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**STRENGTH MODELS
FOR PREDICTING
THE BEHAVIOR OF BOTH
HPFRCC/UHPFRC OVERLAYS
AND BONDING INTERFACE
OF RETROFITTED
NSC-HPFRCC/UHPFRC
COMPOSITE STRUCTURES**

PhD Thesis

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“Non c’è nulla interamente in nostro potere, se non i nostri pensieri”.
René Descartes

“The measure of intelligence is the ability to change”.
Albert Einstein

Abstract

In the field of retrofitted cement-based composite structures, a damage process along the interface between the concrete substrate and the cement-based overlay material can be observed after a period of service. High Performance Fibre-Reinforced-Cementitious-Composite and Ultra-High Performance Fibre-Reinforced-Concrete, also known as HPFRCC and UHPFRC respectively, are better suited than Normal-Strength-Concrete (NSC) for repairing and reinforcing this kind of structures, thanks to their better performances. But, as far as the mechanical properties of HPFRCC/UHPFRC could be better, the risk of incompatibility between two different bonded cement-based materials might be high, if adequate considerations are missing. In addition a wider application of HPFRCC/UHPFRC in the field is still limited for two main reasons. First, the manufacturing cost of such products, especially for UHPFRC, is much higher than that of ordinary concretes (from 5 to 10 times higher). This is mainly due to the presence of metallic fibers embedded within the cementitious mixture. Second, the continuous innovation of both the matrix and fibers features modifies the mechanical properties of the fibrous mortar, in comparison with its previous version. Thus, the data obtained by testing the previous version of the product do not have sufficient standing anymore. This calls for further testing so that the renewed formulation of the fibrous mortar meets the design requirements. A solution can be found by developing a proper strength model that predicts a formulation of the fibrous mortar, in terms of matrix strength and fiber properties, suitable for each kind of application, ensuring the performances requested by designers, e.g. a first cracking strength higher than 6 MPa, a hardening post-cracking behavior, etc. Most concerning models found in Literature were fitted for a specific data-set for predicting the compressive and the splitting tensile strengths of NSC/HPCC materials reinforced by adding steel fibers within the cementitious mixture. Such models are limited on two aspects. First, they do not provide enough information about the mechanical performances requested by designer, e.g. the post-cracking tensile response is unknown because of the nature of the splitting tension test. Second, such models were developed only on NSC/HPCC materials studied with few types and dosages of fibers. This makes these models reliable only for predicting the mechanical response of fibrous mortars whose formulation, in terms of matrix strength and fiber properties, was very similar to that investigated.

The first part of this PhD work thesis presents a generalized model that predicts the compressive, tensile and bending flexural responses of any HPFRCC/UHPFRC, as the matrix strength and the fiber properties change. The model was validated on the basis of the experimental data recorded by testing both lab specimens and full-scale structural beams made of different advanced cement-

based fiber-reinforced materials. The proposed model not only extends the range of application of the previous models to the family of UHPFRC materials, but also introduces the prediction of both the toughness and the post-cracking tensile response. In addition, an analytical calculation based on the "Composite Material Theory" (CMT) was implemented into the model in order to estimate the bending flexural response once the compressive and the tensile behaviors are predicted by the model. Consequently the model can also correlate the results provided by the different mechanical tests of characterization requested for a given product; for example, if the compressive and the tensile responses of a given HPFRCC or UHPFRC are measured or predicted by the model, the latter can estimate the bending flexural response, reducing both time and costs of characterization. Moreover, the model permits to minimize the manufacturing costs of the commercial product, by detecting the optimal type and the minimal dosage of fibers necessary to meet the mechanical performances required, in terms of compressive, tensile, flexural strengths and toughness. This statement was confirmed by using the model in a practical case, whose purpose was to virtually optimize the type and the dosage of fibers embedded in an existing UHPFRC structure.

The rehabilitation of existing concrete composite structures with a layer of HPFRCC/ UHPFRC can provide higher performances to the structure, in comparison with composite structures rehabilitated with a layer of standard concrete overlay. However, the risk of incompatibility between two different cement-based materials might be high, if adequate bonding conditions are not ensured. The bond failure at the interface between the concrete substrate and the cement-based overlay material is caused by both the volume changes between bonded materials and the effects of the loading conditions on the composite member. Whatever the cause, the peeling traction (mode I) and shear stresses (mode II) mutually arise along the interface according to mixed-mode. This phenomenon especially occurs near to edges of discontinuities of the overlay, e. g. boundary locations and joints. Current bond test methods permit to evaluate the loading bond capacity, even though they cannot measure the coupling factor between mode I and mode II. This can result in an improper estimation of the bond response in the field. Due to a lack of experimental data, the existing models propose theoretical formulations of the coupling factor, which are fitted on specific cases and difficult to generalize. Another aspect that may lead to an improper estimation of the effective bond response concerns the assumption that the substrate surface is properly prepared prior to the application of the overlay. An optimal substrate preparation mainly depends on both the moisture level of the substrate surface and the fresh properties of the latter. Both factors call for further investigations, since controversial opinions are found in Literature.

The second part of this PhD work thesis presents a theoretical cohesive model that predicts the loading bond capacity of an existing concrete structure overlaid with a layer of HPFRCC/UHPFRC, by taking into account the coupling factor between mode I and mode II, the moisture level of the substrate surface prior to the application of the overlay and the fresh properties of the latter. Different bond tests were designed by Author in order to accurately calibrate the parameters implemented into the modeled interface, in particular, the peeling-crack (mode I), the shear-slip (mode II) responses and the coupling factor between mode I and II. An independent experimental campaign was carried out to validate the theoretical results. A Finite-Element (FE) model was developed in

order to numerically reproduce the propagation of the damage process at the interface subjected to mixed-mode stresses. The FE interface was modeled with a nonlinear cohesive law which couples the relationships of mode I with mode II on the basis of the experimental data.

Some parts of this PhD work thesis reflect the content reported in the following works:

- Savino V, Lanzoni L, Tarantino AM, Viviani M. *Tensile constitutive behavior of high and ultra-high performance fibre-reinforced concretes*. Construction and Building Materials 186, 2018, pp. 525 – 536;
- Savino V, Lanzoni L, Tarantino AM, Viviani M. *Simple and effective models to predict the compressive and tensile strength of HPFRC as the steel fiber content and type change*. Composites Part B 137, 2018, pp. 153 – 162;
- Savino V, Lanzoni L, Tarantino AM, Viviani M. *An extended model to predict the compressive, tensile and flexural strengths of HPFRCs and UHPFRCs: Definition and experimental validation*. Composites Part B 163, 2019 pp. 681 – 689;
- Savino V, Lanzoni L, Tarantino AM, Viviani M. *A cohesive model to predict the loading bond capacity of repaired/reinforced concrete structures stressed to mixed modes*. Cement and Concrete Composites, Under Submission, 2019;
- Savino V, Lanzoni L, Tarantino AM, Viviani M. *A cohesive FE model for simulating the cracking/debonding pattern of composite NSC-HPFRC/ UHPFRC members*. Construction and Building Materials, Under Submission, 2019;
- Savino V. *Modèle prédictif visant à optimiser les composants du BFUP en réponse aux exigences d'application*. In: Proceedings of the Troisième journée d'étude BFUP- Béton Fibré Ultra-Performant: concevoir, dimensionner, construire, Fribourg (Switzerland), 2019 pp. 101 – 111.

Sommario

Nelle strutture ripristinate a base cementizia, dopo un certo periodo di servizio si osserva talvolta un processo di danneggiamento lungo l'interfaccia tra il substrato in calcestruzzo e lo strato superiore di protezione. Il Composito-Cementizio Fibro-Rinforzato ad Alte-Prestazioni e il Calcestruzzo Fibro-Rinforzato ad Altissime-Prestazioni, noti rispettivamente come HPFRCC e UHPFRC, sono più adatti dei Calcestruzzi Normo-Resistenti (NSC) per riparare e rinforzare questo tipo di strutture, grazie alle loro prestazioni di gran lunga superiori. Tuttavia, per quanto le proprietà meccaniche dei HPFRCC/UHPFRC possano essere eccellenti, il rischio di incompatibilità di adesione tra due diversi materiali a base cementizia può essere alto, senza adeguate considerazioni. Inoltre una più estesa applicazione di tali materiali nella pratica è ancora limitata per due ragioni principali. In primo luogo, il costo di produzione di tali prodotti, specialmente quando si parla di UHPFRC, è molto più alto rispetto al rispettivo richiesto per i calcestruzzi ordinari (da 5 a 10 volte superiore). Questo è dovuto soprattutto alla presenza di fibre metalliche immerse nella miscela cementizia. In secondo luogo, l'innovazione continua delle proprietà della matrice e delle fibre comporta una modifica continua delle proprietà meccaniche del materiale composito, rispetto alla sua versione precedente. Di conseguenza, i dati ottenuti testando la versione precedente del prodotto non sono più rappresentativi. Ciò richiede test ulteriori affinché la formulazione rinnovata del materiale composito rispetti i requisiti prestazionali richiesti dal progetto. Una soluzione può essere quella di sviluppare un appropriato modello di resistenza che predice una formulazione del materiale composito, in termini di resistenza della matrice e di proprietà delle fibre, adatta a ciascun tipo di applicazione, assicurando le prestazioni richieste dai progettisti, per esempio una resistenza di prima fessurazione superiore a 6 MPa, un comportamento incrudente post-fessurativo, etc. Sono stati trovati in Letteratura alcuni modelli calibrati per un set specifico di dati al fine di predire la resistenza alla compressione e a trazione da splitting di NSC/HPCC rinforzati con fibre d'acciaio. Tali modelli sono tuttavia limitati su due aspetti. Il primo motivo è che non vengono fornite informazioni sufficienti sulle proprietà prestazionali richieste dai progettisti, ad esempio la resistenza a trazione post-picco è incognita a seguito della natura dello splitting test a trazione. In secondo luogo, tali modelli sono stati calibrati solo su materiali di classe NSC/HPCC studiati con pochi tipi e dosaggi di fibre. Ciò li rende affidabili solamente per predire la risposta meccanica di materiali compositi la cui formulazione, in termini di resistenza della matrice e proprietà delle fibre, risulti molto simile a quella del materiale studiato.

La prima parte di questa tesi di dottorato presenta un modello generalizzato che predice la risposta alla compressione, alla trazione e al momento flettente

di qualunque HPFRCC/UHPFRC, al variare della resistenza della matrice e delle proprietà delle fibre. Il modello è stato validato sulla base dei dati di test sperimentali effettuati sia su provini di laboratorio, sia su travi in scala reale composti da differenti materiali fibrorinforzati a base cementizia. Il modello proposto non solo estende il campo di applicazione dei modelli esistenti alla famiglia dei materiali UHPFRC, ma introduce la predizione della duttilità e del comportamento a trazione post-picco. Inoltre, un calcolo analitico basato sulla "Teoria del Materiale Composito" (CMT) è stato implementato nel modello al fine di stimare la risposta a flessione una volta che le risposte alla compressione e alla trazione vengono predette dal modello. Il modello può quindi essere utile per correlare i risultati ottenuti dai differenti test di caratterizzazione richiesti per un dato prodotto; per esempio se le risposte alla compressione e alla trazione di un HPFRCC o UHPFRC sono state misurate o predette dal modello, quest'ultimo può stimare la risposta a flessione, riducendo i tempi e i costi di caratterizzazione meccanica. Inoltre il modello permette di minimizzare il costo di produzione del prodotto commerciale, individuando il tipo ottimale e il dosaggio minimo di fibre necessarie per soddisfare le esigenze prestazionali richieste, in termini di resistenza a compressione, trazione, flessione e duttilità. Questa affermazione è stata confermata applicando il modello in un caso pratico che consisteva nell'ottimizzare virtualmente il tipo e il dosaggio di fibre immerse in una struttura esistente in UHPFRC.

Il ripristino di strutture composite a base cementizia mediante uno strato di HPFRCC/ UHPFRC può fornire alla struttura prestazioni superiori, in confronto a strutture ripristinate con uno strato di calcestruzzo standard. Tuttavia, senza adeguate condizioni all'interfaccia, il rischio di incompatibilità di adesione tra due diversi materiali a base cementizia può essere alto. La crisi nel collegamento di interfaccia tra il substrato in calcestruzzo e il materiale di ricoprimento a base cementizia è causata sia dai cambiamenti di volume tra i materiali collegati, sia dagli effetti delle condizioni di carico sulla struttura composita. Qualunque sia la causa, le tensioni di peeling (modo I) e di taglio (modo II) nascono sull'interfaccia secondo modi misti. Questo fenomeno si manifesta specialmente vicino ai bordi delle discontinuità dello strato superiore, come le zone perimetrali e giunti. I metodi attuali sui test di collegamento permettono di valutare la capacità di carico di quest'ultimo, anche se non è possibile misurare il fattore di accoppiamento tra modo I e modo II. Nella pratica, questo può portare ad una stima impropria della capacità di carico del collegamento. In presenza di dati sperimentali insufficienti, i modelli esistenti propongono formulazioni teoriche sul fattore di accoppiamento che sono adattate a casi specifici e difficili da generalizzare. Un altro aspetto che può portare ad una stima impropria della capacità di carico del collegamento riguarda l'assunzione che la qualità effettiva della superficie di preparazione del substrato sia stata preparata a regola d'arte. Una preparazione ottimale del substrato dipende principalmente sia dal livello di umidità della superficie del substrato prima dell'applicazione dello strato di riparazione/rinforzo, sia dalle proprietà allo stato fresco di quest'ultimo. Entrambi i fattori richiedono ulteriori indagini, dal momento che sono state trovate opinioni controverse in Letteratura.

La seconda parte di questa tesi di dottorato presenta un modello teorico di principio coesivo che predice la capacità di carico del collegamento di interfaccia di una struttura di calcestruzzo esistente ricoperta da uno strato di HPFRCC/UHPFRC, prendendo in considerazione il fattore di accoppiamento tra il modo I e modo

II, il livello di umidità superficiale del substrato prima della posa dello strato superiore e le proprietà allo stato fresco di quest'ultimo. Differenti test sul collegamento sono stati concepiti dall'Autore al fine di calibrare accuratamente i parametri implementati nel modello di interfaccia, in particolar modo le relazioni di peeling-apertura di fessura (modo I), taglio-scorrimento (modo II) e il fattore di accoppiamento tra il modo I e II. Un indagine sperimentale indipendente è stata compiuta per confermare i risultati teorici. Un modello agli Elementi-Finiti (FE) è stato sviluppato al fine di riprodurre numericamente il processo di propagazione di danneggiamento in interfaccia soggetta ad uno stress di modo misto. L'interfaccia FE è stata modellata mediante l'uso di una legge coesiva non lineare che accoppia le relazioni di modo I con quelle di modo II, sulla base dei dati sperimentali.

Diverse parti di questa tesi di dottorato riflettono il contenuto riportato nei seguenti lavori:

- Savino V, Lanzoni L, Tarantino AM, Viviani M. *Tensile constitutive behavior of high and ultra-high performance fibre-reinforced concretes*. Construction and Building Materials 186, 2018, pp. 525 – 536;
- Savino V, Lanzoni L, Tarantino AM, Viviani M. *Simple and effective models to predict the compressive and tensile strength of HPFRC as the steel fiber content and type change*. Composites Part B 137, 2018, pp. 153 – 162;
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- Savino V, Lanzoni L, Tarantino AM, Viviani M. *A cohesive model to predict the loading bond capacity of repaired/reinforced concrete structures stressed to mixed modes*. Cement and Concrete Composites, Under Submission, 2019;
- Savino V, Lanzoni L, Tarantino AM, Viviani M. *A cohesive FE model for simulating the cracking/debonding pattern of composite NSC-HPFRC/ UHPFRC members*. Construction and Building Materials, Under Submission, 2019;
- Savino V. *Modèle prédictif visant à optimiser les composants du BFUP en réponse aux exigences d'application*. In: Proceedings of the Troisième journée d'étude BFUP- Béton Fibré Ultra-Performant: concevoir, dimensionner, construire, Fribourg (Switzerland), 2019 pp. 101 – 111.

Symbols and abbreviated terms

Symbols

A	Area of the overlay cross section
$A_{c,i}$	Nominal cross section area of the cubic specimen
$A_{t,i}$	Nominal cross section area of the dog-bone shaped specimen
E	Young modulus of the overlay
E_f	Young modulus of the steel fiber
G	Shear modulus of the overlay
I	Moment of inertia of the overlay cross section with respect its centroid axis
M	Internal moment applied at the overlay cross section
$M_{R,a}$	Ultimate bending moment prior to the cracking of the fibrous mortar
$M_{R,b}$	Ultimate bending moment after cracking of the fibrous mortar
N	Axial internal force applied at the overlay cross section
$P_{c,i}$	Compressive strength in force of the i fibrous mortar
$P_{t,i}$	Uniaxial tensile strength in force of the i fibrous mortar
$P_{tf,i}$	Indirect tensile strength in force of the i fibrous mortar
R^2	Coefficient of determination of statistical regression analysis
RI	Reinforcing index of the matrix mortar
T	Shear internal force applied at the overlay cross section
V_f	Volume fraction of fibers within the fibrous mortar
$V_{f,crit}$	Critical volume fraction of fibers leading to hardening response
$V_{f,crit,flexion}$	Volume fraction of fibers leading to hardening flexural response
$V_{f,crit,traction}$	Volume fraction of fibers leading to hardening uni-axial response
a_0	Tip of the notch of the prismatic specimen in the mid-span section
$a_q, b_q, c_q, a_\tau, b_\tau, c_\tau$	Coefficients of the analytical relationships of the bonding interface
b	Width of the prismatic specimen
c	Coupling factor between mode I and mode II
d_e	Equivalent diameter of the steel fiber
f_c	Compressive strength of the fibrous mortar
$f_{c,i}$	Compressive strength of the i fibrous mortar
$f_{c,m}$	Mean compressive strength of n fibrous mortar specimens
$f_{c,matrix}$	Mean compressive strength of the matrix mortar
f_{sp}	Mean splitting tensile strength of the fibrous mortar
$f_{sp,matrix}$	Mean splitting tensile strength of the matrix mortar
f_t	Uniaxial tensile stress at the first cracking of the fibrous mortar
$f_{tf,i}$	Indirect tensile strength by 4PBFT of the i fibrous mortar
$f_{tf,m}$	Mean indirect tensile strength by 4PBFT of n fibrous mortar specimens
$f_{t,matrix}$	Uniaxial tensile stress at the first crack of the matrix
$f_{t,i}$	Uniaxial tensile stress at the first cracking of i fibrous mortar specimens
$f_{t,m}$	Mean uni-axial tensile strength of n fibrous mortar specimens
f_{rt}	Residual tensile tension value at 1.5% of the axial strain of the fibrous mortar
h	Height of the prismatic specimen
l	Length of the prismatic specimen

l_f	Length of the steel fibers
l_f/d_c	Aspect ratio of fibers
n	Number of specimens
q	Peeling stress at the interface
u	Slip between the substrate and the overlay
v	Opening between the substrate and the overlay
x	Horizontal coordinate position parallel to the bonding interface
x^*	Neutral axis of the inflected prismatic section
y	Vertical coordinate position perpendicular to the bonding interface
χ	Dimensionless shear factor of the beam cross section
χ^*	Curvature of the inflected prismatic section
ϵ_{mu}	Deformation corresponding to σ_{mu}
$\mu\epsilon$	Micro strain
ν	Poisson's ratio of the overlay
Φ	Diameter of the cross-section specimen
σ	Standard deviation
σ_c	Normal stress acting on the fibrous mortar
σ_{comp}	Normal stress acting on the most compressed region
σ_{cu}	Maximum strength of the fibrous mortar
σ_{mu}	Maximum matrix tension just before the cracking
σ_{fu}	Rupture/pullout stress of the steel fibers
τ	Shear stress at the interface
φ	Relative rotation between the overlay and the substrate members

Abbreviations

3PBFT	Three Points Flexural Bending Test
4PBFT	Four Points Flexural Bending Test
CF	Compressive Force
CM	Compressive Moment
CMOD	Crack Mouth Opening Displacement
CMT	Composite Material Theory
CO	Core specimens
COV	Coefficient Of Variation
CSP	Concrete Surface Profile
CT	Compressive Test
CTOD	Crack Tip Opening Displacement
CU	Cubic specimens
DB	Dog-Bone shaped specimens
DTBT	Direct Tensile Bond Test
DUP	Drilled Un-notched Prismatic specimens
F	Statistic F-test
FE	Finite Element
HF	Hooked-end fiber
HPCC	High-Performance Cementitious-Composite
HPFRCC	High-Performance Fiber-Reinforced-Cementitious-Composite
LVDT	Linear Variable Differential Transformers
LST	Lateral Shear Test

MS	Mean Square
MX	x Mortar type
NP	Notched Prismatic specimens
NSC	Normal-Strength-Concrete
POBT	Pull-Off Bond Test
SFRC	Steel Fiber-Reinforced-Concrete
SS	Sum of Square
POBT*	Modified Pull-Off Bond Test
DSBT	Direct Shear Bond Test
RH	Relative Humidity
SF	Straight fiber
SSD	Saturated Surface Dry
STBT	Shear/Tension Bond Test
STT	Splitting Tension Test
TF	Tensile Force
TM	Tensile Moment
UHPFRC	Ultra-High-Performance Fiber-Reinforced-Concrete
UHPC	Ultra-High-Performance Concrete
UP	Un-notched Prismatic specimens
UTT	Uniaxial Tensile Test
df	Degree of freedom
w/c	Water Cement Ratio

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Part I

A predictive strength model for HPFRCC/UHPFRC materials

Preface of Part I

This PhD work thesis is composed by two parts. The first part is subdivided into four chapters which are briefly described in the following.

The first chapter presents the main issues concerning the application in field of High-Performance and Ultra-High-Performance Fiber-Reinforced-Concrete materials and the interest to adopt a proper strength model for predicting their mechanical behavior. Relevant aspects concerning the available predictive strength models suitable for Steel-Fiber-Reinforced-Concrete materials are discussed in this chapter. The attention is mainly paid to works of different authors who have found some correlations between the formulation of a SFRC material (matrix and fiber features) and its compressive and splitting tensile strengths. Results and shortcomings shall be highlighted.

In order to develop a more extended and suitable model not only for HPFRCC materials, but also for the class of UHPFRC, a proper experimental campaign was carried out for calibrating the proposed model. The experimental program is presented in the second chapter. Different commercial products were characterized according to standard compressive, uni-axial tensile and flexural bending tests, by varying the matrix strength and the fiber properties.

The third chapter includes an explanation about the calibration of the proposed model on the basis of the experimental results presented in the second chapter. Multiple linear regression analysis were carried out in order to develop simple and reliable predictive relationships between the formulation of HPFRCC/UHPFRC and its compressive and tensile responses. The third chapter also presents an analytical calculation based on the CMT that predicts the flexural behavior of the HPFRCC/UHPFRC once the compressive and tensile responses are known. The application of the proposed model in a practical case is presented at the end of the third chapter.

The main conclusions of the first part of this PhD work thesis shall be argued in the fourth chapter.

Chapter 1

HPFRCC and UHPFRC materials : State-of-the-art

1.1 Steel fibers

The use of fibers to enhance the mechanical behavior of brittle materials is not a new concept. Since ancient times straw was used to reinforce sun-baked bricks, as well as horsehairs were used in masonry mortar and plaster to enhance certain mechanical properties. In 1898 the invention of the Hatschek process promoted the commercial use of asbestos fibers within the cement-based matrix. Between 1960 and 1970 new fiber types were introduced and substituted the asbestos fibers. In the last decades some researchers investigated the use of steel fibers reinforcement and demonstrated the advantages of this new technology [1, 29, 54, 69, 78, 91, 104, 107, 108, 110, 111, 113, 122, 132, 142] that led to successfully applications in field [6, 8, 76, 95]. On the hand new shapes and types of steels improved the fibers^{1,2} performances [23, 32, 53, 131], on the other hand new optimized gradation of granular skeleton permitted to obtain the high packing density of the cement-based matrix named High and Ultra-High Performance-Fiber-Reinforced Concretes (HPFRCC and UHPFRC). In Literature steel fibers are usually classified by:

- *Geometry*: several shapes are proposed by the producers e.g. straight, hooked, undulated, flat-end etc;
- *Equivalent diameter*: for fibers that are not circular in cross-section, the equivalent diameter is the diameter of the circle having the same area of the average cross-section of the actual fiber. The greater the equivalent diameter, the greater the flexural strength;
- *Aspect ratio*: is a measure of the slenderness of the fiber defined as the ratio between the length of the fiber and the equivalent diameter (l_f/d_e);

¹FRC based on synthetic fibers has been investigated, as an example, in [79, 98], whereas a recent study about the retrofitting of concrete beams strengthened by a polymer-based mortar has been addressed in [81]. The mechanical performances steel cords and a geopolymeric matrix as strengthening system has been discussed in [8].

²Recent studies about FRCs based on synthetic fibers can be found in [79, 98]

- *Volume fraction*: is the concentration of fibers within a given unit volume of fibrous mortar (V_f);
- *Strength*: is the stress capacity that fiber reaches before failure;
- *Toughness*: is considered as the ability of the fiber to adsorb strain energy before failure.

HPCC and UHPC matrix are characterized by a low tensile strength and ductility that can be improved by adding steel fibers into the fresh cementitious mixture.

1.2 HPCC and UHPC matrix

High-Performance Fiber-Reinforced-Cementitious-Composite materials, also named HPFRCC, are among the breakthroughs of the recent concrete technology. These materials have found their place in the market and in the codes after being thoroughly developed by scientists. Besides using HPFRCC as new material structure its applications as repairing material on deteriorated structures showed significant attention in the last years [34, 75]. Its success promoted the development of a number of commercial products manufactured according to technical data sheets. Obviously, the mechanical behavior of HPFRCC should be known beforehand by the designer in order to ensure that its performances could be suitable in the field. However, due to the heterogeneity of its composition, many parameters affect its mechanical performances, such as fiber properties mentioned in Section 1.1. Factors like the aspect ratio of fibers [160], fiber shape [149], fiber dosage [71, 161], fiber orientation, fiber distribution [160] and the matrix composition proved to strongly influence the mechanical response of the HPFRCC material, to the point that these parameters were successfully used to develop some predictive strength models [106, 135, 152]. This point will be comprehensively discussed in Section 1.5.

In recent years, a more advanced generation of concretes named Ultra-High-Performance Fiber-Reinforced-Concrete (UHPFRC) - whose properties are superior in terms of strength, toughness and durability than HPFRCC [30, 117, 118, 119] - were introduced in several applications, especially in North America, Japan, France, Switzerland and Australia. According to the state-of-the-art report [181] 55 bridges using UHPFRC have been built or are under construction in U.S. and Canada. There are 22 in Europe and 27 in Asia and Australia. In these applications UHPFRC is used as beams and deck panels. UHPFRC has also been used in certain pilot projects of rehabilitation and strengthening such as bridge deck overlays [22, 63] and hydraulic structures [59, 100]. Nevertheless the high manufacturing cost and the missing harmonization of existing codes restrict still its wider usages. Technical guides and professional recommendations, available in United States, Japan, France, and Switzerland, define the UHPC in different ways. According to [183] UHPC is a concrete whose compressive strength exceeds the characteristics of the C110/115 class. Instead, [186] defines UHPC as a material whose compressive and tensile strengths are of a minimum 150 MPa and 5 MPa, respectively. According to [190] UHPC is a material whose compressive strength is higher than 150 MPa, and the size of aggregates and water-binder ratio respect some limits. [187] prescribes the target values for elastic modulus, Poisson's ratio, compressive strength, elastic tensile strength for classifying a

UHPC. A common point in Literature is that the mechanical behavior of the UHPFRC in tension can be strain-softening or strain-hardening, according to its composition [94]. Such a behavior can be simplified in a non-linear path composed by three parts. Part I: linear elastic behavior up to 90 – 95% of the cracking strength. Part II: strain hardening behavior as multiple small cracks occur and stabilize, even though in certain UHPFRC materials multiple cracks are not observed [190]. Part III: softening behavior due to debonding between fibers and cracked matrix [93]. Many researchers discovered the main parameters influencing the mechanical response of the UHPFRC, e.g. [62]. In particular, the matrix properties are highly dependent on various factors such as the dosage of fine mineral mixture [27, 112, 143, 154], the size of aggregates [88, 101], the size effect of the specimen [3, 97], the curing condition [74, 163] and the rate of loading [49, 50]. Not less important is the water-to-binder ratio that strongly affects the classical high strength and the flow-able characteristic of the UHPC matrix with the presence of steel fibers [186, 190]. Other researches also focused on fiber properties that mostly affecting the compressive and tensile strengths, the elastic modulus, the crack resistance and toughness [1, 6, 65, 76, 95]³. Also the splitting and the crushing resistances of UHPFRC materials can be enhanced by using steel fibers [32]^{4,5}. The fiber shape is closely related to the magnitude of the post-cracking strain capacity of UHPFRC. In fact, when UHPC matrix failure occurs, the residual global strength is kept on until fibers bonded with the uncracked matrix are not totally pulled out. Previous studies showed the difference in magnitude of pullout strength when the shape of fibers changes [149]. A comparison of results by using straight and deformed fibers highlighted that hooked-end and twisted steel fibers achieve a maximum fiber stress three times higher than straight steel fibers. In particular, the tensile strength and post-cracking strain capacity of UHPFRC were improved by using deformed steel fibers, as also confirmed in [150]. The aspect ratio l_f/d_e (l = length of fiber, d_e = equivalent diameter) also largely influences the post-cracking behavior of UHPFRC. Adding steel fibers with a high l_f/d_e (97.5) Yoo et al. (2004) [160] improved the fracture energy of about 35% and 121% as compared to fibers of medium (81.5) and low (65) l_f/d_e , respectively. This effect is mainly due to the increase of the bonding area between the matrix and fibers, leading to higher pullout load carrying capacity. Several works, such as [71], showed that the post-cracking flexural strength improved by increasing the volume fraction. In [150] it was observed that by increasing the volume fraction (V_f) of fibers embedded within the UHPC from 1.5 to 2.5%, both the tensile strength and the strain capacity were improved of about two times. Also an increase of the compressive strength was observed in [161], even though this benefit was rather influenced by a proper orientation of fibers in the cementitious mixture. Other works were conducted to evaluate how the fiber orientation influences

³Fracture can be modeled, in a nonlinear framework, following [137], whereas recent contributes about damage mechanics can be found in [82, 83, 84]. Bifurcation and stability in the context of homogeneous finite deformations can be found in [138]

⁴An interesting study about the fiber/matrix interface in a concrete reinforced with steel fibers can be found in [131]. Experimental studies about the beneficial aspects of steel fibers in enhancing ductility of RC columns [53] and the punching shear behavior of slabs [23] have been investigated also.

⁵Recent studies about the mechanical performances of fabric-reinforced cementitious matrix composites can be found in [42, 43, 99, 126], whereas an application of a polymer-based mortar for retrofitting has been reported in [81].

the mechanical response of UHPFRC, e.g. [5, 160]. In [46] it was observed that a properly designed casting placement should improve the fiber orientation, leading to better tensile performances. For horizontal placements the viscosity of the fluid cementitious mixture and the wall effect [89] cause almost all fibers to be aligned perpendicularly to the pouring direction [19] and parallel to its flow in the molding [153].

1.3 Distribution and orientation of steel fibers within fibrous mortars

During the mixing of the fresh fibrous mortar the risk of fiber balling might be high if appropriate aspects are not taken into account. It is worth noting that fiber balling affects the mechanical strength of the hardened material. It creates honeycombs that reduce the performances of the SFRC even in large scale production. These problems are magnified if an inappropriate mixer is used. The use of special mixers and/or a proper selection of fiber type and volume fractions could permit to avoid the fiber balling. A conventional approach is to use multiple fibers bonded together with water-soluble adhesive [28], like the type HF 65 illustrated in Figure 2.1(d).

Experimental evidences observed that the vertical pouring of the fresh mixture into the laboratory molds promotes a certain level of orientation of fibers within the molds. This phenomenon is due to shear stresses which arise in highly viscose mixtures like the cement-based material. Instead, the fiber orientation near to the edge of the molds is mainly promoted by the wall effect [89]⁶ that tends to slightly reduce the V_f of the fibers in molds whose shape is quite complex e.g. dog-bone shaped molds, see Figure 2.2(b). The orientation effects can therefore reduce V_f of the fibers of the fresh cementitious mixture, which can result in a decrease of the mechanical performances of the hardened material. This aspect is a well-known issue, such that some standards propose to cut specimens from large slabs of the appropriate thickness [170].

In applications which involve the use of HPFRCC/UHPFRC, an isotropic and homogenous distribution of fiber is generally assumed. Nevertheless, the mixing, placing and vibration procedures could affect the isotropy. Vibration is not strictly necessary when large quantities of HPFRCC/UHPFRC are cast, but it is advised to increase the regularity of small specimens. Several researchers [39, 134] investigated the effect of the compaction by vibration on both the orientation and the segregation of fibers. Most of these investigations showed that external vibration might produce both planes of preferentially oriented fibers and fiber segregation. Furthermore the torque forces exerted on the fresh cementitious mixture, when is flowed, are generated by both the shear stress and the wall effect, two phenomena that tend to promote preferential orientations. However, in certain cases, the preferential orientation of the fibers is more interesting than isotropic distribution. If the fibers can be oriented perpendicularly to the crack planes, the toughness of the structure will be increased thanks to the higher number of fibers bridging the cracks [125].

⁶Viscoelastic effects in reinforced concrete structure have been accounted e.g. in [36, 37, 38].

1.4 Mechanical characterization of HPFRCC/ UH-PFRC

The norms suggest different standard tests for properly characterizing the mechanical performances of a given SFRC, especially in terms of tensile response [170, 195, 196, 197]. Uni-axial tensile tests (UTT) are preferred to indirect ones, e.g. splitting tension tests (STT), whose estimated strength could be affected by both the size effect [3, 97, 159] and the test set-up. As confirmed in [26, 141] the splitting tension tests revealed that the first tensile crack opened approximately at a third of the height of the specimen in the loading plane, in conjunction with a wedge rupture below the load bearing strips. This observation is in contrast with the basis of the calculation formula for the splitting tensile strength applied today, in which the first tensile crack should begin in the center of the specimen cross section - according to the theory of elasticity -. Standards propose other indirect tensile tests such as four-point bending flexural test on un-notched specimens (4PBFT) [168, 185] and three-point bending test on notched specimen (3PBFT) [173, 189, 185]. Although similar these two tests yield different results. In the work of [102] it was observed that 3PBFT provides values for modulus of rupture 10% higher than 4PBFT. Further investigations pointed out the problem of unexpected crack formation in the vicinity area of the notch of SFRC beams tested to 3PBFT. The notch area is a disturbed stress region because of the influence of both the load and the shear stresses. As consequence, the mechanical response of SFRC materials obtained by 3PBFT is not generally representative for notched SFRC beams [47]. 4PBFT on un-notched SFRC beams permits to prevent the direct influence of the shear stress in the mid-span. However the deflection is influenced also by the formations of shear cracks appearing outside the pure bending region and therefore the results of the tests cannot be accurate [47]. A solution to problems observed in both 3PBFT and 4PBFT can be found in a recent Italian standard [170], that presents a 4PBFT for notched prismatic beams. Results showed the formation of a limited number of cracks due to the presence of the notch. This condition is preferable for characterizing softening SFRC materials for which a single crack is often observed. In addition shear cracks does not occur, thanks to the fact that the section notched is more slender than the un-notched one, limiting the influence of shear.

It is important to note that the continuous innovation of both the matrix and fibers features modifies the mechanical properties of HPFRCC/UHPFRC available in the market, in comparison with its previous version. Thus, the data obtained by testing the previous version of the product do not have sufficient standing anymore. This calls for further testing so that the renewed formulation of the fibrous mortar meets the designer's requirements. Not negligible is the high manufacturing cost of the fibrous mortar, mainly due to presence of metallic fibers. Both aspects can deter the use of these high technology cement-based materials. For this reason, predictive strength models that also permits to optimize the formulation of HPFRCC/UHPFRC might be of great practical interest.

1.5 Predictive strength models for SFRC materials

In Literature few systematic studies deal with the effects of the matrix and fiber properties on the mechanical performances of the SFRC. While many strength models are available for a specific data-set, few researchers aimed to construct a model that predicts the mechanical performance of SFRC by taking into account the matrix and the fiber properties. In order to provide an overview, three different available models are described in the following.

Ramados et al. (2008) [106] developed a linear model to predict the compressive and the splitting tensile strengths of three SFRC materials, by increasing V_f of a given type of steel fiber from 0 to 1.5%. This model is therefore valid if l_f/d_e ratio of the fiber is not changed and if the matrix strength of the SFRC is rated as HPCC :

$$f_c(\text{MPa}) = f_{c,matrix} + 0.0184 \times RI \quad (1.1)$$

$$f_{sp}(\text{MPa}) = f_{sp,matrix} + 0.006 \times RI \quad R^2 = 0.94 \quad (1.2)$$

where $RI = w_f \times l_f/d_e$, w_f = density of fiber / density of SFRC $\times V_f$.

Yazıcı et al. (2007) [152] proposed a multi-linear relationship between the strengths of a given HPCC matrix, by changing l_f/d_e ratio and V_f . The series investigated the effects of three types of hooked-end steel fibers embedded within a given HPCC, whose V_f ranged from 0 to 1.5% :

$$f_c(\text{MPa}) = 50.4869 + 0.0434 \times l/d_e + 1.9667 \times V_f(\%) \quad R^2 = 0.10 \quad (1.3)$$

$$f_{sp}(\text{MPa}) = 2.2121 + 0.0077 \times l/d_e + 1.4233 \times V_f(\%) \quad R^2 = 0.72. \quad (1.4)$$

Sumathi et al. (2015) [135] adopted a second order polynomial equation to regress the relationships between the strengths of a given HPCC matrix reinforced by a given type of hooked-end steel fiber, by varying V_f :

$$f_c(\text{MPa}) = 41.45 + 9.984 \times V_f - 3.837 \times V_f^2 \quad R^2 = 0.91 \quad (1.5)$$

$$f_{sp}(\text{MPa}) = 3.925 + 1.448 \times V_f - 0.042 \times V_f^2 \quad R^2 = 0.99. \quad (1.6)$$

Where $f_{c,matrix}$, f_c , $f_{sp,matrix}$, f_{sp} and RI are the mean compressive strength of the matrix mortar, mean compressive strength of the fibrous concrete, mean splitting tensile strength of the matrix mortar, mean splitting tensile strength of the fibrous concrete and reinforcing index, respectively.

Despite the interesting results, the models above-mentioned present several shortcomings. They predict the tensile response just in terms of splitting tension peak. So, no information about the post-cracking tensile response is provided, which is essential to properly characterize the performances of SFRC materials. Several authors claimed that STT presents shortcomings also in terms of set-up and size effect. Further details are reported in Section 1.4. Moreover, the models above-mentioned were developed only on NSC/HPCC mortars studied with few types and dosages of fibers. This makes these models reliable only for predicting the mechanical response of fibrous mortars whose formulation, in terms of matrix strength and fiber properties, was very similar to that investigated.

1.6 Scope

In the first part of this PhD work thesis an extended model was developed for properly predicting the mechanical behavior of any commercial HPFRCC/UHPFRC, as the matrix and the fiber properties change. In particular, the compressive, uni-axial tensile and the flexural bending responses can be predicted by the proposed model, thanks to a proper regression analysis of data provided by the experimental tests carried out in this investigation. UTT were performed in order to predict the post-cracking tensile response. A further and independent experimental investigation, including standard 4PBFT on both lab specimens and full-scale beam structures, was performed to validate the proposed model. Finally the model was adopted in an existing UHPFRC structure for virtually optimizing the effective formulation of the material used.

Chapter 2

Experimental program

2.1 Materials

Four types of commercial advanced cement-based matrix were investigated, in particular a HPCC (labeled hereafter M1) and three different UHPCs (labeled hereinafter M2, M3 and M4 respectively). An experimental test of characterization permitted to classify these fibrous mortars in terms of strength class and ductility indexes, according to [182, 196], see Table 2.1. The exact composition of the matrix formulation was unknown since the producer did not provide neither the mix design of the matrix, which is simply labeled *premix*, nor the nature of its additives. However, it is well known that the premix of HPFRCC contains cement, small aggregates, sands and silica fume. Instead, UHPC premix does not contains aggregates, but an optimized gradation of granular matter to obtain a higher packing density, compared to the HPCC premix. Table 2.2 shows the compounds to fabricate one cubic meter of each investigated material, according to the producer's prescription. In each premix a specific super-plasticizer was used at high dosage to increase the strength, enhance the durability and give high workability [136]. Because of the presence of the super-plasticizer the water/cement ratio was very low. The producer advised to add in M3 a set of hardening accelerator to shorten the dormant period and to speed up the hydration process.

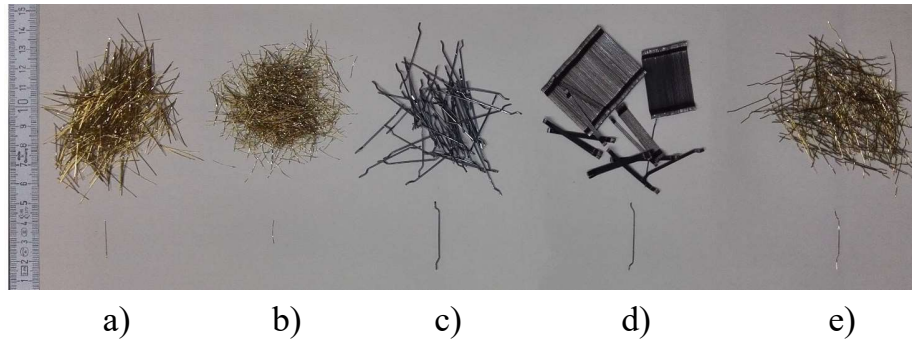


Figure 2.1: Types of steel fibers investigated: a) SF65, b) SF75, c) HF45, d) HF65, e) HF85

Table 2.3 presents in details the fiber properties mentioned before. The fibers were gradually inserted within the fresh cementitious mixture presented above, in order to obtain a good dispersion and minimizing the risk of fiber balling. Although the fiber addition does not modify remarkably the yield stress of

Class F	F2.0	F2.5	F3.0	F3.7	F4.5	F5.5	F6.5	F7.7	F9.0
$f_{1lk,min}$ (MPa)	2	2.5	3	3.7	4.5	5.5	6.5	7.7	9
Index of ductility	Softening			Plastic			Hardening		
	D_{S0}	D_{S1}	D_{S2}		D_P		D_{H0}	D_{H1}	D_{H2}
$D_{0k,min}$	≤ 0.5	≥ 0.5	≥ 0.7		≥ 0.9		≥ 1.1	≥ 1.3	≥ 1.55
Classification							M1	M2	M3
Class							C60/75	C120/140	C110/130
SFRC							HPFRCC	UHPFRC	UHPFRC
Class F							F6.5	>F9.0	F7.7
Class D_0							D_{S1}	D_{S1}	D_{S1}
Class D_1							D_{S0}	D_{S2}	D_{S2}

Table 2.1: Standard classification of M1, M2 and M3

Compounds	kg in 1 m ³ of composite			
	M1	M2	M3	M4
Premix (cement, silica fume, quartz sable)	2226	1970	2296	2183
Super-plasticizer	22.3	39	43.13	25.4
Water	239.5	195	231	185
Accelerator	-	-	10	-
Hooked steel fibers 30/0.35 mm	130 (1.7%)	-	-	-
Straight steel fibers 20/0.3 mm	-	195 (2.5%)	-	-
Straight steel fibers 13/0.175 mm	-	-	296 (3.8%)	-
Straight steel fibers 20/0.15 mm	-	-	-	50 (0.6%)
w/c ratio	0.28	0.17	<0.2	<0.2

Table 2.2: Mix design of M1, M2, M3 and M4 according to the producer's prescription

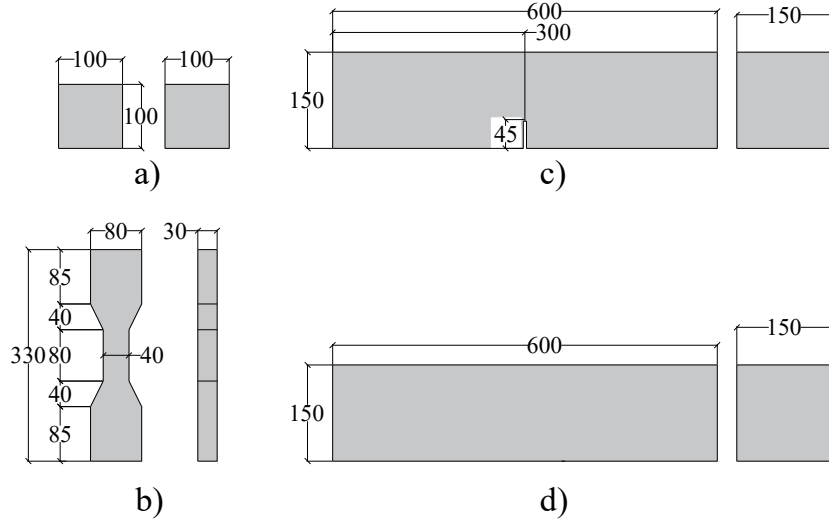


Figure 2.2: a) Cubic specimen, b) dog-bone shaped specimen, c) notched prismatic specimen, d) un-notched prismatic specimen. Scale in millimeter

the fresh cementitious mixture they increase its viscosity¹. A good distribution of the steel fibers in the fresh cementitious mixture was obtained by using a high intensity rotating pan mixer, in particular a 1.5 kW high shear Zyklos type. During filling the molds a visual examination on flow test of the fresh cementitious mixture allowed to detect the formation of fiber balling.

The specimens were made and cured in standard conditions according to [192]. In particular, M1 specimens were also compacted by using a vibrating table. After molding the specimens were covered by a plastic film. 24 hours later specimens were carefully demolded and cured in water ($20 \pm 2^\circ\text{C}$) for 27 days. Then, specimens were prepared for being tested.

M1 and M2 were reinforced by five types of fibers with different shape, l_f/d_e and V_f , see Figure 2.1 and Table 2.3. Data provided by the experimental results were used to calibrate the predictive strength model presented in Section 3. Instead, M3 and M4 were reinforced by the type and dosage of steel fibers suggested by producers. The corresponding experimental data were used to confirm the above-mentioned model. Twenty series of specimens were made. Each series was composed of three cubic and three dog-bone shaped specimens whose matrix was reinforced with a given type of fibers and V_f . For example, the specimens in the first series contained straight fibers with l_f/d_e of 65 and V_f of 1.5%. Also some series of specimens without fibers were made as control. Each series is labeled according to the pattern reported in Table 2.4. While cubic and dog-bone shaped specimens (Figures 2.2(a) and (b)) were tested for developing the predictive relationships for the compressive and tensile responses of the fibrous mortars investigated (see Section 3.1), the prismatic specimens (Figures 2.2(c) and (d)) and a full-scale structural beam (Figure 2.3) were tested for confirming the predictive data provided from the model presented in Section 3.2.

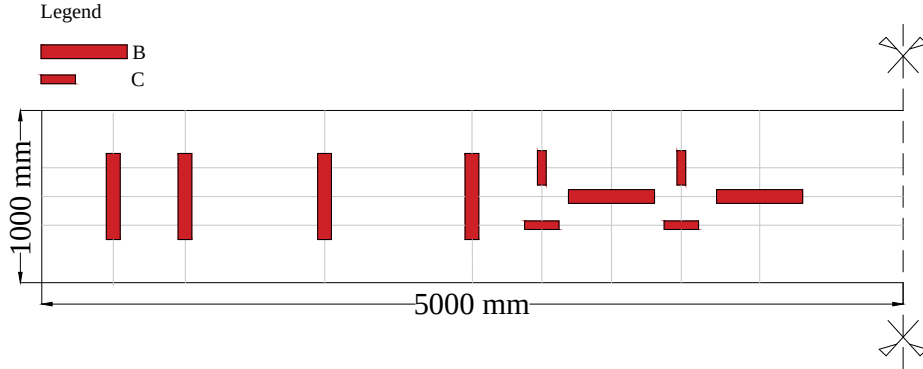


Figure 2.3: Geometry of the full-scale beam and position of the drilled specimens (B: beams, C: cores). Scale in millimeter

A full scale structural $80 \times 1000 \times 10000$ mm beam of M3 was cast in-situ. No thermal treatment was applied after casting. The beam was kept in the molds for at least 20 hours, then it was demolded, protected with plastic sheets and kept inside the precasting plant until 28 days. Six $80 \times 100 \times 500$ mm prismatic and six $\Phi 50 \times 200$ mm core specimens were drilled from the full scale beam

¹The mechanical interaction in time between the hosting structure and the cementitious reinforcement is addressed in [36, 37, 38].

in order to measure the uni-axial tensile and the bending flexural responses. The geometry of the full scale beam and the position of the drilled specimens are illustrated in Figure 2.3. In order to compare the performances of core and cast specimens, a M3 series of $100 \times 100 \times 100$ mm cube and dog-bone shaped specimens were made in lab according to standard conditions. In order to understand the resulted scatter in strength between core and cast (cylinder) specimens a proper investigation is presented in Section 2.3.5.1.

Code (-)	V_f (%)	l (mm)	d_e (mm)	l_f/d_e (-)	Weight (kg)	Strength (MPa)
SF65 ¹	1.5, 2, 3, 4, 5	20	0.3	65	1.10×10^{-5}	1200
SF75	1.5, 2, 3, 4, 5	13	0.175	75	2.45×10^{-6}	1250
HF45 ²	1.5, 2, 3, 4, 5	35	0.75	45	1.12×10^{-4}	1200
HF65	1.5, 2, 3, 4, 5	35	0.55	65	6.54×10^{-5}	1200
HF85	1.5, 2, 3, 4, 5	30	0.35	85	2.27×10^{-5}	1000

Straight Fiber (1), Hooked-end Fiber (2).

Table 2.3: Properties of steel fibers

2.2 Testing method

The mechanical behavior of the material presented in Section 2.1 was characterized according to standard tests in order to develop the predictive strength model presented in Section 3. Specifically, the experimental tests were carried out for measuring the compressive strength on cubic specimens [193], the uni-axial tensile response on dog-bone shaped specimens [170] and the flexural strength, via 4PBFT, on notched [170] and un-notched beam specimens [197]. The geometry of specimens is displayed in Figure 2.2.

Under saturated-surface-dry conditions the measure of density of cubes and dog-bones specimens prior to testing allowed to understand whether the shape and the size of the molds could influence the distribution and the amount of steel fibers embedded within the matrix. In addition, after testing, specimens were cut for a visual examination of the orientation and distribution of steel fibers.

2.2.1 Compressive tests (CT)

A force controlled testing machine (Perrier type 138 – 5000 kN) was used to measure the compressive strength of the cubic specimens according to the [193]. The loading ratio was set equal to 0.6 MPa/s.

2.2.2 Uniaxial tensile tests (UTT)

A displacement controlled computerized universal testing machine (Walter Bai type LVF-200 kN) was used to measure the uni-axial tensile response of the core and dog-bone shaped specimens according to the [170]. UTT were carried out by controlling the axial elongation rate along the longitudinal axis of the specimen, which was monitored by a high precision strain gauge. A *HBM* 100 mm long strain gauge was placed in each side of the specimen, see Figure 2.6(a). The

strain rate was set to 0.05 ± 0.01 mm/min. The end sides of the specimen were larger than the central part, so the hydraulic grips held the specimen during the test and ensured that the failure occurred in the monitored zone by the gauge, see Figure 2.4. The hydraulic grips also provided rotation support to both the specimen ends, in order to keep the specimen and the load axis aligned. This device prevented that any bending moment stress increased into the specimen. In order to avoid any sliding between the hydraulic grips and the dog-bone shaped specimen during the test a set of aluminum plates were glued to the specimen's surface as proposed by [187] and a special clamp was designed for ensuring a perfect load transmission. The aluminum plate surfaces glued on specimens were observed at the microscope prior and after the UTT showing neither signs of groove deformation nor sign of slipping, see Figure 2.5. During the experimental campaign the gripping device was redesigned by Author in order to speed up the set-up. The recorded results confirmed no change in terms of uni-axial tensile response of the dog-bone shaped specimen between the use of the old and the new gripping device. The renewed device is illustrated in Figure 2.7(c).

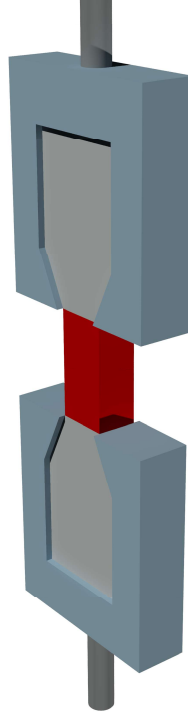


Figure 2.4: Monitored zone (in red) on the specimen whose both ends are grabbed by the new gripping device

2.2.3 Four-points bending flexional tests (4PBFT)

The testing machine mentioned in Section 2.2.2 was also used for carrying out the standard four-point bending flexural tests (4PBFT) on prismatic specimens described in Section 2.1. The specimen was placed over two cylindrical supports. Two top rollers were placed above the specimen. The specimen was notched in

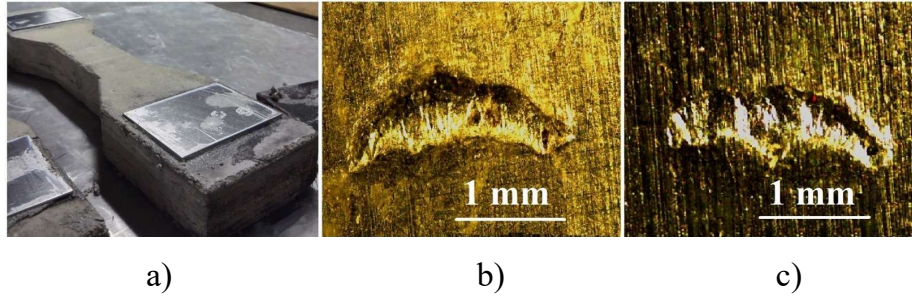


Figure 2.5: a) Aluminum plate glued on the dog-bone shaped specimen, b) holes in the aluminum plate prior to UTT, c) holes in the aluminum plate after UTT

the mid-span (45 mm deep) according to the [170], see Figure 2.2(c). As already discussed in Section 1.4 the notched specimens are preferred for testing softening SFRC materials. The reason is that just a single crack is often observed, due to the presence of the notch, which also makes easy measuring the opening crack. The crack opening was measured both on the bottom and each side of the specimen. Bottom measurement is called Crack Mouth Opening Displacement (CMOD). Side measurement is called Crack Tip Opening Displacement (CTOD), according to [195]. For hardening SFRC materials, in which multi-cracks can be observed, tests on un-notched prismatic specimens are preferable. So, for the investigated materials showing multi-cracks standard four-point bending flexural tests were carried out according to [197]. The geometry dimension of these specimens is illustrated in Figure 2.2(d). The loading rate of $0.05 \pm$

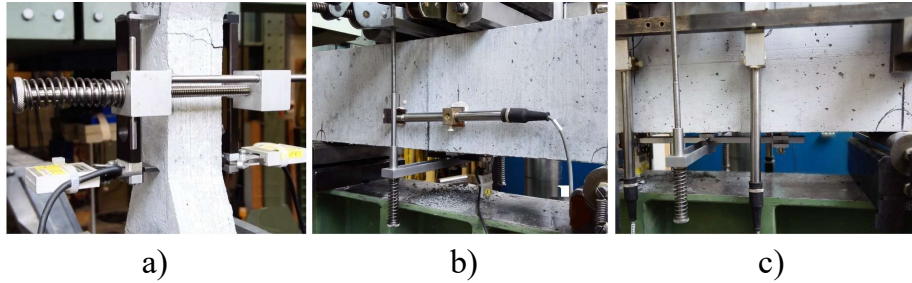


Figure 2.6: a) The strain gauge is placed in the central zone of the dog-bone shaped specimen, b) the strain gauge is placed at the bottom of the notched beam, a LVDT is placed on each side, c) the strain gauge is placed at the bottom of the mid-span of the un-notched beam, four LVDTs are placed under each roller loading

0.01 mm/min was applied on the CMOD and along the bottom for notched and un-notched specimens, respectively. Sensors used to monitor both the CMOD and CTOD were respectively a 100 mm long strain gauge and a couple of LVDT placed on both sides, see Figure 2.6(b). For un-notched beams a 200 mm long strain gauge was placed at the bottom of the mid-span section and four LVDTs were used to measure the localized displacement under the top rollers, see Figure 2.6(c).

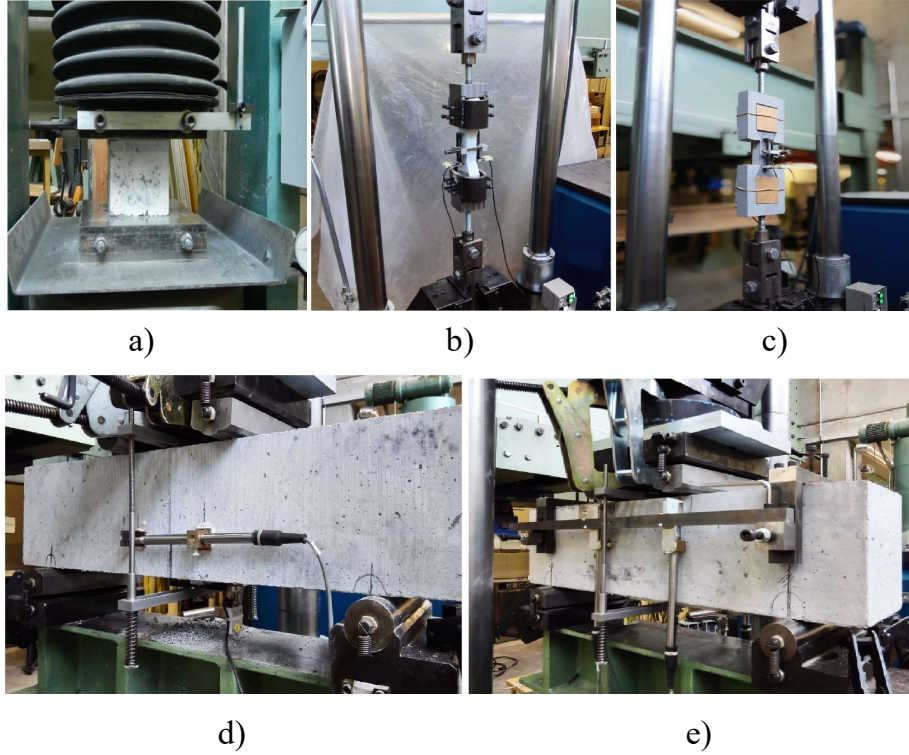


Figure 2.7: Test machines: a) machine Perrier type 138-5000 kN set-up for CT, b) machine Walter Bai type LVF-200 kN set-up for UTT with old gripping device, c) with renewed gripping device, d) machine Walter Bai type LVF-200 kN set-up for 4PBFT on notched and e) un-notched prismatic specimens

2.3 Experimental results

The experimental data were properly subdivided in series, in order to better identify the most relevant results. The series are labeled according to the pattern displayed in Table 2.4.

M1, M2, M3, M4	Type of cement-based matrix
SF, HF	Straight (steel) fibers, hooked-end (steel) fibers
45, 65, 75, 85	Aspect ratio
0, 1.5, 2, 3, 4, 5	Fiber dosages
CU, DB, CO	Cubic, dog-bone, core specimens
NP, UP, DUP	notched, un-notched, drilled un-notched prismatic specimens

Table 2.4: Specimen identification

2.3.1 Fiber distribution and orientation within the matrix

During the mold filling it was observed a phenomenon of fiber balling in the fresh cementitious mixture of series M1-HF65-5 and M1-HF85-5. The fresh

cementitious mixture M2 reinforced with a V_f superior to 3%, including the types HF45 and HF85 for any V_f , showed the same phenomenon as well, see Figure 2.8. In accordance with the discussion reported in Section 1.3 such series had to be discarded from the investigation. This observation highlighted the impact of the l_f/d_e ratio on the distribution of fibers in the fresh cementitious mixture which may play a key role as the matrix packing density and/or the V_f increase. After the mixing the vertical pouring of the fresh cementitious mixture into dog-bone-shaped molds promoted a certain level of orientation of fibers along the main axis of the specimen. By measuring the hardened-state density in each series it was found that the average density of dog-bone shaped specimens was slightly lower than cubic ones, for any increase of V_f , see Figure 2.9. This observation confirms as discussed in Section 1.3.

After testing, the specimens were cut near to the failure sections showing a preferential orientation of fibers, see Figure 2.10(a). The pouring direction on the dog bone specimen caused a fiber orientation along the main axis of the specimen due to both the shear stress (arising also by the friction of the mortar with the bottom of the mold) and the wall effect. This phenomenon was virtually unavoidable since size ratio between the thick of the specimen and the fiber length was similar. In particular, the cross-section of the specimen was 30×40 mm and the average length of the studied fibers was of about 30 mm. The cubic molds were vertically filled and the fibers were aligned mainly in horizontal planes and perpendicular to the pouring direction, see Figure 2.10(b). Specimens of M1-HF45 and M1-HF65 series presented a slight fiber segregation, see Figure 2.10(c). This could be explained considering that even if the shape of these two fibers is similar to the others they are heavier, which caused the segregation process. The specific weight of fibers investigated is reported in Table 2.3.



Figure 2.8: Phenomenon of fiber balling occurred in M1-HF65-5, M1-HF85-5 and M2-HF85 series

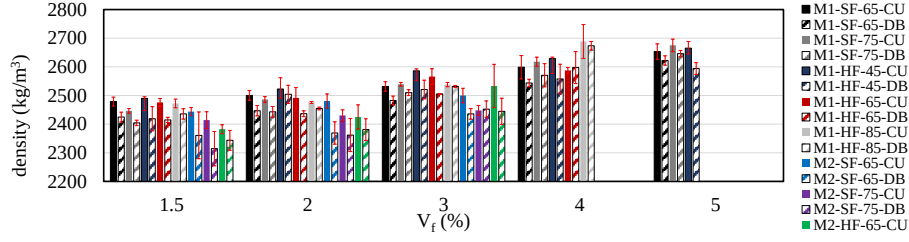


Figure 2.9: Comparison of the specimens' density for series investigated. The red bars represent the standard deviation of a population of three data points

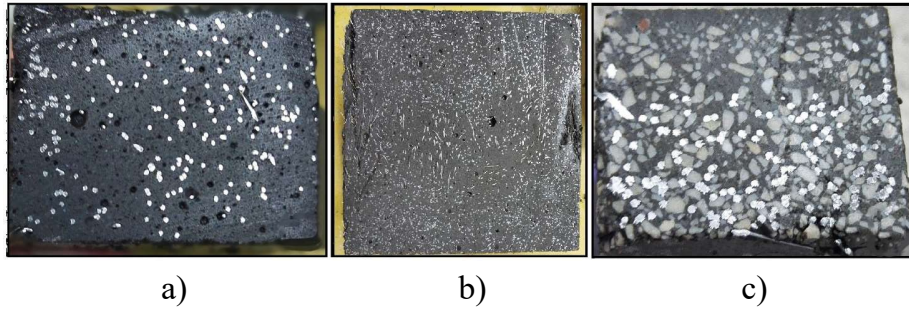


Figure 2.10: Fiber orientation and distribution on: a) dog-bone shaped, b) cubic specimen. c) Specimens of M1-HF45 and M1-HF65 series presented a clear phenomenon of fiber segregation

2.3.2 Influence of the HPFRCC/UHPFRC mix design on the compressive strength

Table 2.5 reports the compressive strength $f_{c,i}$ of cubic specimens, the mean strength $f_{c,m}$ of n tested specimens, and the respective Coefficient-Of-Variation (COV).

$$f_{c,i} = \frac{P_{c,i}}{A_{c,i}} (MPa) \quad f_{c,m} = \frac{\sum_{i=1}^n f_{c,i}}{n} (MPa) \quad COV = 100 \cdot \frac{\sigma}{n} (\%) \quad (2.1)$$

Where $P_{c,i}$, $A_{c,i}$ and σ are the maximum load recorded during the CT, the cross section area of the uniformly loaded specimen and the standard deviation, respectively. The specimens showed the same cracking pattern for any matrix used. As shown in Figure 2.11 the fibers conferred a high level of confinement in the fibrous mortar which prevented a brittle failure, retaining the damage mechanism². It was observed that fibers were mainly aligned in horizontal planes and perpendicular to the pouring direction, counteracting the cracks opening during the test.

The compressive strength values recorded on M1 series are provided in Table 2.5. The results showed an evident increase of compressive strength due to the presence of steel fibers within the cementitious mixture, see Figure 2.12. It

²Fracture can be modeled, in a nonlinear framework, following [137], whereas recent contributes about damage mechanics can be found in [82, 83, 84]. Bifurcation and stability in the context of homogeneous finite deformations can be found in [138].

is worth noting that such an increase was 2% for low l_f/d_e (series HF45) and 36% for high l_f/d_e ratios (series HF85). The small increase of $f_{c,m}$ in series HF45 was probably due to the certain degree of segregation of the fibers which cannot confer an homogeneous distribution of toughness to the cross-section of the specimen. It can be also observed the key role of the amount of fibers on the compressive strength of the fibrous mortar. In fact, as V_f increased $f_{c,m}$ increased as well, for any series. Considering the presence of fiber balling in the fresh cementitious mixture of series HF65_5 and HF85_5, 4% of V_f seems to be the optimal dosage of fibers for reinforcing the M1 matrix with HF. Instead, when SF were used the compressive strength increased further until 5% of V_f , showing no sign of fiber balling.

For M2 series, the fibers increased the compressive strength $f_{c,m}$ as both l_f/d_e and V_f increased. In previous works, the UHPFRC compressive strength was not affected significantly by the fiber presence [157]. Nevertheless the compressive strength is also strongly influenced by the homogeneity of the fiber dispersion that can be achieved by detecting the optimum V_f . In other works [161] an increase of compressive strength was observed up to a V_f of 3%. In [105] the investigated UHPFRC recorded the highest compressive strength with a V_f of 2%, whereas in [163] the compressive strength continuously increased with an increase of V_f until to 4%. Similar results were also observed in [40]. This trend was clearly observed in this investigation as well. In particular, M2-SF75 series showed an increase of compressive strength of about 39%. In M2-SF65 series the increase was 30%. For fibers with the same l_f/d_e and low V_f (1.5%), M2-HF series showed lower increase of strength, compared to M2-SF series. As V_f increased, M2-HF series showed higher gain of strength, see Figure 2.12. Further details are drawn in Table 2.5. Since V_f higher than 3% caused problems of both fiber balling and workability in the mix, $V_f = 3\%$ is considered the upper limit for achieving the best mechanical performances for reinforcing UHPFRC materials. For all series the favorable orientation of fibers within the cementitious mixture permitted to gain toughness, well counteracting the opening cracks under compressive stresses.

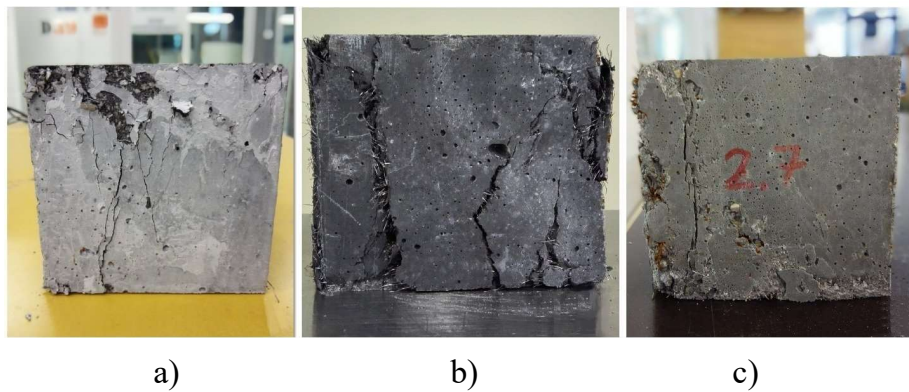


Figure 2.11: Cracking patterns of cubic specimens for series after the CT: a) M1-HF85-1.7, b) M2-SF75-3.8, c) M3-SF65-2.5

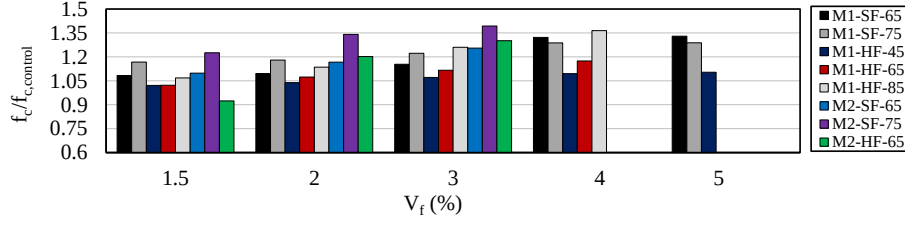


Figure 2.12: Effect of matrix strength, l_f/d_e ratio and V_f on $f_{c,m}$

2.3.3 Influence of the HPFRCC/UHPFRC mix design on the uni-axial tensile response

Table 2.5 reports the first cracking tension $f_{t,i}$ of dog-bone shaped specimens, the mean strength $f_{t,m}$ of n tested specimens, and the respective COV.

$$f_{t,i} = \frac{P_{t,i}}{A_{t,i}} (MPa) \quad f_{t,m} = \frac{\sum_{i=1}^n f_{t,i}}{n} (MPa) \quad (2.2)$$

being $P_{t,i}$ and $A_{t,i}$ the measured first cracking load recorded during the uni-axial tensile test and the area of the nominal cross section, respectively. The testing program recorded the uni-axial tensile stress - strain / cracking path. The axial strain measured by the gauge was automatically converted in elongation, considering of course, the characteristic length of the strain gauge (100 mm). The experimental curves showed in Figure 2.14 reported the UTT results of the series M1-HF85-1.7, M2-SF75-3.8 and M3-SF65-2.5, whose fiber properties corresponded to those prescribed by the producers of M1, M2 and M3 matrix respectively. The experimental curves could be stylized by three idealized branches, according to the definition of Naaman et al. (2006) [94]. Part I: linear elastic behavior up to 90 – 95% of the cracking strength. In this phase the contribution of the fibers could not be decoupled, since the matrix characteristics are governing the behavior of the fibrous mortar. Part II: the cracking peak load is followed by a strain hardening branch strongly influenced by the contribution of the fibers. In M1 series, some multiple cracks were observed. Instead, a single large crack was observed in the central area of the specimens in M2 and M3 series, as seen in Figure 2.13. Part III: softening behavior due to debonding between fibers and cracked matrix which ends with a residual strength corresponding to 1.5% of axial strain. The length of this branch depends on the pull-out resistance of fibers bridging the cracks. It is also noticed that some drops of tensile stress in the Part III were recorded in M2-SF75-3.8 and M3-SF65-2.5 series. In Literature this phenomenon is deemed to occurs by one or a combination of the following mechanisms:

- *Fiber failure.* It is the less common phenomenon and it happens usually in long fibers, where the bond strength is so high that the fiber can plasticize and reach rupture.
- *Fiber pull-out.* This is the mechanism that dissipates less energy and it's the most common in straight fibers.
- *Fiber bridging.* Fibers link the two edges of the crack and the stress can

be transferred through them. This is the most common phenomenon in non-straight fibers. The energy is absorbed by the deformation of the fiber.

- *Fiber/matrix debonding.* Like the fiber failure, this happens usually in long and well bonded fibers.
- *Matrix crack.* The fiber can contrast the propagation of micro-cracking through the matrix spreading the energy of the crack on a wider surface.

Such a drop of tension for M2-SF75-3.8 and M3-SF65-2.5 series was due to the low pullout strength of fibers used in these series, which were SF. The pull-out mechanism was prevented in M1-HF85-1.7 series, where HF were used. The hooked-ends of the fibers prevented the total extraction of the fibers ensuring a higher degree of load redistribution. The comparison in terms of pullout strength between straight and hooked-end fibers here investigated confirms as found in [149]. Furthermore, the presence of hooked-end fibers in M1-HF85-1.7 series led to the multi-cracking behavior. A multi cracking behavior is always desirable in both HPFRCC and UHPFRC materials since it indicates an augmented toughness, necessary to have high rotational capacity and good stress redistribution in hyperstatic structures. As discussed in Section 2.3.1 the pouring direction of fresh cementitious mixture in the dog-bone shaped molds promoted an alignment of fibers along the main axis of the specimen, and therefore perpendicular to cracking planes. Such a condition permitted to increase the toughness of the fibrous mortars investigated, which can be assessed as the areas under the graphs in Figures 2.14(a), (b) and (c). This benefit was also due to the good distribution of fibers in the cross section.

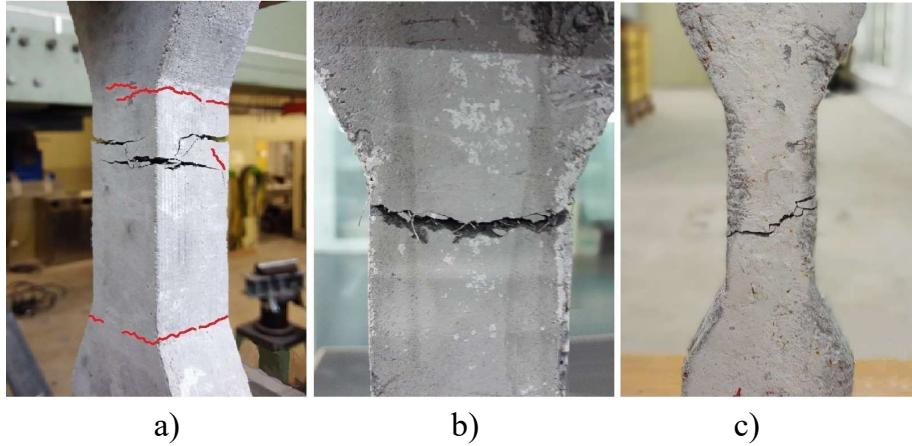


Figure 2.13: Uniaxial tensile failure pattern for series: a) M1-HF85-1.7, b) M2-SF75-3.8, c) M3-SF65-2.5, whose mix design corresponds to that prescribed by the producers

As mentioned above, the series M1-HF85-1.7, M2-SF75-3.8 and M3-SF65-2.5 were investigated for targeting the fibrous mortars according to the current standard. The main experimental campaign aimed to calibrate the predictive model presented in Section 1.4. For this purpose, both commercial HPFRCC (M1) and UHPFRC (M2) materials were characterized, via CT and UTT, by

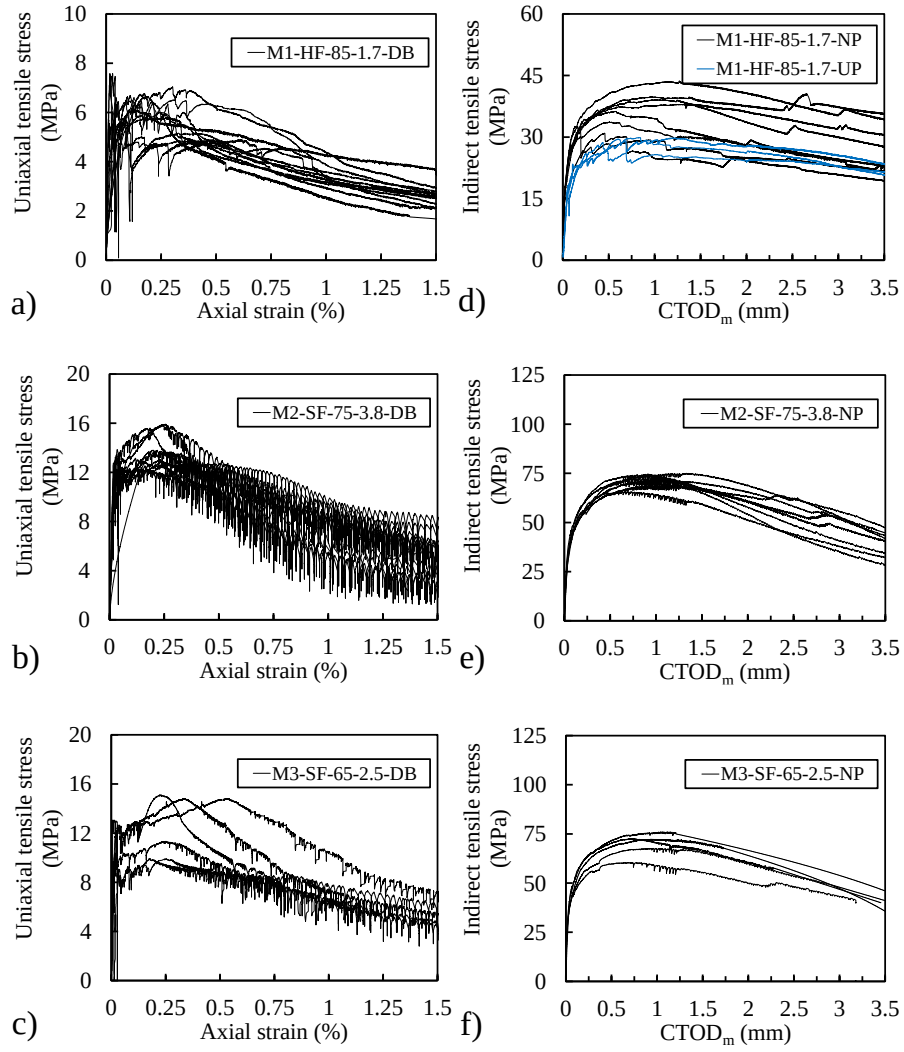


Figure 2.14: UTT and 4PBFT results for series: a, d) M1-HF85-1.7, b, e) M2-SF75-3.8, c, f) M3-SF65-2.5, whose mix design corresponds to that prescribed by the producers

varying the fiber properties. From the CT results the parameter measured was the compressive strength (f_c). From UTT results two main parameters were measured: the tensile strength (f_t) and residual tensile strength (f_{rt}), as illustrated in Figure 2.15. f_t was defined as the peak of uni-axial tensile tension value recorded in the experience. In this investigation, f_t value was reached for uni-axial strain values ranging from 0.01 to 0.25%. UTT data results are reported in the Appendix A1. f_{rt} was the tension value corresponding to the 1.5% of axial strain. The mean strength value of both f_t and f_{rt} was calculated on a population of three and nine specimens for fiber-reinforced and plain concrete materials, respectively. The ratio between $f_{t,matrix}$ and f_t of matrix reinforced

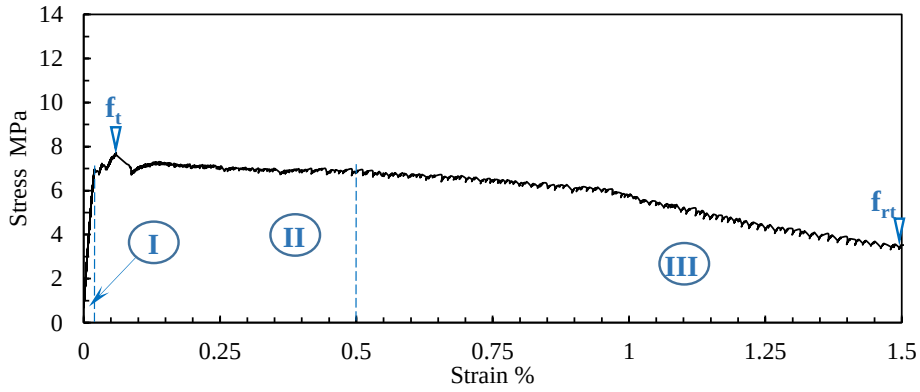


Figure 2.15: UTT mechanical parameters

with steel fibers is presented in Figure 2.16 for any series investigated.

$f_{t,matrix}$ value of M1 was 3.3 MPa. An increase of f_t in the range of 3 – 128% was observed in M1 series. In particular, for M1-HF series f_t increased with increasing both l_f/d_e ratio and V_f . Even though the fiber segregation observed in series M1-HF45 and M1-HF65 provided a slight increase of strength, especially for V_f ranging from 1.5% to 3%, such a phenomenon was less pronounced for higher V_f ($> 3\%$), leading to higher gains of strengths. A better increase of tensile strength was observed in series M1-HF85. Also M1-SF series highlighted the benefit of straight fibers into the cementitious mixture. In particular M1-SF65-4 series showed the highest value of f_t , in relation to all M1 series recorded. But a drop of strength was detected for both M1-SF65-5 and M1-SF75-5 series. Due to the high dosage (5%) fibers were too close to develop an efficient bridging effect in the cracking section.

In M2 series, $f_{t,matrix}$ value was 6.1 MPa. The maximum ratio $f_t/f_{t,matrix}$ reached a value of about 114%, see Figure 2.16. M2-SF series denoted a good increase of f_t by increasing both l_f/d_e and V_f until the upper limit of 3%. Less pronounced was the gain in performance recorded on M2-HF-65 series. The trend of strength improvement was similar to that seen in M1 series even though the ratio magnitude was slightly lower. Before first cracking occurs, the stress-strain behavior was not significantly influenced by the shape of tested fibers, even though the higher number of SF in the cross-section, compared to that one in HF series, led to larger increasing of the first cracking strength, see Table 2.5. As reported in Section 2.3.1 the fibers embedded into the cementitious mixture M1 and M2 were mainly oriented along the main axis of the dog-bone-shaped

specimen, which perpendicularly to the crack plane conferred gain of toughness in post-cracking. After the matrix crack initiation, the stress was absorbed by the fibers bridging the cracks. Whereas the strain of SF caused the debonding from the matrix and consequently a total pull out, the debonding of hooked-end fibers in HF series was preceded by the straightening of hooked-ends that retained the fiber from being pulled out, thus extending the energy-absorbing capability. From a point of view of toughness both SF and HF are well-adapted to dissipate strain energy. Even if HF can offer the benefit described above the energy-absorbing capability of fibers also depends on both the fiber size and the amount of fibers bridging the crack plane. In the investigated series HF were more efficient in increasing f_{rt} of M1 matrix, especially for high V_f values, see Figure 2.17. In particular, the ratio between f_{rt} and $f_{t,matrix}$ of both M1 and M2 series ranged from 0.2 to 1.26 and 0.24 to 0.78, respectively. It is interesting to note that f_{rt} of M1 reinforced by SF decreased when l_f/d_e and V_f increased, in contrast with M2-SF series. Instead for M1 reinforced with HF, f_{rt} improved with increasing l_f/d_e ratio and V_f , as also observed in M2-HF series. The series M1-HF45 showed a small increase of f_{rt} as V_f increases. More pronounced was the gain of residual strength observed in series M1-HF65 and M1-HF85 in which a remarkable increase of f_{rt} was observed with increasing V_f , see Figure 2.17. In fact, thanks to both the hooked-end shape and the high l_f/d_e these fibers enhanced the crack bridging process improving the post-cracking behavior. It can be resumed that SF conferred a higher ductility for V_f values of 1.5 and 2% to both M1 and M2 matrix, even though a slight reduction was detected as the amount of fibers increased. Series M1-SF75 showed no obvious improvement of ductility by varying V_f . Instead M1-HF series clearly denoted that the toughness improves for high l_f/d_e and V_f . The higher increase of toughness in SF series with low dosages, compared to HF series, was due to the fact that SF investigated were smaller than HF, so for a given V_f a higher number of SF properly bridged the cracks and conferred more toughness to the section. By contrast, in M1-HF series whose V_f was high enough, e.g. 3, 4 or 5%, better gains of toughness were recorded. In M2 series SF conferred best performances for any V_f . This observation denoted that higher packing density of cement-based mixtures (i.e. UHPFRC) can improve the bonding strength of the fiber-matrix interface to the point that the benefit in terms of energy-absorbing capability provided by HF become less relevant. These results confirmed the fact that some researchers concentrated their effort on HPCCs reinforced with HF [106, 135, 152] and UHPC reinforced with SF [71].

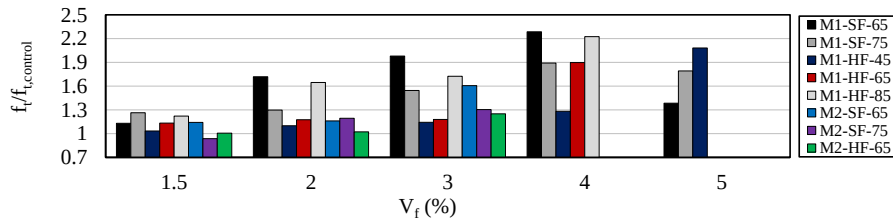


Figure 2.16: Effect of matrix strength, l_f/d_e ratio and V_f on f_t

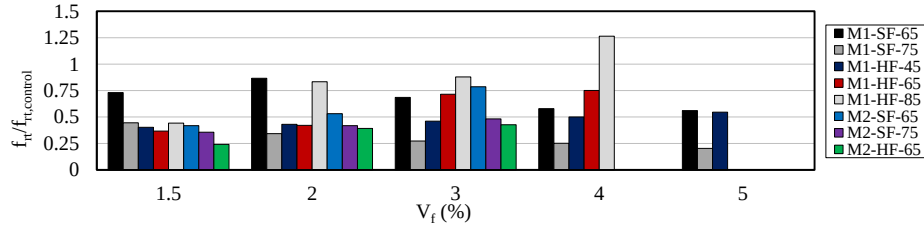


Figure 2.17: Effect of matrix strength, l_f/d_e ratio and V_f on f_{rt}

2.3.4 Influence of the HPFRCC/UHPFRC mix design on the flexural bending response

A series of prismatic specimens of M1-HF85-1.7, M2-SF75-3.8 and M3-SF65-2.5 were tested via 4PBFT, see Section 2.2.3. Since M2-SF75-3.8 and M3-SF65-2.5 series showed a single crack failure during the UTT, a 4PBFT on notched beams was preferable, as also discussed in [47]. Instead, in M1-HF85-1.7 series a 4PBFT on un-notched beams were carried out, due to the fact that a multi-cracking pattern was detected during the UTT, as outlined in Section 1.4. Also for bending tests, a single large crack occurred in the mid-span of M2-SF75-3.8-NP and M3-SF65-2.5-NP. Instead multi-cracks occurred in M1-HF85-1.7-UP series. Even though the notched beam specimens did not show a multi-cracking behavior, a great toughness magnitude was evident, see Figure 2.14. High toughness was also denoted for M2-SF75-3.8-NP and M3-SF65-2.5-NP series, due to the typical high packing density of UHPFRC which improves the bonding matrix-fibers, and therefore the pullout strength. This benefit was reached thanks to a proper distribution and orientation of fibers within the cross section, confirmed by a visual examination of the cross-section after testing. The indirect tensile strength $f_{tf,i}$ can be estimated via 4PBFT, assuming that all materials remained in the elastic field, according to the Equation 2.3 reported in [170]. Results displayed in Table 2.5 and Figure 2.18 show that even though the indirect tensile behavior was higher than that measured by UTT, similar diagrams were observed. A proper correlation will be presented in Section 3.2, in compliance with the Composite Material Theory.

The average first cracking strength $f_{tf,m}$ of n tested specimens and the respective COV are also reported in Table 2.5

$$f_{tf,i} = \frac{P_{tf,i}l}{b(h-a_0)^2} (MPa) \quad f_{tf,m} = \frac{\sum_{i=1}^n f_{tf,i}}{n} \quad (2.3)$$

where $P_{tf,i}$ is the first cracking load recorded during the test, l , b , h , a_0 are the standard geometrical beam dimensions, see Figure 2.2.

2.3.5 Tensile and flexural responses of a full-scale UHPFRC beam

A specific experimental investigation of a full-scale UHPFRC beam was carried out. The formulation of the beam material corresponds to that of the M3-SF65-2.5 series. The experimental graphs recorded by UTT and 4PBFT on specimens extracted from the full-scale beam are reported in Figure 2.19. A series of

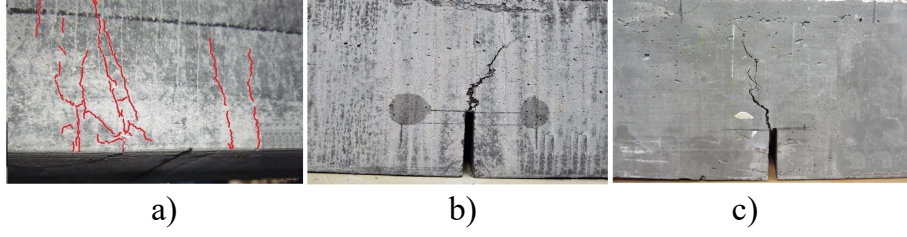


Figure 2.18: Flexural bending failure pattern on series: a) M1-HF85-1.7, b) M2-SF75-3.8, c) M3-SF65-2.5, whose mix design corresponds to that prescribed by the producers

dog-bone shaped specimens was made in lab in order to compare the results with those of the cores. The $f_{t,m}$ of cores was of 8.6 MPa, about 25% lower than the values recorded for dog-bone shaped specimens (11.42 MPa). Also the toughness was lower. This difference can be ascribed to the fact that the mold's shape of cast specimens encouraged a certain degree of fibers orientation, in particular fibers were mainly aligned along the longitudinal axis of the specimen. Such an alignment improves the mechanical response of the specimen whose cracking plane is perpendicular to the fibers bridging the crack. Instead, in the cores drilled from the full scale structural beam fibers were randomly oriented. Also the fact that cores extraction and conditioning promoted a certain degree of damage on the specimens may have influenced this result³. An investigation concerning such an aspect is reported in Section 2.3.5.1.

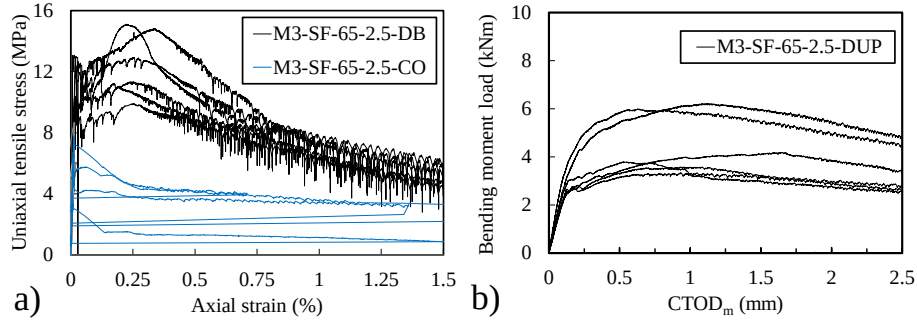


Figure 2.19: a) UTT results for cores and dog-bone shaped specimens of M3-SF65-2.5 series, b) 4PBFT results for un-notched prismatic specimens of M3-SF65-2.5 series cut from the full scale structure

2.3.5.1 Performance comparison between cast and cored specimens

A study on the loss of compressive strength on cores was conducted on the M4 series, whose UHPFRC was designed for precast non-structural elements. The mix design is listed in Table 2.2. Fifteen cylinders $\varnothing 110 \times 220$ mm were cast along with three $\varnothing 100 \times 200$ mm cylinders and a $200 \times 210 \times 600$ mm beam. The beam was further instrumented with two fiber optic sensors placed parallel

³Mechanical modeling of damage can be found, as an example, in [82, 83, 84, 139].

to the shorter side of the mold at a distance of 52 mm from the side. The two sensors were placed about 45 mm apart in order to have one sensor on the center of the specimen and one on the side, thus avoiding the bits of the barrel ripping the sensors, see Figure 2.20. In Figure 2.21(a) the deformation of the sensors

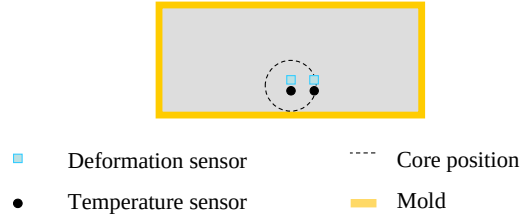


Figure 2.20: Placement of the sensors at the core position

is expressed in micro strain ($\mu\epsilon$), while in Figure 2.21(b) the temperatures are shown for the central and the side sensors. The temperature sensors seemed to demonstrate that the water used for cooling the barrel did not have any visible effect in terms of deformations between core and skin of the core (thus producing a self-strain). A jump of some degrees of temperature was visible after 30 minutes of coring in the external part of the core due to an adjustment of the flow of cooling water. The external layer of the concrete tended to contract while the core expanded. At the end of the coring phase the specimens were left in the socket and the water was stopped. In the space left by the barrel there was no water because it escaped from the bottom immediately after the barrel was extracted. From that moment no further deformations were observed.

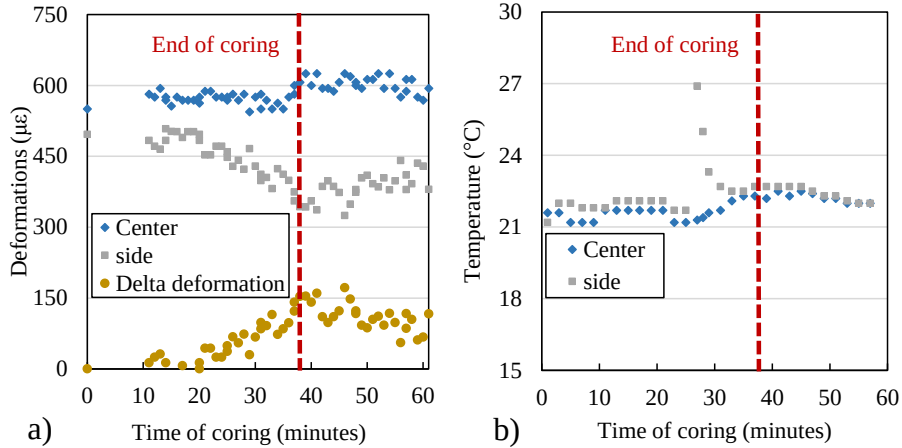


Figure 2.21: a) Deformation and b) temperature of the side and center sensors at the coring

A possible explanation concerning the distortion of the section can be found both in the release of a self-restrain and in the effect of the coring barrel. The eventual water pressure on the side and the temperature of the cooling water

did not seem to have any effect on this concrete. The compressive strength of cores with length-diameter ratio equal to 2 was tested. The core strength was higher at early age (the higher temperatures reached in the cored beam allow for a faster hydration) and lower of more than 10% at 28 days, thus confirming that the strength retrogression occurred also in this case, see Figure 2.22(a). In order to confirm that both the specimen's size and the molds' type were taken into account. A 28-compressive test was carried out on four different types of specimens, i.e. two cores and two cast cylinder of different diameter. The diameters of the cores were 94 and 74 mm respectively, whose length was the double of the diameter in both cases. The diameters of the cast cylinders were 110 and 100 mm respectively, whose length was the double of the diameter in both cases. Cast cylinders were coated with paper box and plastic molds respectively. Figure 2.22(b) shows that cores had similar strength in average and cast cylinder as well. This permitted to check if this potential source of error was having any effect. The average weight of all cores was 2510 kg/m^3 while the average weight of the cylinders was of about 2550 kg/m^3 . This difference highlighted that the concrete had the same degree of compaction in both cylinders and cores. It can therefore be affirmed that the extraction of cores in UHPFRC provoked a distortion of the section that can be quantified in about $100 \mu\epsilon$. This distortion was not due to the coring but by the extraction of the core from the structural element. However this distortion and the torsion promoted by the barrel coring the concrete might provoke enough micro-cracking in specimens of UHPFRC with low fibers content. This parameter should be taken carefully into account whenever the strengths of UHPFRC in the structure and in specimens are compared.

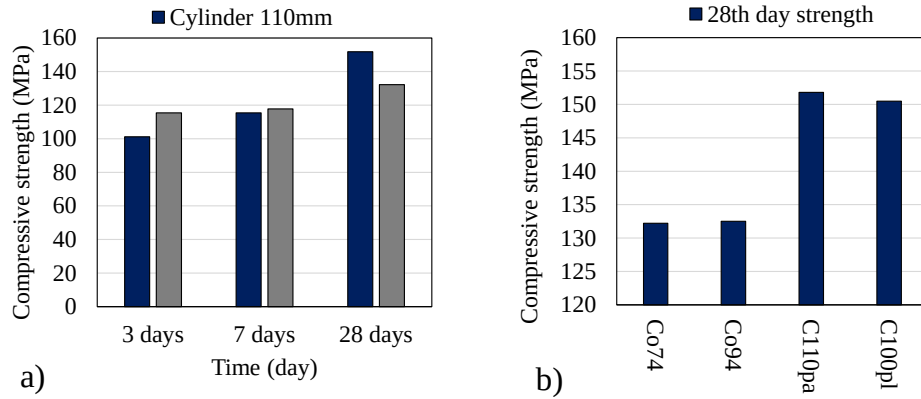


Figure 2.22: a) Compressive strength of cylinder and core specimens at 3, 7 and 28 days, b) compressive strength at 28 days of different size of specimens and molds types

Series	Type of specimen	CT			Type of specimen	UTT			Type of specimen	4PBFT	
		$f_{c,m}$ (COV)	Rel.	MPa (%)		$f_{t,m}$ (COV)	Rel.	MPa (%)		$f_{rt,m}$ (COV)	$f_{lf,m}$ (COV)
M1-0	CU	75.85(9.63)	100.00		DB	3.30(9.58)	100.00	—(—)	NP	—(—)	
M1-SF65-1.5	CU	82.17(8.57)	108.33		DB	3.73(11.27)	113.03	2.41(8.70)	NP	—(—)	
M1-SF65-2	CU	83.11(5.84)	109.57		DB	5.67(12.48)	171.82	2.86(24.20)	NP	—(—)	
M1-SF65-3	CU	87.53(2.48)	115.40		DB	6.53(7.51)	197.55	2.26(23.94)	NP	—(—)	
M1-SF65-4	CU	100.26(1.57)	132.18		DB	7.54(14.36)	228.30	1.91(25.26)	NP	—(—)	
M1-SF65-5	CU	100.85(1.06)	133.62		DB	4.57(22.41)	137.64	1.85(16.43)	NP	—(—)	
M1-SF75-1.5	CU	88.56(1.26)	116.76		DB	4.17(4.79)	126.36	1.47(35.49)	NP	—(—)	
M1-SF75-2	CU	89.50(2.46)	118.00		DB	4.28(1.93)	129.70	1.13(54.03)	NP	—(—)	
M1-SF75-3	CU	92.71(2.55)	122.51		DB	5.10(23.95)	154.55	0.90(23.91)	NP	—(—)	
M1-SF75-4	CU	97.65(2.66)	128.74		DB	6.24(6.30)	189.09	0.83(26.99)	NP	—(—)	
M1-SF75-5	CU	97.74(3.05)	128.86		DB	5.91(12.20)	179.09	0.67(23.13)	NP	—(—)	
M1-HF45-1.5	CU	77.47(1.07)	102.14		DB	3.41(5.03)	103.33	1.33(22.09)	NP	—(—)	
M1-HF45-2	CU	78.79(5.01)	103.88		DB	3.63(6.41)	110.00	1.42(38.26)	NP	—(—)	
M1-HF45-3	CU	81.25(2.19)	107.12		DB	3.77(14.36)	114.24	1.52(6.21)	NP	—(—)	
M1-HF45-4	CU	83.06(2.48)	109.51		DB	4.23(29.11)	128.18	1.65(23.06)	NP	—(—)	
M1-HF45-5	CU	83.70(2.96)	110.35		DB	6.87(11.73)	208.18	1.80(10.27)	NP	—(—)	
M1-HF65-1.5	CU	77.52(5.67)	102.20		DB	3.74(12.75)	113.33	1.21(22.59)	NP	—(—)	
M1-HF65-2	CU	81.43(2.96)	107.36		DB	3.88(11.37)	117.58	1.39(26.22)	NP	—(—)	
M1-HF65-3	CU	84.66(7.07)	111.62		DB	3.89(5.00)	117.88	2.36(19.97)	NP	—(—)	
M1-HF65-4	CU	89.06(7.73)	117.42		DB	6.26(41.48)	190.21	2.48(1.41)	NP	—(—)	
M1-HF85-1.5	CU	81.03(6.41)	106.83		DB	4.03(5.44)	122.12	1.46(41.37)	NP	—(—)	
M1-HF85-1.7	CU	86.42(5.06)	113.93		DB	6.3(11.52)	190.91	2.48(35.85)	NP	34.52(16.25)	
M1-HF85-1.7	CU	—	—		DB	—(—)	—	—	UP	29.27(6.44)	
M1-HF85-2	CU	86.15(3.07)	113.58		DB	5.43(4.47)	164.55	2.75(10.05)	NP	—(—)	
M1-HF85-3	CU	95.60(1.78)	126.42		DB	5.69(12.26)	172.42	2.90(12.85)	NP	—(—)	
M1-HF85-4	CU	103.52(1.44)	136.48		DB	7.34(6.34)	222.42	4.17(38.88)	NP	—(—)	
M2-0	CU	103.95(6.23)	100.00		DB	6.10(8.54)	100.00	—(—)	NP	—(—)	
M2-SF65-1.5	CU	114.14(8.88)	109.80		DB	6.96(3.88)	114.24	2.55(23.53)	NP	—(—)	
M2-SF65-2	CU	121.30(5.47)	116.68		DB	7.07(11.17)	116.05	3.24(20.06)	NP	—(—)	
M2-SF65-3	CU	130.47(1.76)	125.51		DB	9.79(8.27)	160.61	4.79(14.20)	NP	—(—)	
M2-SF75-1.5	CU	127.41(8.25)	122.57		DB	5.71(13.41)	93.70	2.17(15.67)	NP	—(—)	
M2-SF75-2	CU	139.36(2.83)	134.06		DB	7.28(3.40)	119.40	2.55(15.67)	NP	—(—)	
M2-SF75-3	CU	144.84(1.27)	139.33		DB	7.95(1.85)	130.35	2.94(17.69)	NP	—(—)	
M2-SF75-3.8	CU	148.61(2.54)	142.96		DB	13.06(8.49)	214.1	2.33(44.66)	NP	71.52(10.20)	
M2-HF65-1.5	CU	96.08(15.60)	92.43		DB	6.13 (11.58)	100.57	1.47(16.33)	NP	—(—)	
M2-HF65-2	CU	124.92(2.79)	120.17		DB	6.23(4.65)	102.21	2.39(17.57)	NP	—(—)	
M2-HF65-3	CU	135.31(1.82)	130.16		DB	7.62(6.30)	125.01	2.60(2.31)	NP	—(—)	
M3-0	CU	167.32(4.28)	100.00		DB	9.17(12.07)	100.00	—(—)	NP	—(—)	
M3-SF65-2.5	CU	188.00(5.21)	112.36		DB	11.42(17.71)	187.27	4.59(14.95)	NP	66.69(9.31)	
M3-SF65-2.5	CU	—	—		CO	8.60(12.13)	—	—(—)	DUP	23.72(29.99)	

Table 2.5: Data results provided from CT, UTT and 4PBFT

Chapter 3

A predictive strength model for HPFRCC/UHPFRC

The experimental results of M1 and M2 series, listed in Table 2.5, were used to develop an extended predictive strength model suitable for any HPFRCC/UHPFRC. The model took into account the matrix strength and the fiber properties (shape, V_f and l_f/d_e) as variables. A multi-linear regression analysis with 95% confidence intervals was computed in order to obtain the numerical relationships between the matrix strength, the fiber properties and compressive (f_c), uni-axial (f_t) and residual tensile (f_{rt}) strengths of any HPFRCC/UHPFRC. The processing of regression analysis is reported in the Appendix A2.

3.1 Compressive and tensile relationships

The experimental data confirmed a different behavior between the mortar reinforced with SF and that reinforced with HF, see Section 2.3. For this reason the proposed model is composed of two sub-models, one suitable for HPCC/UHPC reinforced by SF, see Equations 3.1-3.3, and the other one for HPCC/UHPC reinforced by HF, see Equations 3.4-3.6. Experimental observation denoted that in SF series, f_t decreased with V_f exceeding 4% (Figure 2.16), so the linear model of f_t does take into account a range of V_f from 0 to 4%. Instead the decrease of f_{rt} as both l_f/d_e ratio and V_f increase (Figure 2.17) was clearly observed for V_f ranging from 1.5 to 5%.

As discussed in Section 2.3.1, when adopting 5% of V_f , two out of three tested HF did not show an homogeneous distribution within the fresh cementitious mixture M1, therefore available data for this dosage were not enough to be taken into account into the model. For this reason the model does take into account a range of V_f from 0 to 4% for M1-HF series. For the same reason the predictions for M2 does take into account a V_f ranging from 0 to 3% for both SF and HF series

$$f_{c,SF} = 1.07f_{c,matrix} + 5.89l_f - 388.75d_e + 3.38V_f, R^2 = 0.92 \quad (3.1)$$

$$f_{t,SF} = 0.98f_{t,matrix} - 0.93l_f + 59.64d_e + 1.27V_f, R^2 = 0.91 \quad (3.2)$$

$$f_{rt,SF} = 0.16f_{t,matrix} - 0.13l_f + 16.41d_e - 0.09V_f, R^2 = 0.81 \quad (3.3)$$

$$f_{c,HF} = 1.03f_{c,matrix} + 0.3l_f - 32.7d_e + 6.28V_f, R^2 = 0.92 \quad (3.4)$$

$$f_{t,HF} = 0.96f_{t,matrix} + 0.05l_f - 5.09d_e + 0.88V_f, R^2 = 0.91 \quad (3.5)$$

$$f_{rt,HF} = 0.03f_{t,matrix} + 0.09l_f - 4.23d_e + 0.56V_f, R^2 = 0.88. \quad (3.6)$$

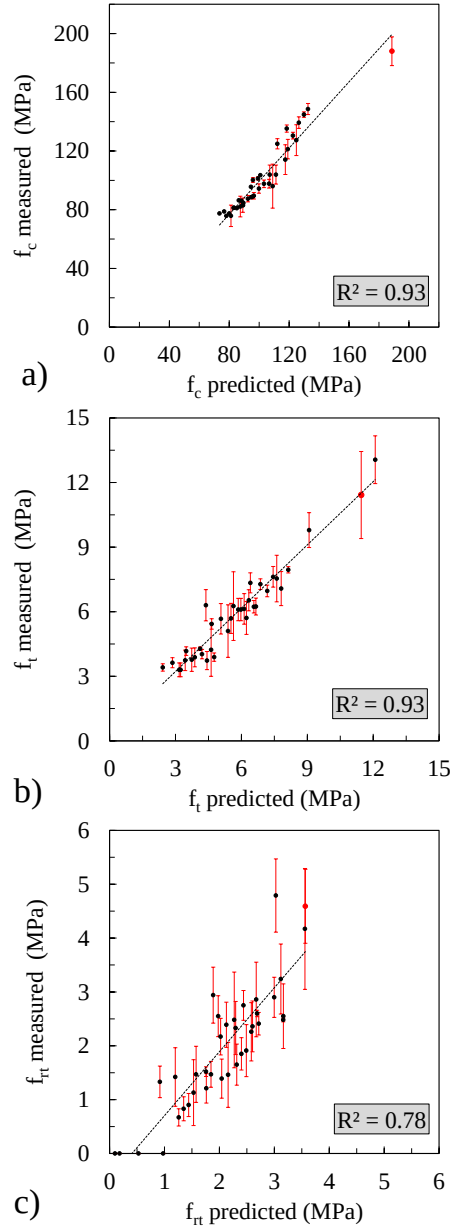


Figure 3.1: Correlation between measured (see Table 2.5) and predicted values (see Equations 3.1-3.6) of : a) compressive, b) uni-axial and c) residual tensile strengths of the proposed model

Figure 3.1 shows the relationships between the predicted (by using the Equations 3.1-3.6) and the average strength values measured on a population of three specimens, for each series of M1 and M2. Since matrix strength and fiber properties of the UHPFRC full-scale structural member (M3-SF65-2.5) were known beforehand, the model was used to predict its compressive and tensile strengths by using the Equations 3.1-3.3. Theoretical and experimental results of M3-SF65-2.5 series were also plotted in Figure 3.1 (red points). The red vertical bars on each point represent the deviation standard of the strength measured on a population of three specimens. The coefficients of correlation (R^2) between measured and predicted values were 0.93, 0.93 and 0.78 for f_c , f_t and f_{rt} , respectively. The high R^2 values confirmed a clear relationship between the parameters involved (matrix strength, shape of fibers, l_f/d_e and V_f) and the compressive and tensile responses of HPFRCC and UHPFRC materials investigated.

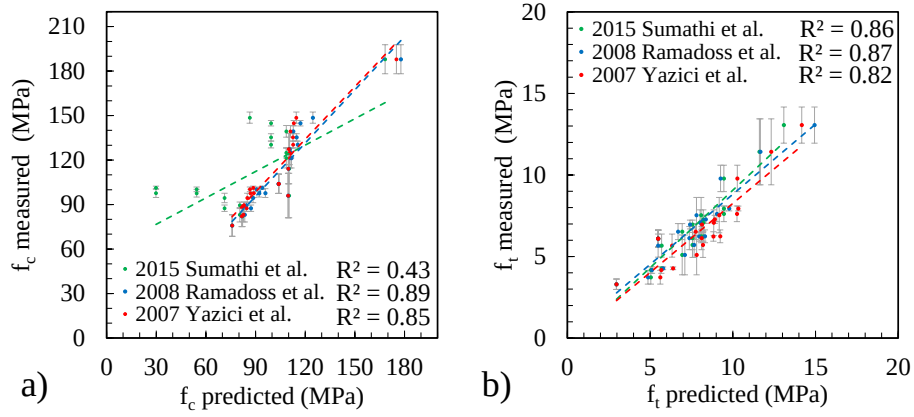


Figure 3.2: Correlation between measured and predicted values provided from the previous models for the a) compressive and b) the equivalent uni-axial tensile strengths

In Figure 3.2 the models found in Literature (see Section 1.5) were used to predict the experimental results presented in Section 2.3 in order to compare the benefit of the model developed in this PhD work thesis. The models available in Literature were developed by studying a limited range of l_f/d_e (only one type of fiber has often been studied), V_f (from 0 to 1.5%) and matrix strength. In fact these models are suitable only for Normal-Strength-Concrete (NSC) and/or HPFRCC materials. In Figure 3.2(a) some values are obtained just extending the prediction over the limits of the models themselves. The compressive strength was predicted by using the Equations 1.1, 1.3 and 1.5. Ramadoss et al. (2008) [106] took into account the compressive strength of the matrix as variable of the model i.e. for a $V_f = 0$; $f_c = f_{c,matrix}$. The compressive strength of the matrix of the SFRC materials investigated in [152] and [135] were not a variable but it corresponded approximately to the y-intercept parameter of the model carried out from the Equations 1.3 (~ 50 MPa) and 1.5 (~ 40 MPa), respectively. Substituting the y-intercept value of Equation 1.3 with the matrix strength value here investigated (M1, M2 and M3) it can be observed that also the model of Yazıcı et al. (2007) [152] well agrees with that of Ramadoss et al. (2008) [106].

The values of the compressive strength of the matrix investigated in this work were also implemented into the model of Sumathi et al. (2015) [135] as the y-intercept parameter in the Equation 1.5. In this case, as shown in [135] the compressive strength of the SFRC increased until the fiber dosage did not exceed 1.5 %; for higher V_f , the compressive strength decreased. The poor prediction ($R^2 = 0.45$) was likely due to the low strength of the matrix investigated in [135] that was an ordinary concrete. In fact, within the ordinary concretes the substantial presence of aggregates makes the consolidation of matrix-fiber weaker than in HPCCs, leading to a decrease of the strength and toughness of the reinforced concrete as V_f increases [133]. Although the proposed model could not be accurately suitable for ordinary concretes (matrix strength lower than 50 MPa), for advanced concrete materials (HPFRCC/UHPFRC) it showed higher R^2 values compared with the predictions of compressive strength observed on the models of Ramadoss et al. (2008) [106] and Yazıcı et al. (2007) [152].

The tensile strength of SFRC materials can be measured either by indirect methods, such as STT, or by direct ones, such as UTT. The previous models were calibrated on the basis of STT data, see Equations 1.2, 1.4, 1.6. Although STT is simple to perform and presents low fluctuations of the experimental results no information about the post-cracking behavior can be provided. In addition, several authors questioned its applicability for properly estimating the uni-axial tensile strength [26, 66, 141]. In fact, the actual codes prescribe UTT methods to characterize the physical properties of softening/hardening SFRC materials used in field [170, 187, 197]. Accordingly, in this work a UTT method was used for calibrating the proposed model and predictions with higher R^2 values were obtained, compared to those of available models, see Equations 3.1 - 3.6. However it can be interesting to compare the uni-axial tensile strength predicted by the proposed model with that predicted by the previous ones. In order to convert the splitting tensile strength predicted in the previous models into uni-axial tensile strength it can be appropriate to use the formula proposed in [171] that defines the following relationship:

$$f_{t,m} = 0.9 \times f_{sp} \quad (3.7)$$

where $f_{t,m}$ is the mean uni-axial tensile strength and f_{sp} is the mean splitting tensile strength. $f_{t,matrix}$ of the mortar investigated by Ramadoss et al. (2008) [106] was a variable of the model (see Equation 1.2). In the models of Yazıcı et al. (2007) [152] and Sumathi et al. (2015) [135], see Equations 1.4 and 1.6 respectively, the value of $f_{t,matrix}$ was measured via STT (4.06 and 4.00 MPa, respectively). Therefore this parameter into the previous models can be modified by using the $f_{t,matrix}$ ($V_f = 0$) of the mortars investigated in this PhD work thesis, converted by Equation 3.7. The comparison of the modified models is showed in Figure 3.2(b). The proposed tensile strength model well agrees with the models found in Literature, although the proposed model is more reliable and extends the prediction of previous models for UHPFRC as well, for hooked and straight fibers whose l_f/d_e and V_f range from 45 to 85 and 0 to 4% respectively. Furthermore the tensile model, developed by UTT, offers a more realistic prediction with high levels of reliability, in accordance with actual codes and practices, compared to the STT adopted in the previous models.

3.2 Bending flexural relationship

3.2.1 The Composite Material Theory applied to SFRC materials

Once the compressive and tension responses of investigated fibrous mortars were predicted by Equations 3.1 - 3.6, it was possible to compute their flexural bending response according to the analytical approach based on the CMT. This calculation is governed by the following hypothesis:

- Perfect alignment of the fibers in the direction of the stress;
- Perfect adherence of the fibers to the matrix guaranteed by the friction;
- Poisson ratio equal to zero, both for the matrix and fiber.

CMT applied to SFRC materials claims that the critical volume of fibers that might promote a strain-hardening behavior can be obtained computationally by equating the normal stress acting on the composite σ_c before and after cracking. Before cracking the normal stress acting on the composite is:

$$\sigma_c = V_{f,crit} E_f (1 - V_{f,crit}) \sigma_{mu} \quad (3.8)$$

whereas after the specimen cracks, the matrix does not contribute any longer to sustain the load and stresses are transferred to the active fibers bridging the cracked section. Then, in this case the tension of the composite becomes

$$\sigma_c = V_{f,crit} \sigma_{fu}. \quad (3.9)$$

By equating (3.8) and (3.9) the critical fiber volume that promotes a hardening behavior under uni-axial tensile stress is obtained

$$V_{f,Cr} = \frac{\sigma_{mu}}{\sigma_{fu} - E_f \epsilon_{mu}}, \quad (3.10)$$

where σ_{mu} is the maximum matrix tension just before the cracking; ϵ_{mu} is the corresponding deformation which, according to the assumption of perfect adhesion, is the same for steel fibers; σ_{fu} is the failure tension of the fibers (rupture or pull-out); E_f is the elastic modulus of the steel fibers.

If the volume of the fibers in the composite exceeds the critical threshold $V_{f,crit}$, a hardening post-peak behavior will be expected. If the volume of fibers is below the critical threshold, then a softening post-peak behavior will be observed. In order to calculate the critical volume of fibers $V_{f,crit}$ is firstly necessary to model the stress distribution in the cross-section. The pre-peak stress distribution in the cross section can be modeled as showed in Figure 3.3(a), with the tension stresses σ_{mu} and σ_{comp} having the same value. The post-peak behavior can be modeled as showed in Figure 3.3(b), where σ_{cu} is the ultimate strength of the composite reached in post-cracking phase. If the neutral axis depth is defined to be equal to $h/4$, the two ultimate bending moments (pre-peak $M_{R,a}$ and post-peak $M_{R,b}$) can be expressed as follows:

$$M_{R,a} = \frac{\sigma_{mu} b h^2}{6}, \quad M_{R,b} = \frac{\sigma_{cu} b h^2}{32}, \quad (3.11)$$

where h and b are the dimensions of the cross section.

By equating the moments it follows:

$$\sigma_{cu} = 0.41\sigma_{mu} \leftrightarrow \sigma_{mu} = 2.44\sigma_{cu}, \quad (3.12)$$

and

$$M_{R,b} \geq M_{R,a} \leftrightarrow \sigma_{cu} \geq 0.41\sigma_{mu}. \quad (3.13)$$

The fibers volume that is the boundary between hardening and softening behaviors in flexion can be obtained by inserting (3.12) into (3.9)

$$V_{f,crit,flexion} = 0.41 \frac{\sigma_{mu}}{\sigma_{fu}} \cong 0.41 V_{f,crit,traction}. \quad (3.14)$$

Eq. (3.14) proves that the hardening behavior in bending stress is also obtained reducing the volume of fibers requested to obtain the hardening behavior under uni-axial tensile load. Figures 3.4(a) and (b) show the uni-axial tensile stress - strain and the indirect tensile stress - strain curves, respectively obtained by applying (3.8) and (3.9) for uni-axial tensile stress-strain conditions and (3.11)a and (3.11)b for indirect tensile stress-strain conditions, under the hypothesis that the volume of the fibers is equal to $V_{f,crit,traction}$. As consequence a plastic plateau post-peak is observed in Figure 3.4(a) and a hardening post-cracking behavior with a maximum tension strength of 2.44 times the uni-axial tensile peak is observed in Figure 3.4(b). Figures 3.4(c) and (d) show the curves under the hypothesis that the volume of fibers is equal to $V_{f,crit,flexion}$. In this case a softening behavior occurs under the uni-axial tensile stresses with the stress decreasing to 0.41 times the maximum stress. Instead, a plastic plateau is observed under indirect stress-strain curves. Experimental data confirm the correlation between the uni-axial and the indirect tensile behaviors suggested by the CMT, as showed in Figure 2.14. This important result was obtained thanks to a proper evaluation about which test methods that better highlight the correlation mentioned above.

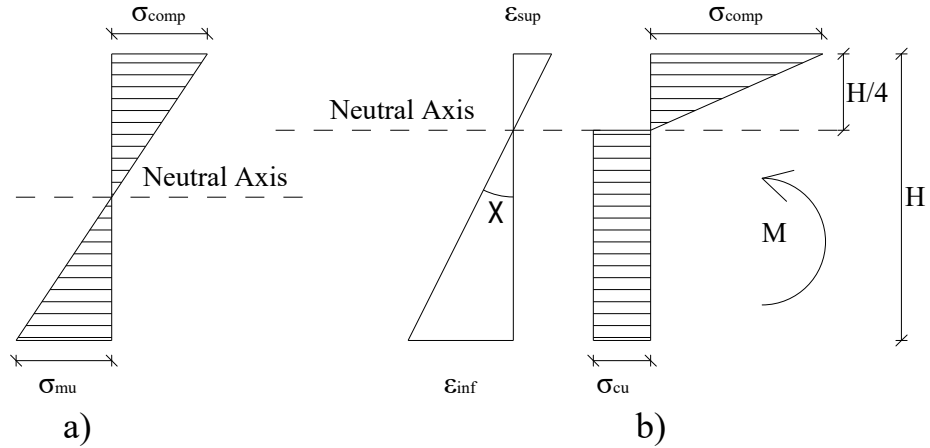


Figure 3.3: Idealized stress and strain distributions on the cross section of a bent beam: a) stress distribution before the matrix cracking, b) stress distribution in a post-cracking phase

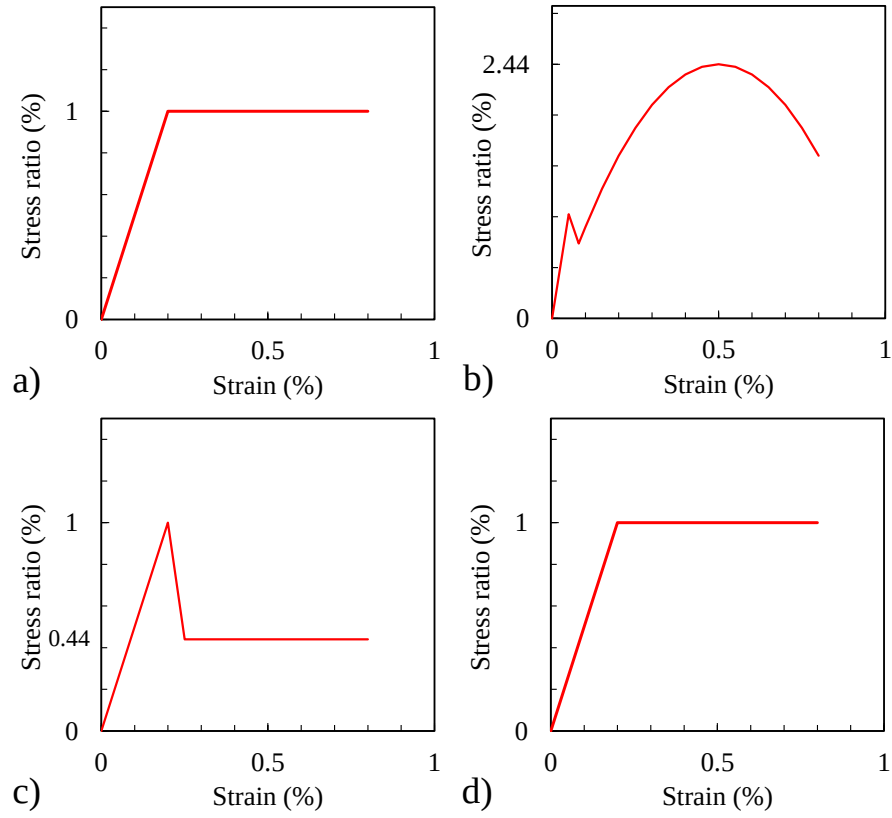


Figure 3.4: a) Schematic uni-axial tensile, b) indirect tensile curves computed in a mortar reinforced with a volume of fibers equal to $V_{f,crit,traction}$, see (3.12). c) Schematic uni-axial tensile, d) indirect tensile curves computed in a mortar reinforced with a volume of fibers equal to $V_{f,crit,flexion}$, see (3.13)

As mentioned above, the compressive (f_c) and the uni-axial tensile (f_t and f_{rt}) responses of both the core specimens of the full-scale structural beam and the cast specimens of the series M1-HF85-1.7, M2-SF75-3.8 and M3-65-2.5 were measured (Table 2.5) and compared with those predicted by the proposed model, showing a great agreement, see Figure 3.1. Consequently it was possible to compute the flexural bending response of SFRC materials investigated according to the analytical approach presented in Section 3.2.1. For this purpose a non-linear model was adopted for modeling the compressive response predicted, see Figure 3.5(a). The compressive axial deformation values corresponding to both the peak and the residual strengths were defined according to [172]. A bi-linear model was adopted for modeling the uni-axial tensile response predicted, see Figure 3.5(b). The tensile axial deformation values corresponding to f_t and f_{rt} were the same detected in the experimental investigation, see Section 2.3.3.

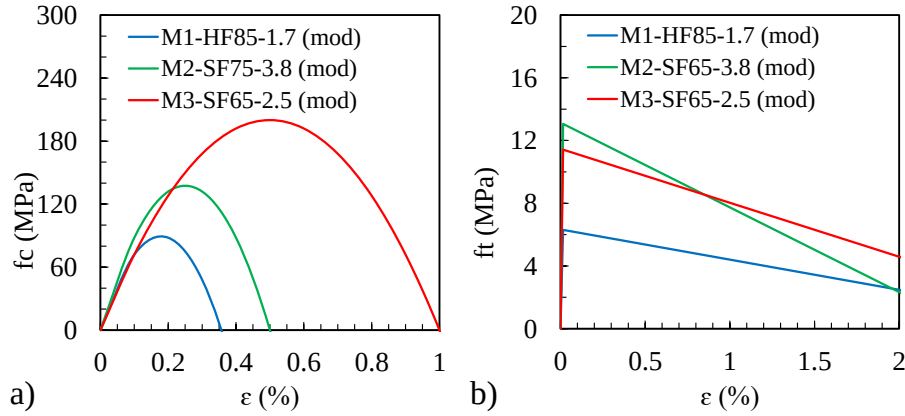


Figure 3.5: Modeling of the constitutive laws for: a) compressive response - the peak was predicted by the proposed model, the branch was defined according to the [172]-, b) uni-axial tensile response - the key points of both stress (f_t and f_{rt}) and deformations were calibrated on the basis of the predictions provided from the proposed model -

Finally, the predicted compressive and the uni-axial tensile responses were used for analyzing the incremental deformation and stress fields on a plane section subjected to the bending flexural load. The Euler-Bernoulli hypothesis was adopted for. A numerical solver named *Mathematica* was adopted for computing the flexural response of such a beam in terms of bending moment vs CTOD. The theoretical calculations implemented into the solver are reported in Appendix A3 with the addition of some preliminary remarks. The calculation was performed for reproducing the experimental 4PBT carried out on the investigated series.

A comparison between the theoretical and the experimental results in terms of bending flexural load vs CTOD is illustrated in Figure 3.6. It is worth noting that the prediction of the flexural response of the core specimens of the full-scale structural beam slightly overestimated the real response, see Figure 3.6(c). This was due to the fact that in core specimens a not-preferential orientation of fibers occurred, contrary to the cast specimens of the same material (M3-65-2.5). In addition, the lower performances recorded on core specimens were also due to

the light damage of the latter during the coring and extraction, as claimed in Section 2.3.5.1. Both conditions reduced the mechanical performance at the hardened-state of the SFRC. In fact, an overestimation of the model was observed only for 4PBFTs carried out on core specimens. Instead, the 4PBFTs carried out on cast specimens showed a good agreement with the theoretical results, see Figure 3.6. The figure also shows that the lower R^2 value corresponding to the prediction of the f_{rt} (see Figure 3.1(c)), compared to the prediction of f_c and f_t , slightly reduced the reliability of the predicted post-peak flexural response, especially for M2-SF-75-3.8 series, illustrated in Figure 3.6(b). Such a result was due to the high scatter of tensile strength recorded in the investigation, see Figure 2.14.

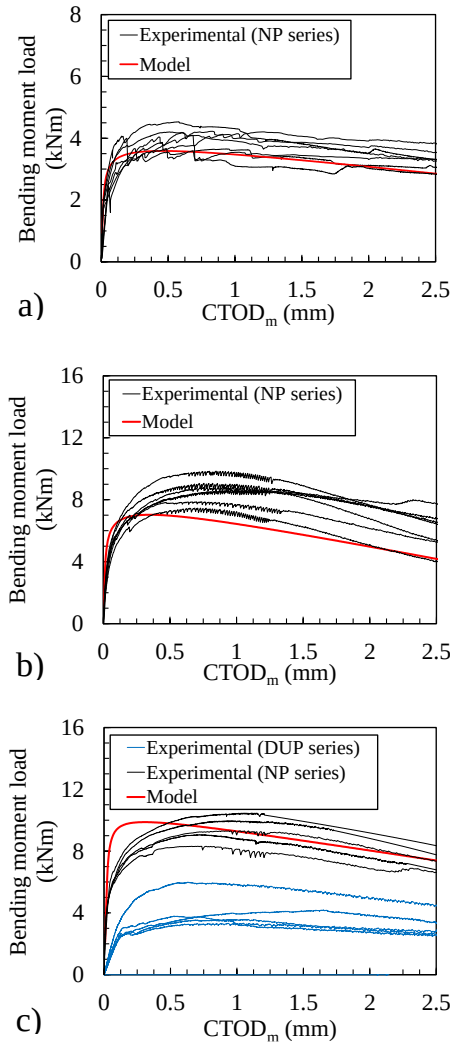


Figure 3.6: Validation of the proposed model by comparing the predictive with the experimental results of the series a) M1-HF85-1.7, b) M2-SF75-3.8, c) M3-SF65-2.5

3.3 Application of the predictive model on an existing UHPFRC structure



Figure 3.7: Photo of the Olympic Museum at Lausanne. The roof in UHPFRC was completed in 2013

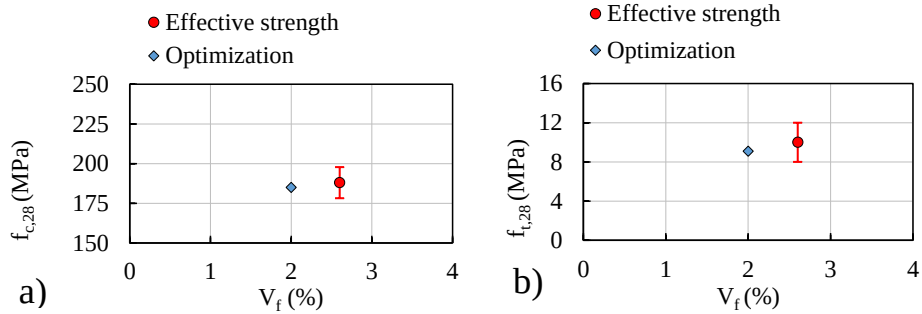


Figure 3.8: Virtual optimization of the UHPFRC mix design used to build the roof of the Olympic Museum at Lausanne, in terms of a) compressive, b) uni-axial tensile strengths

In order to provide an example of application of the proposed model, an existing UHPFRC structure was considered for this purpose, in particular the new roof for the Olympic Museum at Lausanne (Switzerland) showed in Figure 3.7. The new roof was built to cover the existing building (finished in 1990). The concerning work is further described in [92]. The roof is made of a commercial UHPC, named BSI, reinforced with discrete steel straight fibers with 65 and

2.6% of l_f/d_e and V_f respectively. The control quality of such a material was carried out via both 28-days compressive and 28-days uni-axial tensile tests. The compressive strength was 188 ± 9.8 MPa. The uni-axial tensile strength was about 10 ± 2 MPa. The model presented above was therefore used for optimizing the type and the fiber dosage adopted within the formulation of the existing material, in order to ensure the same performances measured at the hardened state. For this purpose, an algorithm was implemented into the model for finding the optimal solution in terms of fiber properties (fiber dosage, aspect ratio, shape), which was detected considering all commercial fibers here investigated. The prediction ensured the same performances measured in the existing case by changing the current set of fibers (SF-65-2.6) with the type SF-75-2. It is important to note a reduction of V_f of about 30%.

Despite the promising results of application of the model, some aspects should be considered. The results of the model were interesting even though real testing at the hardened state of the optimized UHPFRC formulation would be necessary to confirm that. In addition, the amount of fibers predicted by the model was lower than the existing one, which can reduce the risk of fiber balling. However, any change of the fiber properties may affect the fresh properties of the fibrous mortar and the model does provide no information about such properties. Consequently, preventive tests on the fresh optimized fibrous mortar are recommended.

Chapter 4

Conclusions

The main conclusions of the first part of this PhD work thesis are drawn in the following:

- The influence of the formulation, in terms of matrix and fiber features, of commercial HPFRCC and UHPFRC materials on their compressive, tensile and flexural bending responses at the hardened state was investigated. The experimental results confirmed that the mechanical behavior of the tested materials is specifically influenced by the matrix strength, the shape of steel fibers, l_f/d_e ratio and V_f . This results confirmed the need to develop a generalized predictive model useful for the practitioners;
- New linear mathematical relationships between the formulation of the fibrous mortar (matrix properties, shape of steel fibers, l_f/d_e ratio and V_f) and its mechanical performances at the hardened state were established. While previous models predict only the peak of the splitting tension strength of ordinary reinforced concrete and HPFRCC materials, the proposed model can also predict the post-cracking behavior and the toughness, which represent the most relevant performances requested by designers. In addition, the prediction of the proposed model showed higher R^2 values compared to the previous ones, providing reliable results for SFRC materials whose compressive and uni-axial tensile matrix strengths are superior than 50 and 3 MPa, respectively, i.e. for any HPFRCC and UHPFRC;
- Once the compressive and uni-axial tensile responses were predicted by the proposed model, an analytical calculation based on the CMT permitted to estimate the flexural bending response of both a full-scale structural UHPFRC beam and different marked UHPFRC prismatic specimens. A good agreement with the experimental results was evident, confirming the reliability of the proposed model. In order to make the prediction of the post-peak flexural bending response more accurate, the regression analysis on more experimental data could reduce the scatter of the post-cracking response under uniaxial tensile stress, which is still high for certain fibrous mortars;
- A great agreement between experimental and predictive data confirmed that the model can be used to correlate the results provided by the different mechanical tests of characterization requested for a given product;

for example, if the compressive and tensile responses of a given matrix of HPCC or UHPC are measured, the model can estimate both the flexural bending response and the increase of performance as steel fibers are added within the cementitious mixture. In addition the model can be used for detecting the optimal V_f , shape and l_f/d_e ratio of fibers to meet the designers' requirements, in terms of compressive and tensile responses, for any commercial HPFRCC/UHPFRC. An example of application of the model on an existing UHPFRC structure confirmed this statement. In particular, the model identified the minimal dosage of fibers necessary to attain the strength requested by designers, reducing the costs of manufacturing as well. The predictive results permitted in fact to reduce the fiber dosage from 2.6% to 2% by volume, by substituting SF65 with SF75. The results of the model were interesting even though real testing at the hardened state of the optimized UHPFRC formulation would be necessary to confirm that. In addition, any change of the fiber properties may affect the fresh properties of the fibrous mortar and the model does provide no information about such properties. Consequently, preventing tests on the fresh optimized fibrous mortar are recommended;

- In order to both minimize the scatter results recorded during the characterization tests of the HPFRCC/UHPFRC studied and measure a representative mechanical response, adequate tests were carefully set-up by Author. In particular, a specific device was designed for UTT in order to prevent the arising of bending stresses during the tensile loading. In order to avoid the problem of unexpected crack formation in the vicinity area of the notch of SFRC beams, which is typically observed in 3PBFT, 4PBFT on notched and un-notched specimens were carried out;
- In this investigation the specific weight of the studied mortars increased with using steel fibers up to 300 kg/m³. The compressive and the peak of tensile strengths increased between 2 – 42% and 3 – 222%, respectively when fibers were added to the plain HPCC/UHPC. This result showed the efficiency provided by both SF and HF added into the mortars investigated. The use of HF influenced considerably the toughness of the mortar. In particular, as the fiber dosage increased for both high (85) and low (45) aspect ratio cases an improvement in toughness was observed;
- The mixing, casting and vibration procedures influenced the distribution, orientation and segregation of fibers. As a consequence, the presence of both slight segregation and fiber balling were observed in series containing heavy fibers and high dosages, respectively. For these series a loss of both strength and toughness at the hardened state occurred;
- The fiber orientation strongly affected both the peak strength and the energy-absorbing capability of the post-cracking behavior, as seen by comparing the experimental results of cast and core specimens. In fact, in the cast specimens the mold's shape encouraged a certain degree of fibers orientation, while in the cores drilled from the full scale structural beam fibers were randomly oriented. Another relevant effect of scatter in strength between cast and core specimens can be explained by considering the distortion of the section in the release of a self-restrain or in the effect of the coring barrel;

- The mechanical test results were also used to classify the HPFRCC/UHPFRC investigated, that were rated as low ductility SFRC. This might be the effect of the standard test protocols that are still strongly sensitive to the high scatter of SFRC performances.

Part II

A predictive cohesive bonding model for NSC slabs overlaid with a layer of HPFRCC/UHPFRC

Preface of Part II

The second part of this PhD work thesis is subdivided into five chapters.

The Literature concerning the problem of bonding failure of retrofitted cement-based composite structures is summarized in the first chapter. This kind of composite structures, which include concrete slabs, bridge decks and pavements, are frequently stressed to bending loads in the practice. Near to zones of maximum negative moment the bending load induces a high stress at the interface. If in these zones there exist some discontinuities of the overlay (boundary locations, joints and cracks) the cracking/debonding along the interface might occur. A current issue concerns the prediction of such a phenomenon in the field when advanced materials like HPFRCC and UHPFRC are used for rehabilitating such structures. Numerous efforts are demanded for properly assessing the effective load capacity of the bond at the interface. For this purpose, the development of a proper cohesive model could be of a great interest.

An experimental campaign was carried out for calibrating the cohesive parameters employed into the proposed model. In addition, an independent bond test was carried out for confirming the theoretical results, by reproducing a realistic case of cracking/debonding failure. Details concerning the experimental set-up are explained in the second chapter.

The third chapter presents the development of the model above-mentioned. Theoretical results are compared with those measured by the experimental tests. The fourth chapter presents a FE model calibrated on the basis of the recorded experimental data. The FE simulations of a cracking/debonding failure permitted to observe the propagation process of the fracture along the interface. The main conclusions of the second part of this PhD work thesis are drawn in the fifth chapter.

Chapter 5

The risk of bond failure in cement-based composite structures: State-of-the-art

Nowadays design codes for building constructions are becoming more stringent and precise than the past, e.g. in regard of out-of-plane shear design and earthquake-capacity design. In a landscape of morose economic growth, the budgets of the governments for the infrastructures are often reduced. Nevertheless ageing of the structures do not stops especially for elements highly exposed to both harsh environment and ever increasing loading such are the bridge decks, pavements, tunnels, piles and linings. The quest of efficient and cost-effective rehabilitation methods, able to extend the service life of the structure, is therefore a priority for all researches and public administrations. The use of state-of-the art for overlay materials and techniques of rehabilitation is one of the most interesting answers found by the scientific community which are adopted by many engineers. However, a current issue highlighted in the field of retrofitted cement-based composite structures concerns the risk of premature bond failure near to zones of discontinuities of the overlay. Such a phenomenon is often observed in the practice, after a period of service. HPFRCC and UHPFRC are among the breakthroughs in the recent concrete technology which could provide a solution in this challenge. In particular, the application of HPFRCC as repairing material on deteriorated structures showed significant attention in last years [34, 75]. Instead for the UHPFRC, despite the better performances compared to HPFRCC in terms of strength, ductility and durability [30, 117, 119], the high manufacturing cost and the missing harmonization of existing codes restrict still its wider usages. Nevertheless, several applications involving the use of UHPFRC have been gradually increasing, also in certain pilot projects of rehabilitation and strengthening, such as bridge deck overlays [22, 63] and hydraulic structures [59, 100]. Beside its higher strength, low permeability and high energy absorption [22], compared to HPFRCC, UHPFRC has also shown high bond strength and good adherence to substrate made of NSC [2, 67, 140]. Further research also denoted that during the service life of these composite concrete, in case of failure in the overlay (HPFRCC or UHPFRC), the cracks in the matrix are retained by steel fibers which can transmit the force through the

fracture, thus the built-in peak stress at the interface decreases, reducing the width of the cracks [57]. Furthermore, steel fibers reduce the volume changes of the overlay, limiting eigenstrains at the interface which permits to increase the loading bond capacity. Further increases of the loading bond capacity can be also achieved by installing a series of dowel bars properly anchored in both the substrate and the HPFRCC/UHPFRC overlay. Dowel bars have to be deformed plastically prior to carry a relevant part of the load. But since they are more rigid than the overlay, they carry the load only after the bond has broken [130]. This solution, even if increases the global capacity of the bond, does not prevent a premature deterioration. These findings revealed that HPFRCC/UHPFRC can be suitable for properly retrofitting cement-based composite structures, even though a proper estimation of the loading capacity at the interface should be requested [2, 22, 96, 155].

The term *interface* relies on the penetration and hardening of the overlay inside the open cavities at the surface of the concrete substrate, resulting in a physical anchorage [14]. Two main factors can deteriorate such adhesion:

- i) Different volume changes between overlay and substrate (thermal expansion coefficient, elastic modulus);
- ii) Effects of the external mechanical loading [56].

For both cases, it is possible to distinguish different patterns of failure: i) the peeling stress perpendicular to the interface exceeds the tensile strength, which causes cracking (mode I); ii) the shear stress along the interface exceeds the respective strength, which causes debonding (mode II); iii) a mix of mode I and mode II [58]. The latter usually occurs in retrofitted cement-based composite structures, such as slabs, bridge decks and pavements.

These composite structures are frequently stressed to bending loads, see Figure 5.1. Near to zones of maximum negative moment the bending load induces a high stress at the interface of mode I which, detrimentally, adds its effects to mode II. If in these zones, there exist some discontinuities of the overlay, e.g. boundary locations, joints or cracks, the cracking/debonding failure might occur [56], see the schematic illustration in Figure 5.2. In such conditions the bond failure is governed by a mutual interaction between mode I and mode II. Several bond tests were designed in order to assess the loading bond capacity in cement-based composite structures, even though they cannot measure the mutual interaction between mode I and mode II, see Section 5.2. Not less important is that the premature bond failure also depends on the quality of the surface preparation of the substrate prior to the application of the overlay. In order to clarify the effect of both parameters, a relevant summary is presented in the following.

5.1 Influence of the substrate preparation on the development of bond strength

The preparation of the substrate surface is a process by which a clean, sound and properly roughened surface is obtained on the concrete substrates [146]. The soundness and roughness of the substrate strongly influence the bond strength development. Both parameters seem to depend on the removal methods of



Figure 5.1: Switzerland 2018 [21] - Retrofitting of a bridge deck member by using a commercial UHPFRC

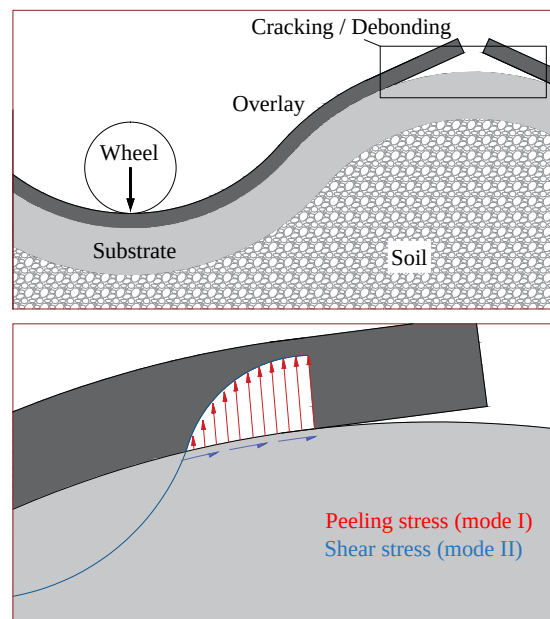


Figure 5.2: Schematic illustration of the cracking/debonding path along the interface of concrete pavements repaired by a layer of cement-based overlay, due to loading conditions [56]

the deteriorated concrete [15, 130]. The most common removal methods used in the field include demolition of a layer of the existing concrete by impact, high pressure water, or mixture of them. The impact methods are based on the use of breakers (jackhammers, sandblasting, etc.) to fracture and spall the unsound concrete. Care shall be exercised in order to minimize the damage to the sound concrete. The advantage of such a method is that rougher surfaces of the interface are provided, which is beneficial to the bond strength [90]. The drawback is that the heavy impact performs micro-cracks on the concrete surface, which have a detrimental effect on the bond strength [55, 148]. According to some authors is possible to remove the micro-cracks and provide a sound surface by using sandblasting (with an appropriate sand) after jack hammering [24]. Another removal method used in the practice is the hydro-jetting. The use of a small jet of water driven at high speeds permits to attain a pressure of more than 300 MPa. Hydro-jetting literally disintegrates unsound or deteriorated concrete and ensures a substrate with a sound and rough surface profile. The roughness profile by hydro-jetting is less pronounced than jack hammering, but no micro-crack is observed [90]. Kauw et al. (1997) and Silfwerbrand (2000) [73, 127] discussed the effect of hydro-jetting on the quality of the concrete surface. They concluded that hydro-jetting is a good method to remove unsound concrete, but a minimal compressive strength of the substrate is requested to avoid the rupture of sound concrete too. In particular, Bissonette et al. (2008) [15] observed that lower compressive strengths of the substrate led to lower bond strengths. The interface roughness also plays a key role on the development of bond strength. Research findings reported in [128] showed that the roughness profile provided by sandblasting resulted in maximum gains of tensile bond strength, compared to other removal techniques; values of roughness beyond this threshold did not show relevant gains of strength. In [10] it was stated that the interface roughness is of major importance for shear bond mechanisms than tensile mechanisms. Several researchers also investigated the influence of the carbonation of the existing substrate on the development of bond strength. According to [60, 121] the substrate carbonation reduced the bond strength. By contrast, in [17] the bond strength was not affected by surface carbonation. Further investigations showed that for carbonated substrate surfaces prepared by not detrimental technique (e.g. sandblasting) no loss in bond strength were recorded. Conversely, for substrate surfaces prepared by impact technique (e.g. chipping hammer), a reduction of bond strength of about 16% was observed [14]. Such an effect was due to the presence of micro defects of the surface caused by the impact of the hammer, further aggravated by the carbonation process and related volume changes. Recent investigations pointed out that moisture conditioning of the concrete substrate surface at the time of application of the overlay strongly affects the development of bond strength. Certain authors stated that a *dry* substrate condition prior to overlay led to better performances of the bond than *saturated-surface-dry* (named hereinafter SSD) conditions; in certain cases, SSD treatment was even detrimental [11, 145]. Different observations were presented in [87, 144], claiming that SSD condition improved the bond strength. Other authors suggested that the optimal saturation level ranges from 55 to 90% [16]. These findings show that such a phenomenon is still controversial. From a theoretical point of view, in the case of dry conditions, concrete substrate surface tends to adsorb water from the fluid overlay. A densification of the micro-structure in the interfacial zone is possible and a flowing overlay properly

fills the asperity of the roughened interface [9]. After a proper curing such a densified zone creates a stronger bond between substrate and overlay. If too much water is removed from the overlay at the interface, the risk of insufficient hydration rises up, which leads to a weak bond. In this case the interfacial zone becomes a site of premature cracking/debonding under shear/tension stresses. In the case of wet conditions, the film of water on the substrate tends to dilute the overlay at the interface. This phenomenon increases the water/binder ratio of the overlay, which leads to lower material strength, low bond strength and increased shrinkage as well. In the worst case scenario, free water at the interface can even prevent the cement particles of the overlay to fill the pores of the substrate, increasing the risk of premature bond failure.

5.2 Influence of test methods on the estimation of the loading bond capacity

Current specifications in the concrete repair technology suggest that bond strength is defined as the tensile strength measured at the interface via *pull-off* bond tests (named hereinafter POBT) [167, 169]. POBT is the most accepted method used among practitioners, being easy to set-up both in situ and in the laboratory. However, test results can be affected by both the eccentricity in the load application and the damage during the coring. A solution can be found by using a *direct tension* bond test (named hereinafter DTBT) [85]. A limit of both tests concerns the fact that if the bond strength is higher than tensile strength of bonded materials the failure will not be at the interface, but it will be localized to the weaker zone, which could be the substrate or the overlay. In these conditions no information about the bond strength would be revealed. Not less important is that in real structures the external loading and the differential volume changes between substrate and overlay not only cause tension stress at the interface, but also shear stress. Furthermore, bond response in tension differs from that in shear. For instance, high interface roughness can affect the shear bond strength, but not necessary the tensile strength too. The latter is more sensitive to the transverse anchorage in pores and voids than vertical nooks [10]. Such considerations make POBT insufficient to characterize the bond strength in most cases.

In many practical cases the interface is subjected to pure tension only at the small zones close to edges. By contrast, shear stresses at the interface occur along the entire interface, e.g. in composite slabs subjected to bending loads. In these cases, it might be more interesting the evaluation of the shear bond strength. Several shear test methods were developed, even though none of these is still accepted as standard [129]. Among shear tests, the *slant shear* is the most used in the researches; the set-up is easy, the reliability of the results is good. Nevertheless several authors indicated as the primary drawback of the slant test the unrealistic loading conditions applied to the interface [35]. Secondly, the failure of the interface depends on the angle of the plane with respect to the load. In addition, this test proved to be relatively insensitive to the surface preparation and roughness. In fact, bond failures occurred only for smooth surfaces [4, 35]. A more realistic loading condition can be reproduced via *lateral shear* tests (named hereinafter LSBT), but a disadvantage is the presence of a bending moment at

the interface, due to the shear force eccentricity, which affects the shear strength. In order to prevent such an inconvenient Silfwerbrand (2003) [129] developed a *twist-off* test. A torque load provides an equivalent pure shear strength at the interface. Clearly, the real force applied to the structure is not a torsion on the interface, but this test can simulate the behavior of the interface under direct shear stress. Although, according to the theory of brittle material strength, like concrete, the failure plane is not parallel to the torque plane, but it has an inclination around 45 degrees. In the case of bonded materials subjected to torsion torque, the plane of failure does not correspond with the plane of the interface. In fact, results in [129] showed that among 58 specimens, only one of them failed at the interface. Further investigations recorded a percentage of failure at the interface of about 30% and the corresponding bond failure pattern was not localized at the plane of the interface but appeared to be uniformly distributed along the length of the tested cores [14]. In [12] a different method, named *direct shear* (named hereinafter DSBT) was used, solving the problem of geometric incompatibility of the twist-off test. In such a test, the shear load is totally transferred along the interface, which permits to reduce the bending moments and tensile forces arising at the interface. The geometric configuration of the DSBT is compatible with the debonding plane of the interface, contrary to the twist-off test; test results hence provide the direct shear bond strength, which is not measurable with other shear tests. Care should be taken on the alignment of the test apparatus, otherwise interface bending stresses might arise. In practical situations the bond failure between substrate and overlay propagates in a mixed mode of stresses at the interface, i.e. *mode I* (peeling) combined with *mode II* (shear) [58]. Current bond tests can assess the bond performances subjected to solely mode I (pull-off) or to a given combination of mode I and mode II (slant/shear test). Few research focused on a finely prediction of the realistic mixed-mode stress, by coupling pull-off and slant/shear tests and adopting some empirical bond failure envelopes such as the Griffith theory and the Mohr-Coulomb criterion. Despite the efforts carried out by these authors, attempts according to the Griffith theory have proved unsuccessful, while the Mohr-Coulomb criterion should be appropriate with stress states within a material rather than at the interface between two bonded together [4]. In conclusion, by adopting current bond tests the risk to improperly estimate the effective loading bond capacity could be high, since they cannot directly measure the coupling factor during a realistic condition of mixed-modes stress occurring along the interface. Furthermore, more fundamental approaches to predict the bond failure are now requested, via cohesive models, even though, in a lack of data, the interaction between mode I and II is still theoretically estimated [58].

5.3 Numerical FE investigations

It is well known that the interaction between the composite concrete members represents a relevant challenge in a numerical modeling, due to the complicated nature of their adhesion [130]. Concerning studies have been very limited in the past. In the work of [77] the interface of a composite NSC-UHPFRC member was modeled but no experimental data were available to confirm the results provided by their model. The investigation reported in [2, 114] aimed to predict the behavior of composite NSC-UHPFRC members using a 3D FE analysis under

the assumption of a perfect bond at the NSC-UHPC interface, even though such an assumption increases the risk to overestimate the ultimate capacity of the composite structure. The efforts accomplished in the above-mentioned works have been shown relevant outcomes, but a calibration of the FE data on the basis of the experimental results was often missing, or experimental investigations were carried out without any numerical simulation. Recently, some works proposed FE models calibrated on the basis of the experimental data. It is interesting to note that different approaches for modeling the interface were proposed in these works. In [156] it was proposed an equivalent beam elements to represent the bond behavior at the interface, instead in [45] an interface material between the concrete layers was used. Both models proved to be efficient and were validated by comparing their results with a three-point flexural test. It is important to note that these models reduced the overestimation of the bond performances, compared to “tie” models, in particular when either no surface preparation or no bonding agent are employed in certain cases [20, 45]. Despite their good efficiency the models found are complex and risk to reduce the interest of the practitioner which tends to simplify its numerical simulations by using “tie” interface model [70, 116]. However, with this assumption, the behavior of the interface cannot be directly observed when the cracking/debonding failure begins, so the risk to overestimate the bond performances increases.

5.4 Scope

The aims of the second part of this PhD work thesis are listed in the following:

- i) Development of a theoretical cohesive model able to properly predict the loading bond capacity of retrofitted cement-based composite structures. In particular, the model can predict the load-slip behavior of bonded materials subjected to mixed mode stresses, by taking into account the coupling factor between mode I and mode II, the moisture conditioning of the substrate prior to the application of the overlay and the fresh properties of the latter. The peeling-crack (mode I), the shear-slip (mode II) relationships and the coupling factor between mode I and mode II, implemented into the modeled interface, were calibrated on the basis of the experimental data provided from POBT*, DSBT and tension/shear bond test (TSBT) respectively. These bond tests were accurately designed by Author. TSBT provides infinite combinations of bond tension/shear ratio, in order to encompass all possible stress states encountered in the practice, typically near to discontinuities of the overlay (boundary locations and joints) where the cracking/debonding path begins. A further bond test (labeled in the following LSBT) was designed by Author in order to reproduce a practical case of cracking/debonding pattern and compare the experimental with the theoretical results;
- ii) Investigation of aspects affecting the quality of the substrate preparation prior to the application of the overlay, like the substrate moisture conditioning, the fresh properties of the overlay and roughness profile of the interface. Dry, 75% and SSD conditions were taken into account. One marked HPFRCC and one marked UHPFRC were used for overlaying the

substrate. In order to properly reproduce the rehabilitation of cement-based composite structures in the practice, the hydro-jetting method was adopted to prepare the substrate prior to the application of the overlay. The roughness profile was carefully analyzed;

- iii) Development of a simple and accurate FE model able to reproduce the propagation of the damage process at the interface subjected to mixed-mode stresses. The FE interface was modeled with a nonlinear cohesive law which couples the relationships of mode I with mode II on the basis of the experimental data. Additional experimental tests on both NSC substrate and HPFRCC/UHPFRC overlay were performed to calibrate the FE model.

Chapter 6

Experimental program

6.1 Materials and Methods

Several concrete slabs were cast, exposed to weather conditions for 90 days and then subjected to the surface treatment by hydro-jetting. The roughness profile of the surface was measured by photo-scanning. Then, the substrate was prepared to the application of the overlay. Two commercial advanced fibrous mortars were used as overlay, one HPFRCC and one UHPFRC. After 28 days of curing, the specimens were prepared and tested. Further details are reported in the following subsections.

6.1.1 Substrate

The substrate was made using a commercial self-compacting normal-strength-concrete (NSC) reinforced by steel bars. Both concrete and steel bars were compliant with the Swiss standards and are largely used to build slabs on grade, bridge decks, pavements, slabs etc. The material properties of the NSC are reported in Table 6.1. Thirty slabs of NSC were cast. Twelve 150 mm cubes of NSC were made as well, in order to determine the compressive strength of the substrate, which is fundamental to estimate the adequate water pressure magnitude of hydro-jetting, see Section 5.1. The geometry of the specimens differs from one another, according to the corresponding test configuration, see Section 6.1.4 and Figure 6.1. In order to consider the effect of different environmental conditions on existing concrete members, concrete slabs were cast outdoors, both in summer and winter (Switzerland). After molding, concrete slabs were covered by a plastic sheet and cured in the lab under constant environmental conditions (). After 28 days of curing, concrete the slabs and cubic specimens were demolded. Cubic specimens were tested under compressive load, according to European standards [180]. Instead, slabs were placed outdoors and exposed to external weather conditions (sun, wind, rain) ranging from 24 °C and 74% RH in summer and 7 °C and 87% RH in winter, for additional 90 days.

6.1.2 Treatment prior to overlay

After 90 days of external curing, the upper surface of slabs was treated by hydro-jetting, by using a Hammelmann 2500 bars machine, see Figure 6.2(d).

Properties	NSC	M1	M2
Maximum aggregate size (mm)	16	6	2
w/c ratio	0.4	0.28	0.17
Slump test (mm)	700 ^(a)	240 ^(b)	250 ^(b)
Slump test T500 (mm)	—	60 ^(c)	517 ^(c)
Compressive strength (MPa)	$59 \pm 3.3^{(d)}$	$78 \pm 3.1^{(d)}$	$147 \pm 5.3^{(d)}$
Tensile strength (MPa)	$2.9^{(e)}$	$6.5 \pm 0.9^{(f)}$	$14.6 \pm 1.12^{(f)}$
Modulus of Elasticity (MPa)	35000 ^(g)	$38300 \pm 200^{(h)}$	$47100 \pm 200^{(h)}$
Poisson's ratio	0.2	0.17 ⁽ⁱ⁾	0.2 ^(k)

According to [178](a), [177](b), [179](c), [180](d), [172](e), [170](f), [174](g), [176](h), [86](j), [64](k),

Table 6.1: Material properties

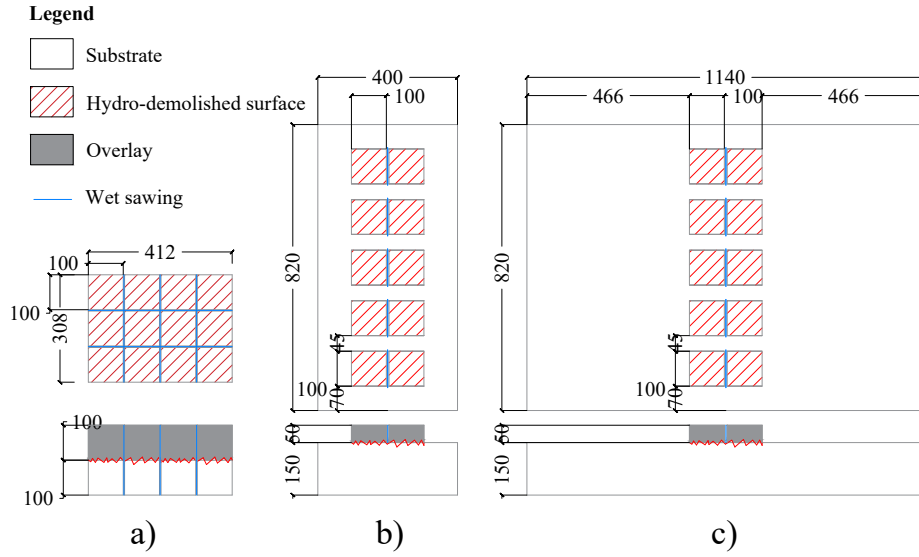


Figure 6.1: Layout of specimens prepared for bond testing: a) DSBT and DTBT, b) POBT*, c) TSBT and LSBT

The water pressure of hydro-jetting was adjusted on the basis of the substrate compressive strength, in order to avoid issues discussed in Section 5.1. Practical observations suggest that a proper hydro-jetting can be achieved when the ratio between water pressure and compressive strength of the substrate ranges from 2.5 to 3.5, for a standard distance between the nozzle and the concrete surface. Since the compressive strength of NSC was 59 ± 3.3 MPa, water pressure was set to 1800 bars. A concrete layer of 50 ± 10 mm was removed by hydro-jetting. The resulting surface was rough and sound, as confirmed by a visual examination, see Figures 6.2(g), (h) and (i). There exist some methods to measure the roughness profile. The best suited method seems to be the Concrete Surface Profile Chips (CSP), in accordance with [184]. However, the roughness surface magnitude achieved by hydro-jetting is beyond the limits of measure of the current set of chips [103]. Furthermore, CSP method does not give a quantitative evaluation of the geometry profile. For these reasons, a photogrammetric method, which does not depend on the roughness magnitude, was preferred to analyze the roughness

profile; in addition, it permits to produce reliable and accurate quantitative data, although it needs intensive work processing. The *AgisoftPhotoScan* software was adopted to process digital images and generates 3D spatial data of the scanned surfaces. Processed data provided the roughness altitude in 4 points per square millimeters. Since the interface area of each specimen was 100×100 mm, a population of 40000 points was used to calculate the average roughness and its standard deviation.

Once roughness profile of the interface was scanned, the substrate was prepared to the application of the overlay. In order to quantify the influence of the substrate moisture conditioning on the bond strength, the whole range of possible moisture levels was taken into account, in particular *dry*, 75% and *SSD*. Dry-type surface was reached by curing the substrate surface for 14 days at laboratory conditions of $23 \pm 2^\circ\text{C}$, $55\% \pm 5\%$ RH, as also seen in [9]. During the curing, the surface substrate was covered with a plastic sheet, in order to slow down the carbonation process. 75% and SSD conditions were achieved by keeping wet the substrate surface for 24 hours, then the surface was manually dried with towels to attain the SSD conditions. Several hours later, at laboratory conditions, a moisture level of 75% was reached. The surface moisture level was measured by a superficially encased relative humidity probe, in particular a *ROTRONIC* type *HC2 – AW*. The digital probe signal was processed by a *HygroPalm23* multifunction hand-held indicator, which provided the temperature, RH and time of measure, see Figures 6.3(a) and (d).

6.1.3 Application of the overlay

The moisture level of the substrate surface prior to the application of the overlay is not the only parameter that affects the development of the bond strength. The ability to densify the interfacial zone, which leads to better bond performances, also depends on both the water movement contained in the overlay (w/c ratio) and the capacity of the fresh overlay material to flow and fill cavities of the roughened substrate surface. In order to better understand the influence of w/c ratio and fluidity of the overlay on the bond strength, two commercial overlay materials, with different properties, were used for this purpose. These overlay materials were presented in the first part of this PhD work thesis. In fact, they correspond to M1-HF45-0.3 (named hereafter M1) and M2-SF75-3.8 (named hereafter M2), respectively. Their mix design and material properties are reported in Tables 2.2 and 6.1, respectively. Once the desired moisture level of the substrate surface was achieved, the overlay was manually poured and compacted, see Figures 6.3(b) and (e). The overlay was covered by plastic sheets and cured for additional 27 days. Then, the specimens were demolded and divided in two or more items, in order to extend the number of specimens to be tested, see Figures 6.3(c) and (f).

6.1.4 Testing method

An overview about the available bond test methods is reported in Section 5.2. In order to properly characterize the bond capacity of repaired concrete structures, the interface of specimen was tested to adequate load configurations, which included the modified pull-off bond test (POBT*), the direct tensile bond test (DTBT), the direct shear bond test (DSBT) and the tensile/shear bond test

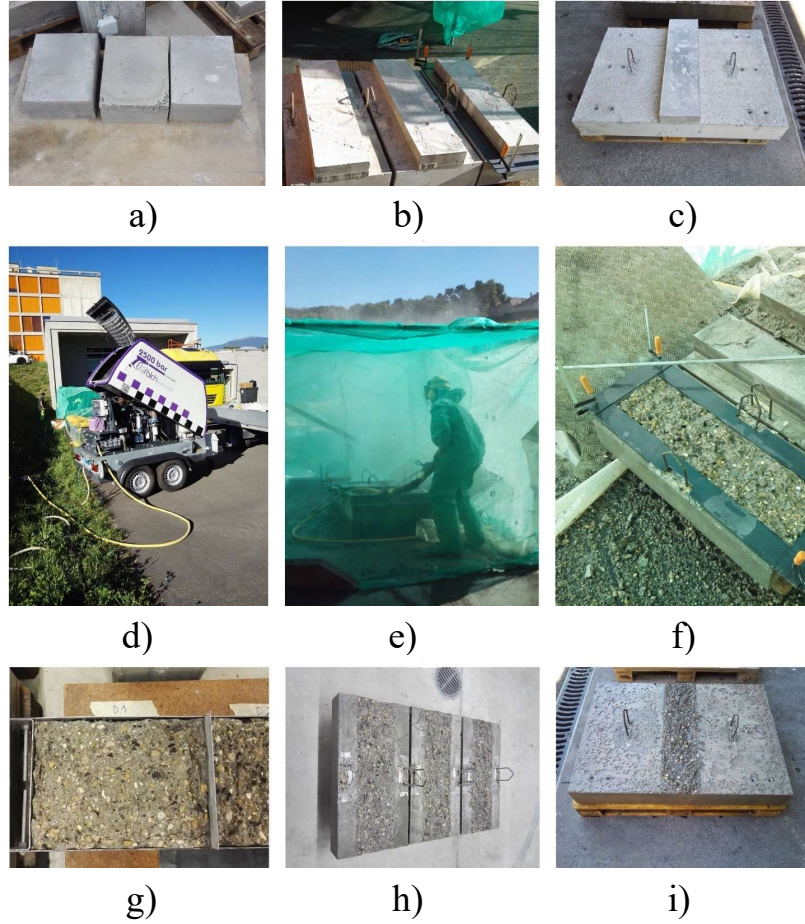


Figure 6.2: a, b, c) After 90 days of exposition to weather conditions the concrete slabs were prepared to the hydro-jetting, d) Hammelmann 2500 bars machine used for hydro-jetting the surface of the concrete slabs, e, f) hydro-jetting site, g, h, i) roughness profile of the interface, provided by the hydro-jetting, prior to the application of the overlay

(TSBT). The core POBT is considered the most common method for evaluating the bond tensile strength in the field [146]. In this study a modified testing equipment was developed in order to measure not only the failure load, which is the only information provided by standard POBT, but also the initial stiffness and the opening crack corresponding to the peak of tensile bond strength. Such data were used to calibrate some key cohesive parameters of mode I, see Section 7. For this purpose, the specimen was not drilled from the composite slab, which is the typical condition for carrying out standard POBT, but thanks to a particular set-up of the mold's shape designed by Author, the overlay zone of the specimen was totally exposed once the latter was demolded, see Figures 6.3(b) and (c). This enabled to place a couple of LVDT on both sides of the specimen for measuring the opening crack at the interface, see Figure 6.4(a). LVDT measures

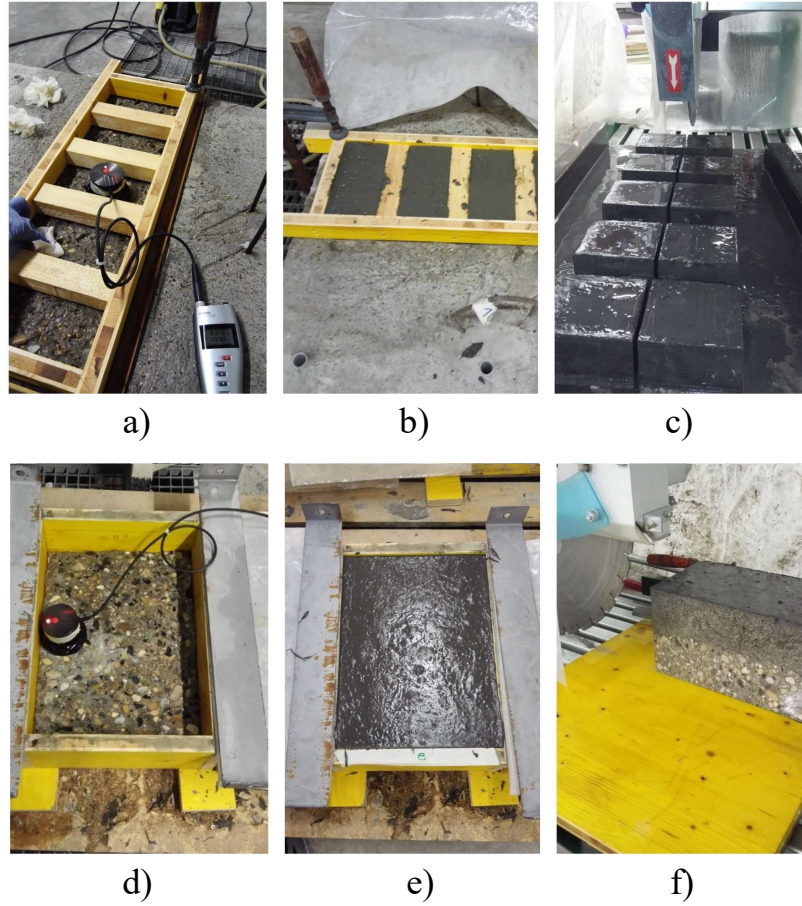


Figure 6.3: a, d) Substrate surface moisture measurements prior to the application of the overlay, b, e) the overlay was manually poured and compacted once placed on the substrate, c, f) the specimens were cut and prepared to be tested

a range of displacement detected within the system “substrate-interface-overlay” subjected to pull-off loads, which does not encompasses just the opening crack arising along the interface, but also the deformation of bonded materials. This drawback was therefore minimized, since the majority of the deformability of the system was at the interface, as confirmed by measuring both the initial stiffness and the peak of strength observed in Fig. 6.9, which are much lower than those of bonded materials, see Table 6.1.

The POBT* equipment included a cylinder *RACH-202* which transferred the tensile load to the interface surface. The pressure in the cylinder was provided by a lightweight hand pump type *P-141*. Since the pull-off strength might decrease as loading rate slows down [18], in the POBT* the loading rate was set to 0.003 ± 0.002 MPa/s, which is much lower than standard suggestions [175]. The load cell *C6A*, setup on the cylinder, detected the loading rate, with an accuracy of 2%. A pin system fixed to the specimen’s head (overlay zone) was properly designed to prevent the effect of load eccentricity at the interface. The POBT* is illustrated in Figure 6.4(a). A series of specimens was also investigated via

DTBT, see Figure 6.4(b), in order to confirm the reliability of the POBT*. A displacement controlled computerized universal testing machine (Walter Bai type LVF-200 kN) was used for DTBT. Details of testing machine were presented in Section 2.2. In order to observe the post-peak behavior, which could not be detected in POBT*, a strain rate was used to pilot DTBT. With the aim of having comparative rates between POBT* and DTBT, an opening crack rate was set to 0.07 ± 0.01 mm/min.

If for bond tensile tests is possible to transfer a pure stress on the interface (mode I), things are different when a shear bond stress (mode II) is demanded. The disadvantage of most common shear methods is the occurrence of a bending moment due to the shear force eccentricity applied at the interface, as already discussed in Section 5.2. In this work, such a problem was faced by adopting a DSBT setup, as also seen in [12]. Specimen dimensions were the same of those tested in POBT* series, in order to avoid the influence of size effects between different series. The same testing machine presented above was used for DSBT as well. A slip rate of 0.07 ± 0.01 mm/min was set. Four loading/supporting rollers transmitted the direct shear load to the interface. Since the roughness profile provided by hydro-jetting was irregular and slightly different from one specimen to another, see Tables 4, 5, 6, 7, 8, 9, 10, 11 and Figures 5 and 6, a proper position of loading/supporting rollers was fundamental for reducing the risk of load eccentricity. For this reason the loading/supporting rollers were not fixed, but they could be moved in order to minimize such a risk, for each test, see Figure 6.4(c).

In the practice, the unbalanced shear stresses near to discontinuities of the bonded overlay (e. g. slab edge, cracks, joint) lead to the development of tensile stress, perpendicular to the interface. As consequence, a peeling moment is generated and increases with the edge overlay thickness [56]. In order to reproduce this stress condition (mixed mode), which can generate a cracking/debonding failure on the interface, a LSBT was designed by Author and illustrated in Figure 6.4(d). Since the cracking/debonding failure usually begins near to discontinuities of the bonded overlay, the specimen length was defined to represent the edge of this critical region. The specimen dimensions were defined also by taking into account the limited load carrying capacity of the testing system, which is man-portable and it can be easily set-up both in situ and in the lab. A couple of gauge were placed on both sides of the specimen for measuring the average slip at the interface. The testing equipment included a cylinder *RC-102* which transferred the shear load to the interface surface. The pressure in the cylinder was provided by the same lightweight hand pump used in POBT*. The loading rate was set to 0.003 ± 0.002 MPa/s, in order to have comparative results with POBT*. The load cell *CFT*, fixed to the cylinder end, detected the loading rate, with an accuracy of 0.8%.

As already mentioned, the interface of composite cement-based composite structures is subjected to mixed-mode stresses, so the loading bond capacity should be assessed by taking into account the coupling factor between mode I and mode II. For this purpose a TSBT was designed by Author, by coupling the lateral shear with the modified pull-off devices presented above. In the TSBT, the interface was subjected to a fixed tensile load, while shear load increased until the bond failure, see Figure 6.4(e). By varying the loading ratio was possible to encompass all possible tension/shear stress states encountered in the practice. The stress rate was set to 0.003 ± 0.002 MPa/s, in order to have comparative

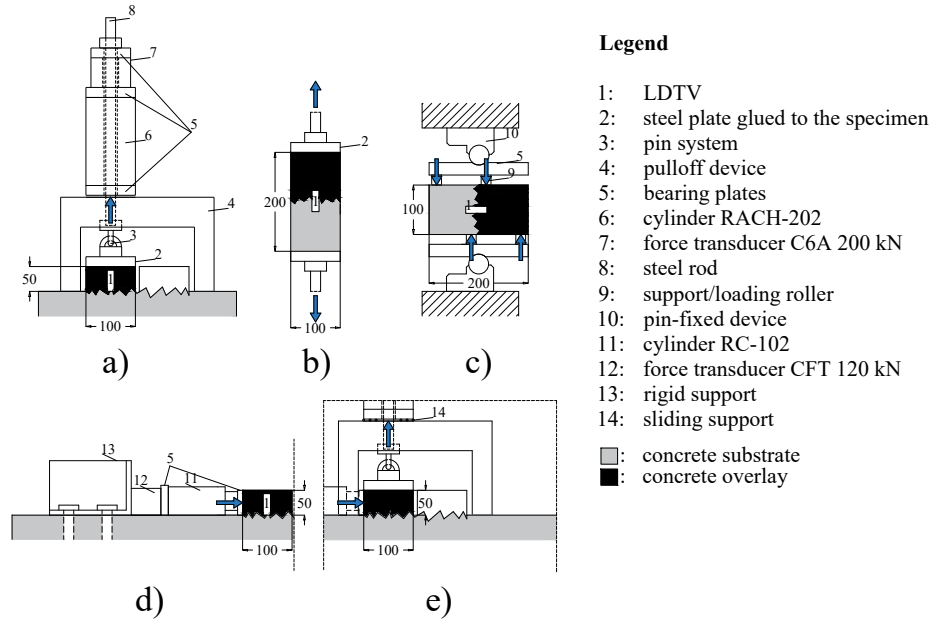


Figure 6.4: Illustration of testing setup for :a) POBT*, b) DTBT, c) DSBT, d) LSBT and e) TSBT

results with those provided from POBT* and LSBT. The experimental data were recorded on a *CATMAN* data acquisition software, with a frequency of 10 Hz.

Besides the bonding properties, further properties that also affect the contact problems include the elastic modulus, the shear modulus, the Poisson's ratio, the geometry of both bonded materials [80] and also the fresh properties of the overlay poured on the substrate surface. Such properties were measured on the basis of the experimental data obtained from single-material tests, see Table 6.1. It is worth mentioning that these tests are independent of any bonding properties. The machine Walter Bai type LVF-200 kN presented in Section 2.2.2 was setup for carrying out the elastic modulus test on M1 and M2 mixtures, according to the [176]. As discussed in Section 6.1.3 the ability to increase the density of the the interfacial zone, which leads to better bond performances, also depends on both the water movement contained in the overlay (w/c ratio) and the capacity of the fresh overlay material to flow and fill cavities of the roughened substrate surface. In order to better understand the influence of both w/c ratio and the fluidity of the overlays M1 and M2 on the bond strength, both a slump test and a slump test T500 were carried out according to [177] and [179] respectively.

6.2 Experimental results

The specimens tested in this investigation were labeled according to the type of test carried out, the type of overlay and the moisture level of the substrate surface prior to be overlaid. In particular:

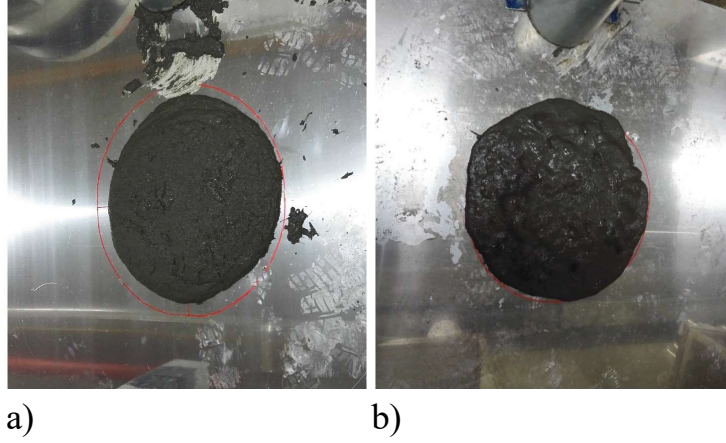


Figure 6.5: Slump test T500 carried out for series : a) M1 (HPCC) and b) M2 (UHPC)

- POBT*, DTBT, DSBT, LSBT, TSBT: stand for the modified pull-off, direct tensile, direct shear, lateral shear, tensile/shear bond test;
- Dry, 75, SSD: stand for dry substrate surface, moisture surface level of 75% and saturated-surface-dry;
- M1, M2: stand for overlay M1 (M1-HF45-0.3) and overlay M2 (M2-SF75-3.8).

By varying the parameters above, thirty series were investigated, each of these series was composed by 8 – 10 specimens. Due to the large amount of data recorded, just some representative results will be illustrated in the following. All results can be found in Appendix A5.

6.2.1 Fresh properties of the overlay

In order to assess the fresh properties of both M1 and M2 the slump test and slump test T500 were carried out, see Figure 6.5. Results of the slump test were almost the same for both M1 and M2, see Table 6.1. Instead the slump test T500 recorded a time of 23 and 50 seconds for M1 and M2, respectively, confirming the higher viscosity of the UHPC (M2), compared to that of the HPCC (M1).

6.2.2 Elastic modulus of the overlay

Figure 6.6 shows the elastic modulus test carried out on M1 and M2 prismatic specimens. Recorded data can be observed in Figure 6.7. Results are reported in Table 6.1.

6.2.3 Roughness profile of the interface

After hydro-jetting, the surface profile of the substrate prior to the application of the overlay was analyzed via photogrammetric survey, as discussed in Section 6.1.2. Figure 6.8 shows an example of roughness profile investigated. The colored area represents the interfacial zone of the substrate surface prior to the application of the overlay. For each series, statistical analysis of the roughness profile were computed. The interface profile magnitude showed agreement with [90]. Table 6.2 lists an example of the main results provided for a given series investigated. Instead all results are found in the Appendix A4, including the scanned surface of all series, see Figures 5 and 6.

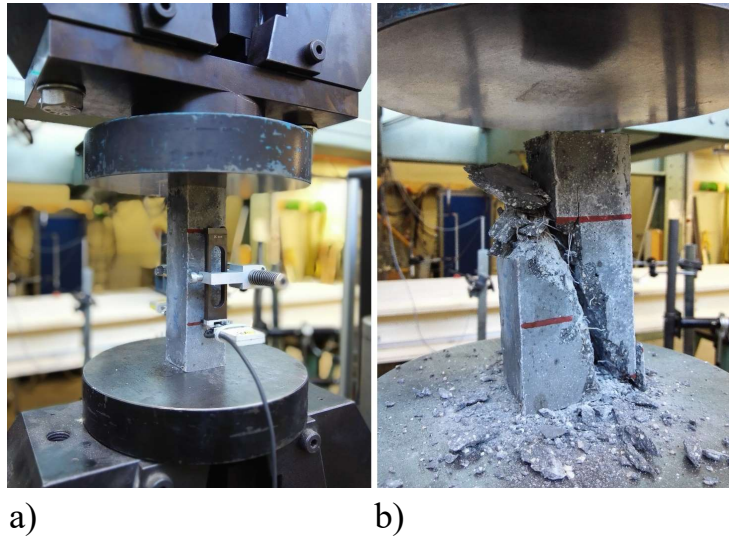


Figure 6.6: a) Elastic modulus test during the cycle of loading/unloading, b) detection of the compressive strength at the end of the elastic modulus test

Interfacial zone	Minimum (mm)	Maximum (mm)	Average (mm)	Stand. Dev. (mm)
TSBT-75-_1	139.1	157.1	150.2	3.13
TSBT-75-_2	135.7	154.0	143.6	3.16
TSBT-75-_3	136.3	156.5	145.9	3.82
TSBT-75-_4	136.4	155.0	147.2	3.90
TSBT-75-_5	137.3	156.5	146.8	3.22
TSBT-75-_6	136.9	153.6	145.7	3.19
TSBT-75-_7	134.5	151.9	143.1	3.05
TSBT-75-_8	135.2	156.0	144.3	3.17
TSBT-75-_9	139.2	154.0	146.3	2.81
TSBT-75-_10	141.2	155.6	147.4	2.89

Table 6.2: Logged roughness profile for a given series (TSBT-75)

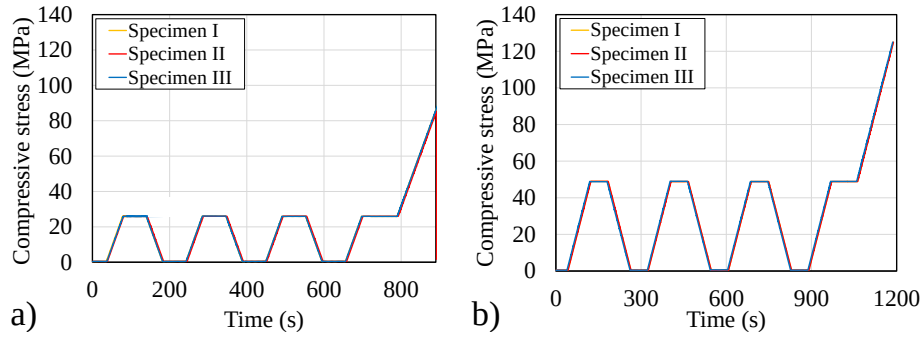


Figure 6.7: Elastic modulus test during the cycle of loading/unloading for series : a) M1 and b) M2

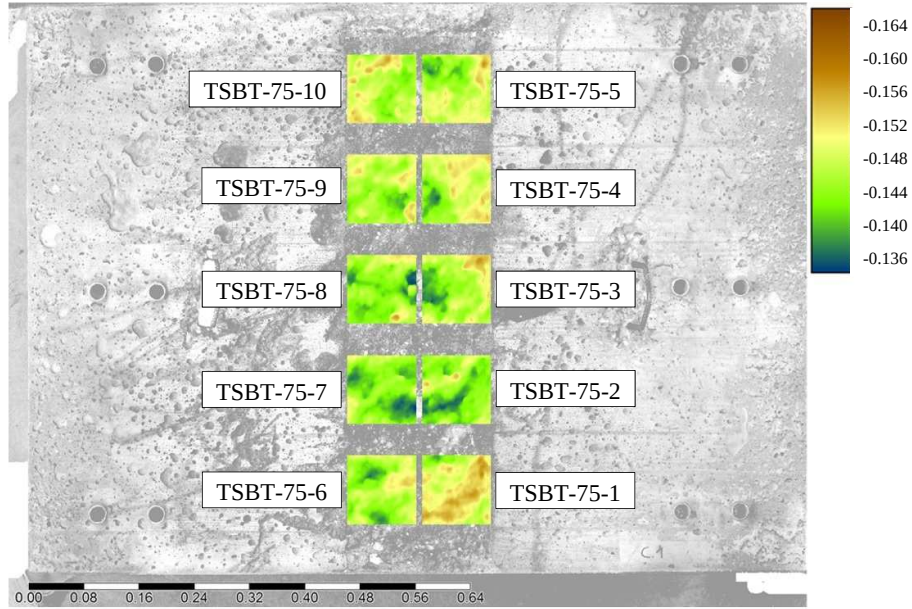


Figure 6.8: Roughness profile scanned in a given series (TSBT-75). Scale in meters

6.2.4 Modified pull-off (POBT*) and direct tensile bond tests (DTBT)

After the photo-scanning, the overlay was poured on the substrate surface, in regard to the specific moisture conditioning discussed in Section 6.1.2. The specimens were cured for 28 days and then prepared for bond testing. The bond test results of few specimens that were not so representative to the average value were discarded.

From data recorded by POBT*, it can be noted that higher saturation levels led to higher tensile bond strength values. Instead, the stiffness of the bond seemed to be not influenced by the moisture level, see Figure 6.9. By a visual

examination of the cracking/debonding surface, POBT*-dry-M2 series showed a clear failure in the overlay near the interfacial zone. Such a condition was less pronounced for POBT*-dry-M1 series, where failure often occurred in the substrate near to the interfacial zone, see Figure 6.10.

The reason could be that the more brittle nature of overlay M2, as compared to overlay M1, induced higher stress concentration along the interface, thus causing a failure in the overlay. However, this aspect explains the cracking/debonding pattern observed in POBT*-dry-M2 series only partially, since in all other series with the same overlay no failure within the overlay was observed. Further analyses seem to confirm that the cracking/debonding pattern was rather related to both the moisture level and the fresh properties of the overlay. From a theoretical point of view, under dry conditions, the concrete substrate surface tends to adsorb water from the fluid overlay. A densification of the micro-structure in the interfacial zone is possible and overlay fills the asperity of the roughened interface [9]. After a proper curing such an interfacial transition zone creates a bond between the substrate and the overlay. If too much water is removed from the overlay at the interface, the risk of insufficient hydration rises up, which leads to a weak bond. This effect is magnified in overlays with very low w/c ratio, as observed in POBT*-dry-M2 series (w/c ratio = 0.17) where the failure took place in the overlay. By increasing the moisture levels the concrete substrate surface adsorbs less water from the fluid overlay, so the hydration in the interfacial zone rises up and creates a stronger bond between substrate and overlay, to the point that in POBT*-75-M1, POBT*-75-M2, POBT*-SSD-M1 and POBT*-SSD-M2 series bond failure was observed in the substrate layer, near to the interface. In terms of strength a clear increase in magnitude was observed in POBT*-75-M1, POBT*-75-M2, POBT*-SSD-M1 and POBT*-SSD-M2 series. This increase was more pronounced in substrates overlaid by UHPFRC (overlay M2), due to its higher sensitivity to the moisture level, as compared to HPFRCC (overlay M1, w/c ratio = 0.28).

The failure pattern detected on all POBT* series is illustrated in Figures 7 - 12 reported in the Appendix A6. In order to confirm the fact that load eccentricity was properly prevented during POBT*, some series of specimens were prepared and tested under direct tensile conditions, see Figure 6.4(b). In few direct tensile tests, failure occurred on the interface between substrate and steel plate glued to the specimen (substrate side). These results were, of course, not useful to characterize the tensile bond capacity, so they were discarded. However, for specimens successfully tested, results in Figure 6.11 confirm a good agreement between POBT*-M2 series with DTBT-M2 series and POBT*-dry-M1 series with DTBT-dry-M1 series, proving the reliability of POBT* designed by Author. The convergence between POBT*-75-M1 with POBT*-SSD-M1 series and DTBT-75-M1 with DTBT-SSD-M1 series was less evident, in particular the direct tensile strengths were slightly lower, compared to pull-off strengths. This could be due to the geometry of specimens in POBT*-series. In POBT*-series, the load can be transferred over a slightly larger interface area, compared to that in DTBT series, which may possibly increased the bond strength, see Figures 6.15(a) and (b). In addition, in POBT*-series, the specimens were subjected to one operation of cutting to be prepared, while specimens in DTBT were subjected to four operation of cutting (one for each side), see Figures 6.3(c) and (f). Such an operation could have stressed the interface prior to testing, especially the interface of DTBT specimens.

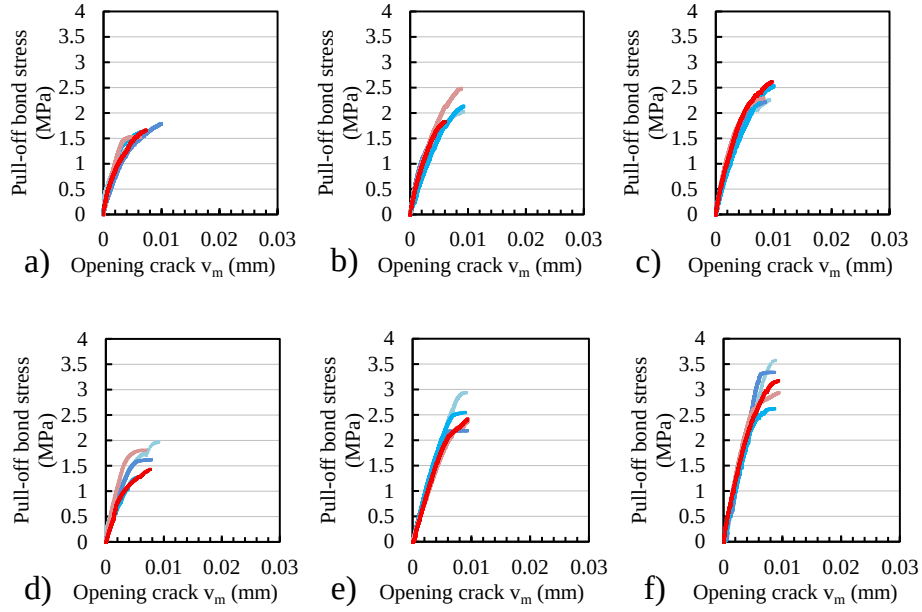


Figure 6.9: POBT* results for series: a) dry-M1, b) 75-M1, c) SSD-M1, d) dry-M2, e) 75-M2, f) SSD-M2

6.2.5 Direct shear bond tests (DSBT)

For DSBT-M1 series, SSD conditions achieved the highest bond shear strength, even though similar strength values were also observed in both DSBT-dry-M1 and DSBT-75-M1, see Figure 6.11. This observation confirms that explained in 6.2.4. Since the w/c ratio of overlay A is not critically low, see Table 6.1, the hydration magnitude of the interfacial zone was high enough to lead similar bond strengths for any change of moisture levels. The same conclusions partly confirmed in [9], even though in such a work some variables were different, e.g. shear test method and surface treatment. Instead, a different response was observed in DSBT-M2 series, where the development of shear strength increased with increasing the moisture level, as also observed in POBT*-M2 series. Figure 6.15 shows some examples of bond failure occurred in series investigated.

6.2.6 Tension/Shear bond tests (TSBT)

In order to quantify the coupling factor between mode I and mode II, all possible stress states encountered in the practice were detected via TSBT, see Section 6.1.4. Interface was subjected to a fixed tensile load value, while shear load increased until the failure. Five combinations of different tensile stress ratios were investigated: no tensile load (only shear load), 25% of tensile bond strength, 50% of tensile bond strength, 75% of tensile bond strength, solely tensile load (without shear load). Figure 6.12 shows the bond failure envelope recorded for substrates repaired with overlay M1 and M2, under dry, 75% and SSD conditions. Each point in the graph represents the result of one specimen. Both DSBT-75-M1 and LSBT-75-M1 series showed lower shear bond values, in comparison with other series. This phenomenon was further pronounced when the interface was

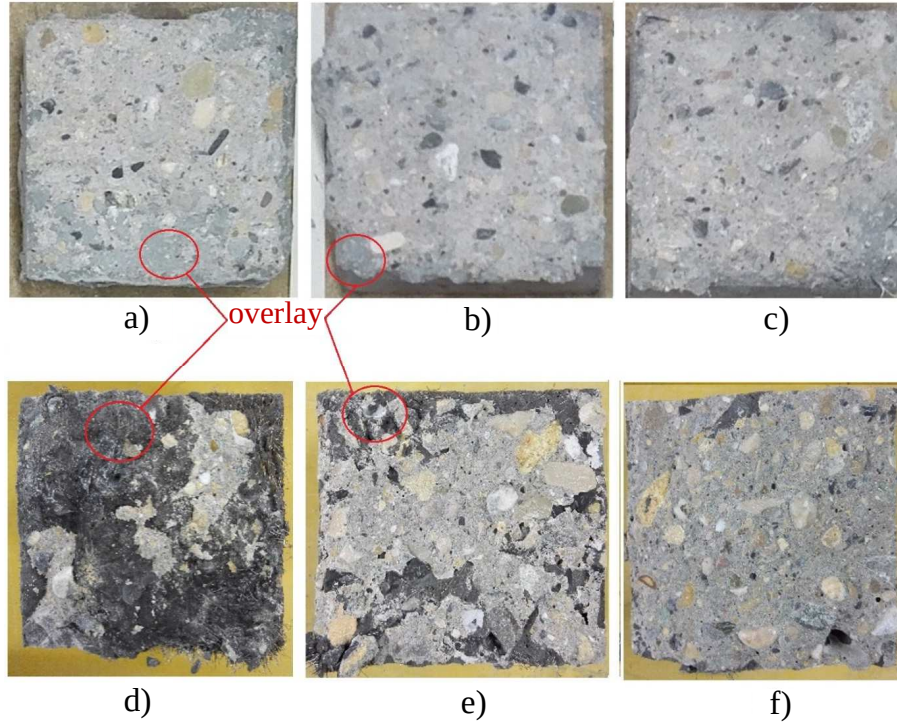


Figure 6.10: Surface failure observed after POBT*: a) dry-M1 series, b) 75-M1 series, c) SSD-M1 series, d) dry-M2 series, e) 75-M2 series, f) SSD-M2 series

also subjected to an increase of tensile load, reducing the loading bond capacity. In fact, results of TS-75-A series in Figure 6.12, confirmed a premature bond failure, for low increases of tensile loading ratios, as compared to TSBT-dry-M1 and TSBT-SSD-M1 series. All other series in Figure 6.12 denotes higher strength values, which permitted to obtain a shape almost linear of the bond failure envelope. In particular, it was observed a linear decrease of bond in shear, or tension, as stress in tension, or shear, increases. It should be taken into account that high scatter of bond strength is expected, because of the high roughness profile of the interface provided by hydro-jetting [10], see Figure 6.8. A linear regression function was adopted to correlate experimental data, including any change of tensile/shear loading ratio here investigated. Even though more complex shapes of the regression functions were analyzed, the coefficient of determination provided by the linear regression was considered high enough to calibrate the cohesive parameters presented in Section 7. In particular, the slope of the linear regression, pointed out in Figure 6.12, depicts the average coupling factor magnitude between mode I and mode II. Not less important is also the influence of the moisture condition, which clearly affects the bond capacity. Experimental data reported in Figure 6.12 denotes an extension of the bond failure region, as the moisture level increases, confirming the trend observed in both POBT* and DTBT, see Sections 6.2.4 and 6.2.5.

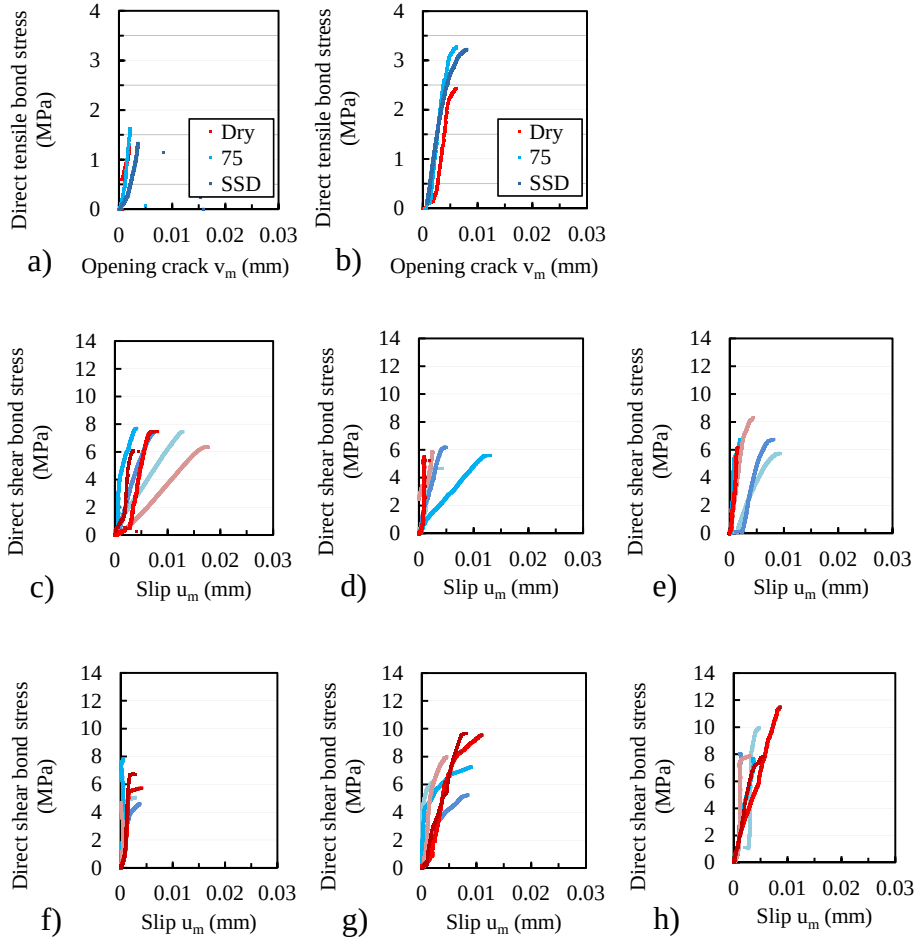


Figure 6.11: DTBT results for series: a) Dry-M1, 75-M1, SSD-M1, b) Dry-M2, 75-M2, SSD-M2. DSBT results for series: c) Dry-M1, d) 75-M1, e) SSD-M1, f) Dry-M2, g) 75-M2, h) SSD-M2

6.2.7 Lateral shear bond tests (LSBT)

In the practice, the unbalanced shear stresses near to discontinuities of the bonded overlay leads to the development of tensile stress, perpendicular to the interface. As consequence, a peeling moment is generated and increases with the edge overlay thickness. In order to simulate such a realistic condition of cracking/debonding pattern, LSBT were carried out. Results are showed in Figure 6.14. Bond strength was clearly affected by both the moisture level and the fresh properties of the overlay, whose trend was similar to that seen in POBT* series. In fact, LSBT-dry-M2 series showed a slight failure in the overlay near to the interfacial zone. Such a condition was less pronounced for LSBT-dry-M1 series, where the failure often occurred in the substrate near to the interfacial zone. By increasing the moisture levels the failure moved away from the overlay and reached the substrate near the interface, see Figure 6.13. Such an effect was more evident for LSBT-M2 series. The failure pattern detected on all LSBT

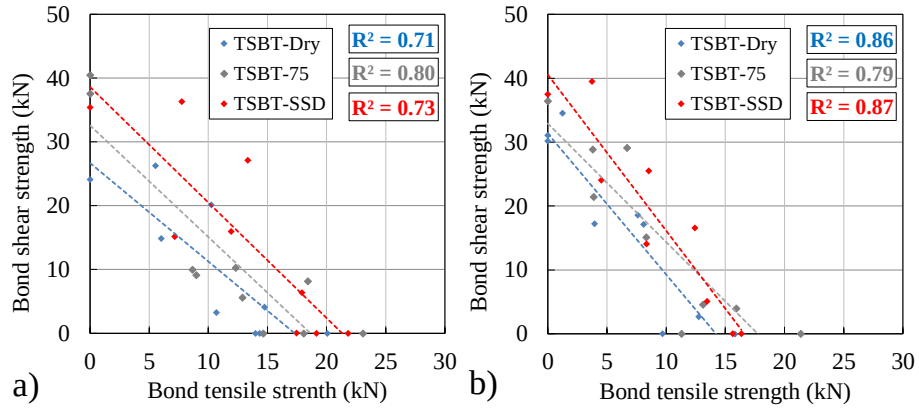


Figure 6.12: TSBT results for: a) NSC slab overlaid by M1, b) NSC slab overlaid by M2

series is illustrated in Figures 13 - 18 reported in the Appendix A6.

Max bond stress and corresponding slip/crack opening reported in Figures 6.9, 6.11 and 6.14 are listed in Table 6.3.

6.2.8 Influence of the roughness profile on the bond strength

A correlation between the roughness profile provided by the hydro-jetting and the bond strength measured from the tests presented above is illustrated in Figure 6.16. The bond strength was compared with the coefficient of variation of roughness profile (COV), expressed as the ratio between standard deviation and average value. It was observed that the bond strength tends to slightly decrease with the roughness profile COV values higher than 6%, for any moisture level; such values were exceeded in few cases. It can be stated that the opposite trend of increase of strength observed in UT-A and UT-B series is casual, since most data points confirmed that the bond strength is rather affected by both the moisture level and w/c ratio of the overlay, than the roughness COV.

Details about the experimental data recorded from bond tests presented above are listed in Tables 12, 13, 14, 15, 16 and 17 reported in the Appendix.

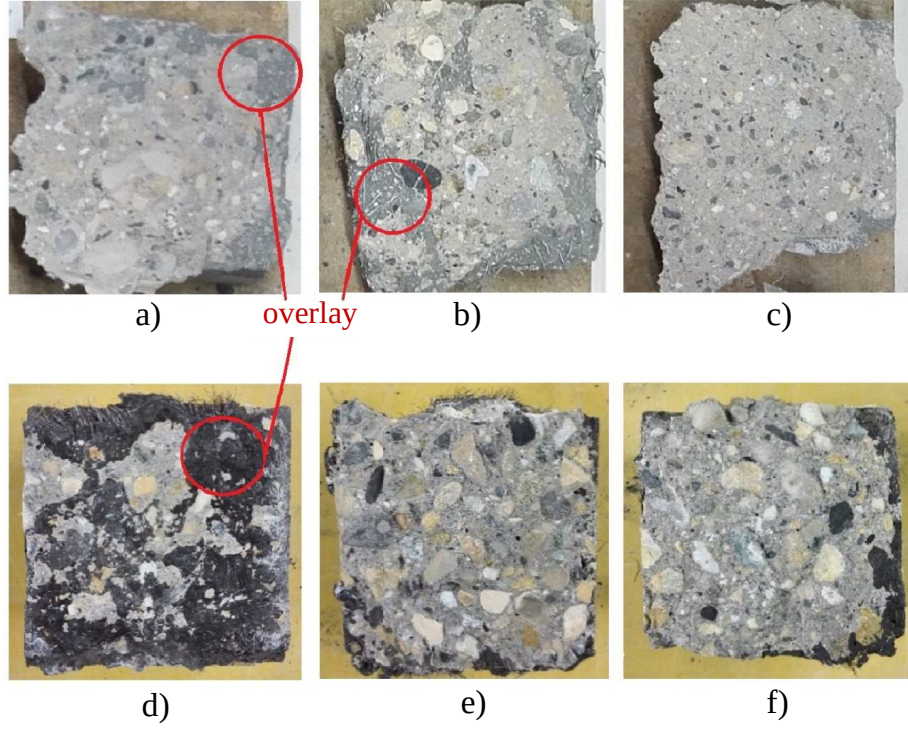


Figure 6.13: Surface failure observed after LSBT for series : a) dry-M1, b) 75-M1, c) SSD-M1, d) dry-M2, e) 75-M2, f) SSD-M2

Series	Overlay M1		Overlay M2	
	q_{max} / τ_{max} (MPa)	$v(q_{max}) / u(\tau_{max})$ (mm)	q_{max} / τ_{max} (MPa)	$v(q_{max}) / u(\tau_{max})$ (mm)
POBT*-dry	1.6 ± 0.15	0.007 ± 0.002	1.62 ± 0.28	0.007 ± 0.001
POBT*-75	2.06 ± 0.27	0.008 ± 0.002	2.47 ± 0.25	0.008 ± 0.001
POBT*-SSD	2.38 ± 0.18	0.009 ± 0.001	3.1 ± 0.39	0.008 ± 0.001
DTBT-dry	1.3	0.002	3.24	0.004
DTBT-75	1.64	0.002	3.4	0.005
DTBT-SSD	1.49	0.002	3.45	0.006
DSBT-dry	7.09 ± 0.68	0.009 ± 0.001	5.77 ± 1.29	0.002 ± 0.001
DSBT-75	5.57 ± 0.55	0.005 ± 0.003	7.64 ± 1.79	0.007 ± 0.003
DSBT-SSD	6.77 ± 0.87	0.005 ± 0.003	8.81 ± 1.56	0.004 ± 0.003
LSBT-dry	2.00 ± 0.33	0.017 ± 0.004	2.42 ± 0.33	0.019 ± 0.005
LSBT-75	1.76 ± 0.26	0.02 ± 0.004	2.77 ± 0.45	0.028 ± 0.004
LSBT-SSD	1.94 ± 0.28	0.02 ± 0.004	3.73 ± 2.37	0.02 ± 0.005

Table 6.3: Bond max stress and corresponding slip/crack opening recorded in POBT*, DTBT, DSBT and LSBT series

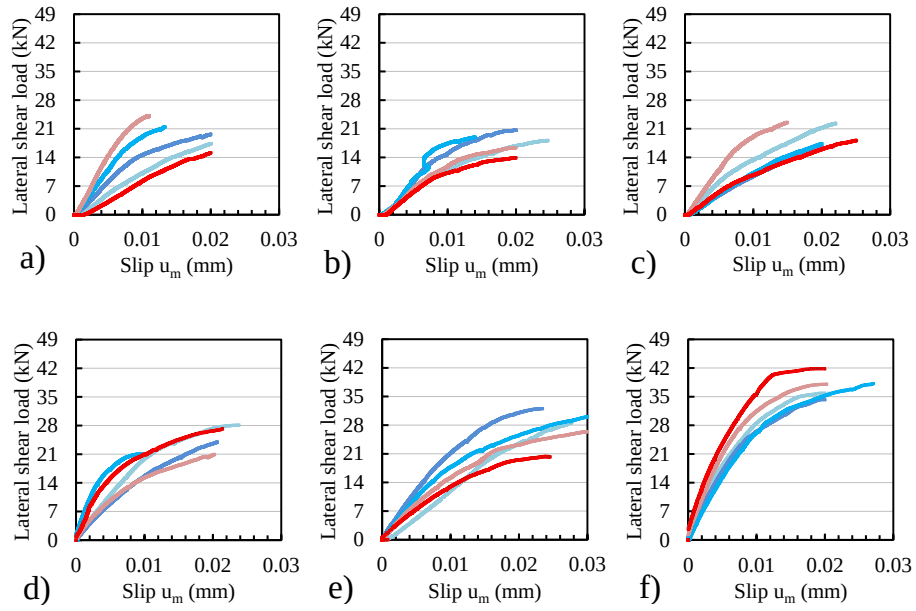


Figure 6.14: LSBT results for series: Dry-M1 (a), 75-M1 (b), SSD-M1 (c), Dry-M2 (d), 75-M2 (e), SSD-M2 (f)

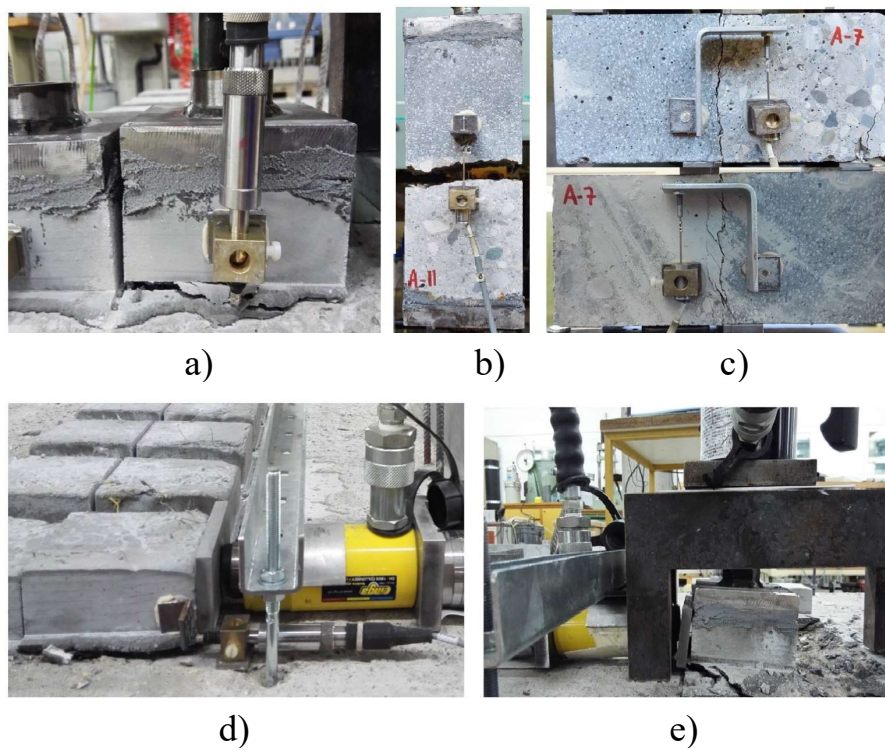


Figure 6.15: Overview of bond failure pattern observed in: a) POBT*, b) DTBT, c) DSBT, d) LSBT, e) TSBT

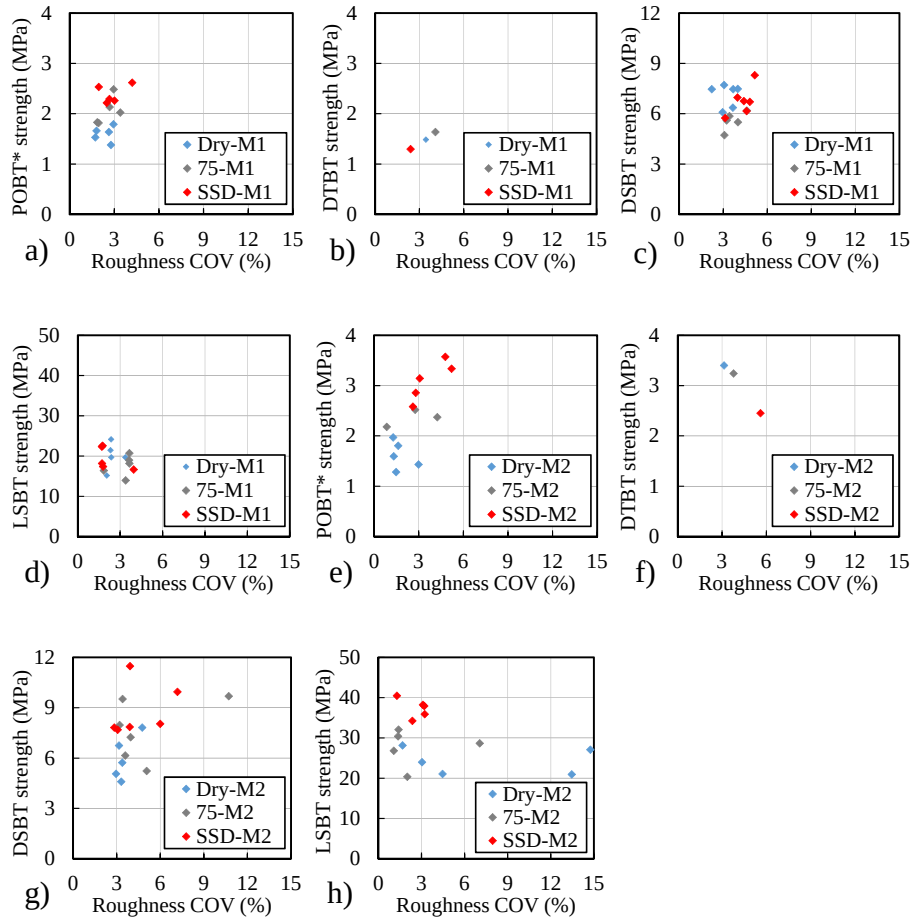


Figure 6.16: Correlation between bond strengths and roughness COV for series: a) POBT*-M1, b) DTBT-M1, c) DSBT-M1, d) LSBT-M1, e) POBT*-M2, f) DTBT-M2, g) DSBT-M2, h) LSBT-M2

Chapter 7

Theoretical model

The cracking/debonding pattern encountered in the practice near to discontinuities of the overlay, was simulated by LSBT on concrete slabs repaired by different overlays, see Section 6.2.7. In the experimental test, a concrete slab 200 mm thick was reinforced by 50 mm of HPFRCC/UHPFRC. The edge side of the overlay was subjected to incremental lateral shear load, until the cracking/debonding failure occurred along the interfacial zone. Experimental results confirmed that the majority of the deformability of the system “overlay-interface-substrate” is at the interface, so the material non-linearity of bonded layers was neglected to develop the cohesive model. In order to provide a generalized model, some bonded material properties should, however, be taken into account, in particular the elastic modulus, the shear modulus, the Poisson’s ratio and the geometry of both bonded materials [43, 44, 45, 80, 156]. Consequently, the deformability of the overlay can be analytically represented with a Timoshenko beam of finite length, which depicts the overlay, bonded to a substrate supposed to be a member infinitely rigid (MIR), according to the approach investigated in [80]. The assumption of MIR for modeling the substrate is explained in Section 8.1.3. The beam of length l , with rectangular cross section of height h and width b , is subjected to horizontal displacement u_l imposed at the edge of the beam. u_l represents the slip measured at the interface in LSBT series, see Figures 6.4(d) and 6.15(d). Axial forces are positive if rightward directed, shear forces are positive if upward directed, bending moments are positive if counterclockwise, as showed in Figure 7.1. The equilibrium equations of the beam are:

$$N^I + \tau = 0, \quad T^I + q = 0, \quad M^I - T + \tau \frac{h}{2} = 0. \quad (7.1)$$

where N , T and M are the internal forces applied at the beam cross section. τ and q represent the shear and peeling traction, respectively, which arise at the interface between the beam and the rigid support. Prime denotes differentiation with respect to coordinate x . Kinematic assumptions for Timoshenko beam together with constitutive relations provide:

$$\frac{M}{EI} = \varphi^I, \quad u^I = \frac{N}{EA} + \varphi^I \frac{h}{2}, \quad v^I = -\varphi + \frac{\chi T}{GA}. \quad (7.2)$$

where E denotes the Young modulus of the beam, A and I are the area and the moment of inertia of the beam cross section respectively, G represents

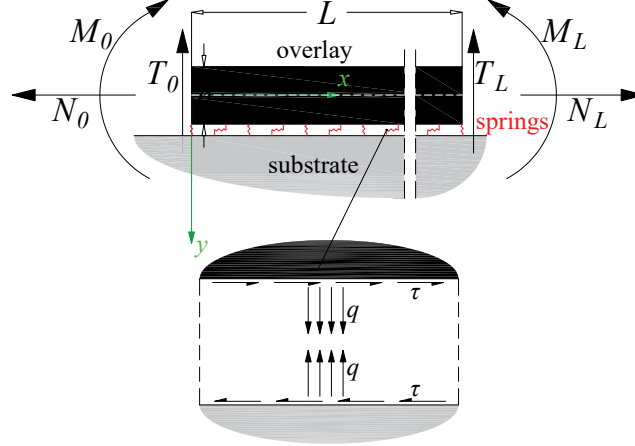


Figure 7.1: A Timoshenko beam (overlay) bonded to a MIR (substrate) via translational springs

the shear modulus of the beam, χ is the dimensionless shear factor, which is $\chi = 6(1 - \nu G/E)/5$, ν is the Poisson's ratio of the beam. $u(x)$ is the relative horizontal displacement between beam and rigid support at $y = h/2$ (slip at the interface), $v(x)$ represents the relative vertical displacement between beam and the rigid support along the y axis (crack at the interface), $\varphi(x)$ denotes the relative rotation of the beam cross section, positive if counterclockwise. The cross section of the beam is assumed to preserve its planarity after bending. The interface was modeled of a series of translational springs, which link substrate and overlay together. Vertical and horizontal springs govern mode I ($q(x)-v(x)$) and mode II ($\tau(x)-u(x)$), respectively. Once stress along the interface reaches the peak of strength, the residual strength ensured by the interlocking mechanism is defined by a softening curve. Such a curve goes to zero when cracking/debonding spreads along the whole interface. This phenomenon concerns both mode I and mode II. Experimental results recorded in P and DS series, see Section 6.2, permitted to calibrate both the initial stiffness and the peak of strength of both mode I and mode II. The missing information concerning the softening curve after peak were derived from few data found in Literature. In particular, Granju et al. (2004) [58] proposed a limiting value of opening v_{res} beyond which there is no more interlocking in mode I, similar to the limit slip u_{res} in mode II. As a first approximation it was assumed that $u_{res} = v_{res}$ equal to 0.05 mm, as observed in [58], one of the few available research. By coupling experimental data presented in Section 6.2 (POBT*, DSBT and TSBT series) with those derived from the Literature, it was possible to fit a numerical function. This function was finally adopted to describe the constitutive laws governing both mode I and mode II, see Equation 7.3.

$$q(v) = a_q(e^{b_q v(x)} - e^{c_q v(x)}), \quad \tau(u) = a_\tau(e^{b_\tau u(x)} - e^{c_\tau u(x)}) - c q(v). \quad (7.3)$$

Where a_q , b_q , c_q , a_τ , b_τ and c_τ are coefficients of the analytical relationships that best fit the experimental curves recorded in series POBT*, DTBT and

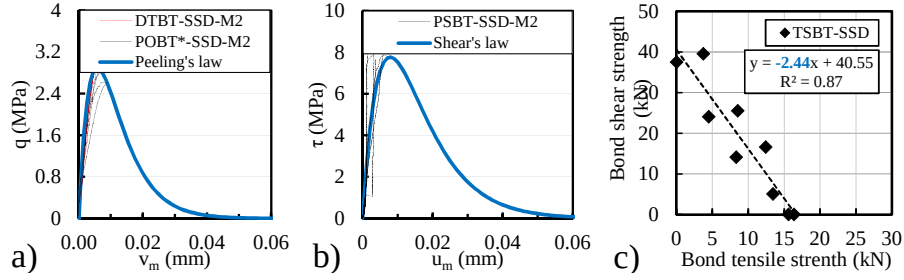


Figure 7.2: Example of parametrization of constitutive laws governing the modeled interface for: a) mode I , b) mode II, c) coupling factor magnitude, for dry-M2 series

DSBT; while c is the coupling factor between mode I and mode II, provided from TSBT series. The latter represents the slope of the linear regression showed in Figure 7.2(c). The parameters for all series studied are listed in Table 18 in the Appendix A7, including the fitted curves illustrated in Figures 19 and 20.

The equilibrium conditions of the beam are drawn in the following:

$$\begin{aligned} N(x) &= N_l + \int_x^l \tau(s)ds, & T(x) &= -T_l + \int_x^l q(s)ds, \\ M(x) &= M_l + T_l + \frac{h}{2} \int_x^l \tau(s)ds + \int_x^l q(s)(x-s)ds, & \text{for } |x| \leq l. \end{aligned} \quad (7.4)$$

In order to calculate the stress and kinematic fields at the interface subjected to mixed-mode, a fourth-order ordinary differential equation system was defined, substituting the Equation 7.2 in the Equation 7.1. The analytic form of such a system is drawn:

$$\begin{cases} EA[u^{II}(x) - \frac{h}{2}(-v^{III}(x) - \frac{xq^I(x)}{GA})] + \tau(x)b = 0 \\ EI(-v^{IV}(x) - \frac{xq^{II}(x)}{GA}) + \tau^I(x)b\frac{h}{2} + q(x)b = 0 \end{cases} \quad (7.5)$$

Six boundary conditions are defined in the following:

$$\begin{cases} u(x=l) = u_l \\ N(x=0) = 0 \\ T(x=0) = 0 \\ T(x=l) = 0 \\ M(x=0) = 0 \\ M(x=l) = 0. \end{cases} \quad (7.6)$$

Such a problem cannot be solved in a closed form. The code *MATLAB* was used to provide a numerical solution, by increasing the lateral shear load, in order to reproduce the experience recorded in LSBT series. Finally the theoretical results provided by the model were compared with the experimental data recorded by LSBT, see Figure 8.7. It is important to note that data results

provided by LSBT, which were used for confirming the model predictions, were independent from data results provided by POBT*, DSBT and TSBT, which were used for calibrating the model. Consequently this aspect should be taken into account when the comparison between experimental and theoretical curves is observed in Figure 8.7.

Chapter 8

FE model

This chapter presents a simple cohesive FE model able to simulate the cracking/debonding failure reproduced from the experimental investigations presented in Section 6.2.7. The interface was modeled with a series of vertical and horizontal truss elements which govern the loading bond capacity and the propagation of the cracking/debonding path along the interface, on the basis of the energy balance concept of the fracture mechanism [109]. The mechanical response of the truss elements was provided by a nonlinear cohesive law which couples mode I (tension-crack) with mode II (shear-slip) behaviors. The nonlinear cohesive law was defined on the basis of the experimental data recorded in LSBT.

8.1 Modeling of composite NSC-HPFRC/ UH-PFRC structures

8.1.1 The code Straus7

The commercial software package *Straus7* [7, 25, 51, 61, 115] was adopted for simulating a realistic case of cracking/debonding failure (Figure 5.2) between a NSC substrate and a layer of HPFRCC/UHPFRC. The code provides a series of tools and functions for obtaining the approximate solution of the problem investigated, according to steps listed below:

1. Defining the geometric and loading characteristics of the retrofitted cement-based composite structure;
2. Defining the material characteristics of the bonded materials (constitutive equations);
3. Dividing the physical system into elements and nodes (meshing of both single materials and the interface);
4. Forming the element matrices (stiffness, mass and damping elements) and vectors (nodal load and restraint elements);
5. Assembling the element matrices into global matrices and the element vectors into global vectors;

6. Solving the global equilibrium equations for the primary unknown variables and generating element results (i.e. cinematic and stress field at the interface);
7. Generating other useful result data such as transformation and extrapolation of results;
8. Investigating and interpreting the result data.

Step 1 to 3 are performed with the pre-processor; step 4 to 6 are performed with the solver; step 7 to 8 are performed with the post-processor. The approximated solution provided by *Straus7* considers the three physical laws of the continuum mechanic: equilibrium, compatibility and material law. The equilibrium equation system, which is formulated on the basis of the principle of virtual work, is a direct result of considering force equilibrium and displacement compatibility at the nodes. The global equations are solved by direct or indirect methods. The direct method uses the Crout's algorithm to decompose the global matrices into a triangular form. Instead, the indirect method uses the Pre-conditioned Conjugate Gradient algorithm [198]. The solutions yield one or more vectors of nodal displacements which are used to determine element stress and strain and nodal reactions in the structural solution.

8.1.2 Modeling of the FE interface

In the composite concrete members the link layer between the substrate and the overlay is usually composed by a porous media thin layer of 100 μm consisting of weak hydrates such as calcium hydroxide [11] and plays a key role in the bond strength [13, 41, 151]. In addition, the high roughness profile provided by the hydro-jetting may improve the bond performances thanks to the interlocking mechanism achieved by removing the deteriorated concrete [11]. This aspect is considered into the model presented in the following. In *Straus7*, when two surfaces are bonded a fictitious layer of zero thickness exists between them. The contact condition/effect to be physically modeled requires that the interface provides infinite stiffness so that the two surfaces cannot go penetrate each other. In addition, deformations around the interface area occur within the material on both sides of the interface [198]. For this purpose, the interface was modeled by so-called *master-slave* elements. This kind of link, which is a module given by the code, is not a real element like *beam* or *plate*. However, it provides the physical condition discussed above, and forces a node (slave) to follow another one (master) in the selected displacement directions. A smooth geometrical interface was modeled, since the effect of the real roughness profile affecting the bond strength was taken into account by properly calibrating the stiffness of the master-slave links on the basis of the experimental results. The stiffness of the the master-slave link, in regard with the relative displacements of its facing nodes (opening crack v and slip debonding u), was calibrated by pin-connecting the master-slave link to *truss* elements of 100 μm length, whose value corresponds to the average thick of the interface layer, as mentioned earlier. An illustration is reported in Figure 8.1. In the code, a truss element is a straight element pin-connected at its joints, which carries only tensile and compressive forces along its axial direction. In a truss element all loads can be applied to its joints and not between them. The mechanical behavior of the truss elements was

properly calibrated on the basis of the experimental data, in order to govern the peeling-crack (mode I) and the shear-slip (mode II) responses at the interface. For this purpose, a nonlinear cohesive law was adopted.

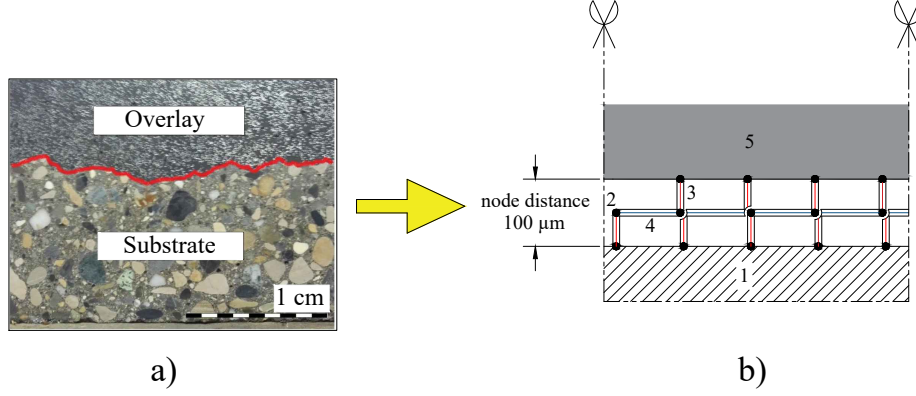


Figure 8.1: a) Interface profile observed by cutting the retrofitted cement-based member, b) modeling of the FE interface between (1) the substrate and (5) the overlay. The contact condition/effect between the two materials was modeled by using (2) master-slave elements. The peeling-opening (mode I) and the shear-slip (mode II) relationships at the interface were modeled by using (3) vertical and (4) horizontal truss elements, respectively

Such a law can be defined into the code by selecting the function *Stress vs Strain Table*. In this table it is possible to calibrate the mode I and mode II relationships that will govern the mechanical response of the vertical and horizontal truss elements, respectively. This technique permits to directly calibrate the nonlinear response of the interface face to peeling and shear stresses induced by the external load, until the peak of bond strength is reached. Then, the propagation of cracking/debonding begins and the frictional effect ensures a residual bond strength prior to reach the complete failure [58, 68, 116]. This important phenomenon cannot be directly accounted by nonlinear cohesive laws governing tied nodes. However, the residual strength provided by the frictional effect can be taken into account according to the energy balance concept of the fracture mechanism. While the cracking/debonding path increases, the peak of bond strength is kept constant in the truss elements, until the value of energy absorbed by the overall FE interface attains the value of the maximum energy absorbed by the real interface, prior to the complete failure. This approach requests, of course, several iterations for properly calibrating the cohesive law on the basis of the experimental test data recorded in LSBT, see Section 6.2.7. The construction of such a law into the code is illustrated in Figure 8.2. The branches b-c in Figure 8.2(a) and a-b/e-f in Figure 8.2(b) correspond to the plateau of strength discussed above, which is reached when the propagation of cracking/debonding begins.

In the first iteration, the mode I and mode II relationships were calibrated on the basis of the results recorded in DTBT and DSBT series. As mentioned above, the final expressions of the cohesive law were fitted through an iterative process, until the simulated load - slip/debonding curve agreed with those provided from LSBT, see Section 6.2.7. The FE parameters of the nonlinear cohesive law showed

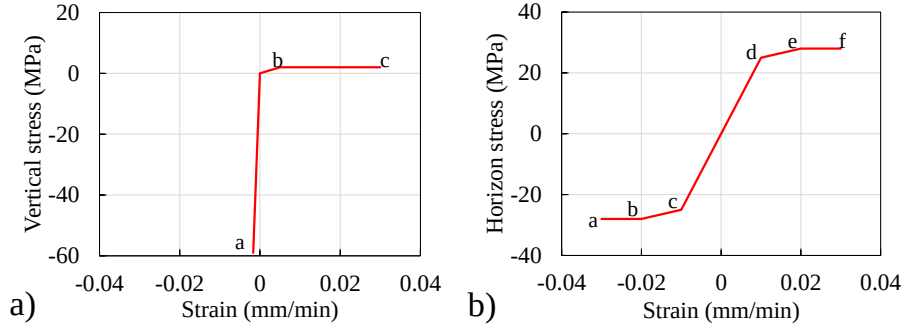


Figure 8.2: *Straus7* environment. Series LSBT-dry-M1: The function stress vs strain permitted to construct the nonlinear cohesive law governing the mechanical response of the FE interface subjected to (a) normal and (b) shear stresses. In each figure, the points in which the slope of the function stress vs strain changes are labeled according to letter a, b, c, etc.

in Figure 8.2 are reported in Table 8.1, for all series. In order to consider the fact that the flexural deformation of the overlay produces a compressive stress along certain regions of the interface, a compressive-strain law was implemented into the model, see the branch a-0 in Figure 8.2(a). Such a law considered as point of peak the compressive strength of the weaker material between the substrate and overlay, i.e. the NSC substrate. The corresponding strain was derived as the ratio between the compressive strength and the elastic modulus in compression, both experimentally measured, see Table 8.2. For both signs of slipping (left towards or right towards) the horizontal truss response does not change in terms of absolute values, so $\tau(a) = -\tau(f)$, $\tau(b) = -\tau(e)$ and $\tau(c) = -\tau(d)$.

Series	Vertical truss						Horizontal truss					
	v(a)	v(b)	v(c)	$\sigma(a)$	$\sigma(b)$	$\sigma(c)$	u(a)	u(b)	u(c)	$\tau(a)$	$\tau(b)$	$\tau(c)$
	mm/mm			MPa			mm/mm			MPa		
LSBT-dry-M1	-0.00169	0.005	0.03	-59	1.5	1.5	-0.03	-0.02	-0.01	-28	-28	-25
LSBT-75-M1	-0.00169	0.005	0.03	-59	1.2	1.2	-0.03	-0.02	-0.01	-30	30	-18
LSBT-SSD-M1	-0.00169	0.005	0.03	-59	1.6	1.6	-0.03	-0.025	-0.01	-33	-33	-16
LSBT-dry-M2	-0.00169	0.005	0.03	-59	1.7	1.7	-0.03	-0.02	-0.01	-40	-40	-26
LSBT-75-M2	-0.00169	0.005	0.03	-59	2.1	2.1	-0.03	-0.025	-0.01	-45	-45	-20
LSBT-SSD-M2	-0.00169	0.005	0.03	-59	3	3	-0.03	-0.02	-0.01	-66	-66	-50

Table 8.1: Parameters implemented into the constitutive laws governing the mechanical response of the FE interface

8.1.3 Modeling of the single-materials (substrate and overlay)

Besides the bonding properties, further properties that also affect the contact problems include the elastic modulus, the shear modulus, the Poisson's ratio and the geometry of both bonded materials [44, 80]. On the basis of the experimental data obtained from single-material tests, such properties were implemented into the FE model, see Table 8.2. It is worth mentioning that tests on single-materials are independent from the bonding properties of the composite element.

Series	b	l	h	Overlay			h	Substrate	
				E	ν	f_c		Modeled as	f_c
	mm	mm	mm	GPa	adim.	MPa	mm		MPa
LSBT-dry-M1	100	100	50	38.3	0.17	78	200	MIR	59
LSBT-75-M1	100	100	50	38.3	0.17	78	200	MIR	59
LSBT-SSD-M1	100	100	50	38.3	0.17	78	200	MIR	59
LSBT-dry-M2	100	100	50	47.1	0.2	147	200	MIR	59
LSBT-75-M2	100	100	50	47.1	0.2	147	200	MIR	59
LSBT-SSD-M2	100	100	50	47.1	0.2	147	200	MIR	59

b: width of the overlay.

l: length of the overlay.

h: thick of the overlay/substrate.

E: Elastic modulus measured according to the [176].

ν : Poisson's ratio for HPFRC and UHPFRC estimated according to [86] and [64], respectively

f_c : Compressive cubic strength measured according to the [180]

MIR: Modeling of the substrate as a member infinitely rigid.

Table 8.2: Parameters implemented into the constitutive laws governing the mechanical response of the FE bonded materials

The elastic modulus E of the substrate was assumed to be 25700 MPa, according to the formulation reported in [191]. The surface area of both the substrate A_{sub} and the overlay A_{overl} along the vertical plane was 200×820 mm and 50×100 mm, respectively. So, both E_{overl}/E_{sub} and A_{overl}/A_{sub} ratios were about 1.6 and 0.03, respectively. The resulting stiffness ratio $EA_{overlay}/EA_{substrate}$ was about 0.045. Consequently, the substrate could be modeled as a member infinitely rigid (MIR).

In order to consider the non-linear response of the contact problem, a non linear static analysis was carried out. In *Straus7*, an algorithm based on the modified Newton-Raphson method was used to solve the nonlinear equation system. The algorithm uses an iteration procedure within each load increment to ensure that the equilibrium of forces is maintained, within a specified allowance, at the end of each load increment. An incremental displacement history was assigned to the edge side of the overlay in order to simulate the cracking/debonding failure at the interface, near to zone of discontinuities of the overlay (e.g boundary locations, joints and crack). Each increment of displacement u , corresponding to the step 1, step 2, etc. of the analysis, was small enough (0.0005 mm) to make the nonlinear analysis efficient. It is worth noting that the resulting stress field was uniformly distributed along the width of the interface. This permitted to further simplify the numerical calculations, by selecting the function of *2-D stress plane* and modeling the layer of the overlay as a group of *plate*¹ items.

In the field of FE models better solutions are given for meshes which have both simple geometrical shape and sides' ratio close to 1 [198]. A 4-node quadrilateral type was therefore employed for modeling the meshes of the overlay. The mesh size of 1 mm was adopted, due to the fact that the independence of the calculation result versus the mesh size was reached when the node-to-node distance was ≤ 1 mm. The overlay size was of course defined in respect to the size of the experimental specimen, see Table 8.2.

¹The plate item is defined in *Straus7* as a surface element with a given thickness.

8.2 FE simulations and Results

The FE program does not provide solutions when constitutive laws governing the bonding behavior at the FE interface are described by softening branch post peak, which would be adequate for brittle materials. In order to compare the preliminary FE results, see Figure 8.4, with the theoretical model described in Section 7 the constitutive laws of the FE interface were firstly defined by a monotonic elastic-linear behavior. The same laws were implemented into the theoretical model. The convergence was good, as can be seen in Figure 8.4.

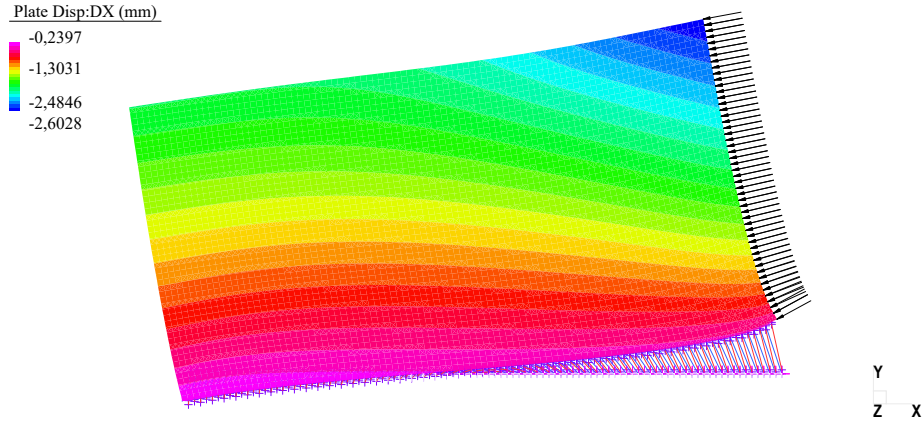


Figure 8.3: FE simulation of the cracking/debonding pattern for a generic series (LSBT-dry-M1), according to the assumption of elastic-linear response of the FE interface

Then, the constitutive laws governing the bonding behavior at the FE interface were adapted according to the approach explained in Section 8.1.2. For each increase of displacement assigned to the edge side of the overlay specimen, see Figure 8.5, the stress field arising along the FE interface was analyzed. In each step, the integration of the internal shear stress along the FE interface, see Figure 8.6(b), should be equal to the experimental load F_x uniformly applied to the edge side of the overlay specimen, see Figure 8.6(c). Instead the integration of the internal stress perpendicular to the interface should be equal to zero in order to respect the vertical static equilibrium, see Figure 8.6(a). The cohesive FE permitted to observe the propagation of the cracking/debonding failure along the interface, as the external load increased. Figure 8.6(b) denotes that since the step 30 (slip $u = 0.015$ mm) the propagation of the cracking/debonding path almost involved the overall FE interface, in fact in this region the stress remained constant and did not increase further (see step 40). This statement can be confirmed by observing the experimental curve in Figure 8.6(c). In particular, the slip value corresponding to the drop in force just after the peak load recorded in the experimental curves almost corresponds to the total propagation of the cracking/debonding path along the modeled interface. In addition, for all series, the experimental-to-numerical peak load ratio $F_{x,exp} / F_{x,FE}$ showed good accuracy. Also the initial stiffness was accurately predicted. Figure 8.7 denotes a good prediction of the cracking/debonding pattern of NSC-M1/M2 materials subjected to mixed-mode stress, confirming the reliability of both theoretical

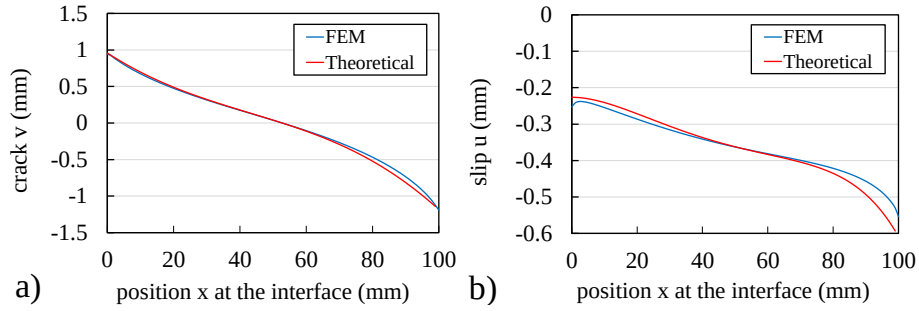


Figure 8.4: Comparison between the results provided by FE and theoretical simulations, both computed for a generic series (LSBT-dry-M1), according to the assumption of elastic-linear response of the modeled interface

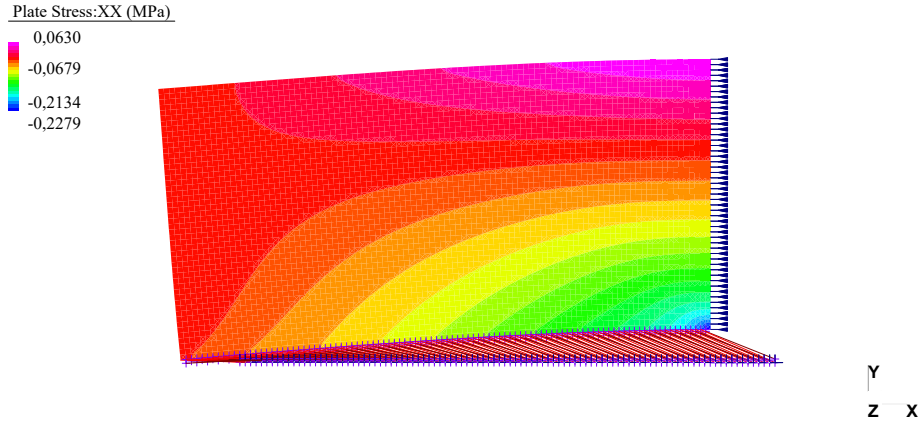


Figure 8.5: FE simulation of the cracking/debonding pattern stressed to mixed-mode, observed in the experimental investigation (series LSBT-dry-M1): Step 1 ($u = -0.0005$ mm)

and FE models developed in this PhD work thesis. In Figures 8.6 and 8.7 the gray lines represent the experimental curves recorded for each tested specimen. Instead, the black curve represents the experimental average derived from a polynomial regression analysis of the experimental data. Such a curve was considered sufficiently representative because of its high R^2 value obtained for all series investigated.

The experimental results also confirmed that the bond strength was clearly affected by both the moisture conditioning and the fresh and hardened properties of the overlay. In particular, lower moisture levels led to lower bond strength values when UHPFRC was used as overlay. By contrast, when HPRC was used as overlay the bond strength value was almost the same under any moisture levels. Such a difference was due to the different w/c ratio between the two overlays investigated. The low w/c ratio of UHPFRC (< 0.2) made this type of overlay highly sensitivity to the moisture level, especially under dry conditions. In such a condition an insufficient hydration of the fresh overlay in the interfacial zone occurred. This reduced the densification of the bond and its strength as well,

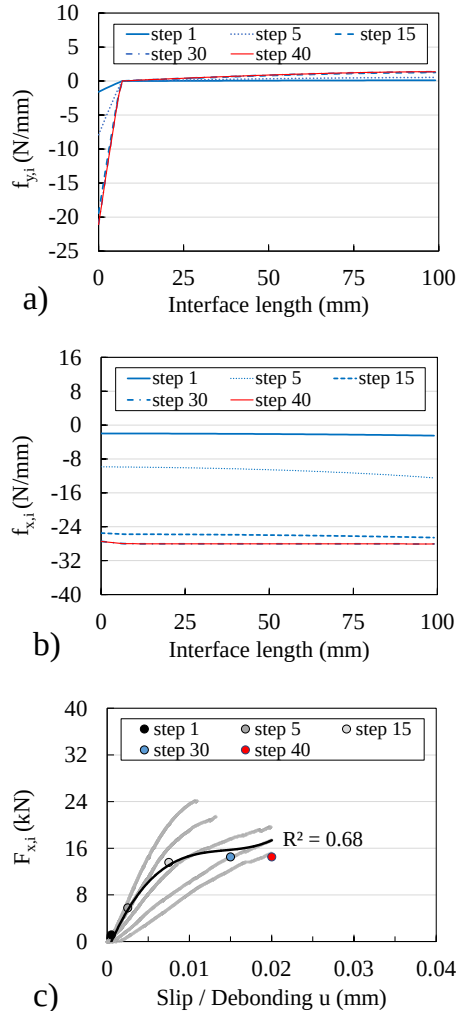


Figure 8.6: Analysis of the stress field on the modeled interface in series LSBT-dry-M1 recorded by a nonlinear static analysis: a) peeling stress (mode I), b) shear stress (mode II). Load vs slip/debonding curve: c) comparison between simulation and experimental results (LSBT)

showing a slight failure in the overlay, see Figure 6.13. Instead, in the HPFRCC series, the bond strength was less sensitive to the moisture level because of its higher w/c ratio (> 0.2). Also the hardened properties of the overlays affected the bond load capacity. In particular the bond strength ratio between the use of UHPFRC and HPFRCC as overlay was about 1.6.

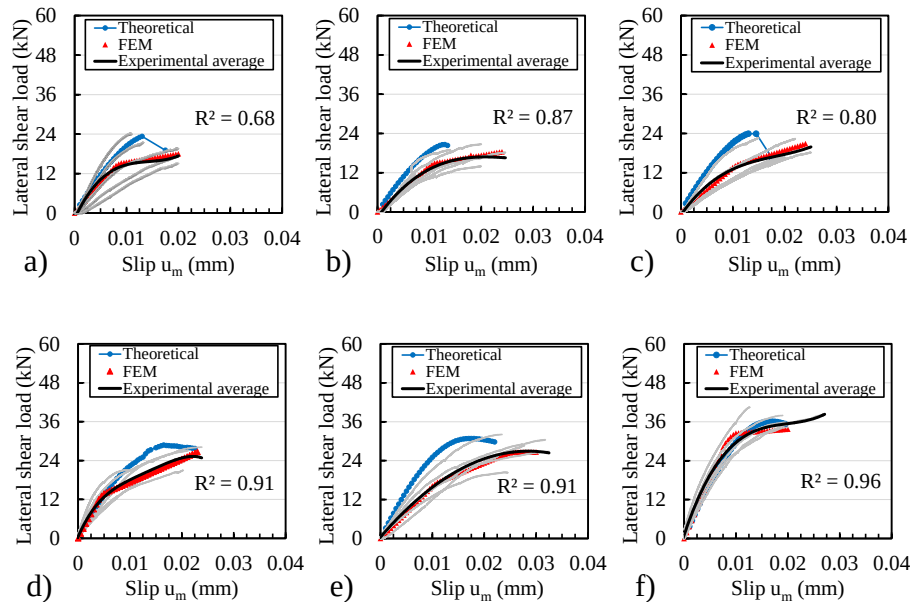


Figure 8.7: Comparison of experimental, theoretical and FE cohesive model results of cracking/debonding pattern for LSBT series : a) dry-M1, b) 75-M1, c) SSD-M1, d) dry-M2, e) 75-M2, f) SSD-M2

Chapter 9

Conclusions

The main conclusions of the second part of this PhD work thesis are drawn in the following:

- The aim of this work was to develop a cohesive model able to properly predict the loading bond capacity of retrofitted NSC - HPFRCC/UHPFRC composite structures, whose interface is subjected to mixed-mode stresses. The experimental investigation permitted to validate both the proposed model and highlight the different cracking/debonding patterns observed in the system “overlay-interface-substrate”, by varying both the moisture conditions of the substrate prior to the application of the overlay and the properties of the latter;
- The mechanical parameters implemented into the proposed model took into account mode I (tension-crack), mode II (shear-slip) and their coupling factor. A modified pull-off (POBT*), a direct shear (DSBT) and a tension/shear (TSBT) devices were designed by Author to properly calibrate the constitutive laws governing mode I, mode II and their coupling factor at the interface. The model also took into account the moisture level of substrate surface prior to the application of the overlay and the fresh properties of the latter. The theoretical results were compared by an independent experimental investigation (LSBT) confirming the reliability of the proposed model;
- A potential limit of the presented model is due to the assumption of the substrate member as a rigid support, which is appropriate in the field of retrofitted cement-based composite structures, where a thin layer of overlay is used;
- The bond testing device designed by Author provided reliable results. In addition, it was designed to be man-portable and easy to use both in situ and in the lab. An increase of the load carrying capacity would permit to test interface longer than 100 mm, in order to better reproduce the propagation of mixed stress along the interface of full scale rehabilitated concrete composite members;
- A FE model was developed in order to simulate the stress field and the propagation process of cracking/debonding path along the interface. The

numerical simulations were validated by the experimental results provided by LSBT series. In particular the peak load and stiffness of the numerical curves well agreed with the experimental ones. The approach to model the FE interface via link elements governed by a nonlinear cohesive law, on the basis of the energy balance concept of the fracture mechanism, proved to be effective and efficient;

- Experimental data highlighted the influence of the moisture condition level on the development of high bond capacity, answering to the controversial opinions found in recent studies. In particular, dry conditions can be detrimental, for overlays with very low w/c ratios (< 0.2), which is typical for UHPFRC materials. It was also observed that in dry conditions the cracking/debonding pattern failed in the UHPFRC overlay. Detrimental effects were not observed for HPFRCC overlays, whose sensitivity to low moisture levels is reduced because of their higher w/c ratios ($> 0.2 - 0.3$). However, SSD conditions should not be considered the best solution in all cases. If both moisture level and w/c ratio of the overlay are relatively high, the risk to develop a weak bond returns growing, since the excess of water along the interface increases the w/c ratio of the fresh overlay reducing the development of high bond strength values.

Appendices

A1 Uniaxial tensile response vs Axial strain of the fibrous mortar by varying the matrix and fiber properties

The UTT results presented in Section 2.3.3 were recorded on a *CATMAN* data acquisition software, with a frequency of 10 Hz. The recorded data were plotted and can be observed in Figures 1, 2 and 3.

A2 Regression analysis data output

As discussed in Section 3 the experimental results of M1 and M2 series listed in Table 2.5 were used to develop a new and more generalized predictive linear strength model for any HPFRCC/UHPFRC, taking into account the matrix strength and the fiber properties (shape, dosage and l_f/d_e) as variables. A

	$f_{c,SF}$	$f_{t,SF}$	$f_{rt,SF}$	$f_{c,HF}$	$f_{t,HF}$	$f_{c,HF}$
Regression analysis						
Multiple R	1.00	1.00	0.96	1.00	1.00	0.98
R square	1.00	0.99	0.91	0.99	0.99	0.96
Adjusted R square	0.92	0.91	0.81	0.92	0.91	0.88
Standard error	7.97	0.52	0.79	7.83	0.57	0.46
Observations	18	16	16	17	17	17
Variance analysis						
df regression	4	4	4	4	4	4
SS regression	202573.0	634.2	80	146198.8	450.6	71.3
MS regression	50643.2	158.5	20	36549.7	112.7	17.8
F regression	797.4	588.9	32.1	595.5	350.9	83.9
Significance F regression	0.0	0.0	0.0	0.0	0.0	0.0
df residual	14	12	12	13	13	13
SS residual	889.1	3.2	7.5	797.8	4.17	2.8
MS residual	63.5	0.3	0.6	61.4	0.32	0.2
df total	18	16	16	17	17	17
SS total	203462.1	637.4	87.5	146996.6	454.8	74.1

Table 1: Regression and variance analysis parameters for predicting the compressive (f_c), uni-axial (f_t) and residual tensile (f_{rt}) strengths of any HPFRCC/UHPFRC. Calculation processed by the software *EXCEL*

multi-linear regression analysis with 95% confidence intervals was computed in

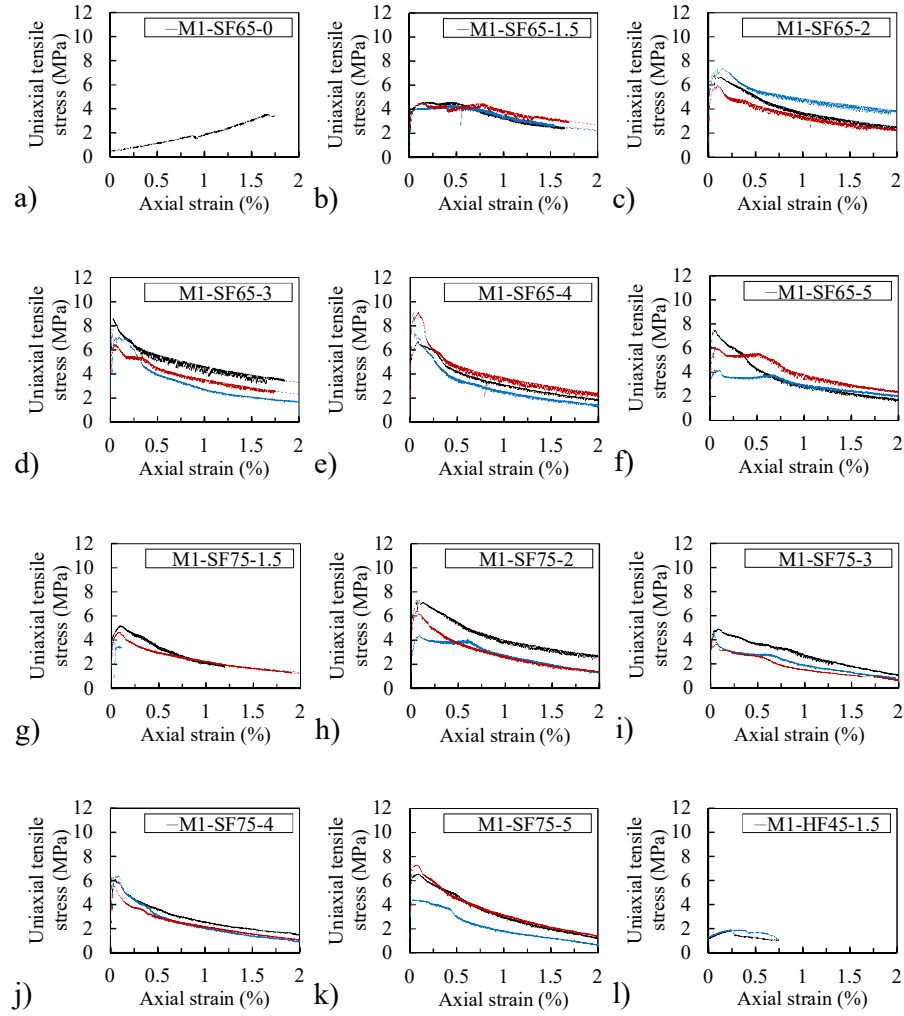


Figure 1: UTT carried out on specimens by varying both matrix and fiber properties

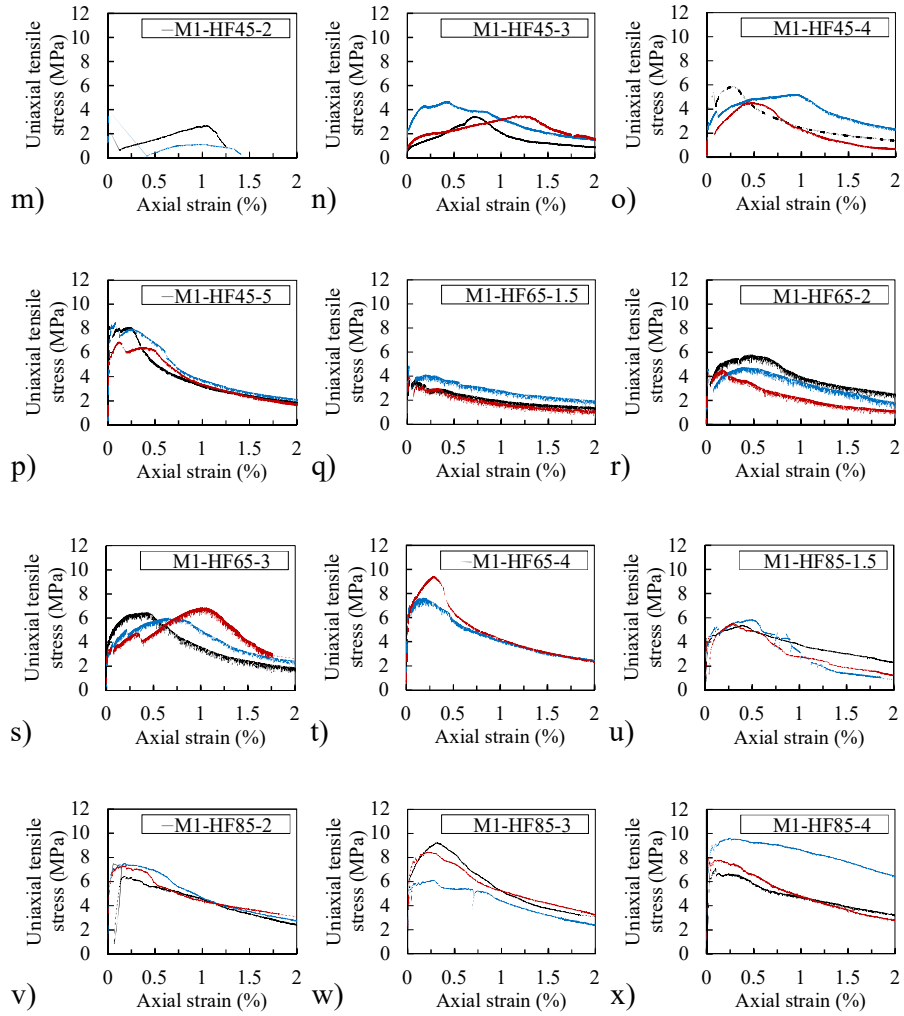


Figure 2: UTT carried out on specimens by varying both matrix and fiber properties

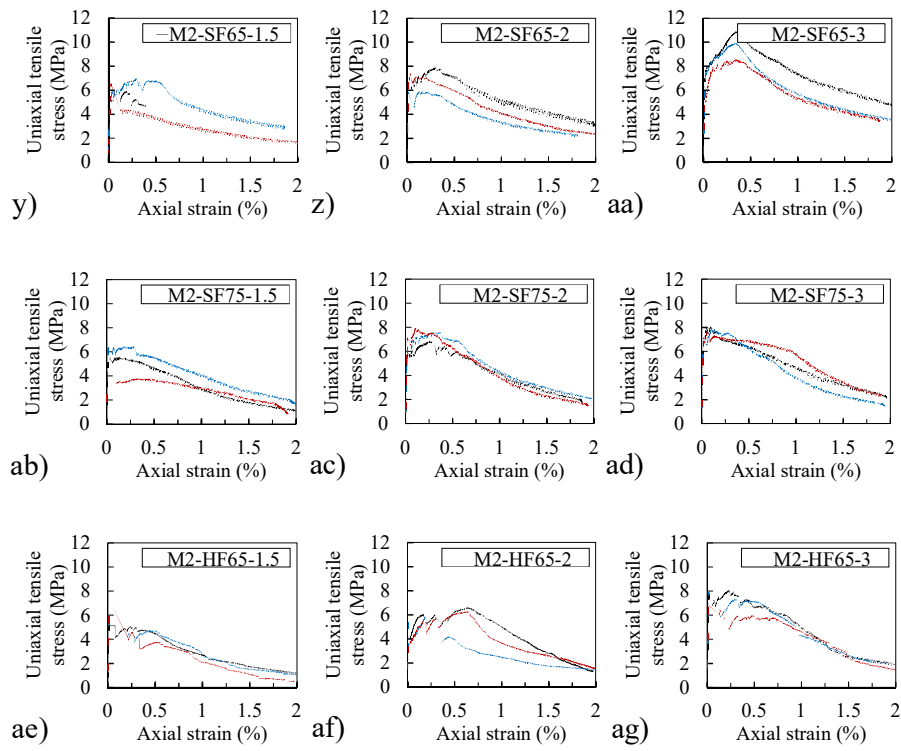


Figure 3: UTT carried out on specimens by varying both matrix and fiber properties

order to obtain the numerical relationships between the matrix strength, the fiber properties and compressive (f_c), uni-axial (f_t) and residual tensile (f_{rt}) strengths of any HPFRCC/UHPFRC. For achieving such analysis the matrix

Dependent variable	Coefficients	Std Error	t Stat	Lower 95%	Upper 95%
$f_{c,SF}$					
Variable x 1	1.07	0.06	18.49	0.94	1.19
Variable x 2	5.89	3.36	1.75	-1.32	13.09
Variable x 3	-388.75	211.77	-1.84	-842.94	65.44
Variable x 4	3.38	1.72	1.96	-0.31	7.06
$f_{t,SF}$					
Variable x 1	0.98	0.06	16.13	0.85	1.11
Variable x 2	-0.93	0.23	-3.97	-1.44	-0.42
Variable x 3	59.64	4.76	4.04	-27.48	91.79
Variable x 4	1.27	0.16	7.79	0.91	1.62
$f_{rt,SF}$					
Variable x 1	0.16	0.09	1.77	-0.04	0.36
Variable x 2	-0.13	0.36	-0.36	-0.90	0.65
Variable x 3	16.41	22.44	0.73	-32.48	65.29
Variable x 4	-0.09	0.25	-0.36	-0.63	0.45
$f_{c,HF}$					
Variable x 1	1.03	0.06	17.58	0.90	1.15
Variable x 2	0.30	0.40	0.75	-0.56	1.15
Variable x 3	-32.70	17.44	-1.87	-70.39	4.99
Variable x 4	6.28	2.15	2.92	1.63	10.94
$f_{t,HF}$					
Variable x 1	0.96	0.07	13.53	0.81	1.12
Variable x 2	0.05	0.03	1.94	-0.01	0.12
Variable x 3	-5.09	1.26	-4.03	-7.82	-2.36
Variable x 4	0.88	0.16	5.65	0.55	1.22
$f_{rt,HF}$					
Variable x 1	0.03	0.06	0.48	-0.10	0.15
Variable x 2	0.09	0.02	3.71	0.04	0.13
Variable x 3	-4.23	1.03	-4.12	-6.45	-2.01
Variable x 4	0.56	0.13	4.41	0.29	0.84

Table 2: Analysis of variables for predicting the compressive (f_c), uni-axial (f_t) and residual tensile (f_{rt}) strengths of any HPFRCC/UHPFRC. Calculation processed by the software EXCEL

strength, the shape of fibers, the dosage and the l_f/d_e are selected as dependent variables of the regression analysis. The data parameters of such analysis are reported in Tables 1 and 2.

A3 Analytical calculation of the flexural bending load - CTOD response of prismatic HPFRCC/UHPFRC beams

The flexural bending load vs CTOD response of prismatic HPFRCC/UHPFRC beams was theoretically calculated by an analytical approach presented in Section 3.2. The solver *Mathematica* was used to model the constitutive laws in compression and tension of a given HPFRCC/UHPFRC cross section, in order to calculate its bending flexural response. The concerning calculations are described in the following.

First of all, a 3-D coordinate system was adopted in order to define the orientation of the prismatic element subjected to the bending flexural load, as illustrated in Figure 4. The sectional analysis denotes that the mechanical response in the

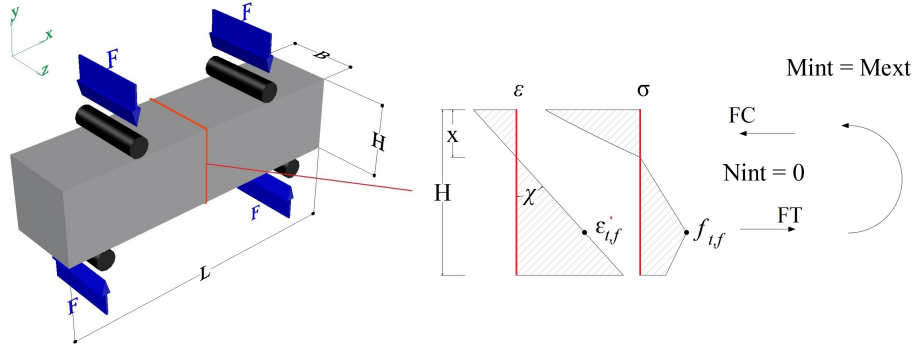


Figure 4: Geometric model of the prismatic fibrous mortar beam under four-point flexural loading

plane x-z is negligible. So, the analysis can only involve the element behavior in the plane x-y. Both geometrical and mechanical parameters requested in the calculations are listed below:

- width b , height h , notch a_0 and length L of the SFRC prismatic section;
- matrix strength in compression f_{c0} and traction f_{t0} , fiber properties (l , d_e , V_f)
- compressive response of the SFRC (f_c , ϵ_{c2} , ϵ_{cu2}) ;
- uni-axial tensile response of the SFRC (f_t , ϵ_{tel} , ϵ_{tu}) .

The Table 3 shows the parameters listed above, which were implemented into the solver.

The constitutive laws “parabola-rectangle” and the “bi-linear” models were adopted for the compressive and tensile regions, respectively, see Equations A3.3 and A3.4. By integrating both the laws in respect of the y-axis the resulting compressive and tensile forces are provided, respectively, in Equations A3.5 and A3.6. A second integration provides the respective resulting moments, see Equations A3.7 and A3.8.

$$c = -10^{-10} \quad (\text{A3.1})$$

Geometrical and mechanical parameters of the SFRC beam									
Series M1-HF-45-0.3									
	mm				MPa				% ₀₀
b	150	l (mm)	30	f_{c0}	75.85	ϵ_{c2}			$2 + 0.085(f_c - 50)^{0.53}$
h	150	d (mm)	0.35	f_c	$1.03f_{c0} + 0.3l - 32.7d + 6.28V_f$	ϵ_{cu2}			$2.6 + 35((90 - f_c)/100)^4$
a_0	45	V_f (%)	1.7	f_{t0}	3.3	n			$1.4 + 23.4((90 - f_c)/100)^4$
L	600	-	-	f_t	$0.96f_{t0} + 0.05l - 5.09d + 0.88V_f$	ϵ_{tel}			0.2
-	-	-	-	$f_{t,r}$	$0.03f_{t0} + 0.09l - 4.23d + 0.56V_f$	ϵ_{tu}			25
Series M2-SF75-3.8									
	mm				MPa				% ₀₀
b	150	l (mm)	13	f_{c0}	103.95	ϵ_{c2}			$2 + 0.085(f_c - 50)^{0.53}$
h	150	d (mm)	0.175	f_c	$1.1f_{c0} + 5.9l - 388.7d + 3.4V_f$	ϵ_{cu2}			$2.6 + 35((90 - f_c)/100)^4$
a_0	45	V_f (%)	3.8	f_{t0}	6.1	n			$1.4 + 23.4((90 - f_c)/100)^4$
L	600	-	-	f_t	$f_{t0} - 0.9l + 59.6d + 1.3V_f$	ϵ_{tel}			0.2
-	-	-	-	$f_{t,r}$	$0.2f_{t0} - 0.1l + 16.4d - 0.1V_f$	ϵ_{tu}			24
Series M3-SF65-2.5									
	mm				MPa				% ₀₀
b	150	l (mm)	20	f_{c0}	167.32	ϵ_{c2}			$2 + 0.085(f_c - 50)^{0.53}$
h	150	d (mm)	0.3	f_c	$1.1f_{c0} + 5.9l - 388.7d + 3.4V_f$	ϵ_{cu2}			$2.6 + 35((90 - f_c)/100)^4$
a_0	45	V_f (%)	2.5	f_{t0}	9.17	n			$1.4 + 23.4((90 - f_c)/100)^4$
L	600	-	-	f_t	$f_{t0} - 0.9l + 59.6d + 1.3V_f$	ϵ_{tel}			0.25
-	-	-	-	$f_{t,r}$	$0.2f_{t0} - 0.1l + 16.4d - 0.1V_f$	ϵ_{tu}			35

Table 3: Input parameters implemented into *Mathematica*

$$\epsilon(y) = \chi y \quad (\text{A3.2})$$

Where c represents a numerical coefficient near to 0; $\epsilon(y)$ represents the linear strain field assumed along the cross-section; χ is the curvature of the cross-section; y represents the vertical position along the height of the cross-section.

$$f_c(\chi, y) = f_c \left(1 - \left(1 - \frac{\chi y}{\epsilon_{c2}} \right)^n \right) \text{UnitStep}[\epsilon_{c2} - \chi y] + f_c \text{UnitStep}[\chi y - \epsilon_{c2}] \quad (\text{A3.3})$$

$$f_t(\chi, y) = \chi y \frac{f_t}{\epsilon_{tel}} \text{UnitStep}[\epsilon_{tel} - \chi y] + \left(f_t + \chi y \frac{f_{tu} - f_t}{\epsilon_{tu} - \epsilon_{tel}} \right) \text{UnitStep}[\chi y + c - \epsilon_{tel}] \quad (\text{A3.4})$$

$$CF(\chi, x) = B f_c \left(\frac{n \epsilon_{c2}}{\chi + n \chi} + \frac{(\epsilon_{c2} - x \chi)}{\chi} \left(- \text{UnitStep} \left[x - \frac{\epsilon_{c2}}{\chi} \right] + \frac{\epsilon_{c2}^{-n} (-(1+n) \epsilon_{c2}^n + (\epsilon_{c2} - x \chi)^n) \text{UnitStep} \left[-x + \frac{\epsilon_{c2}}{\chi} \right]}{1+n} \right) \right) 10^{-3} \quad (\text{A3.5})$$

$$\begin{aligned}
TF(\chi, x) = B \frac{1}{2\chi} & \left(\epsilon_{tel} f_t + \frac{f_t(-\epsilon_{tel}^2 + (H-x)^2\chi^2) UnitStep\left[-H+x+\frac{\epsilon_{tel}}{\chi}\right]}{\epsilon_{tel}} + \right. \\
& + UnitStep\left[H-x+\frac{c-\epsilon_{tel}}{\chi}\right] \left(-(c-\epsilon_{tel}+H\chi-x\chi) \left(-2f_t + \right. \right. \\
& \left. \left. \frac{f_{tu}-f_t}{\epsilon_{tu}-\epsilon_{tel}}(c-\epsilon_{tel}-H\chi+x\chi) \right) + (c-\epsilon_{tel}) \left(\frac{f_{tu}-f_t}{\epsilon_{tu}-\epsilon_{tel}}(c-\epsilon_{tel}) + \right. \right. \\
& \left. \left. -2f_t \right) UnitStep\left[\frac{c-\epsilon_{tel}}{\chi}\right] \right) \Bigg) 10^{-3}
\end{aligned} \tag{A3.6}$$

$$\begin{aligned}
CM(\chi, x) = B & \left(If\left[x\chi < \epsilon_{c2}, \frac{1}{2(1+n)(2+n)\chi^2} \left(\epsilon_{c2}^{-n} (2f_c(\epsilon_{c2}-x\chi)^{1+n}(\epsilon_{c2}+ \right. \right. \right. \\
& \left. \left. + (1+n)x\chi) - f_c\epsilon_{c2}^n)(2\epsilon_{c2}^2 - (1+n)(2+n)x^2\chi^2) \right), \right. \\
& otherwise \int_0^x \left(f_{cy} UnitStep[\epsilon_{c2}-y\chi] + \right. \\
& \left. - f_c y \left(1 - \frac{y\chi}{\epsilon_{c2}} \right)^n UnitStep[\epsilon_{c2}-y\chi] + \right. \\
& \left. \left. + f_c y UnitStep[-\epsilon_{c2}+y\chi] \right) dy \right] 10^{-6}
\end{aligned} \tag{A3.7}$$

$$\begin{aligned}
TM(\chi, x) = B \frac{2f_t}{6\chi^2} & \left(\frac{1}{\epsilon_{tel}} \left(\epsilon_{tel}^3 + (-\epsilon_{tel}^3 + (H-x)^3\chi^3) UnitStep\left[-H+x+ \right. \right. \right. \\
& \left. \left. + \frac{\epsilon_{tel}}{\chi} \right] \right) + UnitStep\left[H-x+\frac{c-\epsilon_{tel}}{\chi}\right] \left((c+ \right. \\
& \left. -\epsilon_{tel})^2 \left(\frac{f_{tu}-f_t}{\epsilon_{tu}-\epsilon_{tel}}(2c-2\epsilon_{tel}) - 3f_t \right) + \right. \\
& 3(H-x)^2 f_t \chi^2 + 2 \left(\frac{f_{tu}-f_t}{\epsilon_{tu}-\epsilon_{tel}} \epsilon_{tu}-\epsilon_{tel} (H-x)^3 \chi^3 + \right. \\
& \left. \left. - (c-\epsilon_{tel})^2 \left(\frac{f_{tu}-f_t}{\epsilon_{tu}-\epsilon_{tel}}(2c-2\epsilon_{tel}) - 3f_t \right) UnitStep\left[\frac{c-\epsilon_{tel}}{\chi}\right] \right) \right) \Bigg) 10^{-6}
\end{aligned} \tag{A3.8}$$

A3.1 Bisection method

The resulting forces and moments depend on both the curvature χ^* and the neutral axis x^* of the inflected prismatic section. These kinematic parameters are both unknown. In order to solve such a problem the carrying load capacity of the inflected prismatic section should be determined on the basis of the mechanical

and geometrical properties of the SFRC prismatic section, regardless of the entity of the external loads. It is worth noting that the variability of both χ^* and the x^* leads to existence of ∞^2 congruent configurations. Among these, only one configuration also respects the equilibrium conditions. In other words, the deformation field of the inflected prismatic section is such that the internal stresses are in equilibrium with the external loads, once the constitutive laws of the section material are known. It can be stated that the position of x^* is inside the section, since the static translational equilibrium N must be 0. The reason is due to the fact the only bending loads are applied on the prismatic section. The following conditions have to be respected.

$$\begin{cases} N_{int}(\chi^*, x^*) = 0 \\ M_{int}(\chi^*, x^*) = M_{est} \end{cases} \quad (\text{A3.9})$$

Since $N_{int}(\chi^*, x^*)$ can be represented from a continuous function, by fixing one of the two kinematic parameters, in this case the curvature χ^* , the first equation of the System A3.9 becomes an equation in one variable (x^*). According to the bisection method is possible to find the solution in which $N_{int} = 0$. Once the neutral axis x^* is calculated, the second equation of the System A3.9 permits to calculate the resistance moment M of the cross-section. By increasing the χ^* value, step-by-step, the procedure above-mentioned permits to calculate the resistance moment - curvature of the cross-section. The last step is attained when the ultimate increase of χ^* causes the failure of the cross-section. Alternatively, the procedure is also stopped when a too high value of χ^* is reached, e.g. $\chi^* > 10^{-3}$, which would be unrealistic. A further instruction in the calculation permits to show the x^* position, the kinematic and stress magnitude in both the top and the bottom of the cross-section, for a given χ^* value. The estimation of the CTOD value is derived according to the equation 9.3 reported in [170]. The resistance moment - CTOD curves are calculated and reported in Figure 3.6.

A4 Roughness profile of the interface measured via photogrammetric survey

As discussed in Section 6.2.3 the surface profile of the substrate surface prior to the application of the overlay was analyzed via photogrammetric method. For each series, statistical analysis of roughness profile were computed. Tables 4, 5, 6, 7, 8, 9, 10 and 11 list the concerning results.

Figures 5 and 6 show the results of the photogrammetric scanner of the interface profile of each specimen provided by hydro-jetting.

A5 Experimental data recorded from bond tests

The experimental results of bond tests discussed in Section 6.2 are summarized in Tables 12, 13, 14, 15, 16 and 17.

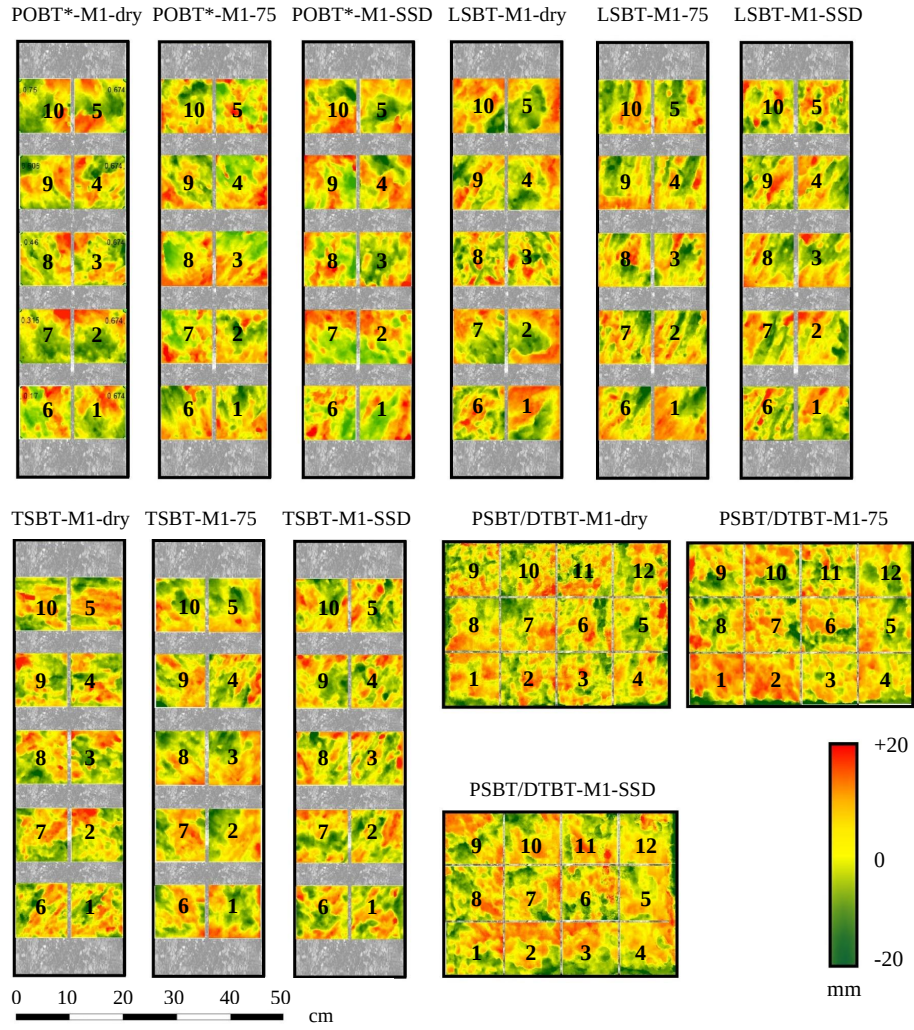


Figure 5: Roughness profile scanned in M1 series

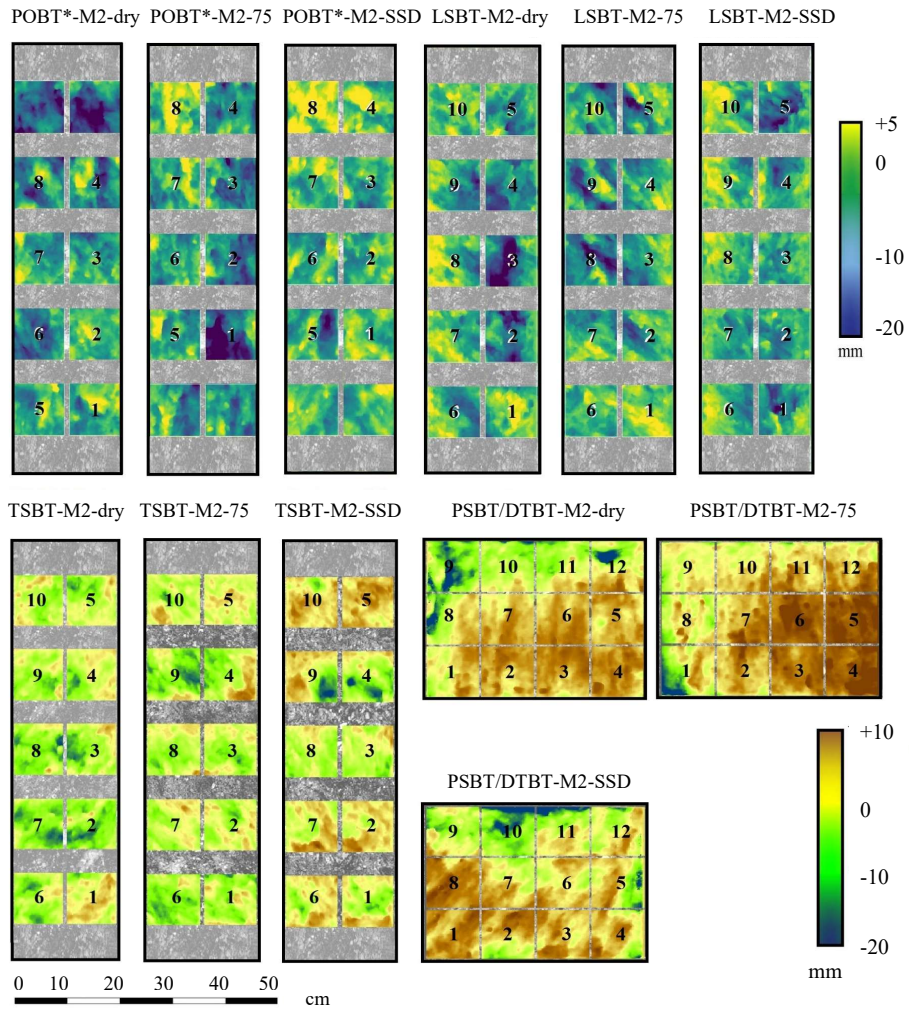


Figure 6: Roughness profile scanned in M2 series

Interfacial zone Series	Minimum (mm)	Maximum (mm)	Average (mm)	Stand. Dev. (mm)
POBT*-dry-M1-1	2.6	23.9	11.3	4.0
POBT*-dry-M1-2	-0.8	23.9	11.2	5.8
POBT*-dry-M1-3	-3.9	21.3	10.9	4.0
POBT*-dry-M1-4	-1.0	20.1	10.1	3.0
POBT*-dry-M1-5	-2.3	18.7	6.2	3.4
POBT*-dry-M1-6	-3.0	16.1	8.0	2.4
POBT*-dry-M1-7	-4.1	17.6	3.2	3.7
POBT*-dry-M1-8	-4.6	13.4	2.9	3.7
POBT*-dry-M1-9	-7.9	5.9	-1.4	2.2
POBT*-dry-M1-10	-7.8	6.1	-0.9	2.6
POBT*-75-M1-1	-11.9	4.3	-3.0	2.8
POBT*-75-M1-2	-12.4	6.4	-1.7	2.6
POBT*-75-M1-3	-8.4	7.2	-0.9	2.8
POBT*-75-M1-4	-12.0	5.9	-2.8	3.1
POBT*-75-M1-5	-15.1	6.7	-2.6	4.1
POBT*-75-M1-6	-11.0	6.6	-2.1	3.4
POBT*-75-M1-7	-6.4	9.6	0.6	2.7
POBT*-75-M1-8	-11.0	8.2	-1.4	3.2
POBT*-75-M1-9	-10.5	8.5	-2.0	3.4
POBT*-75-M1-10	-10.6	8.6	-1.0	3.3
POBT*-SSD-M1-1	-16.5	1.0	-5.5	3.3
POBT*-SSD-M1-2	-17.0	5.4	-4.7	3.7
POBT*-SSD-M1-3	-18.0	-0.2	-7.1	3.0
POBT*-SSD-M1-4	-15.9	2.4	-4.9	3.7
POBT*-SSD-M1-5	-14.2	2.6	-5.5	2.7
POBT*-SSD-M1-6	-12.4	6.1	-6.3	2.8
POBT*-SSD-M1-7	-15.4	2.7	-4.8	3.2
POBT*-SSD-M1-8	-15.6	3.1	-6.2	3.6
POBT*-SSD-M1-9	-4.4	8.3	1.0	1.8
POBT*-SSD-M1-10	-7.6	12.1	0.8	3.4

Table 4: Roughness profile of the interface of POBT*-M1 series

A6 Bonding failure pattern observed in POBT* and LSBT series

The failure pattern of POBT* and LSBT series, presented in Sections 6.2.4 and 6.2.7, is illustrated in Figures 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 15, 16, 17 and 18.

A7 Coefficient of the fitted cohesive bonding laws

The coefficients of the analytical relationships discussed in the section 7 are listed in Table 18 for all series investigated. The concerning curves are illustrated in

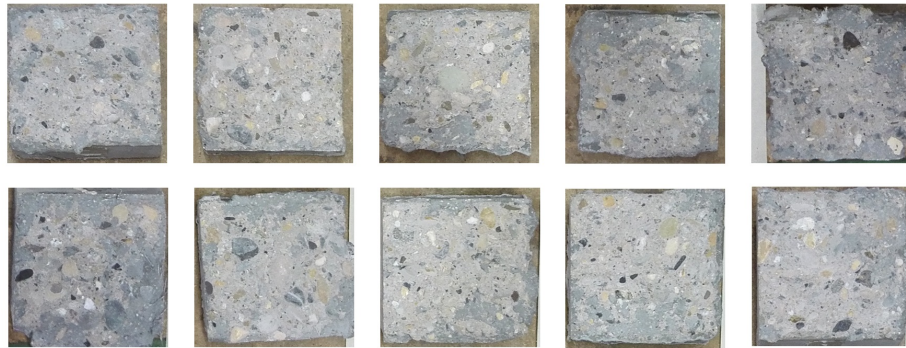


Figure 7: Failure pattern of POBT*-dry-M1 series

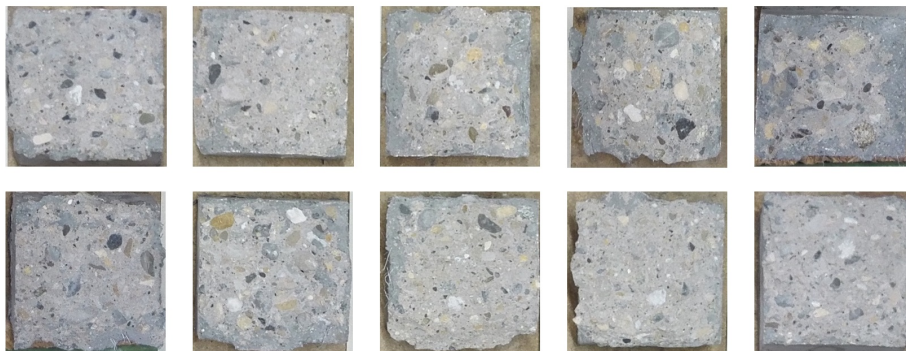


Figure 8: Failure pattern of POBT*-75-M1 series

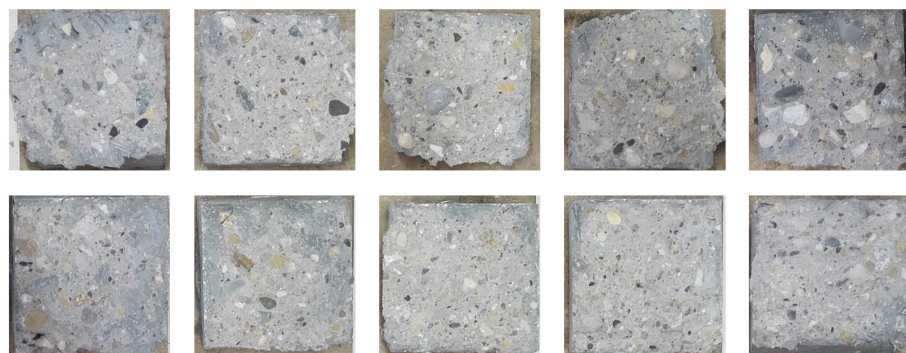


Figure 9: Failure pattern of POBT*-SSD-M1 series

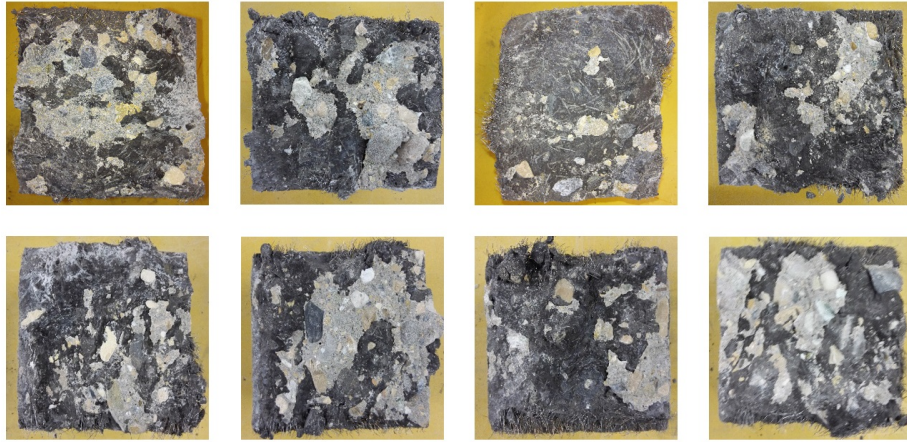


Figure 10: Failure pattern of POBT*-dry-M2 series

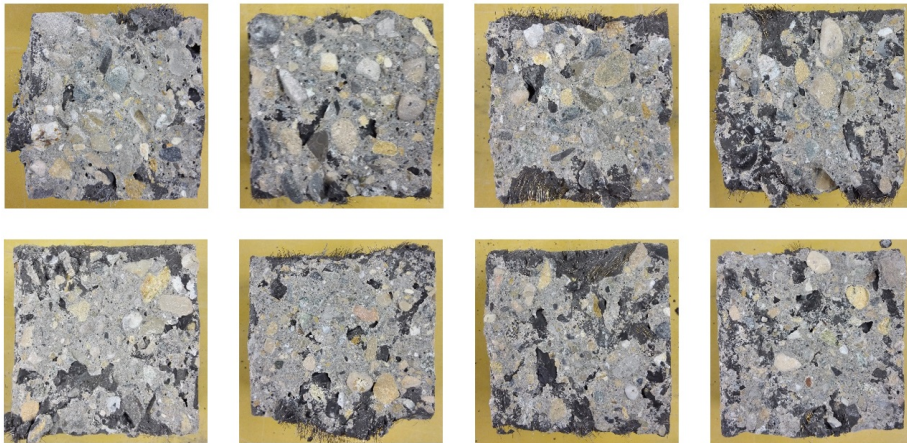


Figure 11: Failure pattern of POBT*-75-M2 series

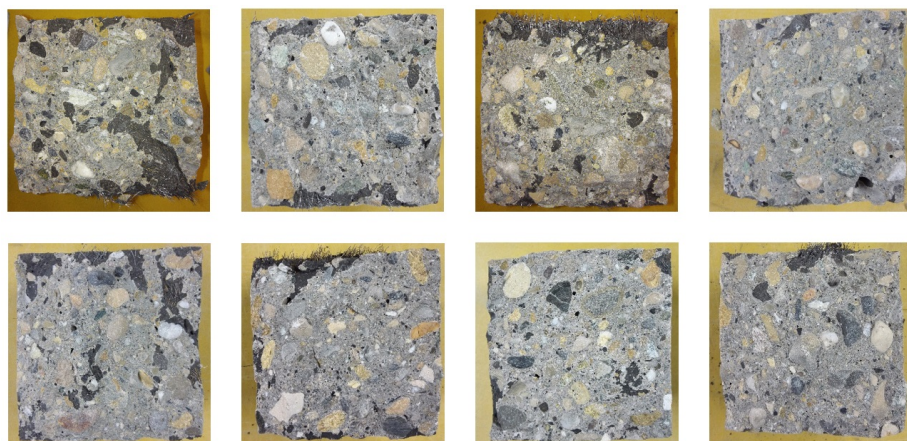


Figure 12: Failure pattern of POBT*-SSD-M2 series

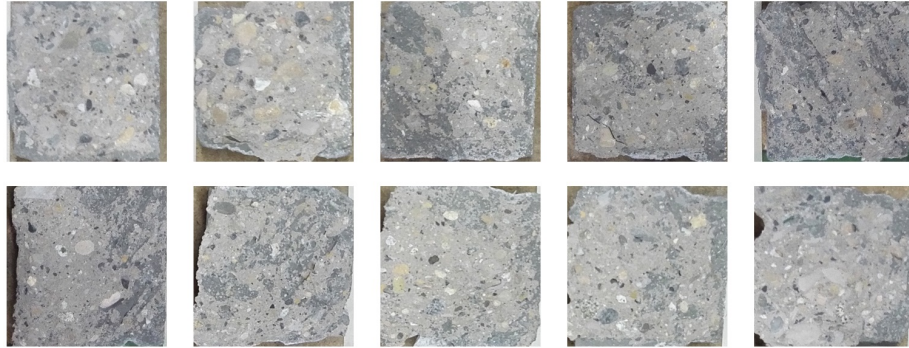


Figure 13: Failure pattern of LSBT-dry-M1 series

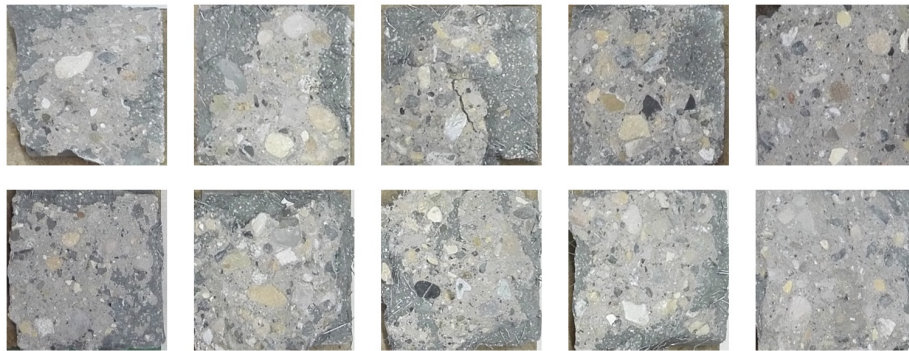


Figure 14: Failure pattern of LSBT-75-M1 series



Figure 15: Failure pattern of LSBT-SSD-M1 series

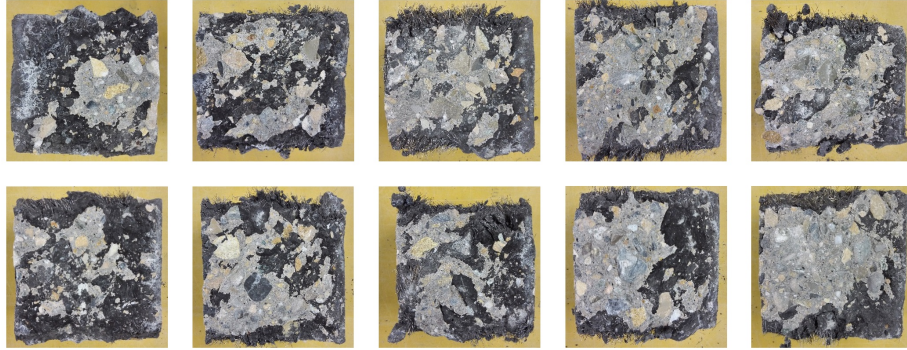


Figure 16: Failure pattern of LSBT-dry-M2 series

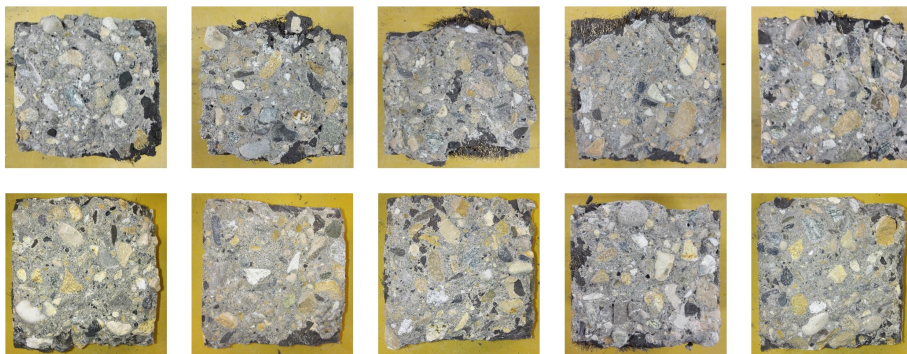


Figure 17: Failure pattern of LSBT-75-M2 series

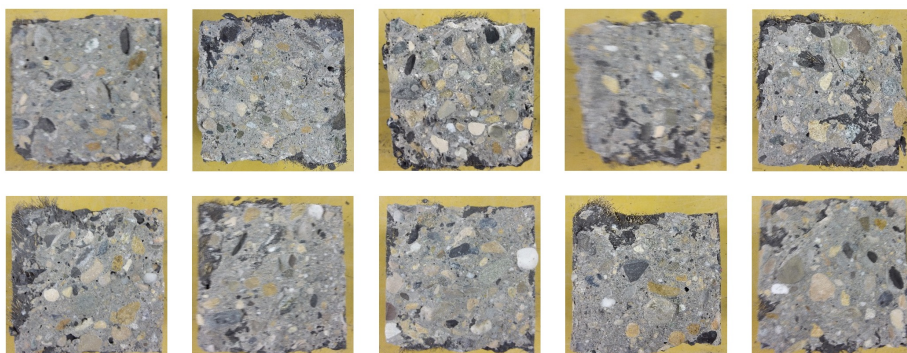


Figure 18: Failure pattern of LSBT-SSD-M2 series

Interfacial zone Series	Minimum (mm)	Maximum (mm)	Average (mm)	Stand. Dev. (mm)
POBT*-dry-M2-1	-12.2	8.0	1.6	3.8
POBT*-dry-M2-2	-9.4	9.0	0.9	2.3
POBT*-dry-M2-3	-13.8	4.0	3.7	3.4
POBT*-dry-M2-4	-15.9	13.0	3.8	5.0
POBT*-dry-M2-5	-10.8	10.0	2.4	3.4
POBT*-dry-M2-6	-14.7	3.0	6.1	3.4
POBT*-dry-M2-7	-12.3	11.0	2.4	3.7
POBT*-dry-M2-8	-17.8	7.0	5.9	4.3
POBT*-75-M2-1	-23.8	6.0	10.3	6.6
POBT*-75-M2-2	-16.0	8.0	6.1	4.0
POBT*-75-M2-3	-18.0	5.0	5.5	4.1
POBT*-75-M2-4	-15.3	9.0	4.6	3.7
POBT*-75-M2-5	-15.6	8.0	2.4	3.6
POBT*-75-M2-6	-10.8	8.0	2.5	3.1
POBT*-75-M2-7	-11.5	9.0	1.8	3.7
POBT*-75-M2-8	-11.6	12.0	2.0	4.2
POBT*-SSD-M2-1	-6.9	7.0	0.2	2.9
POBT*-SSD-M2-2	-8.0	7.0	2.0	2.9
POBT*-SSD-M2-3	-11.4	8.0	2.4	3.5
POBT*-SSD-M2-4	-9.1	11.0	9.1	3.9
POBT*-SSD-M2-5	-18.0	7.0	3.3	4.6
POBT*-SSD-M2-6	-10.2	8.0	1.6	3.4
POBT*-SSD-M2-7	-7.9	9.0	0.2	3.3
POBT*-SSD-M2-8	-5.5	14.0	4.0	3.6

Table 5: Roughness profile of the interface of POBT*-M2 series

Figures 19 and 20.

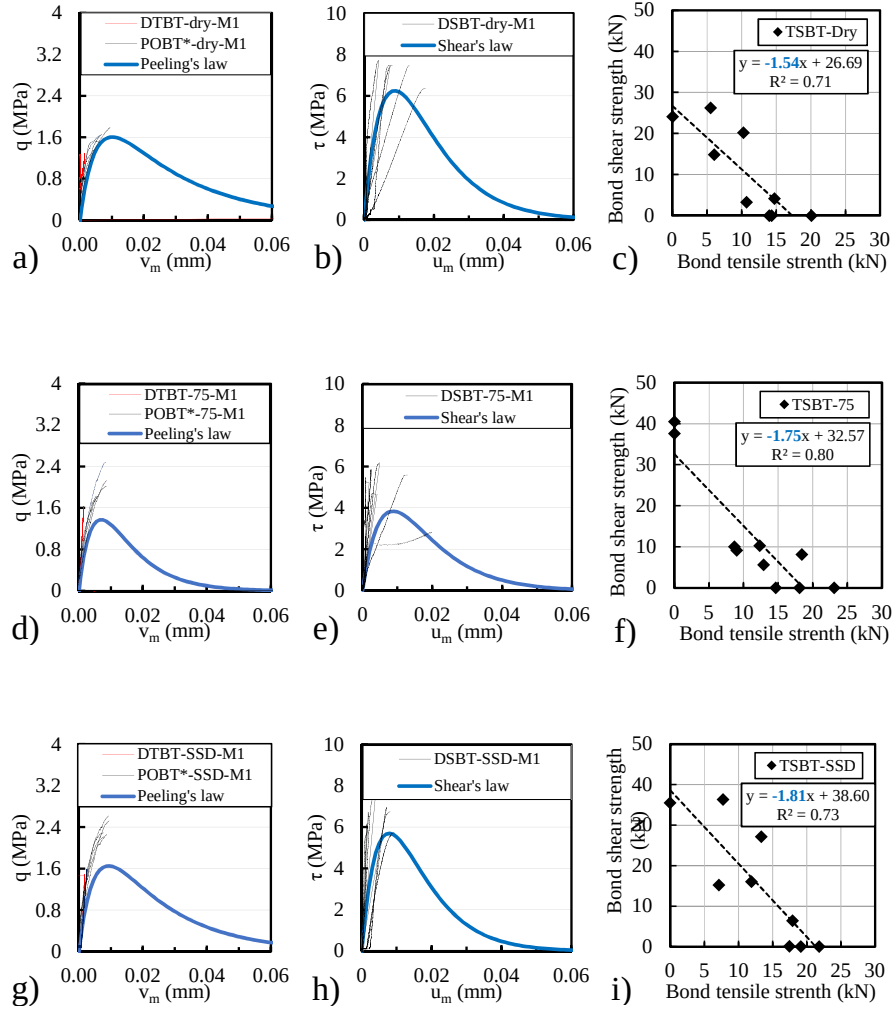


Figure 19: Parametrization of constitutive laws governing the modeled interface of M1 series, in terms of a) mode I - dry, b) mode II - dry, c) coupling factor I-II - dry, d) mode I - 75, e) mode II - 75, f) coupling factor I-II - 75, g) mode I - SSD, h) mode II - SSD, i) coupling factor I-II - SSD

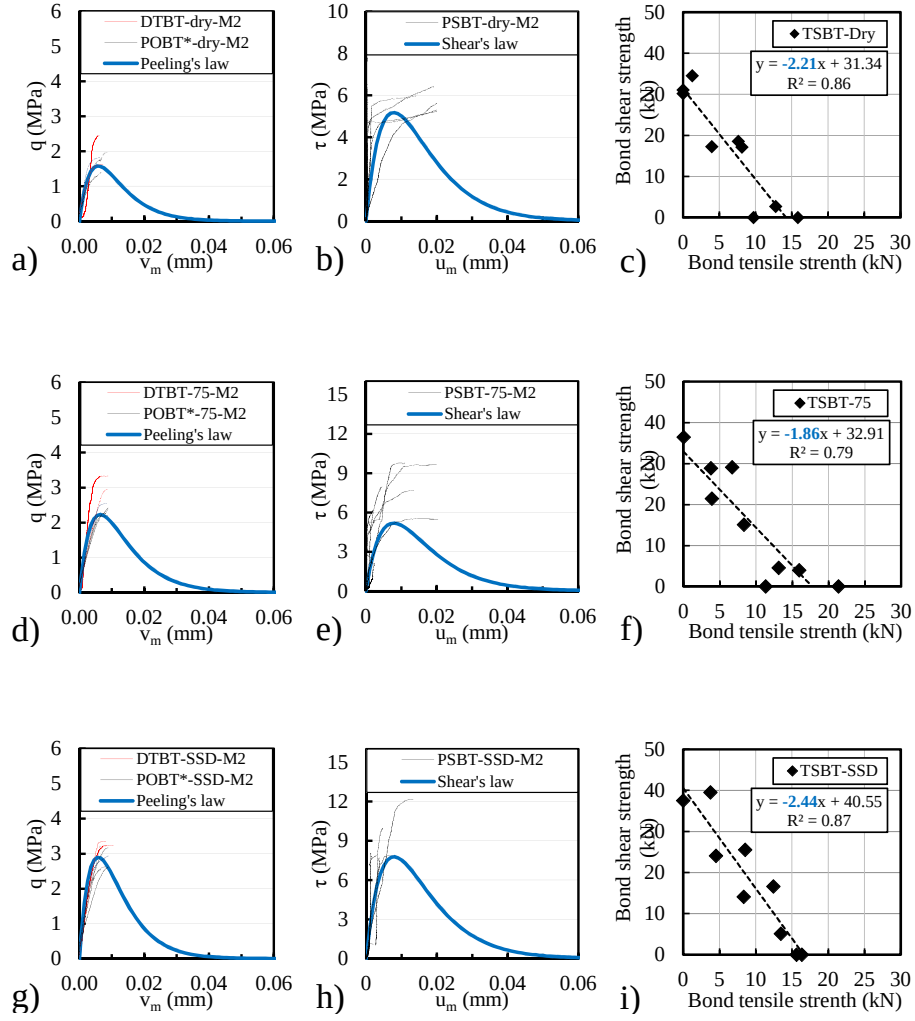


Figure 20: Parametrization of constitutive laws governing the modeled interface of M2 series, in terms of a) mode I - dry, b) mode II - dry, c) coupling factor I-II - dry, d) mode I - 75, e) mode II - 75, f) coupling factor I-II - 75, g) mode I - SSD, h) mode II - SSD, i) coupling factor I-II - SSD

Interfacial zone Series	Minimum (mm)	Maximum (mm)	Average (mm)	Stand. Dev. (mm)
LSBT-dry-M1-1	-16.6	2.9	-3.9	3.6
LSBT-dry-M1-2	-12.2	7.0	-2.6	4.2
LSBT-dry-M1-3	-12.2	7.0	-2.6	4.2
LSBT-dry-M1-4	-11.9	5.7	-2.9	3.2
LSBT-dry-M1-5	-7.6	4.8	-2.1	1.9
LSBT-dry-M1-6	-8.1	5.3	-1.0	2.2
LSBT-dry-M1-7	-12.7	3.5	-5.2	3.1
LSBT-dry-M1-8	-17.7	5.3	-4.6	5.3
LSBT-dry-M1-9	-7.8	5.9	-1.8	1.9
LSBT-dry-M1-10	-17.4	4.5	-4.4	5.0
LSBT-75-M1-1	-14.9	0.8	-7.5	3.0
LSBT-75-M1-2	-15.5	-1.3	-8.9	2.5
LSBT-75-M1-3	-18.9	-2.7	-10.2	2.6
LSBT-75-M1-4	-18.3	-1.4	-10.0	2.6
LSBT-75-M1-5	-16.1	-2.4	-7.8	2.0
LSBT-75-M1-6	-18.4	-0.4	-8.7	2.7
LSBT-75-M1-7	-12.4	0.9	-5.6	2.6
LSBT-75-M1-8	-15.9	-3.0	-9.1	2.2
LSBT-75-M1-9	-17.2	-3.3	-9.3	2.3
LSBT-75-M1-10	-25.2	-5.3	-13.1	4.2
LSBT-SSD-M1-1	-17.5	4.6	-7.1	3.4
LSBT-SSD-M1-2	-12.4	-0.3	-5.5	1.9
LSBT-SSD-M1-3	-9.6	5.3	-2.4	2.3
LSBT-SSD-M1-4	-14.9	3.5	-6.1	2.9
LSBT-SSD-M1-5	-11.1	5.2	-4.0	2.7
LSBT-SSD-M1-6	-15.2	2.3	-7.7	2.9
LSBT-SSD-M1-7	-8.8	7.5	-2.2	2.7
LSBT-SSD-M1-8	-13.4	3.9	-5.2	3.0
LSBT-SSD-M1-9	-12.5	6.8	-1.0	1.9
LSBT-SSD-M1-10	-11.1	7.1	-2.7	3.2

Table 6: Roughness profile of the interface of LSBT-M1 series

Interfacial zone Series	Minimum (mm)	Maximum (mm)	Average (mm)	Stand. Dev. (mm)
LSBT-dry-M2-1	-11.3	13.0	1.1	3.9
LSBT-dry-M2-2	-16.9	2.0	6.2	4.3
LSBT-dry-M2-3	-22.7	4.0	8.7	6.0
LSBT-dry-M2-4	-14.7	5.0	6.5	3.5
LSBT-dry-M2-5	-14.8	7.0	3.6	3.6
LSBT-dry-M2-6	-11.5	8.0	2.3	4.5
LSBT-dry-M2-7	-10.4	6.0	0.7	3.2
LSBT-dry-M2-8	-12.5	9.0	0.3	4.3
LSBT-dry-M2-9	-15.5	7.0	3.4	4.4
LSBT-dry-M2-10	-7.8	9.0	0.1	2.7
LSBT-75-M2-1	-7.6	8.0	1.0	2.7
LSBT-75-M2-2	-13.2	6.0	2.5	4.0
LSBT-75-M2-3	-14.6	5.0	3.5	4.1
LSBT-75-M2-4	-10.3	7.0	2.2	3.4
LSBT-75-M2-5	-16.2	10.0	3.3	5.0
LSBT-75-M2-6	-7.9	7.0	1.6	2.6
LSBT-75-M2-7	-13.2	6.0	2.2	3.5
LSBT-75-M2-8	-19.0	7.0	6.6	4.2
LSBT-75-M2-9	-15.6	9.0	5.1	5.0
LSBT-75-M2-10	-17.2	7.0	3.5	3.9
LSBT-SSD-M2-1	-20.6	6.0	3.9	4.2
LSBT-SSD-M2-2	-11.2	7.0	3.5	2.9
LSBT-SSD-M2-3	-9.4	4.0	2.5	2.6
LSBT-SSD-M2-4	-13.9	8.0	2.6	4.0
LSBT-SSD-M2-5	-19.3	9.0	6.2	4.4
LSBT-SSD-M2-6	-10.8	8.0	1.7	3.8
LSBT-SSD-M2-7	-10.7	4.0	2.7	2.7
LSBT-SSD-M2-8	-9.2	7.0	0.4	2.7
LSBT-SSD-M2-9	-10.4	10.0	0.5	3.7
LSBT-SSD-M2-10	-12.4	9.0	0.1	3.9

Table 7: Roughness profile of the interface of LSBT-M2 series

Interfacial zone Series	Minimum (mm)	Maximum (mm)	Average (mm)	Stand. Dev. (mm)
DSBT-dry-M1-1	-14.2	10.0	-3.2	3.0
DSBT-dry-M1-2	-9.1	11.7	1.7	3.0
DSBT-dry-M1-3	-2.9	12.3	6.6	2.4
DSBT-dry-M1-4	-4.8	15.7	6.5	3.3
DSBT-dry-M1-5	-11.8	12.4	2.2	4.0
DSBT-dry-M1-6	-6.1	13.2	3.8	3.7
DSBT-dry-M1-7	-8.5	15.3	7.2	3.7
DSBT-dry-M1-8	-4.5	17.0	6.5	3.4
DSBT-dry-M1-9	-13.4	12.6	2.6	4.9
DSBT-75-M1-1	0.0	19.6	10.0	3.5
DSBT-75-M1-2	0.2	16.0	9.7	3.1
DSBT-75-M1-3	-0.2	12.6	6.3	2.2
DSBT-75-M1-4	-14.4	46.8	1.2	4.1
DSBT-75-M1-5	-5.9	21.3	12.6	4.0
DSBT-75-M1-6	-5.3	18.2	13.1	3.4
DSBT-75-M1-7	-7.6	16.4	10.7	3.2
DSBT-75-M1-8	-5.7	47.5	5.2	3.9
DSBT-75-M1-9	-38.0	19.1	10.3	6.4
DSBT-SSD-M1-1	-0.6	16.5	7.3	2.7
DSBT-SSD-M1-2	-1.4	17.1	8.9	3.4
DSBT-SSD-M1-3	0.1	24.3	11.7	3.4
DSBT-SSD-M1-4	-6.1	24.1	11.1	4.5
DSBT-SSD-M1-5	-5.9	19.2	6.0	3.0
DSBT-SSD-M1-6	-5.3	17.9	10.4	3.3
DSBT-SSD-M1-7	-7.6	23.8	13.7	4.0
DSBT-SSD-M1-8	-4.4	22.4	12.6	4.6
DSBT-SSD-M1-9	-12.2	16.7	3.6	3.1
DTBT-dry-M1-1	-6.9	15.1	6.3	3.7
DTBT-dry-M1-2	-6.7	16.0	10.1	2.2
DTBT-dry-M1-3	-3.2	17.3	9.4	3.1
DTBT-75-M1-1	-31.2	19.0	11.9	5.2
DTBT-75-M1-2	-17.0	20.3	12.5	3.1
DTBT-75-M1-3	-50.8	50.2	8.8	4.6
DTBT-SSD-M1-1	-7.6	17.5	5.9	4.4
DTBT-SSD-M1-2	-7.4	17.3	6.5	4.8
DTBT-SSD-M1-3	-11.9	23.6	9.6	5.2

Table 8: Roughness profile of the interface of both DSBT-M1 and DTBT-M1 series

Interfacial zone Series	Minimum (mm)	Maximum (mm)	Average (mm)	Stand. Dev. (mm)
DSBT-dry-M2-1	-9.4	8.2	-2.9	4.8
DSBT-dry-M2-2	-10.0	9.8	1.2	3.3
DSBT-dry-M2-3	-9.4	10.7	2.5	3.7
DSBT-dry-M2-4	-8.4	47.1	3.4	5.2
DSBT-dry-M2-5	-10.8	44.4	0.1	4.0
DSBT-dry-M2-6	-7.4	9.7	1.7	3.3
DSBT-dry-M2-7	-8.8	7.5	-0.4	3.4
DSBT-dry-M2-8	75.4	6.3	-7.0	10.0
DSBT-dry-M2-9	75.9	-2.0	-14.5	12.4
DSBT-75-M2-1	-23.4	5.0	-7.6	6.6
DSBT-75-M2-2	-13.5	15.4	-0.5	3.5
DSBT-75-M2-3	-4.5	11.2	3.5	3.0
DSBT-75-M2-4	-1.6	47.4	9.0	7.6
DSBT-75-M2-5	-1.8	47.0	9.2	6.5
DSBT-75-M2-6	0.2	18.3	6.9	3.3
DSBT-75-M2-7	-7.7	14.2	0.6	3.9
DSBT-75-M2-8	-16.8	6.4	-5.4	3.7
DSBT-75-M2-9	-16.5	39.0	-5.5	2.7
DSBT-SSD-M2-1	-6.0	9.8	1.6	3.0
DSBT-SSD-M2-2	-16.1	12.3	-1.8	5.5
DSBT-SSD-M2-3	-8.0	8.0	-0.6	3.3
DSBT-SSD-M2-4	-20.7	9.8	-2.6	4.6
DSBT-SSD-M2-5	7.5	7.2	-6.1	1.5
DSBT-SSD-M2-6	-10.7	6.7	-3.1	3.3
DSBT-SSD-M2-7	-13.9	7.4	-3.5	4.0
DSBT-SSD-M2-8	-7.6	15.6	3.1	3.3
DSBT-SSD-M2-9	-21.3	3.9	-7.5	3.6
DTBT-dry-M2-1	-27.6	1.5	-9.1	3.4
DTBT-dry-M2-2	-19.8	6.3	-7.8	4.6
DTBT-dry-M2-3	-28.7	40.4	-6.0	7.4
DTBT-75-M2-1	-14.2	5.9	-3.6	3.0
DTBT-75-M2-2	-15.3	9.6	-1.8	3.6
DTBT-75-M2-3	-11.7	46.6	0.6	7.9
DTBT-SSD-M2-1	-31.7	0.8	-13.7	4.8
DTBT-SSD-M2-2	-33.6	0.6	-8.0	6.3
DTBT-SSD-M2-3	7.5	4.6	-5.5	0.8

Table 9: Roughness profile of the interface of both DSBT-M2 and DTBT-M2 series

Interfacial zone Series	Minimum (mm)	Maximum (mm)	Average (mm)	Stand. Dev. (mm)
TSBT-dry-M1-1	-7.1	10.7	0.7	2.8
TSBT-dry-M1-2	-7.8	6.6	0.5	2.9
TSBT-dry-M1-3	-7.3	9.0	-1.0	2.6
TSBT-dry-M1-4	-9.2	9.9	2.2	2.6
TSBT-dry-M1-5	-6.1	11.3	2.6	2.9
TSBT-dry-M1-6	-4.9	10.7	1.7	3.0
TSBT-dry-M1-7	-3.9	8.5	2.4	2.3
TSBT-dry-M1-8	-3.3	11.5	2.9	2.5
TSBT-dry-M1-9	-4.4	5.7	1.5	1.7
TSBT-dry-M1-10	-3.9	7.2	1.8	1.8
TSBT-75-M1-1	-12.0	7.5	-2.5	3.3
TSBT-75-M1-2	-16.3	9.5	-3.1	4.4
TSBT-75-M1-3	-16.2	3.4	-2.4	2.6
TSBT-75-M1-4	-5.5	7.0	1.0	2.1
TSBT-75-M1-5	-16.6	1.2	-7.5	3.6
TSBT-75-M1-6	-13.9	5.2	-3.4	3.9
TSBT-75-M1-7	-13.7	5.3	-2.8	3.4
TSBT-75-M1-8	-8.6	14.6	1.7	3.9
TSBT-75-M1-9	-12.2	5.0	-1.8	2.7
TSBT-75-M1-10	-11.2	4.3	-3.3	3.2
TSBT-SSD-M1-1	-8.6	6.3	-0.6	2.1
TSBT-SSD-M1-2	-6.4	4.1	-0.7	1.6
TSBT-SSD-M1-3	-7.9	9.1	-1.1	2.5
TSBT-SSD-M1-4	-10.1	3.4	-2.8	2.1
TSBT-SSD-M1-5	-12.4	3.4	-4.1	2.3
TSBT-SSD-M1-6	-8.8	2.9	-4.0	1.9
TSBT-SSD-M1-7	-8.8	2.9	-4.0	1.9
TSBT-SSD-M1-8	-14.4	-0.6	-6.3	2.3
TSBT-SSD-M1-9	-11.4	0.7	-5.9	2.2
TSBT-SSD-M1-10	-10.4	4.1	-4.5	2.5

Table 10: Roughness profile of the interface of TSBT-M1 series

Interfacial zone Series	Minimum (mm)	Maximum (mm)	Average (mm)	Stand. Dev. (mm)
TSBT-dry-M2-1	-10.9	7.1	0.2	3.1
TSBT-dry-M2-2	-14.3	4.0	6.4	3.2
TSBT-dry-M2-3	-13.7	6.5	4.1	3.8
TSBT-dry-M2-4	-13.6	5.0	2.8	3.9
TSBT-dry-M2-5	-12.7	6.5	3.2	3.2
TSBT-dry-M2-6	-13.1	3.6	4.3	3.2
TSBT-dry-M2-7	-15.5	1.9	6.9	3.0
TSBT-dry-M2-8	-14.9	6.0	5.7	3.2
TSBT-dry-M2-9	-10.8	4.0	3.7	2.8
TSBT-dry-M2-10	-8.8	5.6	2.6	2.9
TSBT-75-M2-1	-13.6	5.9	3.6	3.3
TSBT-75-M2-2	-11.2	6.5	2.0	3.0
TSBT-75-M2-3	-12.8	6.4	3.1	3.1
TSBT-75-M2-4	-12.7	11.8	2.0	4.9
TSBT-75-M2-5	-7.0	5.4	0.8	2.1
TSBT-75-M2-6	-12.2	4.2	5.7	2.8
TSBT-75-M2-7	-8.5	7.9	1.9	2.8
TSBT-75-M2-8	-11.0	7.6	3.1	3.0
TSBT-75-M2-9	-13.9	4.5	5.8	3.9
TSBT-75-M2-10	-12.3	6.7	2.2	3.1
TSBT-SSD-M2-1	-11.2	9.2	1.3	3.9
TSBT-SSD-M2-2	-6.1	11.4	1.6	3.0
TSBT-SSD-M2-3	-11.0	7.7	2.4	2.9
TSBT-SSD-M2-4	-17.5	6.1	4.4	4.3
TSBT-SSD-M2-5	-3.8	11.3	3.7	2.7
TSBT-SSD-M2-6	-11.5	8.8	0.1	3.4
TSBT-SSD-M2-7	-7.7	14.6	0.8	3.9
TSBT-SSD-M2-8	-9.8	4.8	3.0	2.4
TSBT-SSD-M2-9	-17.0	9.0	1.3	6.1
TSBT-SSD-M2-10	-10.2	10.8	1.9	3.3

Table 11: Roughness profile of the interface of TSBT-M2 series

N. specimen	Speed test MPa/s	Average MPa/s	Peak value kN	Average kN	Stand. Dev. kN
POBT*-dry-M1-1	0.009		13.79		
POBT*-dry-M1-2	0.008		17.89		
POBT*-dry-M1-4	0.009	0.008	16.38	15.99	1.54
POBT*-dry-M1-9	0.009		15.29		
POBT*-dry-M1-10	0.007		16.60		
POBT*-75-M1-1	0.008		20.23		
POBT*-75-M1-2	0.007		18.18		
POBT*-75-M1-7	0.007	0.007	21.32	20.57	2.73
POBT*-75-M1-8	0.007		24.83		
POBT*-75-M1-9	0.007		18.29		
POBT*-SSD-M1-1	0.007		22.59		
POBT*-SSD-M1-2	0.009		22.16		
POBT*-SSD-M1-5	0.007	0.008	25.33	23.83	1.79
POBT*-SSD-M1-6	0.008		22.91		
POBT*-SSD-M1-10	0.007		26.16		
LSBT-dry-M1-3	0.010		19.67		
LSBT-dry-M1-4	0.009		19.67		
LSBT-dry-M1-6	0.008	0.009	21.45	20.01	3.27
LSBT-dry-M1-9	0.007		24.13		
LSBT-dry-M1-10	0.011		15.15		
LSBT-75-M1-2	0.014		18.14		
LSBT-75-M1-4	0.011		20.71		
LSBT-75-M1-8	0.009	0.012	18.98	17.63	2.58
LSBT-75-M1-9	0.012		16.39		
LSBT-75-M1-10	0.012		13.95		
LSBT-SSD-M1-1	0.009		22.32		
LSBT-SSD-M1-2	0.010		16.63		
LSBT-SSD-M1-3	0.010	0.010	17.38	19.41	2.82
LSBT-SSD-M1-6	0.008		22.56		
LSBT-SSD-M1-9	0.012		18.16		

Table 12: Experimental results of both POBT*-M1 and LSBT-M1 series

N. specimen	Speed test MPa/s	Average MPa/s	Peak value kN	Average kN	Stand. Dev. kN
POBT*-dry-M2-1	0.027		19.69		
POBT*-dry-M2-2	0.030		15.95		
POBT*-dry-M2-3	0.034	0.031	12.81	16.16	2.77
POBT*-dry-M2-4	0.055		18.05		
POBT*-dry-M2-5	0.024		14.31		
POBT*-75-M2-3	0.022		28.68		
POBT*-75-M2-4	0.012		21.80		
POBT*-75-M2-5	0.016	0.015	25.19	24.72	2.53
POBT*-75-M2-6	0.016		23.72		
POBT*-75-M2-7	0.013		24.20		
POBT*-SSD-M2-1	0.037		35.72		
POBT*-SSD-M2-2	0.018		33.34		
POBT*-SSD-M2-4	0.013	0.026	25.82	30.97	3.89
POBT*-SSD-M2-5	0.019		28.57		
POBT*-SSD-M2-7	0.015		31.42		
LSBT-dry-M2-2	0.022		28.14		
LSBT-dry-M2-4	0.018		23.97		
LSBT-dry-M2-6	0.019	0.031	21.07	24.25	3.32
LSBT-dry-M2-8	0.023		20.96		
LSBT-dry-M2-9	0.021		27.09		
LSBT-75-M2-1	0.014		28.68		
LSBT-75-M2-2	0.018		32.05		
LSBT-75-M2-3	0.021	0.031	30.41	27.67	4.52
LSBT-75-M2-4	0.022		26.81		
LSBT-75-M2-5	0.021		20.37		
LSBT-SSD-M2-2	0.042		35.89		
LSBT-SSD-M2-3	0.038		34.25		
LSBT-SSD-M2-4	0.041	0.031	38.21	37.35	2.38
LSBT-SSD-M2-7	0.041		37.95		
LSBT-SSD-M2-8	0.038		40.47		

Table 13: Experimental results of both POBT*-M2 and LSBT-M2 series

N. specimen	Speed test	Peak value	Average	Stand. Dev.
	mm/min	MPa	MPa	MPa
DSBT-dry-M1-2	0.10	7.47	7.09	0.68
DSBT-dry-M1-3		7.47		
DSBT-dry-M1-4		7.71		
DSBT-dry-M1-6		6.36		
DSBT-dry-M1-8		7.48		
DSBT-dry-M1-9		6.09		
DSBT-75-M1-3	0.10	4.72	5.57	0.55
DSBT-75-M1-4		6.19		
DSBT-75-M1-6		5.61		
DSBT-75-M1-7		5.85		
DSBT-75-M1-8		5.49		
DSBT-SSD-M1-1	0.10	5.75	6.77	0.87
DSBT-SSD-M1-2		6.75		
DSBT-SSD-M1-3		6.71		
DSBT-SSD-M1-4		8.29		
DSBT-SSD-M1-5		6.16		
DSBT-SSD-M1-6		6.96		
DTBT-SSD-M1-11	0.10	1.30	1.30	-
DTBT-75-M1-12	0.10	1.64	1.64	-
DTBT-dry-M1-11	0.10	1.49	1.49	-

Table 14: Experimental results of both DSBT-M1 and DTBT-M1 series

N. specimen	Speed test	Peak value	Average	Stand. Dev.
	mm/min	MPa	MPa	MPa
DSBT-dry-M2-1	0.10	5.06	6.34	1.59
DSBT-dry-M2-2		5.62		
DSBT-dry-M2-3		4.60		
DSBT-dry-M2-4		7.82		
DSBT-dry-M2-5		4.65		
DSBT-dry-M2-6		5.73		
DSBT-dry-M2-7		9.05		
DSBT-dry-M2-8		6.74		
DSBT-dry-M2-9		7.76		
DSBT-75-M2-1	0.10	11.32	7.64	1.79
DSBT-75-M2-2		7.46		
DSBT-75-M2-3		6.16		
DSBT-75-M2-4		5.23		
DSBT-75-M2-5		7.25		
DSBT-75-M2-6		7.97		
DSBT-75-M2-7		9.53		
DSBT-75-M2-8		9.69		
DSBT-75-M2-9		11.23		
DSBT-SSD-M2-1	0.10	9.94	8.14	1.78
DSBT-SSD-M2-2		4.92		
DSBT-SSD-M2-3		7.68		
DSBT-SSD-M2-4		7.83		
DSBT-SSD-M2-5		8.04		
DSBT-SSD-M2-6		7.69		
DSBT-SSD-M2-7		7.86		
DSBT-SSD-M2-8		11.48		
DSBT-SSD-M2-9		7.82		
DTBT-SSD-M2-10	0.10	3.24	3.24	-
DTBT-75-M2-10	0.10	3.40	3.40	-
DTBT-dry-M2-10	0.10	2.45	2.45	-

Table 15: Experimental results of both DSBT-M2 and DTBT-M2 series

N. specimen	Speed test	Average	LS value	PO value
	MPa/s	MPa/s	kN	kN
TSBT-dry-M1-1	0.010	0.005	24.13	0.00
TSBT-dry-M1-2	0.009		26.26	5.52
TSBT-dry-M1-3	0.008		20.18	10.23
TSBT-dry-M1-4	0.000		0.01	14.32
TSBT-dry-M1-5	0.000		0.00	20.05
TSBT-dry-M1-6	0.000		0.00	13.97
TSBT-dry-M1-7	0.003		4.12	14.74
TSBT-dry-M1-8	0.004		3.26	10.68
TSBT-dry-M1-9	0.009		14.88	6.02
TSBT-75-M1-1	0.013	0.004	37.57	0.00
TSBT-75-M1-2	0.003		9.13	8.99
TSBT-75-M1-3	0.004		10.29	12.31
TSBT-75-M1-4	0.003		8.15	18.42
TSBT-75-M1-5	0.000		0.00	23.06
TSBT-75-M1-6	0.000		0.00	18.06
TSBT-75-M1-7	0.000		0.01	14.65
TSBT-75-M1-8	0.003		5.60	12.88
TSBT-75-M1-9	0.004		9.99	8.66
TSBT-75-M1-10	0.013		40.47	0.00
TSBT-SSD-M1-1	0.012	0.005	35.45	0.00
TSBT-SSD-M1-2	0.008		15.16	7.15
TSBT-SSD-M1-3	0.006		15.99	11.92
TSBT-SSD-M1-4	0.003		6.38	17.92
TSBT-SSD-M1-5	0.000		0.00	21.82
TSBT-SSD-M1-6	0.000		0.00	19.13
TSBT-SSD-M1-7	0.000		0.04	17.46
TSBT-SSD-M1-8	0.007		27.12	13.34
TSBT-SSD-M1-9	0.010		36.32	7.75

Table 16: Experimental results of TSBT-M1 series

N. specimen	Speed test	Average	LS value	PO value
	MPa/s	MPa/s	kN	kN
TSBT-dry-M2-1	0.013	0.009	31.09	0.00
TSBT-dry-M2-2	0.009		0.00	15.82
TSBT-dry-M2-3	0.010		34.55	1.25
TSBT-dry-M2-4	-		-	-
TSBT-dry-M2-5	0.008		18.59	7.59
TSBT-dry-M2-6	0.012		30.18	0.00
TSBT-dry-M2-7	0.001		2.71	12.75
TSBT-dry-M2-8	0.007		17.26	3.95
TSBT-dry-M2-9	0.006		0.02	9.70
TSBT-dry-M2-10	0.012		17.14	8.10
TSBT-75-M2-1	0.010	0.004	36.42	0.01
TSBT-75-M2-2	0.007		28.86	3.79
TSBT-75-M2-3	0.007		29.09	6.70
TSBT-75-M2-4	0.002		4.55	13.13
TSBT-75-M2-5	0.000		0.00	11.31
TSBT-75-M2-6	-		-	-
TSBT-75-M2-7	0.007		21.45	3.90
TSBT-75-M2-8	0.004		15.11	8.32
TSBT-75-M2-9	0.002		3.99	15.94
TSBT-75-M2-10	0.000		0.00	21.37
TSBT-SSD-M2-1	0.011	0.005	37.52	0.00
TSBT-SSD-M2-2	0.007		24.07	4.51
TSBT-SSD-M2-3	0.005		14.10	8.33
TSBT-SSD-M2-4	0.002		5.08	13.45
TSBT-SSD-M2-5	0.000		0.01	16.36
TSBT-SSD-M2-7	0.011		39.52	3.74
TSBT-SSD-M2-8	0.007		25.51	8.54
TSBT-SSD-M2-9	0.005		16.59	12.42
TSBT-SSD-M2-10	0.000		0.01	15.61

Table 17: Experimental results of TSBT-M2 series

Series	a_q	b_q	c_q	a_τ	b_τ	c_τ	c
TSBT-dry-M1	-3	40	200	-65	100	130	1.54
TSBT-75-M1	-5.5	100	200	-40	100	130	1.75
TSBT-SSD-M1	-3.5	50	200	-440	125.5	130	1.81
TSBT-dry-M2	-15	150	200	-400	125.5	130	2.21
TSBT-75-M2	-12	120	200	-400	125.5	130	1.86
TSBT-SSD-M2	-27.5	150	200	-600	125.5	130	2.44

Table 18: Coefficient of the fitted cohesive curves in terms of mode I, mode II and their coupling factor

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