88.7	

Table 6.7 Segregation degree for epoxy-bonded specimens.

Epoxy-bonded beam specimens										
Specimen ID	ST-M2-EB		ST-M2-EB-30		ST-M2-EB-75		ST-M2-EB-120		ST-M2-EB-HC	
Panel ID	Panel 1	Panel 2	Panel 1	Panel 2	Panel 1	Panel 2	Panel 1	Panel 2	Panel 1	Panel 2
$\bar{x}$ (%)	1.55	2.07	1.40	1.40	1.38	1.87	1.64	1.72	1.45	1.50
$\varphi(x)$	1.16	0.85	0.90	0.89	0.86	0.71	0.72	0.82	0.04	0.06
α	0.31	0.43	0.41	0.41	0.42	0.49	0.48	0.44	0.96	0.94
$\bar{\alpha}$	0.37		0.41		0.46		0.46		0.95	
Peak load (kN)	90.1		99.5		108.9		105.2		133.1	

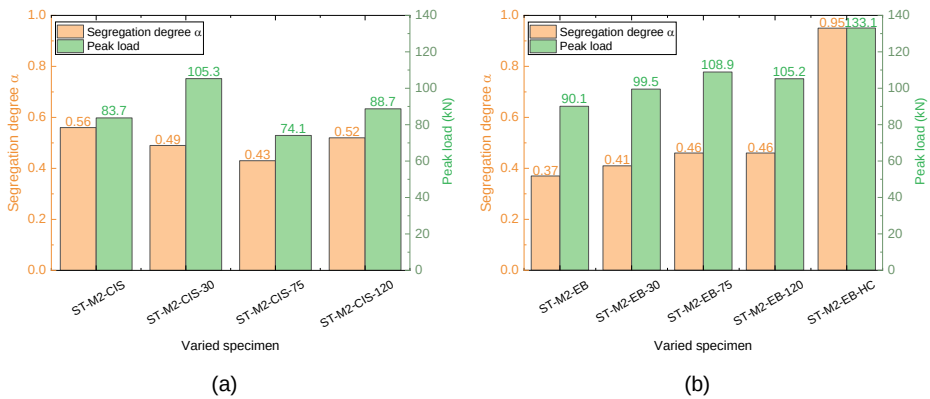


Figure 6.25 Comparison of the segregation degree and the peak load: (a) Cast in-situ specimens; (b) epoxy-bonded specimens.

6.6 Analytical methods for mechanical properties of UHPFRC

Since fiber distribution significantly influences the material properties of UHPFRC, it is necessary to predict the mechanical response of UHPFRC, especially its compressive strength, elastic modulus, as well as the tensile behavior, based on fiber content and orientation. In the current study, due to small thickness of the plate it can be assumed that the fibers are oriented randomly in 2D plane and therefore the mechanical properties mainly depend on fiber content of UHPFRC.

6.6.1 Compressive strength and elastic modulus

Compressive strength

UHPFRC consists of matrix and steel fibers, and its compressive strength can be considered as a linear combination of the contributions from both components. The impact of steel fibers on the compressive strength of UHPFRC is influenced by the fiber volume. When the fiber content is below 2%, the compressive strength increases with higher fiber

content. However, beyond 2%, the flowability is reduced due to large amounts of fibers, and the strength cannot be significantly improved. Consequently, as proposed in [230], the compressive strength of UHPFRC can be estimated using the following formula:

$$f_c = f_m + a_1 f_m \frac{l_f}{d_f} V_f - a_2 f_m \frac{l_f}{d_f} V_f^2 \quad (6.12)$$

where f_c is the compressive strength of UHPFRC; f_m is the compressive strength of the matrix; l_f and d_f are the fiber length and diameter, respectively; V_f is the fiber volume; a_1 and a_2 are two coefficients, which reflect the positive and negative influence of the fibers, respectively, and need to be determined by regression method.

Based on the test results in Table 6.2, as shown in Figure 6.26, the a_1 and a_2 are determined to be 0.41 and 5.68 by fitting analysis and the following equation is obtained:

$$f_c = f_m + 0.41 f_m \frac{l_f}{d_f} V_f - 5.68 f_m \frac{l_f}{d_f} V_f^2; 0 \leq V_f \leq 4\% \quad (6.13)$$

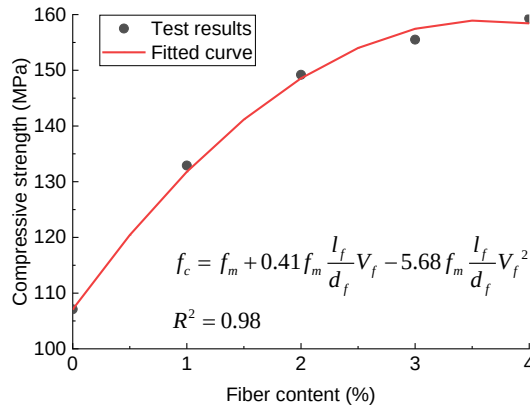


Figure 6.26 Comparison of the test results and predicted results by equation (6.13).

Elastic modulus

Previous studies [231] have shown that there is a correlation between elastic modulus and compressive strength of UHPFRC, and the elastic modulus of UHPFRC increases with the increase in compressive strength. Many empirical equations [230] have been proposed to predict the elastic modulus based on its compressive strength, typically following a similar functional form:

$$E_c = a_3 (f_c)^{a_4} \quad (6.14)$$

where E_c is the elastic modulus of UHPFRC. a_3 and a_4 are two coefficients and can be determined by fitting analysis based on the test results in Table 6.2 (Figure 6.27). The following equation is obtained:

$$E_c = 1.54(f_c)^{2/3} \quad (6.15)$$

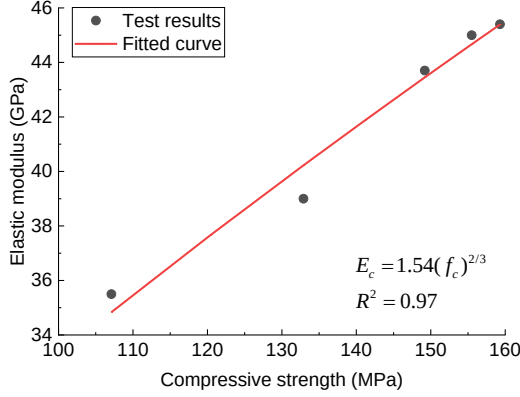


Figure 6.27 Comparison of the test results and predicted results by equation (6.15).

6.6.2 Tensile behavior

The fiber volume in UHPFRC significantly influences its mechanical behavior, leading to two types of responses: strain-softening (low fiber volume, e.g., less than 1% in this study) and strain-hardening (e.g., more than 2% in this study). To achieve strain-hardening behavior in UHPFRC, the minimum (critical) fiber volume ($V_{f,cr}$) must be met [232].

According to Fantilli et al. [233], the $V_{f,cr}$ can be calculated as follows:

$$V_{f,cr} = \frac{2A_f(k_c)^2}{E_c\rho_f k_B} \left(1 + \sqrt{1 + \frac{(E_c)^2 \rho_f k_B}{A_f E_s (k_c)^2}}\right) \quad (6.16)$$

where A_f and ρ_f represent the cross sectional area and perimeter of single fiber, respectively; E_c and E_s represent the E-modulus of matrix and fiber, respectively. k_B and k_c are bond and cohesive parameters, respectively; $k_B = 2550 \text{ MPa/mm}$ for straight steel fibers ($l_f = 13 \text{ mm}$, $d_f = 0.2 \text{ mm}$); k_c , defined as f_{ct} / w_c , equals to 500 MPa/mm in current study.

With the above input parameters, the calculated critical fiber volume is 1.75%. Therefore, only when $V_f \geq V_{f,cr}$ (1.75%) in current study, the strain hardening behavior can be activated. Two constitutive stress-crack opening models, which represent strain softening and strain hardening UHPFRC, SS-UHPFRC and SH-UHPFRC, respectively, are proposed. As shown in Figure 6.28, the tensile stress-strain relation until the peak tensile strength

6.6 Analytical methods for mechanical properties of UHPFRC

(f_{ct} for SS-UHPFRC and f_{pt} for SH-UHPFRC) and the tensile stress-crack opening relation at the softening stage can be described in the two stages for SS-UHPFRC and three stages for SH-UHPFRC as follows:

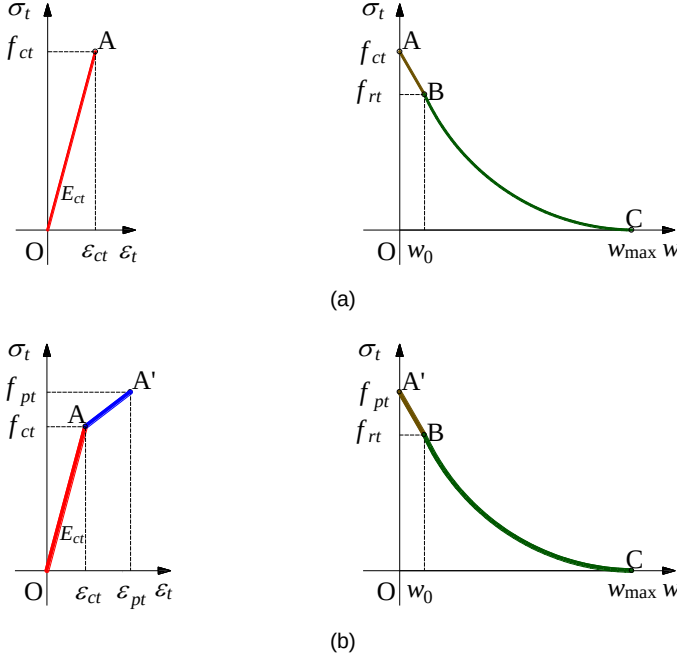


Figure 6.28 Two tensile constitutive models: (a) SS-UHPFRC; (b) SH-UHPFRC.

Stage I: Linear elastic (O-A): when the tensile strain is in the range of $\varepsilon \leq \varepsilon_{ct}$, the tensile stress-strain relation can be described as:

$$\sigma_t = \frac{f_{ct}}{\varepsilon_{ct}} \cdot \varepsilon_t \text{ or } E_{ct} \cdot \varepsilon_t \quad (6.17)$$

Where f_{ct} is the cracking tensile strength (also the peak tensile strength for SS-UHPFRC); ε_{ct} is the elastic limit tensile strain; E_{ct} is the tensile elastic modulus.

E_{ct} can be either calculated as $\frac{f_{ct}}{\varepsilon_{ct}}$ or proposed empirical equation (6.15). Based on the test results in Table 6.2, quadratic equations are proposed to establish the relationship between the V_f and ε_{ct} and the relationship between V_f and σ_{ct} . It should be noted that the fitted equation (6.19) excludes the test results for 3% UHPFRC due to its large discrepancy, and the proposed equations are described as follows:

$$\varepsilon_{ct} = \varepsilon_{mt} + 0.007707V_f - 0.001453V_f^2; 0 \leq V_f \leq 4\% \quad (6.18)$$

$$f_{ct} = f_{mt} + 3.90V_f - 0.65V_f^2; 0 \leq V_f \leq 4\% \quad (6.19)$$

Where f_{mt} is the matrix tensile strength and equals to 6 MPa; ε_{mt} is the matrix tensile strain limit and equals to 0.017% in current study.

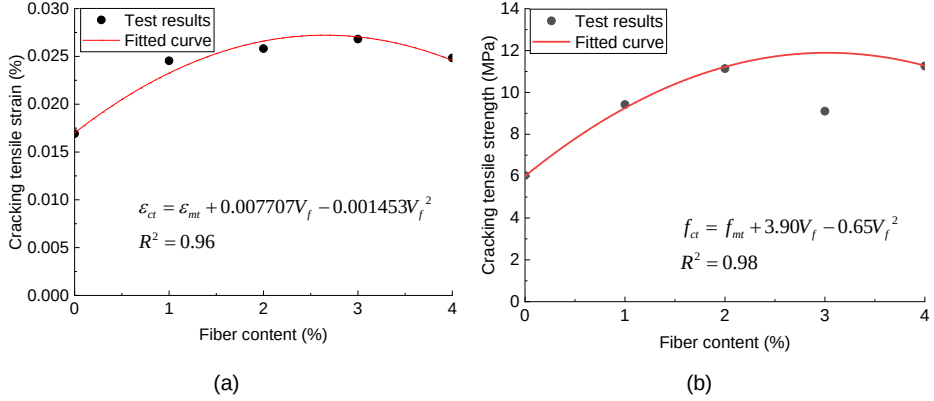


Figure 6.29 Comparison of the test results and predicted results by equation (6.18) and equation (6.19).

Stage II: Strain hardening stage (A-A' only for SH-UHPFRC): This stage involves a multiple cracking process and is described by linear stress-strain relation between cracking point (ε_{ct} , f_{ct}) and peak point (ε_{pt} , f_{pt}). Therefore, when $\varepsilon_{ct} < \varepsilon_t \leq \varepsilon_{pt}$, the tensile stress-strain relation can be described as:

$$\sigma_t = f_{ct} + \frac{f_{pt} - f_{ct}}{\varepsilon_{pt} - \varepsilon_{ct}} \cdot \varepsilon_t \quad (6.20)$$

Where f_{pt} is the peak tensile strength; ε_{pt} is the tensile strain limit.

Based on the test results in Table 6.2, following the same form of empirical equation to predict the f_{pt} as proposed by Wille et al. [234], quadratic equations are proposed to establish the relationship between the V_f and ε_{pt} and the relationship between V_f and f_{pt} , as shown in Figure 6.30 (a) and (b), respectively. It is found that when V_f is increased, both ε_{pt} and f_{pt} increase until the $V_f \approx 3\%$; thereafter, ε_{pt} and f_{pt} decrease if more fibers ($V_f > 3\%$) are added. It should be noted that only when $V_f \geq V_{f,cr}$ (1.75%), ε_{pt} develops; and when $V_f < V_{f,cr}$ (1.75%), $f_{ct} = f_{pt}$. Therefore, empirical equations are proposed as follows:

$$\varepsilon_{pt} = 0.537V_f - 0.086V_f^2 - 0.631; 2 \leq V_f \leq 4\% \quad (6.21)$$

$$f_{pt} = f_{mt} + 3.929V_f - 0.624V_f^2; 0 \leq V_f \leq 4\% \quad (6.22)$$

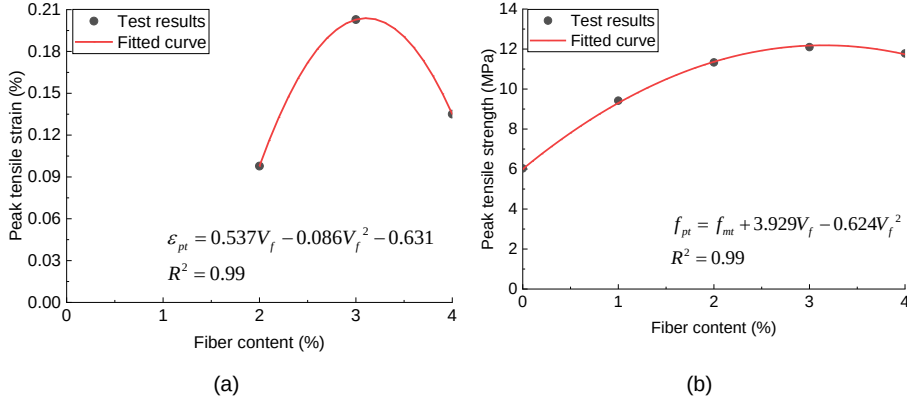


Figure 6.30 Comparison of the test results and predicted results by equation (6.21) and equation (6.22).

Stage III: Strain softening stage (A-B-C for SS-UHPFRC and A'-B-C for SH-UHPFRC): This stage involves a downward stress-crack opening relationship from the peak point $(0, f_{pt})$, through residual strength point (w_0, f_{rt}) , and continue until reaching the maximum crack opening w_{max} . This stage consists of two branches: For the first branch, prior to the residual strength point (w_0, f_{rt}) , smeared cracking occurs, and tensile strength linearly decreases and can be described as:

$$\sigma_t = f_{pt} + \frac{f_{rt} - f_{pt}}{w_0} \cdot w \quad 0 < w < w_0 \quad (6.23)$$

where f_{rt} is the residual tensile strength, w_0 is the crack opening corresponding to the f_{rt} , w is the crack opening.

Based on the tensile test results in Figure 6.8, it is found that when $V_f \geq 2\%$, the measured w_0 is around 0.21-0.36 mm, therefore w_0 is taken as 0.3 mm; when $V_f < 2\%$, a quadratic equation is proposed to establish the relationship between V_f and w_0 as shown in Figure 6.31 (a). Quadratic equation is also used to describe the relationship between V_f and f_{rt} as shown in Figure 6.31 (b). It should be noted that the fitted equation (6.25) excludes the test results for 3% UHPFRC due to its large discrepancy. Therefore, empirical equations are proposed as follows:

$$w_0 = 0.0006 + 0.118V_f + 0.016V_f^2; 0 \leq V_f \leq 2\% \quad (6.24)$$

$$f_{rt} = 2.65 + 4.235V_f - 0.614V_f^2; 0 \leq V_f \leq 4\% \quad (6.25)$$

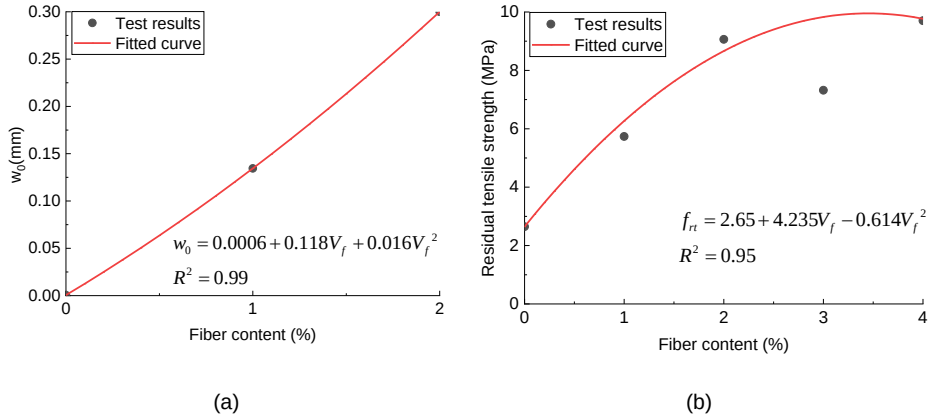


Figure 6.31 Comparison of the test results and predicted results by equation (6.24) and equation (6.25).

After the residual strength point (w_0 , f_{rt}), the crack opening is continuously increased mainly due to the formation of localized crack and a decreasing exponential function [235,236] can be used to describe the fiber pull-out response as follows:

$$\sigma_t = f_{rt} \left[1 + c_1 \left(\frac{w - w_0}{w_{\max}} \right)^3 \right] e^{-c_2 \left(\frac{w - w_0}{w_{\max}} \right)} \quad w_0 < w \leq w_{\max} \quad (6.26)$$

where $w_{\max}$ is the maximum crack opening, and c_1 and c_2 are coefficients obtained by fitting the experimental curve shown in Figure 6.8. After fitting analysis in current study, c_1 and c_2 are obtained for UHPFRC with varying fiber contents, as listed in Table 6.8.

Table 6.8 Fitted parameters for UHPFRC with varying fiber contents.

Mixture	c_1	c_2
UHPC	6.89	5.59
UHPFRC (1%)	15.00	3.88
UHPFRC (2%)	13.33	4.72
UHPFRC (3%)	1.86	3.04
UHPFRC (4%)	27.77	5.86

Based on the tensile test results in Figure 6.8, it is found that when $V_f \geq 1\%$, the measured $w_{\max}$ can be taken as 6.5 mm, that is, half length of the steel fibers; when $V_f \leq 1\%$, a linear equations is proposed to quantify $w_{\max}$ as a function of V_f as follows:

$$w_{\max} = 0.05 + 6.45V_f; \quad 0 \leq V_f \leq 1\% \quad (6.27)$$

6.6 Analytical methods for mechanical properties of UHPFRC

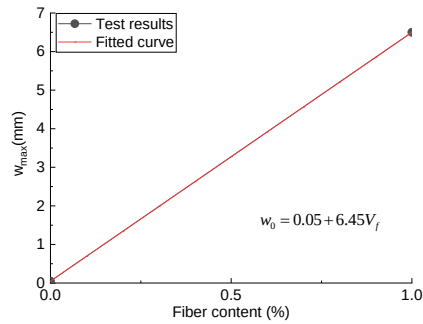
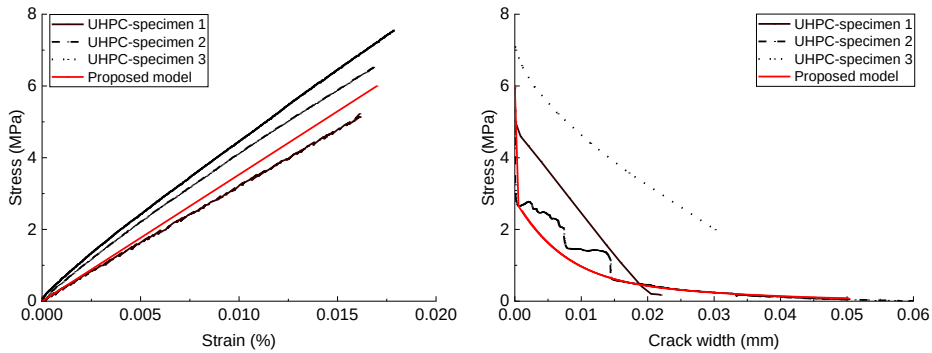
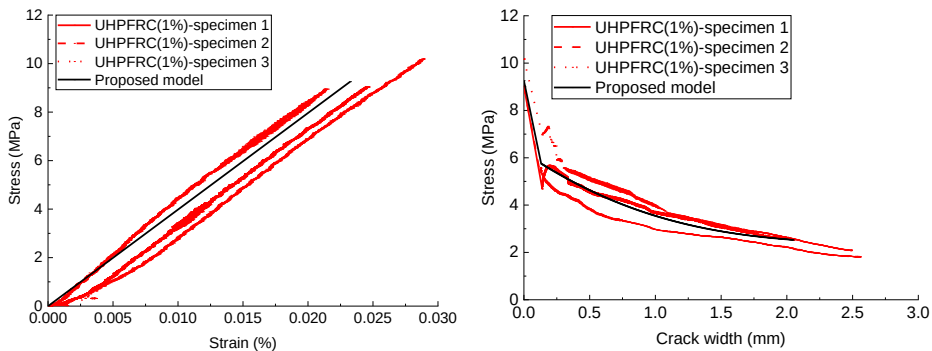


Figure 6.32 Comparison of the test results and predicted results by equation (6.27).

To validate the proposed constitutive models, the predicted tensile curves are compared with experimental results for UHPFRC containing varying amounts of steel fibers, as shown in Figure 6.33. The comparison demonstrates a good overall agreement between the predicted and experimental curves, indicating that the proposed models can provide a reasonably accurate estimation of the tensile behavior of the UHPFRC studied. This suggests that the models are effective in capturing the key characteristics of the tensile response for different fiber contents.



(a)



(b)

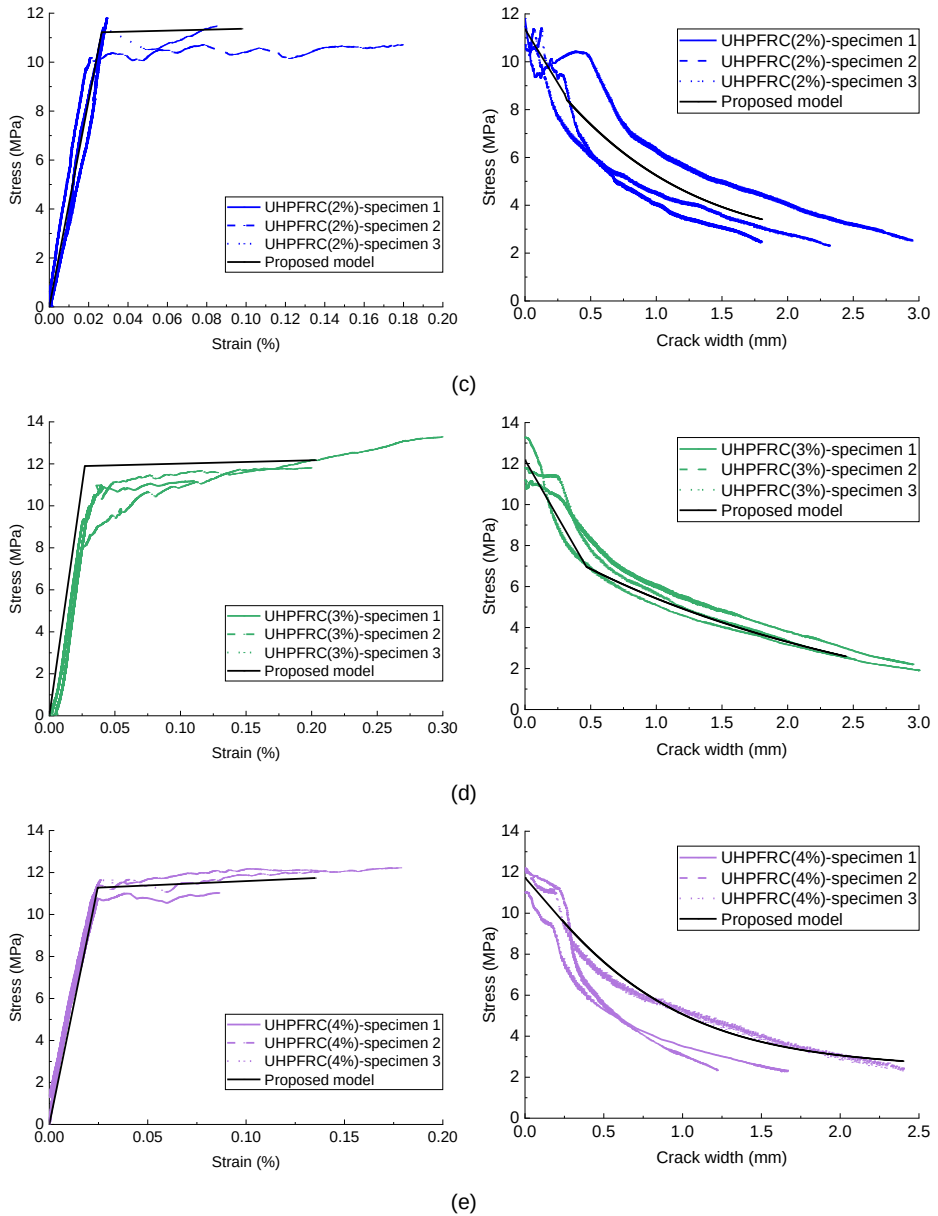


Figure 6.33 Comparison of tensile response between test results and proposed model: (a) UHPC; (b) UHPFRC (1%); (c) UHPFRC (2%); (d) UHPFRC (3%); (e) UHPFRC (4%).

6.7 Conclusion

This chapter explores the use of an electromagnetic NDT method based on induction principles to estimate fiber distribution and orientation in thin UHPFRC elements. Two magnetic sensors are calibrated using small-scale specimens with varying fiber contents.

The impact of measurement conditions (e.g., edge effect, surface roughness, and white paint) is evaluated. The influence of casting direction and vibration on fiber distribution is studied, with the method's accuracy validated by x-ray imaging. After validation, the method is applied to measure fiber distribution in hybrid UHPFRC-RC beams. Finally, based on the assumption of random fiber orientation, analytical models are proposed to predict the main mechanical properties (compressive strength, elastic modulus, and tensile behavior) of UHPFRC based on fiber content. The following conclusions are drawn from this investigation:

(1) The electromagnetic method is effective in estimating fiber content and orientation in thin UHPFRC elements; however, factors such as edge effects and surface roughness can affect measurement accuracy. To ensure reliable predictions, a minimum distance of 50 mm between the measurement point and the element's edge is recommended, when using the I-shaped probe. Additionally, a smooth surface is essential for optimal contact between the magnetic sensors and UHPFRC. A thin layer of white paint has been found to have a negligible impact on inductance measurements.

(2) The influence of casting direction and vibration time on fiber distribution and orientation in UHPFRC is examined. Results indicate that horizontal casting ensures a more uniform fiber distribution, while fibers tend to accumulate at the top during vertical casting as the fiber length is larger than the thickness of the UHPFRC plate. Casting direction has little effect on fiber orientation. The most significant influence is from vibration, with more severe fiber segregation observed as vibration time increases. In addition, it should be noted that the viscosity of the mix (e.g., slump or spread) also affects fiber distribution and orientation. Mixes with lower viscosity are more susceptible to fiber segregation, particularly under vibration.

(3) NDT test results on beams confirm that vertically-cast beams exhibit significant fiber segregation due to vibration, while horizontally-cast beams have a more uniform fiber distribution. A method is then proposed to quantify fiber segregation, revealing a strong association between segregation degree and shear load capacity. Higher segregation leads to lower load capacity, indicating that uniform fiber distribution in UHPFRC effectively enhances the shear resistance in UHPFRC-RC structures.

(4) The accurate prediction of UHPFRC's key material properties, including compressive strength, elastic modulus, and tensile behavior, requires precise fiber content measurements. Based on empirical equations and current study results, new relationships are proposed for predicting compressive strength and elastic modulus as functions of fiber content. Additionally, two tensile constitutive models are developed to describe the behavior of strain-softening and strain-hardening UHPFRC. These models are validated against experimental tensile curves. However, predictive equations are empirical, indicating a need for further development of models with a stronger physical basis.

Chapter 7 Numerical investigation on shear performance of UHPFRC-RC beams

7.1 Introduction

Experimental results on evaluating UHPFRC as a shear strengthening material for reinforced concrete beams are presented in Chapter 4. Although some parameters, such as the interface opening/sliding between UHPFRC and NC, are investigated, their effects have not been sufficiently quantified experimentally. Therefore, the primary goal of Chapter 7 is to numerically study the influence of key parameters on the shear response of UHPFRC-RC beams to better understand the governing mechanisms. The findings will ultimately aid in designing more effective materials by identifying critical material characteristics to focus on during the design process.

The commercial finite element software ATENA 3D is utilized. Section 7.2 details the material constitutive laws, with a focus on the tensile stress-strain relationship of UHPFRC. Section 7.3 validates the established finite element model by comparing it with experimental results, primarily load-deflection responses and failure modes. Upon validation, Section 0 conducts parametric studies examining the influence of UHPFRC-concrete interface properties, the tensile constitutive law of the strengthening material, and the non-uniform fiber distribution in UHPFRC. Finally, Section 7.5 presents the key findings of the numerical investigation.

7.2 Material constitutive laws and finite element model

For the material modeling, a fracture-plastic model has been used to simulate the tensile and compressive behavior of concrete and UHPFRC. For concrete, the compressive strength, elastic modulus and splitting tensile strength as listed in Table 4.1 in Chapter 4 are used as input. For UHPFRC, compressive strength of 120 MPa and elastic modulus of 45 GPa are used. To account for the effect of fibers, the tensile stress-strain relationship of UHPFRC is defined as presented in Figure 7.1. A bilinear relationship is used to simulate the tensile stress-strain behavior of UHPFRC until the peak tensile strength (i.e., linear elastic phase and strain hardening phase). For the softening branch, generally two functions are assumed: linear and bi-linear descending curves (referred to as (1) and (2) in Figure 7.1 (b)) [237]. In this study, the linear descending function is adopted and the crack opening w is converted to strain using the Equation (7.1):

$$\varepsilon_t = \varepsilon_{pt} + \frac{w}{L_t} \quad (7.1)$$

where ε_{pt} is strain capacity at peak tensile strength f_{pt} , w is crack opening and L_t is characteristic length. As suggested by the ATENA manual, the characteristic length L_t (equal to the crack band size) is calculated as element size (h) projected in the crack direction, so $\sqrt{2}h$ for a diagonal crack, as recommended by Dutch Guidelines for Non-

linear Finite Element Analysis of Concrete Structures [238]. To simulate the shear response of beams with epoxy bonded UHPFRC, cast vertically and horizontally (ST-M2-EB and ST-M2-EB-HC), the corresponding tensile stress-strain relationship of UHPFRC, following the trend of tensile tests (Figure 4.7 (a) and Figure 6.8 (c)), are presented in Figure 7.2. Figure 7.2 shows that although the same mixture (UHPFRC2) is used for both beams, the non-uniform fiber distribution in dog-bone specimens that are cast at the same time as ST-M2-EB beam, result in poorer tensile performance compared to those cast with ST-M2-EB-HC beam. Note that dog-bone specimens are always cast in the same way (horizontally). The bilinear stress-strain function with the elastic modulus of 200 GPa and a yield stress of 550 MPa is employed to characterize the rebars.

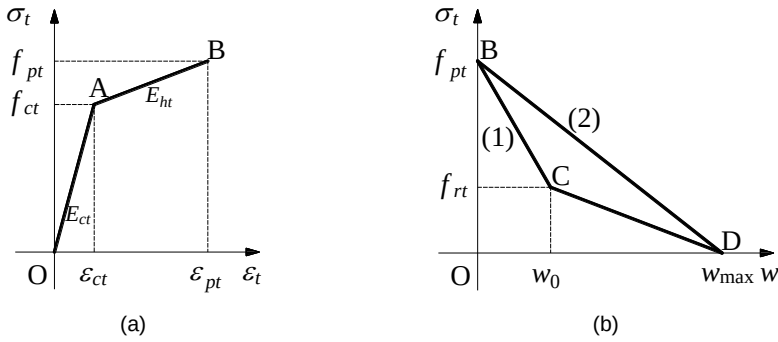


Figure 7.1 Typical tensile stress-strain curve for UHPFRC: (a) Elastic and strain hardening phases; (b) Softening phase.

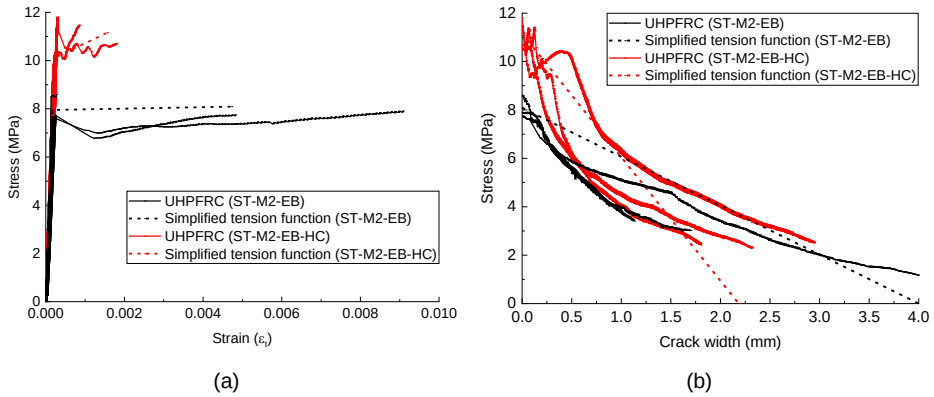


Figure 7.2 Simulated simplified tension function along with the tested tensile response of UHPFRC for ST-M2-EB and ST-M2-EB-HC, respectively: (a) Tensile stress-strain relationship of UHPFRC (elastic and strain hardening phases); (b) Tensile stress-crack width relationship of UHPFRC (softening phase).

In order to further explore the assumptions of the FE model, for ST-M2-EB beam, another simulation method considering the fiber segregation is proposed. As indicated in Chapter 6, severe fiber segregation occurs in the ST-M2-EB beam and therefore, the laminate is

7.2 Material constitutive laws and finite element model

segmented into four distinct layers along its height (Figure 7.3). Based on the fiber distribution for two panels of ST-M2-EB (shown in Table 7.1), these layers are simulated with different fiber percentage. The corresponding mechanical properties can be calculated based on the analytical method proposed in Section 6.6, as summarized in Table 7.2 and shown in Figure 7.4. It should be noted that as the difference in material properties of UHPFRC is negligible when fiber contents are more than 2%, the same inputs are given if $V_f \geq 2\%$.



Figure 7.3 Layered UHPFRC laminates.

Table 7.1 Fiber distribution along the height of ST-M2-EB specimen.

Height of specimen (mm)	ST-M2-EB	
	Panel 1	Panel 2
150-200	0.105%	0.15%
100-150	0.275%	0.88%
50-100	2.22%	2.96%
0-50	>4.10%	>4.46%

Table 7.2 Mechanical properties of UHPFRC with varied fiber content predicted by analytical model from Chapter 6.

Mixture	Compressive strength (MPa)	Elastic modulus (GPa)	First cracking strength (MPa)	Cracking strain (%)	Peak tensile strength (MPa)	Tensile strain capacity (%)
UHPFRC (0.105%)	88.1	35.5	6.4	0.018	6.4	0.018
UHPFRC (0.15%)	89.0	35.7	6.6	0.018	6.6	0.018
UHPFRC (0.275%)	91.7	36.5	7.0	0.019	7.0	0.019
UHPFRC (0.88%)	103.4	39.5	8.9	0.023	8.9	0.023
UHPFRC (>2%)	120	45	10.1	0.022	11.2	0.156

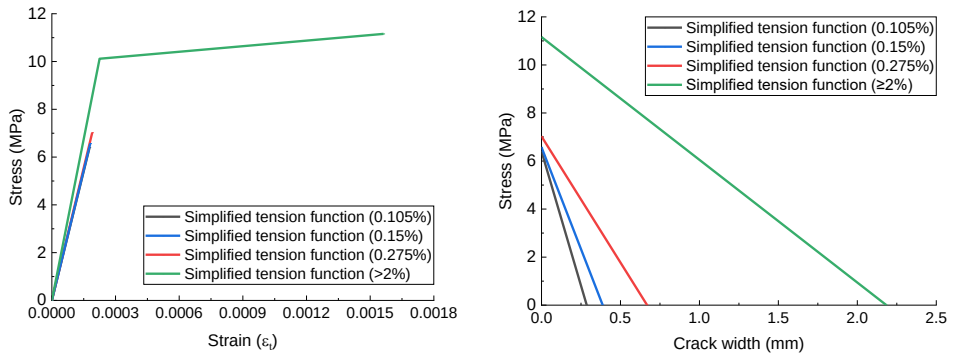


Figure 7.4 Simulated simplified tension function for UHPFRC with varied fiber contents.

After defining the material properties, a 3D FE model is developed for both the reference and UHPFRC-strengthened beams, as shown in Figure 7.5. Given the 10 mm thickness of the UHPFRC strengthening layer, 8-node 3D hexahedral elements with a 10 mm mesh size are used for both UHPFRC and NC. Steel bars are modelled as reinforcement with discrete truss elements. To simulate the experimental setup (Figure 3.10 (a)), two supports (50 mm length, 20 mm thickness) are modelled as hinged and roller supports, and a loading plate (70 mm length, 20 mm thickness) is applied only on the NC part. Both supports and the loading plate are simulated as steel elements with a linear stress-strain relationship, using an elastic modulus of 210 GPa and a Poisson's ratio of 0.3. A perfect bond is assumed between the rebars and NC, as well as between UHPFRC and NC, considering that the epoxy resin provides a good bond. The load is incrementally applied under displacement control with displacement increments of 0.15 mm, with reaction forces and deformations monitored at the mid-span. The Newton-Raphson method is used to solve the nonlinear equations.

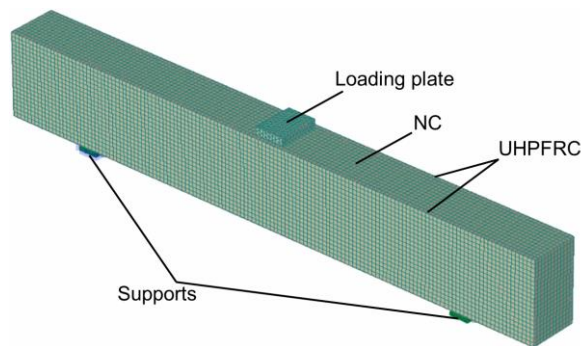


Figure 7.5 Establishment of FE model.

7.3 Validation of the numerical model

Four beams are simulated for validation: two reference beams, without strengthening and with the same width as strengthened beams (RB and RB (120)), and two strengthened

beams with vertically and horizontally cast lamellas (ST-M2-EB and ST-M2-EB-HC). For the ST-M2-EB beam, two methods, namely with uniform and nonuniform properties of UHPFRC lamellas, as described in Section 7.2 are applied to simulate its shear response. Figure 7.6 compares the numerical and experimental results for reference, RB and strengthened ST beams, while Table 7.3 shows the maximum loads and their corresponding deflection obtained from both experiments and simulations. The numerical and experimental results are generally in good agreement, validating the effectiveness of the 3D FE model in capturing the structural behavior of UHPFRC-NC composite beams. The peak load and the load-deflection trends for both RB and ST beams are similar. However, the FE model predicts a higher stiffness than the experiments, resulting in lower mid-span displacements at peak load.

The underestimation of deflection in the simulation may be due to the assumption of a perfect rebar-concrete bond and neglecting the shrinkage effects in the model [239]. Additionally, the use of smeared crack models tends to overestimate stiffness of RC beams, as cracking is distributed uniformly across elements and high shear transfer is maintained across crack surfaces, which delays stiffness degradation compared to experimental observations [240]. In contrast, discrete crack models represent cracks explicitly and capture the initiation and propagation of cracks more realistically, resulting in more accurate stiffness reduction [241]. However, a higher prediction of ductility is seen in ST-M2-EB-HC. A possible reason is that the measured actual fiber content is 1.5% for ST-M2-EB-HC (presented in Table 6.5 in Chapter 6), which is lower than assumed 2% fiber contents. Therefore, with material properties of UHPFRC (2%) as inputs, an overestimation of ultimate load and deformation is observed. For the strengthened beams, additionally, the higher stiffness could be attributed to the assumed homogeneity of UHPFRC in the model. In practice, flaws during casting and non-uniform fiber distribution occur, which are not captured by the model. This assumption of uniform UHPFRC properties may lead to stiffness overestimation, particularly in unreinforced or under-reinforced UHPFRC laminates, as also noted by Feng [242].

The cracking patterns and principal strain contours at failure from the numerical study are compared with experimental observations (Figure 7.7). Only cracks wider than 0.1 mm are shown for clarity. The numerical crack patterns generally match the major shear crack observed in experiments for both reference and strengthened beams. In simulation of ST-M2-EB specimen, both methods can well capture the shear failure patterns. However, the first method, which uses the uniform material inputs for the entire lamella, results in an identical failure pattern on both the front and back sides of the ST beam. In contrast, the second method, utilizing four-layer segmentation, simulates the slight variations in failure patterns between the front and back sides, providing a better match with experimental results. This difference is likely due to the varying degrees of segregation in the two sides of the beam. The numerical results also indicate that applying UHPFRC

alters the shape of the major shear crack and shifts the failure mode from shear to flexural as the shear angle (the angle between the longitudinal axis of the beam and the compressive strut) increases. Therefore, the predicted results are considered acceptable, particularly regarding failure modes and peak loads. The model is used to further understand the influence of key parameters.

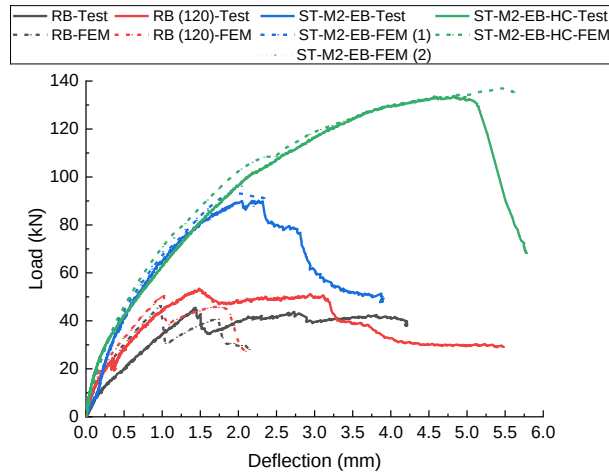


Figure 7.6 Experimental versus numerical load- mid-span deflection response for tested specimens; noting that for ST-M2-EB beam, (1) denotes the simulated results with uniform material inputs from dog-bone specimens and (2) denotes the simulated results obtained by four-layer segmentation.

Table 7.3 Comparison between test and FEM results.

Specimen ID	Peak load			Deflection at the peak load		
	Test (kN)	FEM (kN)	FEM/Test	Test (mm)	FEM (mm)	FEM/Test
RB	45.5	46.4	1.02	1.43	0.98	0.69
RB (120)	53.5	50.6	0.95	1.48	1.02	0.69
ST-M2-EB	90.1	(1) 93.3	1.04	2.26	1.89	0.84
		(2) 96.2	1.07	2.26	2.04	0.90
ST-M2-EB-HC	133.1	137	1.03	4.77	5.47	1.15

7.4 Parametric study

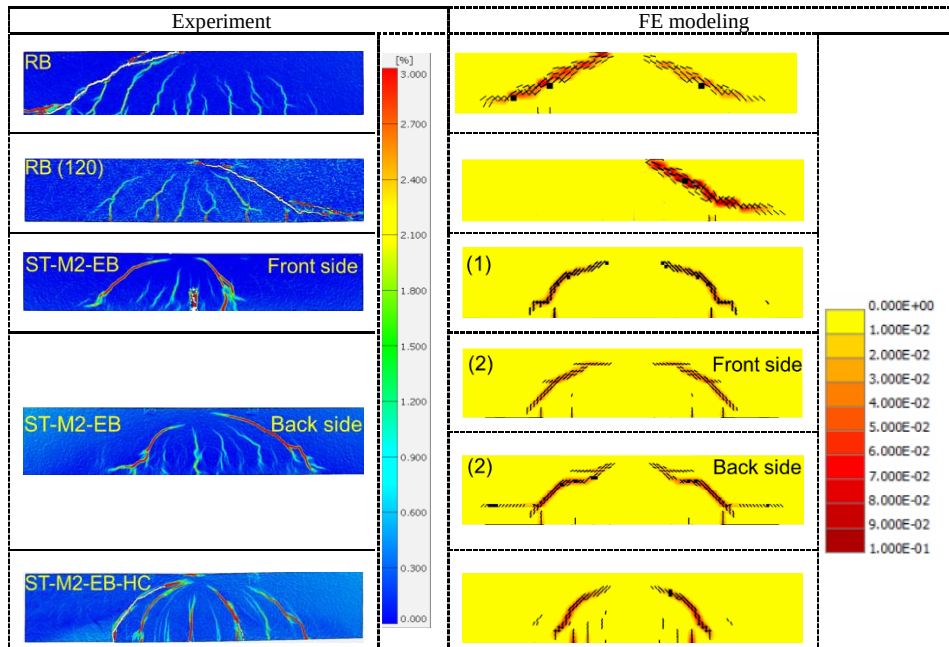


Figure 7.7 Comparison of crack pattern together with principal strain at peak load for tested specimens; nothing that for ST-M2-EB beam, (1) and (2) indicate crack patterns from uniform and four-layer segmentation (non-uniform) properties of UHPFRC lamellas, respectively.

7.4 Parametric study

One of the main goals of this study is to assess the shear strengthening efficiency of concrete beams strengthened with UHPFRC under freeze-thaw (FT) cycles. Despite exposure to 120 FT cycles, no significant degradation in UHPFRC's mechanical properties or the bond at the interface is observed. Previous studies have indicated that significant freeze-thaw cycles can degrade UHPFRC properties and weaken the bond between UHPFRC and normal concrete [125,207,243]. Thus, a parametric study is conducted to explore potential damage that might occur after freeze-thaw cycles, focusing on UHPFRC material properties and the interface. Additionally, the impact of non-uniform fiber distribution in UHPFRC on shear strengthening efficiency is also analyzed.

7.4.1 The effect of interface properties

Based on experimental results, a perfect bond is assumed in FE modeling because the high bond strength between UHPFRC and normal concrete can be ensured with epoxy resin. However, factors such as poor interface preparation, improper curing conditions, and harsh environments can cause debonding in UHPFRC-RC composites. Since interface behavior significantly affects the structural performance of UHPFRC-RC specimens,

it is essential to investigate the impact of interface strength on the effectiveness of UHPFRC strengthening.

The ATENA interface material model treats the interface element as a zero-thickness element, allowing for simulation of the contact between UHPFRC and concrete. As shown in Figure 7.8, the model uses a Mohr-Coulomb failure criterion with a tension cut-off and typical interface behavior in tension and shear, resulting in opening and sliding displacements at the interface. It should be noted that the constitutive behavior of the interface is defined in the local coordinate system of the interface element. The interface sliding direction corresponds to the tangential direction within this local coordinate system, while the normal direction - perpendicular to the interface surface - describes the interface opening or closing (i.e., separation or contact). The normal and tangential directions are automatically determined based on the element geometry and follow the element's deformation during the simulation. Key parameters defining interface behavior include normal and tangential stiffness (K_{nn} and K_{tt}), tensile strength (f_t), cohesion (C), and the coefficient of friction (μ). For numerical stability, minimum values for normal and tangential stiffness ($K_{nn}^{\min}$ and $K_{tt}^{\min}$) are used, typically set to 0.1% of the initial normal and tangential stiffness (K_{nn} and K_{tt}), respectively.

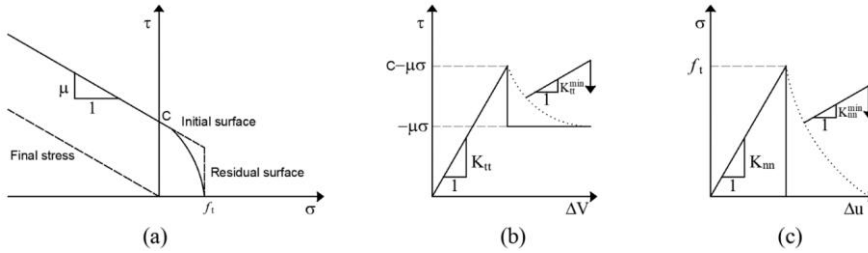


Figure 7.8 Interface material model: (a) Mohr-Coulomb failure criteria with tension cut-off, (b) interface behavior in shear and (c) interface behavior in tension [45].

Using ST-M2-EB-HC as a reference, the interface properties are varied. Parameters for the interface strength between UHPFRC and normal concrete are defined in Table 7.4. All the inputs are provided from previous research by Liu [244], in which the ranges of interface parameters are summarized from the existing test results. To simulate interface debonding and mixed failure modes (a combination of interface failure and concrete fracture), two cases are defined for strengthened beams with UHPFRC: 'weak bond' and 'moderate bond'. Additionally, to represent superior bond properties and to focus on concrete fracture alone, a set of parameters is defined as 'strong bond', reflecting the maximum interface strength summarized in [244].

In addition, to examine the impact of interface stiffness, two additional cases are defined: 'strong bond - low stiffness (SBLS)' and 'strong bond - moderate stiffness (SBMS)'. The

7.4 Parametric study

SBLS case combines the tensile strength (f_t), cohesion (C), and friction coefficient (μ) from the 'strong bond' model with the interface stiffness (K_{nn} , K_{tt}) from the 'weak bond' scenario. The SBMS model uses the same f_t , C , and μ as 'strong bond' but assumes interface stiffness equal to that of the 'moderate bond' case.

Table 7.4 Interface parameters for weak, moderate and strong bonds with varied interface stiffness.

Interface bond type	f_t (MPa)	C (MPa)	μ	K_{nn} (N/mm ³)	K_{tt} (N/mm ³)
Weak bond	2.5	1.6	1.1	100	1
Moderate bond	2.6	6	1.4	500	150
Strong bond	6	10	2	550	350
Strong bond – moderate stiffness (SBMS)	6	10	2	500	150
Strong bond – low stiffness (SBLS)	6	10	2	100	1

Using the ST-M2-EB-HC specimen as a reference, Figure 7.9 shows the simulated load-deflection curves. Varying bond properties affect both the peak load and ductility of the beam, while the initial stiffness remains constant. The peak load decreases as bond strength reduces. The weak bond exhibits the earliest stiffness degradation in the load-deflection response, leading to brittle failure due to premature delamination between UHPFRC and normal concrete.

Moreover, interface stiffness significantly influences the peak load and ductility of the strengthened beam, as seen when comparing the load-deflection responses of strong bonds with varying stiffness levels. Higher stiffness contributes to the highest load, although the deflection capacity is slightly reduced compared to the low stiffness. A lower stiffness (e.g., in SBLS) allows greater displacement at the interface before failure. As a result, SBLS experiences also greater stiffness degradation.

To further understand the role of interface properties, Figure 7.10 shows the final failure patterns in UHPFRC laminates and RC sections, as well as the interface shear displacement after the peak load. With a weak bond strength, the failure patterns in UHPFRC laminate and the RC section are not the same (Figure 7.10 (a)) At lower loads, debonding occurs rather than cracking, as evidenced by significant interface shear displacement (Figure 7.10 (b)). For moderate bond, localized delamination leads to partially incompatible shear cracks between the UHPFRC and RC sections (Figure 7.10 (b)). In the 'SBLS' model, very low interface stiffness results in large interface deformations, which cause shear and splitting cracks to develop rapidly in the RC section, reducing the bonding area and load transfer between UHPFRC and concrete. In contrast, 'SBMS' and 'strong bond' models show fully developed shear cracks at the surface of UHPFRC layers, which, due to strong bond, is reflected from cracks in concrete.

While the effect of bond strength on failure is clear—lower bond strength can lead to early debonding and reduced capacity—the role of interface stiffness is less understood.

Liu [244] suggests that high stiffness might be beneficial, but if stiffness is significantly increased (e.g., ten times higher than in 'strong bond'), it could weaken shear resistance due to potential localized delamination, as noted in previous research [10]. Therefore, a more detailed investigation is needed to explore the relationship between interface stiffness, fracture propagation, and structural response, including peak load, ductility, and failure patterns.

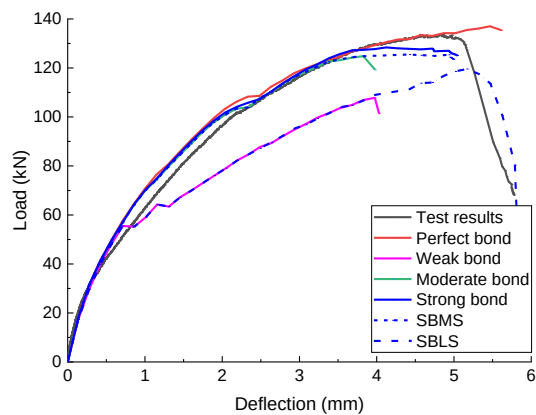
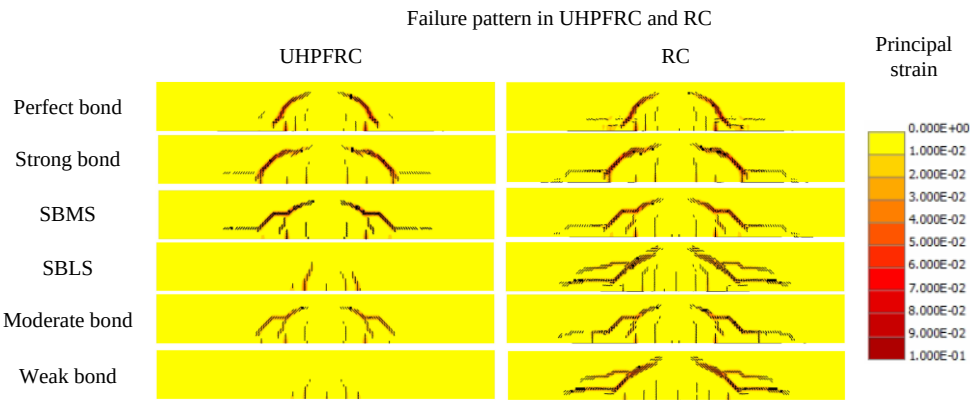


Figure 7.9 Experimental and simulated load-deflection response with different interface properties; SBMS and SBLS refer to Strong Bond with Medium and Low Stiffness, respectively.



(a)

7.4 Parametric study

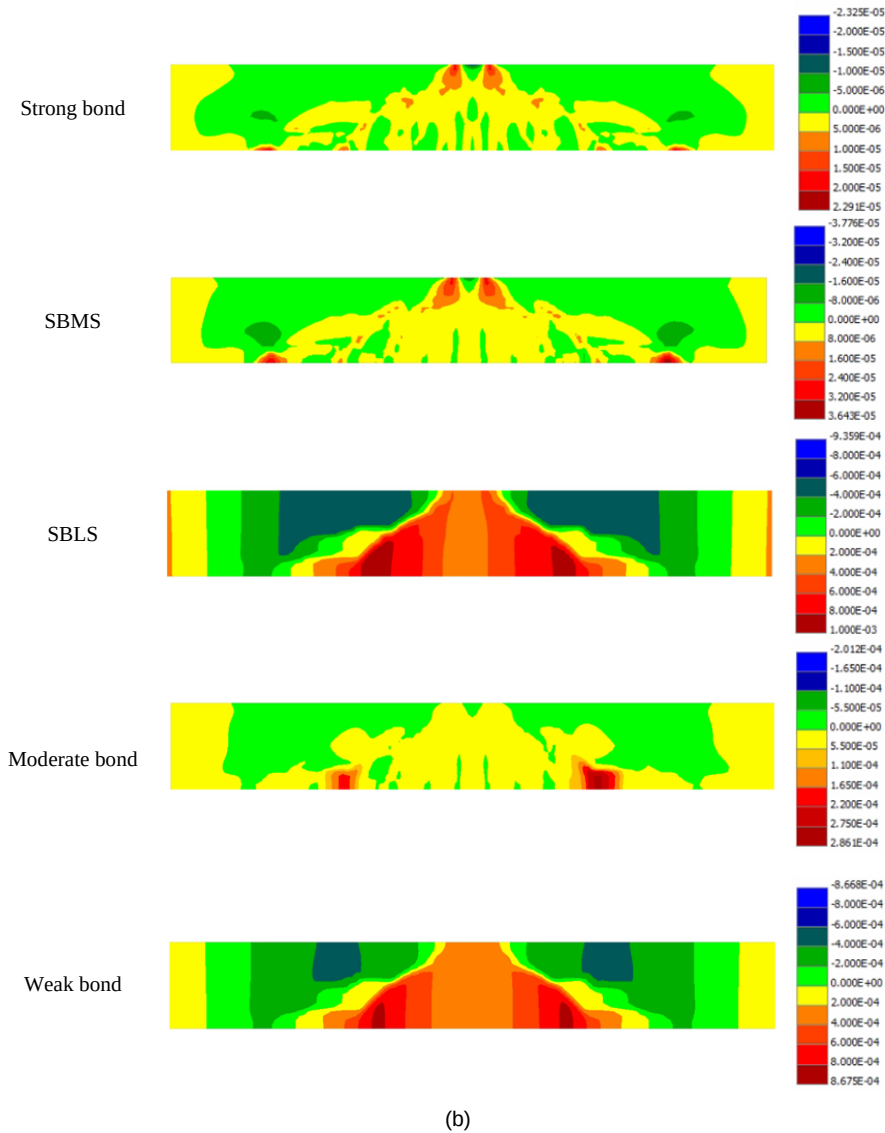


Figure 7.10 Numerical results: (a) obtained final failure patterns (in UHPFRC laminate and RC part) and (b) interface shear displacement [mm] immediately after peak load for different interface properties

7.4.2 The effect of tensile properties of UHPFRC

In evaluating UHPFRC's material properties, the tensile behavior is crucial for the shear performance of strengthened structures. Key tensile properties of UHPFRC include: (1) initial elastic modulus (E_{ct}), (2) peak tensile strength (f_{pt}), (3) strain capacity (ε_{pt}), and (4) maximum crack opening (ω_{max}).

The simulated tension function for ST-M2-EB is used as reference, assuming $E_{ct}=45$ GPa, $f_{ct}=8.0$ MPa, $f_{pt}=8.1$ MPa, $\varepsilon_{pt}=0.45\%$, and $\omega_{max}=4$ mm, as presented in Figure 7.2 (a). It should be mentioned that only the investigated parameter is varied while other parameters are kept constant.

Initial elastic modulus

The elastic modulus of UHPFRC can vary depending on its material composition. Figure 7.11 (a) shows that the elastic modulus ranges from 15 GPa to 60 GPa. Figure 7.11 (b) illustrates how this variation affects the load-deflection response and failure modes. As expected, a higher elastic modulus makes the composite structure stiffer. However, the peak load and ultimate deflection show only minor differences—around 2% for peak load and 16% for deflection. This relationship is not linear; a higher modulus does not consistently increase peak load or deflection capacity. Additionally, the failure patterns and the shape of the dominant shear crack (including shear angles) remain unchanged. Nonetheless, a lower elastic modulus in the strengthening material might be advantageous for shear-critical members, as it enhances the deformational ability of beams and makes shear cracks more noticeable (with larger crack widths) before sudden shear failure occurs.

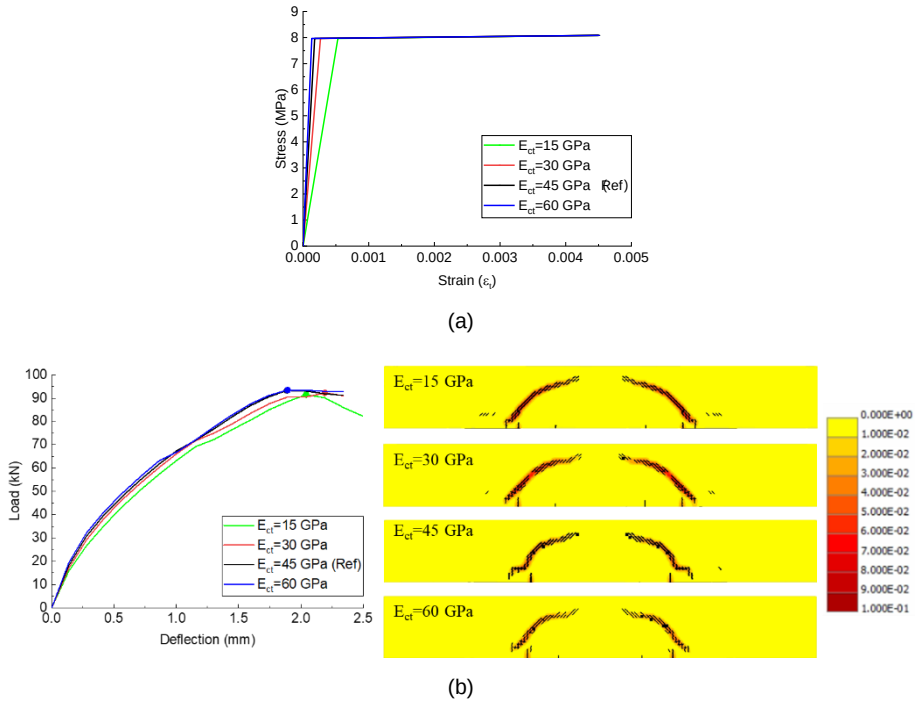
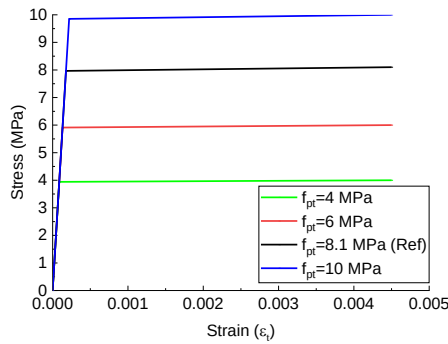


Figure 7.11 Influence of the elastic modulus of UHPFRC on the structural response: (a) Varied elastic modulus, and it is ranged from 15 GPa to 60 GPa; (b) load-deflection response and the corresponding crack patterns with principal strain.

Peak tensile strength

Tensile strength of UHPFRC is a key factor influencing the structural behavior of hybrid beams. Four peak tensile strength values (f_{pt}) are considered: 4 MPa, 6 MPa, 8.1 MPa (as reference), and 10 MPa. The crack tensile strength (f_{ct}) is calculated from peak tensile strength using a constant ratio $f_{pt} / f_{ct} = 1.015$ from reference tension function, as plotted in Figure 7.12 (a).

Figure 7.12 (b) shows the load-deflection curves for hybrid beams with varying peak tensile strengths. Compared to the 4 MPa, peak loads are increased by 8%, 19%, and 33% for tensile strengths of 6 MPa, 8.1 MPa, and 10 MPa, respectively. This indicates that higher tensile strengths in UHPFRC significantly enhance the peak load of the strengthened beams. The initial stiffness remains constant due to the unchanged elastic modulus. As tensile strength increases from 6 MPa to 10 MPa, the beams exhibit larger deformations, with up to a 60% increase seen at 10 MPa. However, this effect is not observed when tensile strength increases from 4 MPa to 6 MPa. This discrepancy is explained by the failure patterns: for all cases except 4 MPa, failure involves only one dominant shear crack. With the strength of 4 MPa, a secondary shear crack develops before the main shear crack fully forms. This secondary crack, caused by the lower tensile strength, reduces stress concentration at the dominant crack site and allows for greater deformation before final shear failure.



(a)

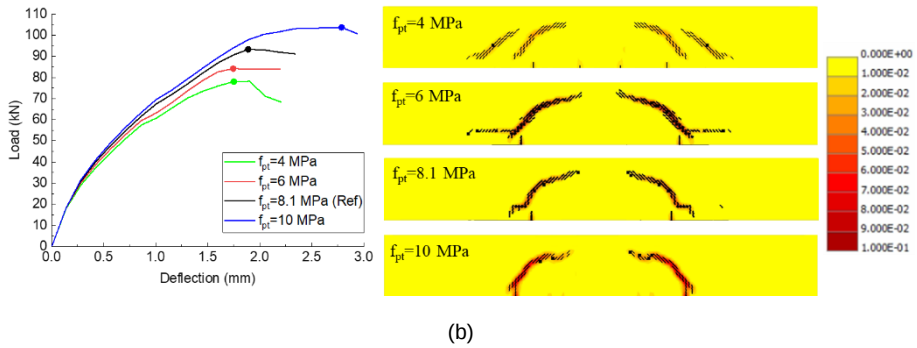


Figure 7.12 Influence of UHPFRC tensile strength on structural response: (a) Varied peak tensile strength, ranging from 4 MPa to 10 MPa; (b) load-deflection response and the corresponding crack patterns with principal strain.

Strain capacity

It has been reported [131] that fiber bridging and strain hardening capacity impact significantly response of UHPFRC members. In a parametric study, the ultimate strain capacity is varied from 0 to 2% (shown in Figure 7.13 (a)). In general, the higher the strain capacity, the higher is the ultimate load and ultimate deflection.

As the strain capacity of UHPFRC (ε_{pt}) increases from 0.45% to 1% and 2%, the shear capacity of the composite beam is increased by 8% and 26%, respectively. A more pronounced enhancement is found in ductility, namely ultimate deflection is increased by 47% and 110% with 1% and 2% strain capacity, respectively, compared to the 0.45% strain capacity.

Comparison of crack patterns reveals that varying the strain hardening capacity does not alter the final failure mode, including the angle and shape of the shear crack. However, with increased strain capacity, particularly from 0.45% to 2%, the width of the dominant shear crack becomes larger, as indicated by a greater number of damaged elements surrounding the cracks, thereby providing more warning before final failure.

In contrast, when the strain capacity is varied from 0% to 0.45%, the shear crack widths remain nearly the same. Additionally, in the case of zero strain capacity, the dominant shear crack is observed to connect with a previously developed flexural crack.

7.4 Parametric study

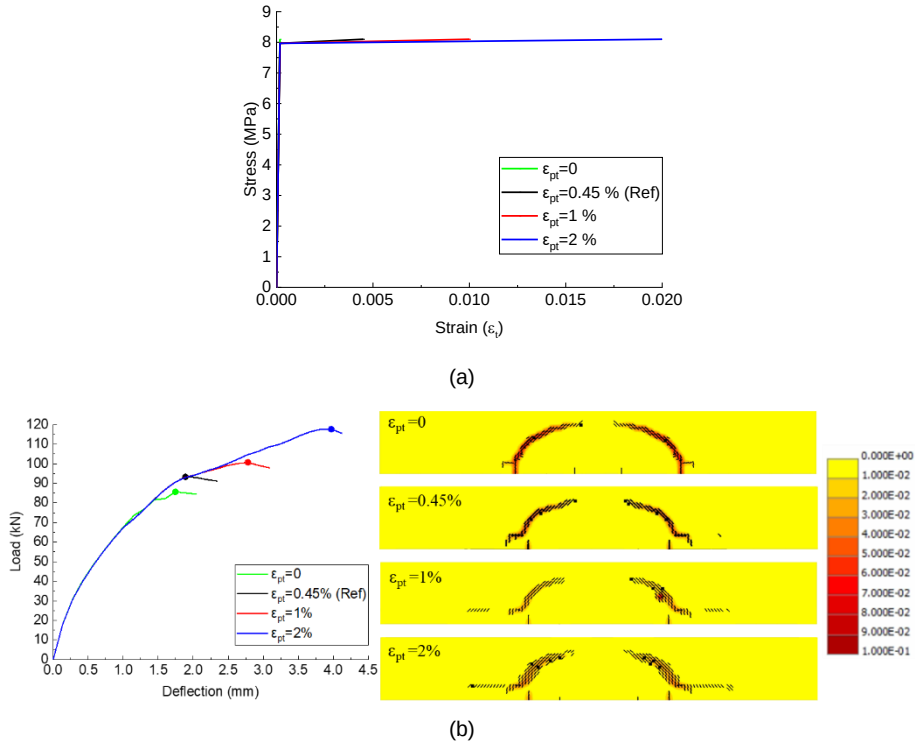


Figure 7.13 Influence of the strain hardening properties of UHPFRC on the structural response: (a) Varied strain hardening capacity, and it is ranged from 0 to 2 %; (b) load-deflection response and the corresponding crack patterns with principal strain.

To investigate whether tensile strength or strain capacity of UHPFRC has a greater impact on the shear performance of UHPFRC-strengthened concrete beams, a comparative analysis is conducted on their effects on peak load and deflection, as shown in Figure 7.14 (a) and (b). The coordinate ranges for the variables tensile strength (f_{pt}) and strain capacity (ϵ_{pt}) in Figure 7.14 (a) and (b) are set based on the scope of the parametric study. As previously mentioned, reference values of tensile strength ($f_{pt}=8.1$ MPa) and strain capacity ($\epsilon_{pt}=0.45\%$) are used as benchmarks.

The comparison reveals trend as follows:

- **Peak Load:** The peak load increases linearly with increases in either tensile strength or strain capacity. However, enhancements in tensile strength leads to a higher increase in peak load than similar improvements in strain capacity, indicating that tensile strength contributes more significantly to the loading capacity of the beams.

- **Deflection:** The deflection behavior exhibits a threshold effect around the reference values. When tensile strength or strain capacity is below its respective reference, deflection increases slowly, with strain capacity showing a slightly greater advantage in enhancing deflection. However, when tensile strength or strain capacity exceeds their reference values, deflection begins to increase more rapidly. Beyond these thresholds, strain capacity becomes more influential, contributing to a more pronounced improvement in deflection.

These findings suggest that while both tensile strength and strain capacity contribute to peak load and deflection enhancements, tensile strength exhibits a stronger influence on peak load, whereas strain capacity becomes more influential in enhancing deflection, especially when beyond the reference values (strength of 8.1 MPa and strain capacity of 0.45%). This emphasizes the importance of optimizing both properties of UHPFRC in the strengthening design of concrete beams with UHPFRC for optimal shear performance.

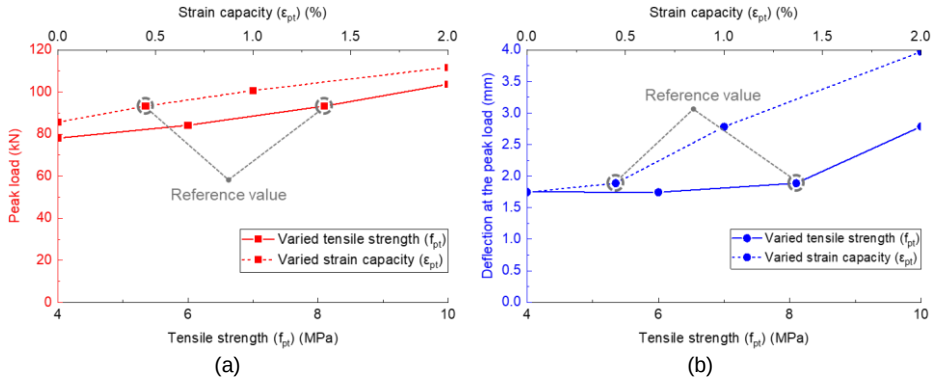


Figure 7.14 Comparison of the effects of tensile strength (f_{pt}) and strain capacity (ϵ_{pt}) on peak load and deflection: (a) Effect on peak load; (b) effect on deflection at peak load.

Softening behavior

The strain softening phase is reported to have an influence on flexural behavior of beams only after the formation of a macro-crack [131,245]. Since the peak load is almost reached when the macro-crack forms, the softening phase might have a minor role in shear-critical concrete members without shear reinforcement. However, due to the bridging effect of steel fibers in UHPFRC, the load can still be transferred after the formation of a macrocrack [246]. Therefore, a parametric study for the softening behavior of UHPFRC is conducted in which the shape of descending branch and the ultimate crack opening are varied, assuming the same tensile behavior for linear elastic and strain hardening phases.

One bilinear softening curve and three linear softening functions are considered, as presented in Figure 7.15. The bilinear function is characterized by the inflection point. Based

7.4 Parametric study

on previous studies [131,247], the ratio of the crack width and tensile stress at the inflection point to the corresponding maximum crack width and peak tensile strength f_{pt} is around 1/3. Therefore, the inflection point can be described as $(w_{max}/3, f_{pt}/3)$ and the softening part is assumed to be bilinear descending curve, from peak point, through inflection point and finally to the ultimate crack opening ($w_{max}=4$ mm). This is a similar ultimate crack opening (w_{max}) as observed in the direct tension test (referred to as Linear – $w_{max}=4$ mm). The linear slopes are characterized by different slopes and maximum crack widths (referred to as Linear – $w_{max}=0.05$ mm, $w_{max}=2$ mm and $w_{max}=4$ mm).

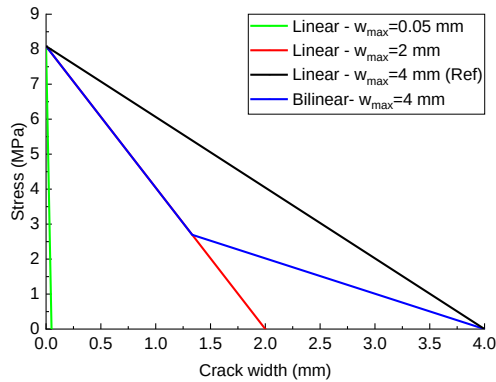


Figure 7.15 Assumed four softening functions of UHPFRC in tension.

The simulated load-deflection response and crack patterns of hybrid beams with varied softening functions, are shown in Figure 7.16. The softening branch of UHPFRC has a negligible effect on the peak load and deflection. However, if the softening branch of UHPFRC is not taken into consideration as represented by softening function linear – $w_{max}=0.05$ mm, lower simulated peak load and deflection are obtained as the contribution of fracture energy of the UHPFRC is greatly underestimated. In addition, it can be clearly seen that the softening branch of UHPFRC will mainly influence the post-peak behavior of the hybrid structure. From the simulated cracking patterns at failure, it can be seen that the beams with assumed softening functions show similar shear failure modes as observed in the experiment (shown in Figure 7.7). Therefore, the UHPFRC softening part has limit contribution in the failure patterns.

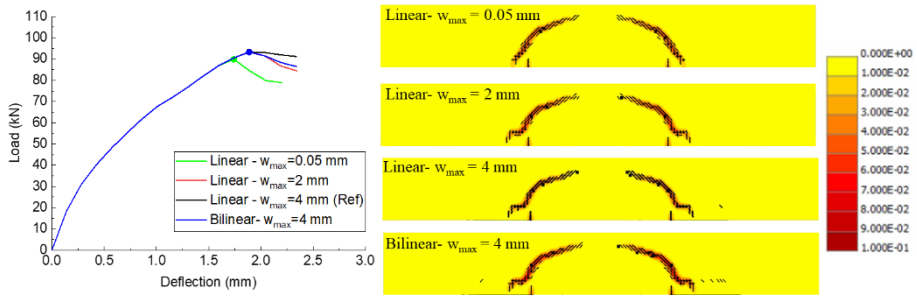


Figure 7.16 Influence of the softening branch of UHPFRC on the load-deflection response and the corresponding crack patterns with principal strain.

7.4.3 The effect of non-uniform fiber distribution in UHPFRC

In Section 4.4.2, the load-deflection response for beams with vertically and horizontally cast lamellas, ST-M2-EB and ST-M2-EB-HC, respectively, is compared. ST-M2-EB exhibits much lower load and deformation capacity, resulting from the non-uniform fiber distribution in UHPFRC laminates, as confirmed by fiber distribution measurements shown in Figure 6.24. To quantify the influence of non-uniform fiber distribution on the shear response of UHPFRC-RC beams, a parametric study is conducted, and five segregation degrees are simulated.

Several assumptions are made: (1) the average fiber content is 2%; (2) both laminates show the same fiber distribution; (3) higher fiber contents exist at the deeper position of the beam; (4) Fiber content along the length is constant. Using the four-layer segmentation simulation method, the laminate is segmented into four distinct layers along its height (Figure 7.3). These layers are simulated with different fiber distribution percentage and corresponding fiber distribution coefficient α (calculated using the Equation (6.10) and (6.11) in Section 6.5.3), shown in Table 7.5.

Table 7.5 Five assumed fiber distributions and corresponding fiber distribution coefficient α .

Height of specimen (mm)	Fiber contents (%)				
150-200	0	0	0	1	2
100-150	0	0	2	2	2
50-100	0	2	2	2	2
0-50	8	6	4	3	2
Average fiber content (%)	2	2	2	2	2
Coefficient of variation	1.73	1.22	0.71	0.35	0
Fiber distribution coefficient α	0.18	0.29	0.49	0.70	1

Based on the material test results shown in Table 6.2 in Chapter 6, the inputs for UHPFRC with varied fiber contents can be obtained. As previously mentioned, the same input val-

7.4 Parametric study

ues are used for cases where $V_f \geq 2\%$ due to the insignificant difference in material properties of UHPFRC when the fiber content exceeds 2%. Three simulated simplified tension functions, which represent the UHPC, UHPFRC (1%) and UHPFRC ($\geq 2\%$), are shown in Figure 7.17.

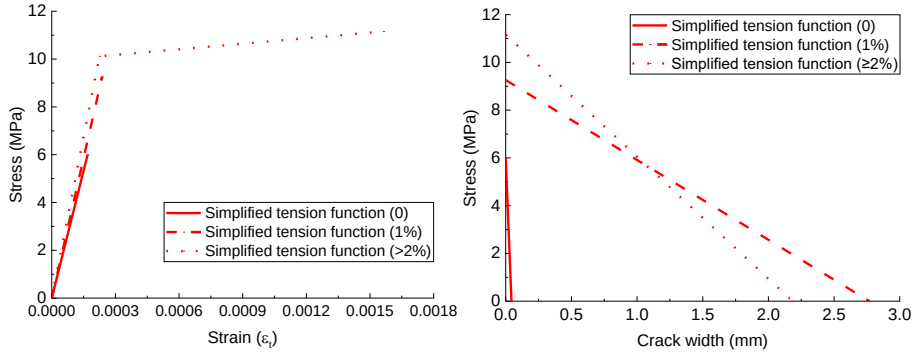
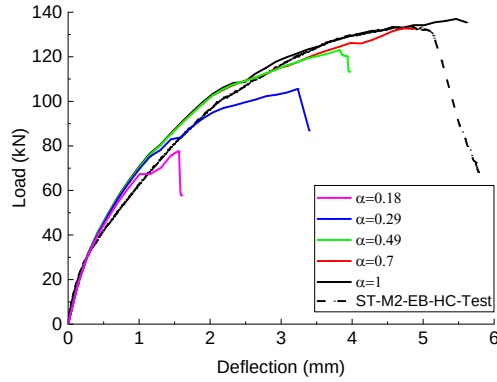


Figure 7.17 Simulated simplified tension function for UHPFRC with varied fiber contents.

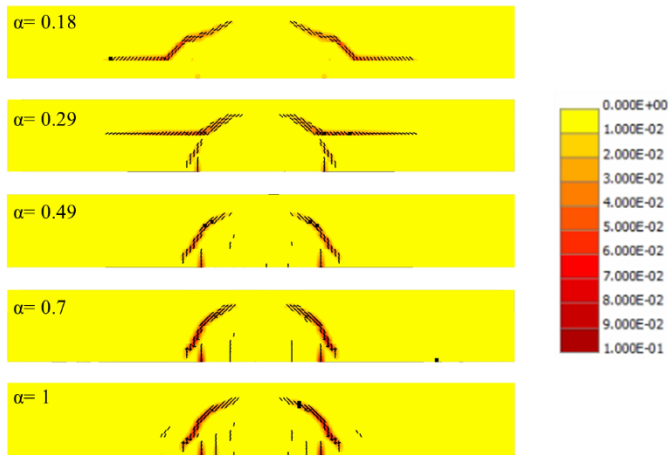
Figure 7.18 shows the simulated load-deflection curves and crack patterns with principal strain. It is evident that the non-uniform fiber distribution greatly influences the peak load and ductility of the hybrid beam. When α is increased from 0.18 to 1 (representing uniform fiber distribution), the peak load and deformation is increased by 77% and 2.5 times, respectively. Compared to the test results of ST-M2-EB-HC, it is found that a better prediction of peak load and ductility is seen when $\alpha = 0.7$ instead of $\alpha = 1$ though the stiffness response is better predicted when $\alpha = 1$ (i.e., uniform fiber distribution). When α is more than 0.49, it does not affect the structural behavior significantly. For the case $\alpha = 0.49$, numerical-to-experimental peak load ratio and ductility ratio are 0.92 and 0.8, respectively. Therefore, it is reasonable to conclude that a threshold value for α exists (0.49 in current study) and the influence of non-uniform fiber distribution can be neglected if above the threshold value.

Uneven fiber distribution will also affect the crack patterns. For varied α (Figure 7.18 (b)), it can be seen that when $\alpha \geq 0.49$, the beams show similar shear failure modes as observed in the experiment. However, when the fiber segregates severely (e.g., with $\alpha = 0.18$ and $\alpha = 0.29$), in addition to shear crack, a splitting crack also initiates and propagates along the weaker side of the UHPFRC. The dominant shear crack when $\alpha = 0.18$ even stops propagating to the bottom of the beams. It should be mentioned that such obvious splitting crack results from significant material difference in assigned layers, which is not observed experimentally. Unlike a sudden fiber content is applied in simulations, the UHPFRC fiber content in the experiments gradually changes. To predict better

the failure pattern for tested beams with fiber segregation, it is recommended to either divide the beams into more layers or define a continuous varied material property of UHPFRC along the height of the beams.



(a)



(b)

Figure 7.18 Influence of the non-uniform UHPFRC fiber distribution on the structural response: (a) load-deflection response; (b) corresponding crack patterns represented by the principal strain distribution.

7.5 Conclusion

In this chapter, numerical simulation is conducted by using commercial software ATENA 3D. A 3D finite element model is established, and its accuracy is validated by comparing the simulated load-deflection response and failure modes with experimental results. Additionally, a parametric study is conducted, focusing on key factors such as the interface properties between UHPFRC and concrete, the tensile constitutive law of UHPFRC, and the effects of non-uniform fiber distribution in UHPFRC. The main conclusions drawn from this study are as follows:

7.5 Conclusion

(1) The interface property is one of the most important parameters affecting the shear strengthening efficiency of UHPFRC. Through the numerical study, it is found that both the bond strength and the interface stiffness will affect significantly the load capacity, ductility and failure patterns. Without proper interface design, the composite structure will fail at low loads and deformations.

(2) The tensile strength and strain hardening capacity of the strengthening material enhance both the load and deformation ability. The tensile strength is more advantageous to increase the load and a more significant deformation improvement is found with increased strain hardening capacity. The elastic modulus of UHPFRC mainly influences the initial stiffness, and the softening behavior of UHPFRC primarily affects the post-peak behavior of the strengthened concrete beam, both having little impact on the shear load and deformational capacity. Still, the peak load and deformational capacity are underestimated if the contribution of UHPFRC softening branch (i.e., fracture energy) under tension is not considered.

(3) Non-uniform fiber distribution affects material properties of UHPFRC, and further determines the load-deflection response and failure pattern of the strengthened beams. More uniform fiber distribution in UHPFRC leads to higher shear capacity and ductility of the strengthened beams. However, a threshold value of fiber distribution factor exists. If the fiber distribution factor is above the threshold value, more uniform fiber distribution has little effect. If it is below, shear resistance decreases and the failure pattern changes. Instead of shear failure, the composite structure fails due to combination of shear and splitting cracks between UHPFRC lamellas with different fiber contents.

Chapter 8 Retrospection, conclusion and future research

8.1 Retrospection

Ultra-high performance fiber reinforced concrete, due to its excellent mechanical properties and superior durability, has been considered as a promising cementitious material for repair/strengthening. However, studies on the strengthening performance of RC structures with UHPFRC in shear are limited. The objective of this research is to investigate the shear behavior of RC structures strengthened with UHPFRC. The study primarily focuses on two key aspects: the performance of the concrete-UHPFRC interface and the material properties of UHPFRC, both of which play crucial roles for the strengthening efficiency of UHPFRC.

In Chapter 3 and Chapter 4, the shear behavior of the reinforced concrete beam strengthened with UHPFRC is investigated. Many parameters are considered, including surface saturation of existing concrete, casting direction of UHPFRC, differential shrinkage, and different bonding techniques between UHPFRC and existing concrete. In addition, in order to simulate the real exposure conditions, beams are also subjected to freeze-thaw cycles prior to mechanical loads. Time-dependent deformation due to restrained shrinkage and structural behavior under mechanical loads including load-(mid-span) deflection response, fracture development, cracking kinematics, and the relative interface displacements, are analyzed.

In Chapter 5, the potential of using active infrared thermography to detect interface delamination between UHPFRC and concrete in strengthened beams is evaluated. A standardized testing method, including a post-processing method for delamination detection of beams strengthened with UHPFRC is proposed using small-scale composite specimens. The developed testing procedures and analytical methods are extended to examine the interface performance in hybrid beams. In Chapter 6, the electromagnetic method is employed to assess non-uniform fiber distribution in thin UHPFRC laminates by estimating fiber content and orientation. The reliability of this method is evaluated, taking into account the edge effect, surface roughness, and white paint. The influence of casting direction and vibration time on the fiber distribution of UHPFRC, is investigated, with accuracy verified through x-ray imaging. Analytical methods are proposed to predict the mechanical properties of UHPFRC based on fiber contents.

In Chapter 7, a numerical study is performed to analyze the impact of key parameters on the shear response of UHPFRC-RC beams. The study specifically investigates the effects of UHPFRC-concrete interface properties, the tensile constitutive law of the strengthening material, and the influence of non-uniform fiber distribution in UHPFRC.

8.2 Conclusions

Based on the current study, the main findings and conclusions can be drawn as follows:

(1) Due to the significant autogenous shrinkage that appears at the early age of UHP-FRC, shrinkage-induced stress must be considered to avoid the risk of early-age cracking or delamination in concrete structures strengthened with in-situ cast UHPFRC (Sections 4.2 and 4.3).

(2) The moisture state of the concrete substrate affects the bond between the UHP-FRC and normal concrete (NC). Air-dry substrate provide better interface performance due to stronger surface interlock, whereas saturated surface dry substrate increase the risk of localized delamination caused by restrained shrinkage (Sections 4.3 and 4.4).

(3) Both UHPFRC and NC show high durability under specified freeze-thaw conditions with no significant material damage after 120 cycles. For reinforced concrete structures strengthened with UHPFRC, freeze-thaw cycles have minimal impact on shear performance, with fiber distribution playing a more critical role in the overall structural response (Sections 4.3 and 4.4).

(4) The bonding technique plays a crucial role in the shear strengthening efficiency of UHPFRC. Compared to cast in-situ and mechanical anchorage techniques, epoxy bonding provides a more robust system with negligible interface displacement, even under freeze-thaw conditions (Section 4.3).

(5) The fiber distribution in UHPFRC panels significantly influences the shear strengthening efficiency and casting direction of the UHPFRC affects its fiber distribution. Horizontally cast panels achieve a more uniform fiber distribution, resulting in higher load, deflection, and energy absorption capacity compared to vertically cast panels. Consequently, vertical casting requires optimization of the mix design to ensure uniform fiber distribution and minimize fiber segregation (Sections 4.3 and 4.4).

(6) Considering the key parameters (i.e., distance between specimen and lamp, optimal heating, and cooling time), a standardized testing procedure using active infrared thermography (IRT) is established to detect the subsurface delamination in UHPFRC-strengthened concrete structures. Two post-processing analytical models—one with prior knowledge of sound areas and one without—are developed, and both methods are effective in detecting the location and size of defects, despite being sensitive to edge effect, possible uneven heating, and environmental noise. While the methods perform well on small-scale specimens, challenges persist when applied to hybrid beams, primarily due to difficulties in achieving uniform heating across the beam surfaces (Sections 5.2~5.6).

(7) The non-destructive electromagnetic method is effective in estimating the fiber content and orientation in thin UHPFRC elements. Factors such as edge effect, surface roughness, and casting direction influence measurement accuracy. The casting direction and vibration time influence the fiber distribution and orientation, with longer vibration times causing more pronounced fiber segregation (Section 6.4).

8.3 Recommendation for future research

(8) The electromagnetic method, validated by x-ray imaging, provides useful insights into fiber distribution in hybrid UHPFRC-RC beams and its influence on beams' shear performance. Analytical models are developed to predict the compressive strength, elastic modulus, and tensile behavior of UHPFRC as a function of its fiber content (Sections 6.5 and 6.6).

(9) The numerical investigation confirms that the interface properties between UHPFRC and NC are critical to the shear performance of the strengthened RC beams with UHPFRC. Both the bond strength and the interface stiffness significantly affect the shear response of the strengthened beams, highlighting the need for a proper interface design to fully activate the UHPFRC strengthening efficiency (Section 7.4.1).

(10) The tensile strength and strain hardening properties of UHPFRC enhance the structural performance, with tensile strength particularly improving load capacity and the strain hardening property mainly improving the ductility. The elastic modulus and softening branch of the UHPFRC have negligible influence on the load and deformational capacities of the strengthened beam. The elastic modulus mainly affects the initial stiffness of the strengthened structure and the softening branch of UHPFRC primarily influences structural post-peak behavior. Still, the peak load and deformation capacity are underestimated if the softening branch is not considered (Section 7.4.2).

(11) The non-uniform fiber distribution affects the load-deflection response and failure patterns of UHPFRC-RC beams. In general, higher uniformity will lead to larger shear capacity and ductility, but there is a threshold value for fiber distribution factor beyond which the effect is minor (Section 7.4.3).

8.3 Recommendation for future research

In order to better understand the shear performance of concrete structures strengthened with UHPFRC, further research is recommended:

(1) The geometry and proportions of the beam in the current study are not realistic (the beam size is much smaller than the representative beams in existing structures), which can have a significant effect on the shear behavior. The role of size effect and possible benefits of shear strengthening with UHPFRC for deep beams should be studied in the future.

(2) For cast in-situ applications with UHPFRC, moisture exchange between UHPFRC and concrete is critical to the shear strengthening efficiency. Besides structural tests in current research, studies at the microscale and mesoscale are needed to systematically study the influence of moisture transport on the bond quality and microstructure development of UHPFRC.

(3) After exposure to specified freeze-thaw conditions in the current study, no obvious damage is observed in the material properties of UHPFRC and concrete, and bond at the interface. However, in practice, for example, concrete bridges will face other harsh conditions that are not tested in this study. To comprehensively assess the robustness of the UHPFRC strengthening system, further investigations such as exposure to frost salt scaling and high temperatures are needed. For example, given that epoxy is sensitive to elevated temperatures, its bond performance under such conditions must be examined to ensure the long-term durability of the system.

(4) With the use of active infrared thermography, the developed standardized testing and post-processing methods are effective in detecting the delamination of hybrid UHPFRC-concrete structures. However, future research is still necessary to further improve the detection accuracy for small delamination thickness and to reduce the influence of noise, edge effect, and uneven heat distribution. For example, always including reference sample next to the investigated sample could potentially improve detection accuracy.

(5) The internal reinforcement in the beams may influence the accuracy of inductance measurements. However, this effect can be minimized if the reinforcement is located deeper than the effective measurement range of the magnetic probe or if an adequate concrete cover is provided. Additionally, calibrating the probes using specimens without reinforcement may not fully reflect real beam conditions. Therefore, it is advisable to perform calibration using specimens that more closely represent actual beam configurations, ensuring reliable measurement performance under realistic conditions.

(6) Based on the fiber contents measured by electromagnetic method, analytical methods are proposed to predict the mechanical properties of UHPFRC. However, some predictive formulas are empirical and physical-based equations still need to be developed.

(7) Pilot tests on a real bridge strengthened with UHPFRC in shear, typically focusing on the interface preparation prior to the application of UHPFRC and examination of fiber distribution in UHPFRC after casting, are suggested to further investigate the on-site application of UHPFRC under real exposure conditions.

8.3 Recommendation for future research

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List of publications

JOURNAL ARTICLES:

- [1] **Huang, Y.**, Grünewald, S., Schlangen E., & Luković, M. (2022). Strengthening of concrete structures with ultra high performance fiber reinforced concrete (UHPFRC): A critical review. *Construction and Building Materials*, 336, 127398.
- [2] **Huang, Y.**, Gu, D., Mustafa, S., Grünewald, S. & Luković, M. (2023). Shear behaviour of reinforced concrete beams strengthened with ultra-high performance fiber reinforced concrete (UHPFRC). *Case Studies in Construction Materials*, 19, e02441.
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