

Bond investigation of iron-based shape memory alloy bars embedded in UHPFRC – Comparison between testing methods

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ABSTRACT

Roughly 50 % of the existing bridges in Switzerland are now over 50 years old. Following a common trend, in most industrialised countries, many bridges are approaching or have reached their design lifespan. Thus, awareness towards retrofitting demands of existing bridges is increasing. In this context, a new prestressed strengthening technique is pro-posed, where a layer of Ultra-High Performance Fibre Reinforced Concrete (UHPFRC) reinforced with Iron-based Shape Memory Alloy (Fe-SMA) bars is cast on top of an existing bridge deck. Besides increasing the deck bending resistance under hogging moments, the innovative method can reduce transverse bending moments in the bridge webs and significantly improve the structure's serviceability thanks to the Fe-SMA prestressing capabilities. As part of the study of the proposed system, and to address the existing gap in knowledge on the bond behaviour between UHPFRC and Fe-SMA, two series of tests were conducted: (i) short pull-out tests (adapted RILEM tests) instrumented with Digital Image Correlation (DIC), and (ii) pull-out tests with longer embedment lengths instrumented with Fibre-Optical (FO) sensors and DIC. For comparison, two types of steel reinforcing bars (B500B and Fe-SMA) were tested. The main results from both pull-out test campaigns are discussed, highlighting the advantages and drawbacks of each of these approaches. The FO sensors used in the long pull-out tests provided new insights into the bond behaviour between UHPFRC and reinforcement steel. Post-processing of the long pull-out tests results revealed average peak bond shear stresses of 28 MPa and 23 MPa for B500B and Fe-SMA steel reinforcing bars, respectively. Conversely, the short pull-out tests yielded unrealistically high bond shear strength values, in some cases overestimating the maximum bond capacity by more than a factor of two. While the bond strengths observed in the long pull-out test align well with the *fib* Model Code predictions, they remained significantly lower than recommended in existing standards, such as the Swiss technical specification SIA 2052.

1. Introduction

1.1. Strengthening of existing structures

Many existing concrete structures have already exceeded, or are approaching, their design service life of typically 50 –100 years. Furthermore, design codes are occasionally revised to accommodate increasing load requirements and incorporate recent research advancements. As a result, many existing structures are significantly deteriorated and/or no longer comply with modern design codes,

requiring interventions. Developing environmentally sustainable and cost-effective strengthening solutions that can be readily applied to ageing structures is thus highly relevant, as it avoids demolition and minimises retrofitting measures with their associated environmental and economic impacts.

Commercially available strengthening techniques include the use of: (i) steel plates bolted or bonded to the parent structure; (ii) Externally Bonded (EB) Carbon-Fibre Reinforced Polymers (CFRP) [1]; and (iii) Near-Surface Mounted (NSM) techniques, where small strips are bonded in previously chiselled grooves on the concrete surface [1,2]. EB-CFRP

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has largely replaced the methods using steel plates due to the CFRP improved corrosion properties and its reduced weight. The NSM method exhibits better bond characteristics compared to EB-CFRP methods, owing to its multi-surface connection to the base-material and the confinement effects of the groove, though it requires substantial preparation of the concrete substrate. Applying any of the described strengthening methods to bridge decks subjected to hogging moments would require complete resurfacing work, including the waterproofing membrane and asphalt layer. Moreover, reduced concrete quality, such as that caused by poor compaction, could impair the bond performance thus diminishing the effectiveness of the intervention. For bridges, in particular, these techniques may result in considerable traffic disruptions. On the other hand, casting a top layer of Ultra-High Performance Fibre Reinforced Concrete (UHPFRC) reinforced with conventional ribbed steel bars offers simplicity, while significantly enhancing resistance and expediting the process [3,4]. In the latter, the application of the sealing membrane may, under certain conditions, be omitted.

All the previously referred solutions are passive, requiring additional structural deformation before developing a load-carrying contribution, and hence mainly activated under traffic loads or at Ultimate Limit States (ULS). On the other hand, the use of prestressed solutions enables to carry a part of the dead load, allowing to reduce transverse bending moments in shear-critical bridge webs [5], partially recover cracks on deteriorated elements or to uplift components exhibiting excessive deformations. Recent studies have shown the possibility of strengthening a concrete structure by installing prestressed CFRP sheets by means of mechanical anchorage and hydraulic systems installed at the sheet fixation [6]. NSM methods can additionally be used to further improve the bond behaviour.

1.2. Current investigation

A novel method for flexural strengthening of bridge decks subjected to hogging moments is currently under development [7]. The method builds upon the previously described technique of casting a top UHPFRC layer reinforced with conventional steel bars. However, the innovative method replaces the conventional steel bars with Iron-based Shape Memory Alloy (Fe-SMA) bars, profiting from the high strength capacity of advanced cementitious materials and the prestressing capabilities of smart alloy materials. Fig. 1a) shows a 3D view of the strengthening system and Fig. 1b) details the application procedure. According to the latter, the end anchorages and mid-surface are cast first (1) and, after hardening, the bars are heated with a direct heating source or with resistive heating (2). The recovery stress that builds up in the Fe-SMA bars is transmitted to the anchorages which must resist the induced stresses. Finally, the surface in-between the anchorages is cast (3). The two materials used in the new system are described below, and the benefits of their combination are outlined.

UHPFRC is a dense cementitious material characterised by enhanced tensile and compressive strengths compared to Normal strength Concrete (NC) [8,9]. By adding steel fibres to its matrix, ductility and fracture energy are greatly improved in comparison to Ultra-High

Performance Concrete (UHPC). When combined with reinforcement to form R-UHPFRC, the tensile resistance and deformation capacity are further increased, ensuring structural reliability and suitability for structural strengthening applications. Additionally, properties such as low permeability, strong bonding with the concrete substrate, and high freeze-thaw resistance make UHPFRC particularly interesting for bridge applications [10]. The dense UHPFRC matrix offers exceptional corrosion protection to the steel reinforcement, allowing for reduced cover requirements and the design of thin overlays, thus minimising the weight increase of the structure after strengthening.

Shape Memory Alloys (SMA) are a group of smart materials that have shown a great potential in several fields of application, including medicine, robotics, automotive, aerospace, and civil engineering [11–14]. Two major groups of SMA exist. After applying an initial deformation above the elastic limit – referred to as prestraining stage – they can either: (i) fully recover their initial shape by simply unloading the initial stress; or (ii) partially recover their initial shape by heating the material above the austenite start temperature A_s , followed by a cooling stage. The described effects result from phase transformations in the lattice structure of these materials, which are only possible due to the SMA's polymorphism, i.e., their ability to have more than one crystal structure. The first group of alloys include for instance Nickel-Titanium (Ni-Ti, Nitinol) alloys with "superelastic" or "pseudoelastic" properties. These are characterised by a reduced stiffness and high cost of production hence low interest for large scale civil engineering applications. The second group of alloys comprises Iron-based Shape Memory Alloys [15, 16]. The heating-cooling cycle is often referred to as activation. The expansion effects during the heating phase are overcome by the phase transformation from the martensite structure (hexagonal close-packed) to the austenite structure (face-centred cubic). During the cooling phase, the retraction phenomenon further increases the shape recovery. If restrained upon heating, these alloys develop internal stresses – often referred to as recovery stresses – which can be used for prestressing applications. These alloys have a special interest in civil engineering applications due to their higher stiffness when compared with Ni-Ti alloys and their lower cost [15–18]. For further insights into the mechanical behaviour of Fe-SMA, see [15–19].

Recent studies [20–22] have demonstrated the feasibility of strengthening concrete structures by either: (i) applying a top layer of low shrinkage mortar prestressed with Fe-SMA ribbed bars (cover replacement method CR), or (ii) installing Fe-SMA ribbed bars in grooves previously chiselled into the concrete substrate, followed by filling the grooves with mortar (NSM method). The use of smart alloys for prestressing applications is particularly attractive for on-site applications, as it eliminates the need for anchors heads, hydraulic jacks or additional mechanical devices. Using UHPFRC overlays instead of mortar promises superior performance for bridge applications, as UHPFRC exhibits significantly improved tensile and compressive strength, ensuring enhanced bond between the ribbed Fe-SMA bars and UHPFRC, while at the same time serving as waterproofing thanks to its low permeability.

Given the high costs of both UHPFRC and Fe-SMA, the proposed

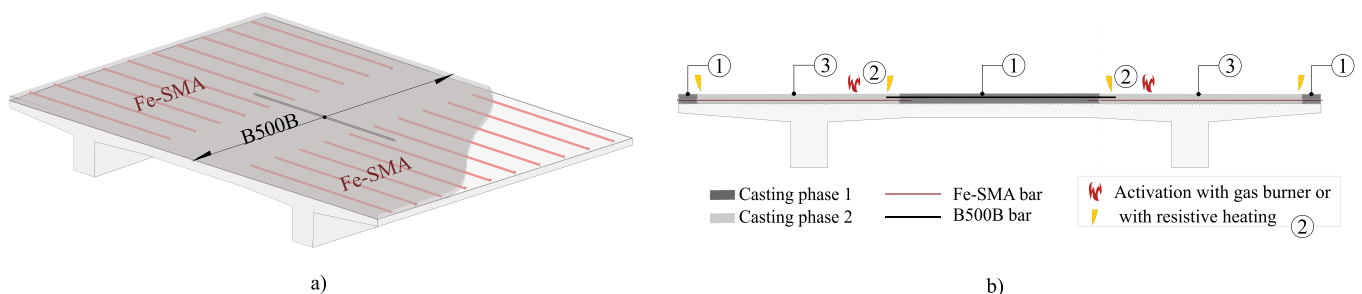


Fig. 1. Strengthening system: a) 3D view; b) application procedure.

system is only suited to a limited range of applications and relies on the synergy of combining the two materials to optimise their quantities, thus minimising intervention costs and ecological impact. Memory steel technology in civil engineering is still relatively recent, with limited industrial experience and production volumes far below typical steel, resulting in higher costs. With the widespread use of Fe-SMA, its unit cost is also expected to decrease, enabling more attractive market solutions.

As part of the study of the innovative strengthening solution, this paper investigates the bond interaction between Fe-SMA bars and UHPFRC, and compares the bond performance to that of conventional steel reinforcing bars embedded in UHPFRC.

1.3. Bond behaviour

Knowledge of bond behaviour is crucial for both Serviceability Limit States (SLS) and Ultimate Limit States (ULS) of Reinforced Concrete (RC) structures. Serviceability is impacted by parameters such as crack spacing, crack widths, and tension stiffening, which all depend on the bond interaction between reinforcing steel and concrete and are responsible for the stiffness degradation of structures. The performance of structures in the ULS can be governed by factors such as the end anchorage resistance of reinforcing bars and the deformation capacity of structural elements, particularly the rotation capacity of plastic hinges. Bond directly influences these factors, thereby affecting the strength and overall ductility of RC structures.

Short embedment length PO tests, recommended by various design standards [23,24] for their simplicity, have been widely used to characterise bond shear stress-slip relationships of reinforcing bars in both NC and UHPFRC [8,25–30]. However, their reliability remains a controversial topic of discussion in scientific research [27,31,32] despite their standardisation and reproducibility. Specifically, the RILEM-type PO test set-up introduces compressive stresses into the concrete specimens via contact with the machine's bearing plate, whereas shear stresses typically develop along with tensile stresses in real structures. Additionally, the specimen geometry often does not reflect the range of cover thicknesses found in real concrete structures. As a result, the actual confinement conditions of real structures are not well reproduced, leading to an overestimation of the bond strength [27,31]. Gross simplifications are also made to facilitate result processing, particularly regarding the assumptions of (i) bond shear stresses uniformly distributed along the nominal bar perimeter (rather than considering the effective force transfer occurring predominantly at the bar ribs) and (ii) constant nominal bond shear stresses and slip along the bonded length. To improve the plausibility of the latter assumption while minimising the influence of steel non-linearity on the bond performance, the embedment length is restricted. Very short embedment lengths can, however, lead to significant scatter in results due to localised effects [27, 31]. As a trade-off, current codes and provisions for normal strength concrete [23,24] recommend a bond length of five times the nominal bar diameter ($5d_s$). When studying the bond behaviour of bars embedded in high strength cementitious materials, such as UHPFRC, the expected bond capacity is higher. Thus, even shorter embedment lengths are often tested [8,29,30,33] to avoid failure by bar rupture, i.e., mobilise the full bond capacity and reach a pull-out type of failure mode.

Similarly, short PO tests have been employed to characterise the bond behaviour between Fe-SMA bars and NC or mortar [20,34,35]. In these applications, Fe-SMA was typically heated to 160–200 °C to activate the shape memory effect. When the bars are fully embedded in the concrete during activation, a substantial amount of thermal energy may be transferred to the surrounding matrix, depending on its thermal conductivity, inducing a non-uniform temperature profile in the concrete element.

Recent studies carried out to characterise the behaviour of UHPFRC under high temperature conditions did not report a decrease on the mechanical properties of UHPFRC under low to moderate temperatures (≤ 200 °C) [36–40], although the bond behaviour was not directly

assessed. Moreover, the thermal boundary conditions during Fe-SMA activation differ from those conducted in the referred studies, potentially leading to more severe localised effects. In fact, a reduction of approximately 20 % in bond resistance has already been observed at approximately 200 °C in both UHPFRC and concrete [7,34].

1.4. Research significance

Despite extensive research on bond behaviour in NC and UHPFRC, a knowledge gap exists regarding the bond interaction between Fe-SMA bars and UHPFRC. Moreover, while typical PO tests have been used for decades, a re-thinking of the methods employed to test bond behaviour is recommended, particularly in the case of UHPFRC. Not only because the commonly used RILEM tests require adaptations to avoid bar yielding, but also because this material is often used for strengthening purposes, where thin UHPFRC layers and small covers are typically employed.

The innovation of this study lies in reproducing the bond conditions as they occur in the actual strengthening system, i.e. R-UHPFRC applied to existing RC, and in showing that a system-oriented investigation of bond can yield significantly different results compared to conventional material-oriented PO tests of steel bars in concrete. In this context, the boundary and confinement conditions provided by the real system are considered to play an essential role in characterising the bond behaviour. This study is possible thanks to the increasing use of high-resolution and non-evasive measurement techniques in civil engineering, such as Distributed Fibre Optical Sensing (DFOS) [41–46], enabling to continuously measure strains along variable embedment lengths, capture bond shear stress profiles, and study phenomena such as bond deterioration and damage progression.

This paper presents and compares the results from an experimental campaign comprising two series of PO tests. The first series – referred to as short PO tests – resembled the RILEM PO test [24] typically used for characterising bond between reinforcing steel and NC. The geometry of the specimens was modified by further reducing the embedment length to account for the improved bond capacity expected in UHPFRC, thereby eliminating the influence of steel yielding on the results. In addition, some bars were eccentrically positioned to reproduce variations in the concrete cover, while staying within the ranges commonly used for UHPFRC overlays. The second series – here referred to as long PO tests – used longer embedment lengths and intended to simulate the actual strengthening method more realistically. The reinforcing steel bars in this series were instrumented with Distributed Fibre Optical Sensing (DFOS) technology, allowing high resolution strain measurements along the entire bonded length and a deeper insight into bond behaviour. The tests were carried out using the interrogator ODiSI-6102 supplied by Luna Innovations [47] and polyimide coated fibres for enhanced accuracy, as reported by [41,42,45].

2. Experimental investigations

2.1. Materials

2.1.1. UHPFRC

2.1.1.1. Mix composition. The recipe used for the UHPFRC mix

Table 1
Mix composition per m³ of UHPFRC.

Components:		
Premix (incl. CEM type I)	2100	[kg]
Admixtures	31	[kg]
Steel fibres - $l_f = 12.5$ mm; $d_f = 0.175$ mm (3.8 % in volume)	300	[kg]
Water	175	[kg]
UHPFRC mix density	2606	[kg/m ³]

preparation was identical in both experimental campaigns. Table 1 summarises the detailed composition required to produce 1 m³ of UHPFRC. Two types of admixtures were incorporated into the mix preparation: Sika Viscocrete Tempo 10, a superplasticizer, to enhance the workability of the mix without increasing its water content; and Sika Set 2, an accelerator, to improve the early-age strength development.

2.1.1.2. Compressive strength. Fig. 2 and Table 2 present the compressive strength development of the UHPFRC, determined on 100 mm cubes. The three mixes used for the short (M0) and long pull-out tests (M1, M2) are identified by different colours in Fig. 2a). Fig. 2b) provides a logarithmic least-squares regression fitting the experimental results, illustrating the development of the average compressive strength over time, $f_{Ucm}(t)$, relative to the compressive strength at 28 days, $f_{Ucm,28}$. Based on this regression, after 3 and 8 days, the mixes reach approximately 76 % and 86 % of the compressive strength at 28 days, respectively. Testing was carried out in accordance with the Swiss standard SN EN 12390 [48–50].

2.1.1.3. Tensile strength and elastic modulus. The tensile strength was calculated through inverse analysis of four-point bending tests performed on six UHPFRC plates in accordance with SIA 2052 [50], Appendix E. The results are presented as average values from the six specimens, with standard deviations given in parentheses. The average maximum tensile strength was $\bar{f}_{Utu} = 14.4$ MPa (+/- 0.1 MPa) registered at a strain of $\bar{\epsilon}_{Utu} = 2.4$ ‰ (+/- 0.91 ‰), while the elastic limit was $\bar{f}_{Ute} = 13.0$ MPa (+/- 0.5 MPa) obtained at a strain of $\bar{\epsilon}_{Ute} = 0.24$ ‰ (+/- 0.01 ‰). The corresponding Young's modulus was $\bar{E}_U = 54.3$ GPa (+/- 0.7 GPa).

Additional material characterization tests were carried out on six 40 × 40 × 160 mm prisms according to 12390-13:2013 [51], which resulted in an average Young's modulus of $\bar{E}_U = 52.7$ GPa (+/- 0.7 GPa).

Both the plate and prism tests were performed for mixture M0 at 28 days. The close agreement between the two Young's modulus values confirms the reliability of the plate-test results.

2.1.1.4. Shrinkage. In Fig. 3, the measured UHPFRC shrinkage is compared to the values predicted by SIA 2052 [50]. The results show good agreement, with measurements falling within the minimum and maximum value ranges of the Swiss technical specification, except for two outliers slightly outside the given range.

2.1.2. Steel reinforcement

The pull-out tests were conducted using bars from two types of reinforcing steel: hot-rolled B500B ribbed bars obtained from coils produced in Switzerland, and straight-length ribbed Fe-SMA (non-prestrained) bars supplied by the company re-fer AG. The manufacturer of the B500B reinforcing bars was identified considering the numerical

system based on rib indentation marks in accordance to SIA 262 – reinforcing steel register [52]. Tensile tests were carried in accordance to ISO 6892-1 [53] for material characterisation. The main geometrical properties of the steel bars used in the tests, defined based on ISO EN 15630-1 [54], and corresponding mechanical properties are summarised in Table 3.

The two experimental campaigns employed Fe-SMA bars with different initial mechanical states. In the short PO tests, the bars were not prestrained, as the primary objective was to examine the influence of heating without triggering activation. In contrast, the bars used in the long PO tests were all prestrained to 4 % to investigate the influence of activation. Fig. 4a) presents the stress–strain curves from tensile tests on three non-prestrained Fe-SMA and three B500B ribbed bars, while Fig. 4b) schematises the distinct stress–strain behaviour of the Fe-SMA bars used in both test campaigns.

2.2. Short pull-out test campaign

2.2.1. Experimental programme

A series of 14 pull-out tests on reinforcing bars embedded in UHPFRC cubes, with short bonded lengths (adapted from [24]), was carried out. Fig. 5 illustrates the test set-up and specimen geometry. The specimens differed in the type of reinforcing steel (non-prestrained Fe-SMA, B500B) and the cover thickness of the steel bar (24 mm, 44 mm, 67 mm), see Fig. 5c); the three different covers aimed at representing three thicknesses of the UHPFRC layer (40 mm, 60 mm and 80 mm) and are denoted as configuration A, B and C. As a further experimental parameter, the influence of heating on the bond resistance was investigated. The heating procedure involved raising the temperature of the as-received (non-prestrained) Fe-SMA bars to ranges typically used for activating this alloy, deemed compatible with concrete applications. Additionally, the influence of the orientation of the steel fibres within the UHPFRC matrix was studied by changing the casting direction and thus the flow of the cementitious material, e.g., parallel (//) to the reinforcing bar axis or perpendicular (⊥) to the latter. Table 4 summarises the experimental programme.

To avoid failure by rupture of the reinforcing bars and capture the bond behaviour without the influence of large plastic bar deformations, the anticipated stresses in the reinforcing bars at bond failure were limited to roughly 60 % of the yield strength of B500B steel, i.e. $\sigma_s = 300$ MPa. Previous studies on the bond interaction between conventional B500B steel reinforcing bars and UHPFRC [8,28–30,33] reported maximum bond shear stresses, τ_b , ranging from 45 to 65 MPa, and the Swiss technical specification SIA 2052 provides indicative values of 35–45 MPa [50]. For the initial design calculations, a value of $\tau_b = 50$ MPa was thus assumed. Based on this assumption, the targeted embedment length is obtained as follows:

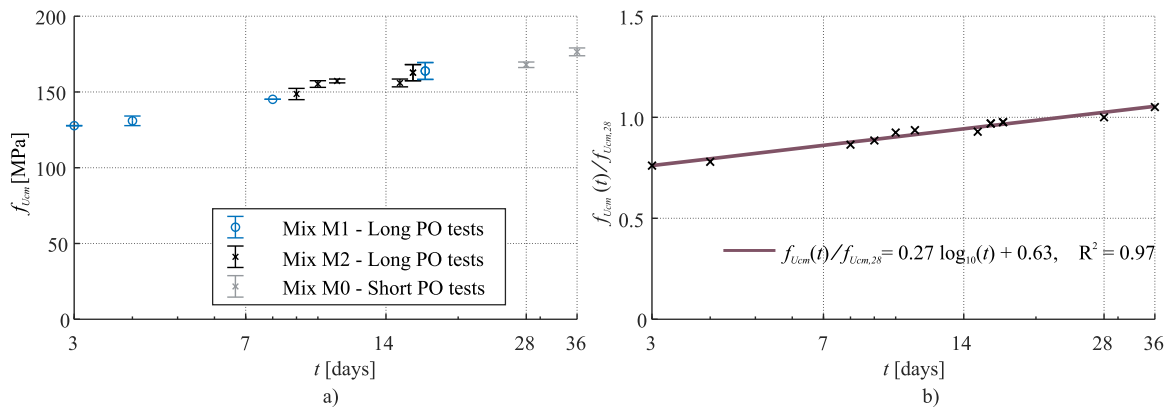
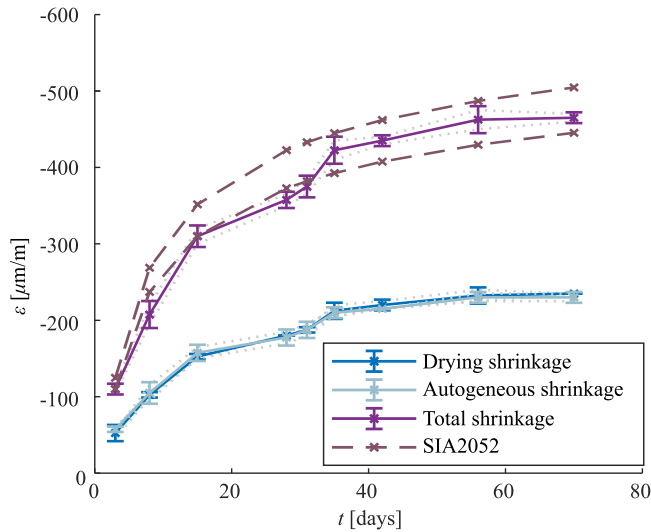


Fig. 2. Compressive strength development of the UHPFRC mixes used in the PO tests.

Table 2

UHPFRC mixtures – compressive strength and density.

Mix	Long PO tests									Short PO tests	
	M1	M1	M1	M2	M2	M2	M2	M2	M1	M0	M0
Age [days]	3	4	8	9	10	11	15	16	17	28	36
No. cubes	2	2	1	2	2	2	2	2	2	4	4
f_{ucm} [MPa]	127.8	131.0	145.2	148.7	155.3	157.3	156.0	162.8	163.9	168.0	176.5
Std (f_{ucm}) [MPa]	0.1	3.2	-	3.7	2.2	1.3	2.6	5.3	5.6	1.8	2.6
ρ [kg/m ³]	2549	2549	2532	2543	2545	2545	2511	2530	2544	2571	2540
Std (ρ) [kg/m ³]	1	3	-	7	4	3	4	3	4	6	12

**Fig. 3.** Shrinkage of the UHPFRC mixture.**Table 3**

Geometrical and material properties of the reinforcing bars.

Steel reinforcement geometry (nomenclature according to ISO 155630-1 [54])		
Steel type	Fe-SMA (not prestrained)	B500B
Nominal diameter, d_s [mm]	16.8	15.9
Rib height 1/4, $a_{1/4}$ [mm]	0.47 (+/- 0.05)	0.69 (+/- 0.10)
Rib height 1/2, $a_{1/2}$ [mm]	0.61 (+/- 0.02)	0.95 (+/- 0.06)
Rib height 3/4, $a_{3/4}$ [mm]	0.52 (+/- 0.05)	0.78 (+/- 0.10)
Rib spacing, c [mm]	7.3 (+/- 0.08)	9.0 (+/- 0.05)
Relative rib area, f_R^*	0.055	0.063
Steel reinforcement properties		
Young Modulus, E_s [GPa]	165 (+/- 15.0)	209 (+/- 2.0)
Tensile strength, R_m [MPa]	754.6 (+/- 4.9)	603.3 (+/- 2.4)
Ultimate strain, ϵ_u [%]	29.4 (+/- 5)	8.0 (+/- 1.5)

* in acc. to the Simpson's formula

$$F_b^{RILEM} = \pi \cdot d_s \cdot l_b \cdot \tau_b < \frac{\pi \cdot d_s^2}{4} \sigma_s \quad (1)$$

$$\rightarrow l_b < \frac{d_s \cdot \sigma_s}{4 \cdot \tau_b} = \frac{d_s \cdot 300 \text{ MPa}}{4 \cdot 50 \text{ MPa}} = 1.5 \cdot d_s$$

where τ_b denotes the uniform bond shear stress, d_s the reinforcing bar nominal diameter, l_b the embedment length and σ_s the stresses in the reinforcing steel.

Since reinforcing bars with a diameter of about 16 mm were used for all tests, an embedment length of 25 mm corresponding to roughly $1.5d_s$ was thus chosen for all specimens. Furthermore, the size of the adapted Rilem cubes was reduced to 150 mm, ensuring the bar in configuration A is aligned with the centre of the cube. Along the unbonded lengths, the bars were encased in a plastic sleeve.

2.2.2. Specimen preparation

After casting, all specimens were stored in a climate chamber at 20 °C and 90 % relative humidity (RH). After 24 h, the cubes were demoulded and stored under identical atmospheric conditions (20 °C, RH = 90 %). Prior to testing, the surface of the specimens was manually smoothed using an abrasive stone and a speckle pattern for DIC measurements was applied to 8 of the specimens, see Fig. 6a) and b).

Resistive (i.e., electrically induced) heating was used to heat the memory steel bars of Specimens S11 to S14 (see Fig. 6c)) to a target temperature of 200 °C, followed by a cooling period to ensure the specimens cooled down to room temperature before testing. A current intensity of approximately 500 A and a voltage of 2–4 V were used to heat the bars within a sufficiently short time while simultaneously being able to control the temperature and manually stop the process once the target temperature was reached. Heating times varied between 6.5 min and 9.7 min. Fig. 7 depicts the heating/cooling curve for Specimen S14, along with the heating times and maximum temperatures recorded for the four heated specimens. Temperature measurements were taken by two thermocouples installed along the embedment length, on opposite sides. In cases where one of the thermocouples was not electrically isolated, temperature readings were taken using only one thermocouple as the second thermocouple could only be connected after the electric current was switched off. This explains why the maximum temperature recorded by one thermocouple was sometimes higher than the target temperature of 200 °C. It is important to note that the Fe-SMA bars were not prestrained prior to heating, ensuring that solely the effects of heating were investigated, with no actual activation taking place.

2.2.3. Test set-up and instrumentation

The tests were conducted using a servo-hydraulic testing machine (Amsler-type). The specimens were positioned on the machine's traverse, the steel bar was clamped at the bottom and the load was applied in displacement control by moving the machine's crossbeam upwards at a rate of 0.008 mm/s. To avoid stress concentrations between the cube and the steel plate, a 3 mm rubber layer was placed in between. Digital Image Correlation (DIC) was used to capture splitting cracks on the UHPFRC surface for Specimens 7–14 (Configurations B and C, small concrete covers). Due to the eccentric position of the bars relative to the centre of the UHPFRC cubes in the configurations B and C, a beam was placed on top of the specimens to counterbalance the resulting residual moments.

A linear variable differential transformer (LVDT) with a measuring range up to 10 mm and an accuracy of +/- 0.01 mm was used to measure the slip at the unloaded bar end during the pull-out test. A load cell with a capacity of 200 kN recorded the applied load. A sampling frequency of 5 Hz was used.

2.2.4. Results and discussion

The nominal bond shear stress-slip relationships were derived based on the common assumptions in such tests, i.e., (i) bond shear stresses uniformly distributed along the nominal bar perimeter and (ii) constant nominal bond shear stress along the short embedment length:

$$\tau_b = \frac{F}{\pi \cdot d_s \cdot l_b} \quad (2)$$

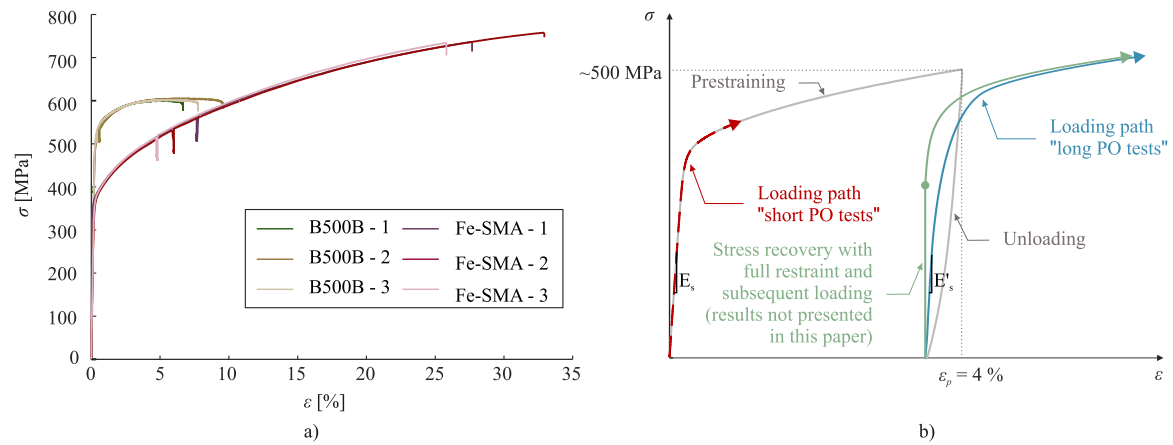


Fig. 4. Material characterisation of 16 mm ribbed reinforcing bars: a) Stress-strain diagrams from tensile tests on non-prestrained Fe-SMA and B500B bars; b) Behavioural differences of the Fe-SMA bars used in the experimental campaigns.

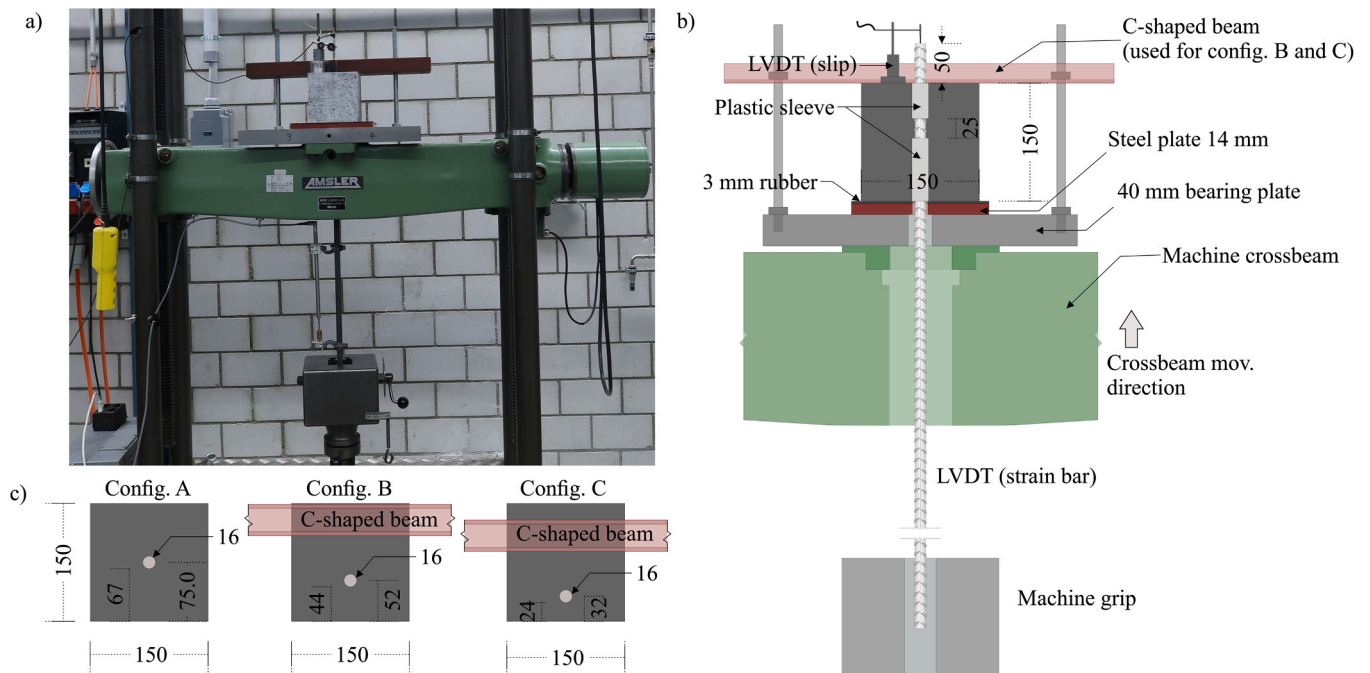


Fig. 5. Test setup: a) picture taken before testing Specimen S1; b) instrumentation used in the experiments; c) Specimen geometry for the configurations A, B and C. Dimensions in mm.

Table 4

Summary of the short embedment length PO tests.

Group	Specimen	Bar material	Configuration (cover [mm])	Heating	Casting direction
PO-1	S1-S2	B500B	A (67)	no	// to bar axis
PO-2	S3-S4	Fe-SMA	A (67)	no	⊥ to bar axis
PO-3	S5-S6	Fe-SMA	A (67)	no	// to bar axis
PO-4	S7-S8	Fe-SMA	B (44)	no	// to bar axis
PO-5	S9-S10	Fe-SMA	C (24)	no	// to bar axis
PO-6	S11-S12	Fe-SMA	B (44)	yes	// to bar axis
PO-7	S13-S14	Fe-SMA	C (24)	yes	// to bar axis

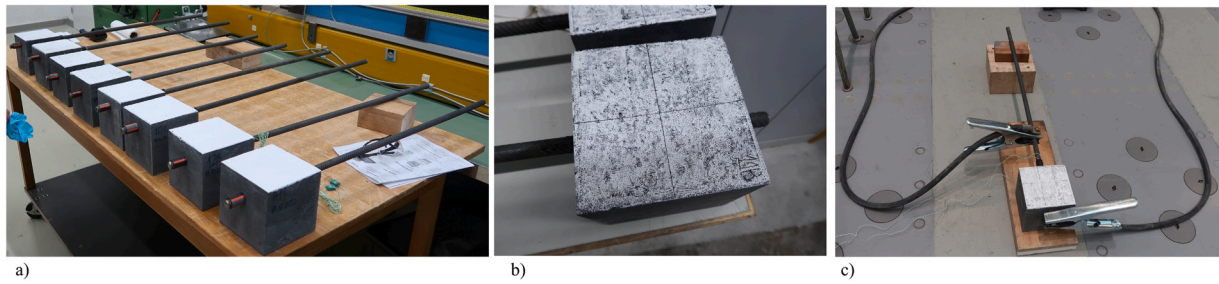


Fig. 6. Sample preparation for the short PO tests: a) preparing the surface of the UHPFRC cubes; b) applying the DIC pattern using the DIC speckling stamp; c) heating the Fe-SMA bars with resistive heating.

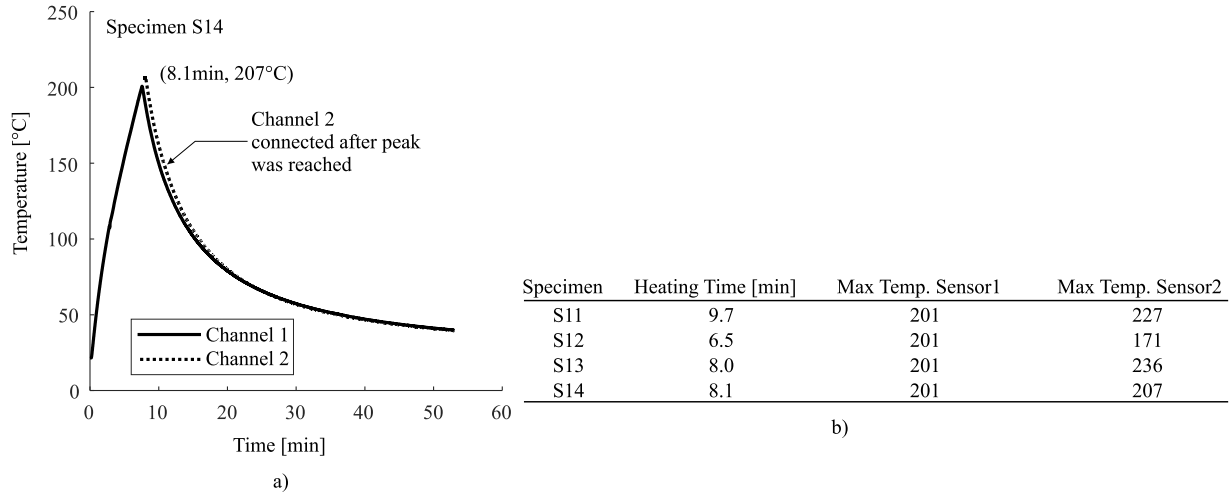


Fig. 7. Heating of Fe-SMA bars: a) exemplary temperature variation with time of the Fe-SMA bar (Specimen S14); b) heating times and maximum temperatures reached in specimens S11 to S14.

Table 5

Main results of short pull-out tests.

Group	Specimen	Age (days)	F_{max} [kN]	F_{max}^- [kN]	$\tau_{b, peak}$ [MPa]	$\tau_{b, peak}^-$ [MPa]	δ_{peak} [mm]	$\delta_{b, peak}^-$ [MPa]
PO1	S1	28	63.9		51.2		1.96	
	S2	28	57.0	60.4	45.6	48.4	1.42	1.69
PO2	S3	28	63.8		48.4		0.38	
	S4	28	70.7	67.3	53.6	51.0	0.47	0.42
PO3	S5	28	66.1		50.1		0.31	
	S6	28	66.3	66.2	50.3	50.2	0.38	0.34
PO4	S7	29	74.7		56.6		0.69	
	S8	29	69.2	72.0	52.4	54.5	0.69	0.69
PO5	S9	29	62.1		47.0		0.14	
	S10	29	63.2	62.6	47.9	47.5	0.31	0.22
PO6	S11	30	59.5		45.1		1.37	
	S12	30	55.7	57.6	42.2	43.6	0.56	0.96
PO7	S13	29	57.7		43.8		0.17	
	S14	30	63.8	60.8	48.4	46.1	0.32	0.25

where τ_b denotes the uniform bond shear stress, F the applied load, d_s the reinforcing bar nominal diameter and l_b the embedment length. Table 5 summarises the main results of all short pull-out tests, including the maximum load measured by the load cell (F_{max}) as well as the corresponding peak bond shear stress ($\tau_{b, peak}$) and slip at maximum load (δ_{peak}) for each specimen, as well as the mean value per group. The bond shear stress-slip relationships obtained for all specimens are presented in Fig. 8a). Fig. 8b) to g) highlight the influence of each experimental parameter on the bond behaviour.

The maximum bond shear stresses, $\tau_{b, peak}$, determined in accordance to (2), range from 42 to 57 MPa, see Fig. 8a), where the lower values were obtained in the specimens which underwent heating prior to

testing – Groups PO6 and PO7, see Fig. 8g). Even in these specimens, however, the bond shear stresses were consistently higher than predicted by the fib Model Code [55] for normal strength concrete without fibres. The bond shear stress-slip relationships according to the fib MC are also shown in Fig. 8a). The latter are based on the following bond stages:

$$\tau_b = \tau_{b, max} (s/s_1)^a \quad \text{for } 0 < s \leq s_1 \quad (3)$$

$$\tau_b = \tau_{b, max} \quad \text{for } s_1 \leq s \leq s_2 \quad (4)$$

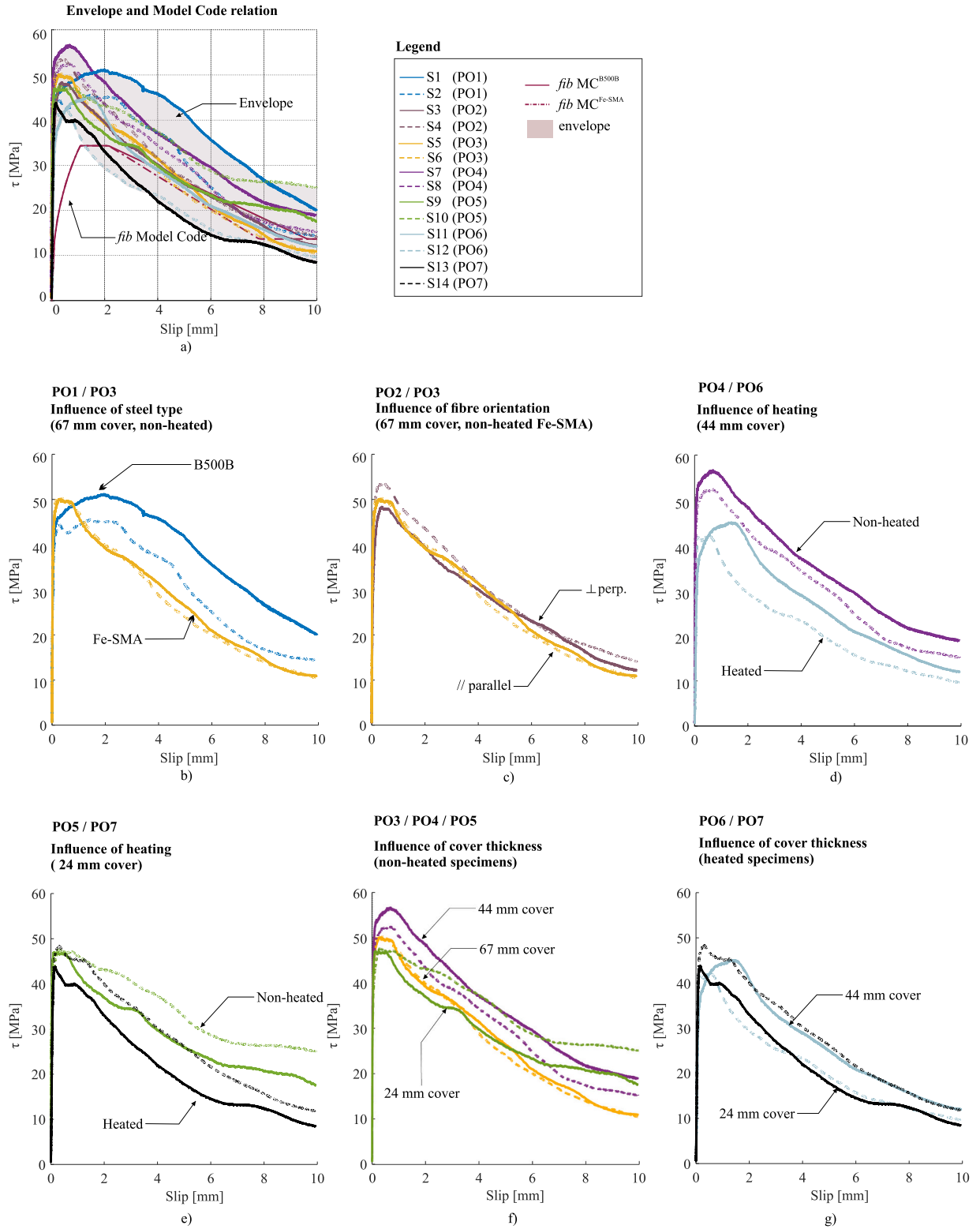


Fig. 8. Bond - slip relationships: a) S1 to S14 curves, envelope and comparisons to fib Model Code [55]; b) to g) comparisons between experimental curves based on different design parameters.

$$\tau_b = \tau_{b,max} - \frac{(\tau_{b,max} - \tau_{bf})(s - s_2)}{s_3 - s_2} \quad \text{for } s_2 \leq s \leq s_3 \quad (5)$$

where $\tau_{bf} = 0.4 \tau_{b,max}$

$$\tau_b = \tau_{bf} \quad \text{for } s > s_3 \quad (6)$$

where s_1, s_2, s_3 denote the boundaries of slip stages in Eqs. (3–6), $\tau_{b,max}$ is the peak bond shear stress calculated for a pull-out failure assuming good bond conditions, τ_{bf} is the frictional bond strength, and α is a fitting parameter. The following values were adopted following the fib MC recommendations: $s_1 = 1.0$ mm, $s_2 = 2.0$ mm, s_3 is taken as the clear distance between ribs (see c in Table 3), $\tau_{b,max} = 2.5\sqrt{f_{cm}}$ and $\alpha = 0.4$.

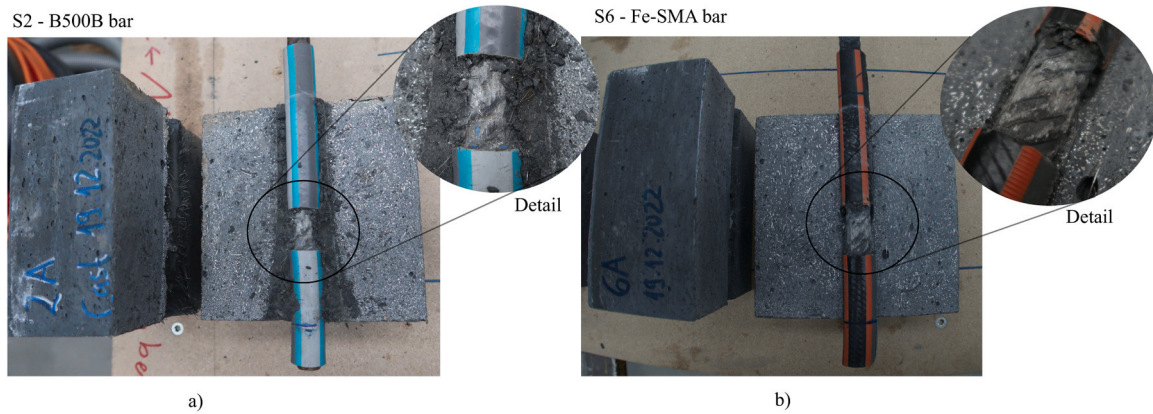


Fig. 9. Inspection of the bond interface in opened cubes, after testing: a) Specimen S2 and embedment region detail; b) Specimen S6 and embedment region detail.

The mean value of the UHPFRC cylinder compressive strength, f_{cm} , was determined from material tests on cubes by multiplying it by a factor of 0.95, as recommended by SIA 2052 [50]. The compressive strength at 28 days, $f_{cm,cube} = 168.0$ MPa (see Table 2), was considered.

The results are discussed in the following subsections, focusing on the key parameters.

Influence of the reinforcing steel type – Fig. 8b)

The maximum peak bond shear stress observed for the B500B bars belonging to Group PO1 ($\tau_{b,peak} = 51.2$ MPa) is slightly higher than that of the Fe-SMA bars in Group PO3. The higher bond strength is in line with the higher relative rib area, f_R , of the B500B bars and their slightly smaller bar diameter compared to the Fe-SMA bars. However, despite the generally lower stiffness and higher ductility of Fe-SMA (see Table 3), which would typically suggest a more ductile bond shear stress–slip response, the experimental results indicate reduced ductility for the Fe-SMA specimens.

Inspecting the bond interface by opening the specimens revealed that the results were affected by geometrical effects. As an example, Fig. 9a) shows the half cubes corresponding to Specimen S2, using B500B bars and Fig. 9b) those of Specimen S6, using Fe-SMA bars. On average, three ribs were found within the embedment length of the B500B samples, compared to four ribs in the specimens using Fe-SMA steel. The small embedment lengths adopted and the closer spacing of the Fe-SMA bar ribs favour the presence of an additional rib in these bars, causing higher nominal bond strengths, increased stiffness and reduced ductility. The visual inspection is in agreement with the rib spacing measured for each bar and presented in Table 3, being 9.0 mm for B500B bars and 7.3 mm for Fe-SMA bars. Fig. 10 illustrates the expected number of ribs within a 25 mm embedment length. These results and observations are compared to those of the long pull-out tests, where no localised effects are

expected, in Section 2.3.

Influence of the orientation of the steel fibres – Fig. 8c)

The bond shear stress–slip curves obtained for Groups PO2 (perpendicular) and PO3 (parallel) show a very similar behaviour. Given the three-dimensional geometry of the specimens, their reduced size, and the influence of the formwork walls, it was not possible to orient the steel fibres as intended, and they are assumed to be randomly distributed in all tests.

Influence of heating – Fig. 8d) and e)

The specimens belonging to Groups PO6 and PO7 (Specimens S11 to S14) were heated with a target temperature of 200 °C, although some registered higher values. After cooling to room temperature, the pull-out tests were carried out. Comparing the results to those of Groups PO4 and PO5 (Specimens S7 to S10), which were not heated but otherwise identical, allows investigating the influence of heating. As previously mentioned, DIC was used to detect the development of splitting cracks in all these specimens (see Fig. 11). Post-processing and image-correlation were carried out using the software VIC-3D 9 from Correlated Solutions Inc. [56] and the crack pattern and crack widths were evaluated with the Automated Crack Detection and Measurement (ACDM) software [57].

The specimens of Group PO6 (44 mm cover), see Fig. 8d), exhibited roughly a 20 % reduction of the peak bond shear stress compared to those of Group PO4 with the same cover, presumably caused by the heating of the Fe-SMA bars. Despite reduced stiffness, probably caused by the higher maximum bar temperature (227 °C), Specimen S11 achieved a higher peak bond shear stress than S12. Splitting cracks were detected on the surface of the UHPFRC cubes of Group PO6, see Fig. 11f) and g). The development of the crack presenting the maximum opening, referred to as governing crack, is illustrated in Fig. 11h), for both specimens 11 and 12. The position of the governing crack is identified in

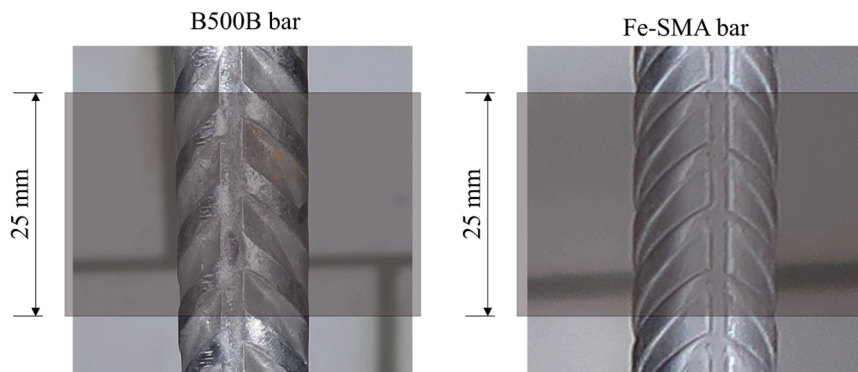
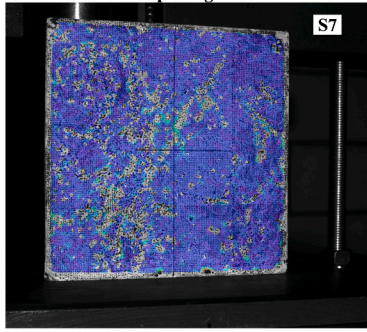


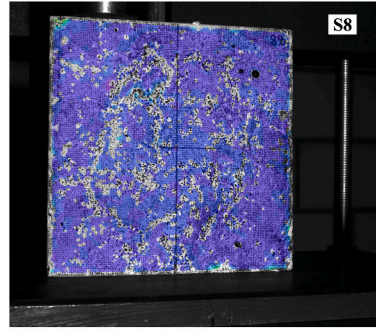
Fig. 10. Expected number of ribs along 25 mm.

PO4

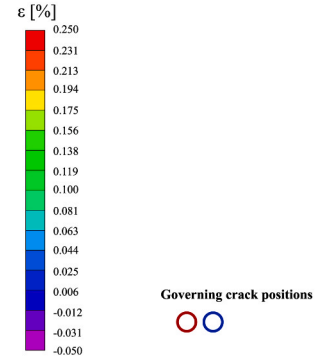
Properties: 44 mm cover; Fe-SMA (non-heated);
Observations: No splitting cracks



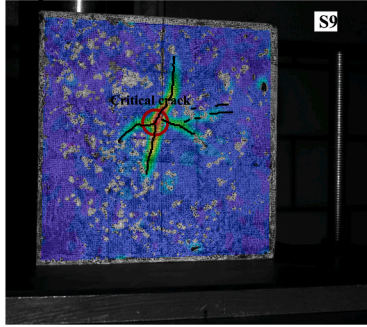
a)



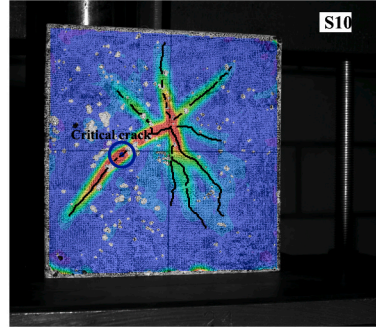
b)

**PO5**

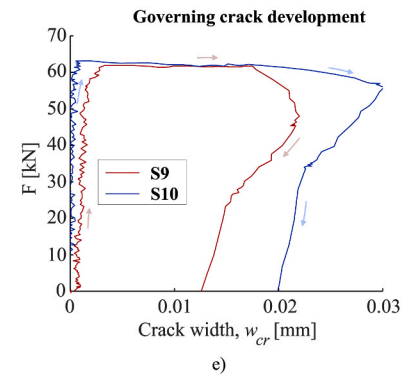
Properties: 24 mm cover; Fe-SMA (non-heated);



c)



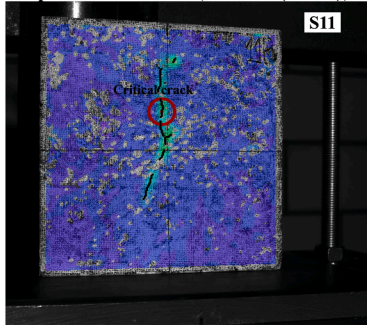
d)



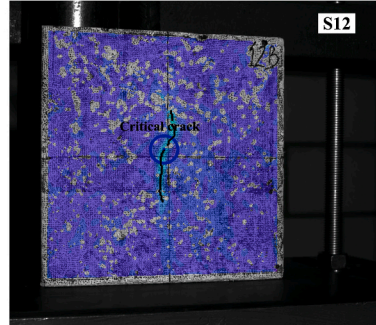
e)

PO6

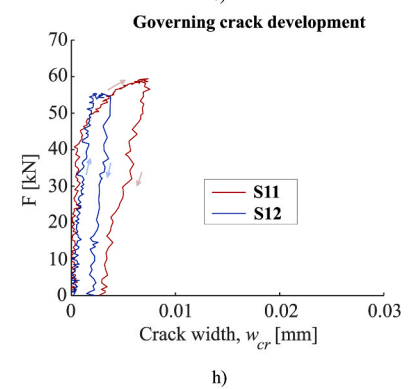
Properties: 44 mm cover; Fe-SMA (heated);



f)



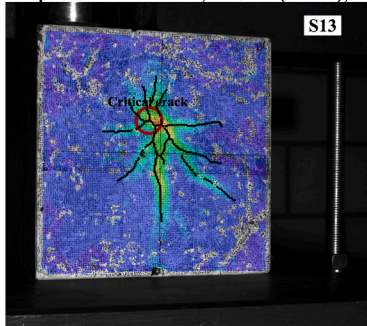
g)



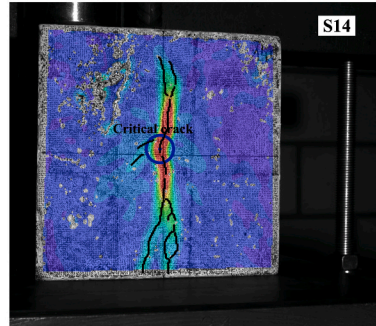
h)

PO7

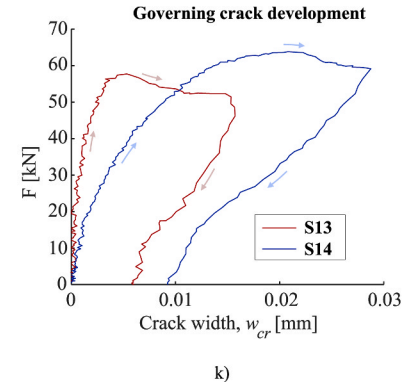
Properties: 24 mm cover; Fe-SMA (heated);



i)



j)



k)

Fig. 11. Strain and crack propagation measurements with DIC at the surface of UHPFRC: a-b) Specimens S7 and S8; c-d) Specimens S9 and S10; f-g) Specimens S11 and S12; i-j) Specimens S13 and S14; e, h, k) Governing crack development with load level. Note that the strain scale is not applicable at cracks.

the corresponding DIC images. No cracks were identified for the specimens of Group PO4 (Specimens S7 and S8), see Fig. 11a) and b), which explains the higher bond strength.

Contrary to the observations in Groups PO4 and PO6, no significant peak bond stress reduction was observed for the specimens with 24 mm

cover (Groups PO5 and PO7), see Fig. 8e), despite that the specimens in Group PO7 also underwent heating. Within this group, Specimen S13 exhibited the lowest peak bond shear stress, likely because the bar reached a maximum temperature of 236 °C, which may have induced additional damage at the bond interface. All 4 specimens exhibited

surface cracks, see Fig. 11c), d), i) and j), which can be attributed to the reduced cover. The governing crack development is illustrated for both groups in Fig. 11e) and k).

Influence of the cover thickness – Fig. 8f) and g)

When analysing the influence of the cover thicknesses in unheated specimens – Groups PO3 (67 mm cover), PO4 (44 mm cover) and PO5 (24 mm cover), illustrated in Fig. 8f) – the lowest peak bond strength was observed in the 24 mm cover specimens, which exhibited splitting cracks visible on the surface, see Fig. 11c) and d). Conversely, the residual bond strength – i.e., the remaining frictional resistance after failure – was higher for Group PO5 and reducing at a slower deterioration rate. The peak bond stress obtained for Group PO4 (44 mm cover) was higher than the values obtained for Group PO3 (67 mm cover), opposing the expected tendency, indicating that increasing the cover beyond the thickness to prevent splitting cracks has a minor effect on the bond strength, as it is governed by pull-out.

In the heated specimens, illustrated in Fig. 8g), – Groups PO6 (44 mm cover) and PO7 (24 mm cover) – no significant differences in the peak stresses were observed, which can be justified by the mixed failure mode in both configurations (pull-out and splitting of the UHPFRC cover).

The governing crack widths measured in the specimens with 24 mm cover (Groups PO5 = unheated and PO7 = heated) reached a maximum of approximately 0.03 mm, with a crack recovery of up to 70 % after failure. However, the load level at which these crack widths developed differed significantly: While the unheated specimens exhibited sudden crack formation at loads slightly above 60 kN, the cracks in the heated specimens developed gradually from the onset of loading. The latter suggests that damage had been caused by the heating procedure. A similar trend was observed for the heated specimens with 44 mm cover (Group PO6), although the surface crack widths were significantly smaller in this case.

2.2.5. Final remarks on the short PO tests

The well-established short PO tests are relatively simple to conduct and reproduce, allowing the comparison of a wide range of parameters such as the influence of cover thickness and heating on the bond behaviour, as analysed in this study. However, very short embedment lengths ($\sim 1.5d_s$) had to be used to avoid bar ruptures and minimise the influence of steel yielding on bond behaviour, which led to localised effects, similarly to what was reported in the literature [27]. This makes it difficult to interpret and compare results for different types of reinforcing steel bars.

The bond strengths obtained in this study are significantly higher than the design values recommended by the *fib* Model Code (MC) for normal strength concrete [42], suggesting that short embedment lengths can lead to overestimated bond resistances, as also noted in [31]. The observed values aligned slightly better with those provided in the Swiss technical specification SIA 2052 [50], ranging from 35 to 45 MPa for typical ribbed bars, though they remain above these indicative values.

Additional limitations of RILEM-type tests, such as the assumption of a constant bond shear stress along the bonded length, were previously mentioned. To further investigate the bond shear-slip relationship of reinforcing steel bars embedded in a UHPFRC layer while addressing these limitations, an additional test campaign with longer embedment lengths was conducted and is presented in the following section.

Table 6
Summary of the long pull-out tests – non-activated specimens.

Specimen	Layer thickness [mm]	Bar material	Casting direction	Hardening time [days]
L1, L2	60	B500B	// to bar axis	17
L3, L4	60	Fe-SMA	// to bar axis	16

2.3. Long pull-out tests

2.3.1. Experimental programme

The test programme comprised a total of 12 specimens and aimed at evaluating the effect of five parameters: (i) layer thickness, (ii) type of reinforcing steel (Fe-SMA, B500B), (iii) activation procedure, (iv) orientation of the steel fibres in the UHPFRC matrix and (v) UHPFRC hardening time. This paper focuses on the four non-activated specimens (see Table 6), allowing to compare the bond shear stress-slip relationships for the two types of steel bars and evaluate the suitability of different testing methods.

2.3.2. Specimen preparation

To overcome the limitations of short pull-out tests, the actual configuration of the envisaged strengthening system was reproduced by hydro-jetting with a roughness of 2–5 mm the top surface of 500 mm wide and 250 mm deep old concrete blocks and subsequently casting a 40–60 mm thick layer of UHPFRC (60 mm in all specimens considered in this paper), with a width of 250 mm and a length of 400 mm, after a reinforcing bar had been positioned with a clear spacing of approximately 5 mm above the roughened surface of the old concrete blocks, see Fig. 12a), b), d) and e). The embedment length, l_b , of 400 mm and part of the free length of the bar were instrumented with fibre optical sensors, enabling to extract strain profiles with high spatial and temporal resolutions, accuracy levels in the range of $\pm 25 \mu\epsilon$ and uncertainty within $\pm 5 \mu\epsilon$ [47]. Temperature fluctuations of approximately $\pm 2^\circ\text{C}$ were observed at the different measurement times, which could introduce additional strain measurement errors of up to $\pm 20 \mu\epsilon$. Polyimide coated fibres were positioned inside 1.6 mm by 1.6 mm grooves carved in two opposite sides of the bar, along the bar ridges (with minimum influence on the rib height), which were then filled with epoxy, see Fig. 12c).

The grooves along the reinforcing steel bars were machined using a planing process, minimising heat generation through lubrication with oil to prevent major changes to the steel's crystal lattice structure and to avoid triggering the activation of Fe-SMA bars, which had been prestrained up to 4 %. Prestraining was carried out prior to the machining process to prevent changes in the groove size caused by subsequent axial elongation. Fig. 13 illustrates the stress-strain curves obtained during prestraining of the Fe-SMA bars used in Specimens L3 and L4.

The compressive strengths of the old concrete blocks were obtained from concrete cores extracted from their bottom, and are summarised in Table 7.

2.3.3. Test set-up and instrumentation

After hardening of the concrete (see Table 6), the reinforcing bars were pulled from the UHPFRC layer while the concrete block was held in place, using the test set-up illustrated in Fig. 14. Displacements of the concrete block were restricted by two steel supports attached to a base plate fixed to the lab strong floor and a stiff steel beam, providing a linear support to the bottom 85 mm of the concrete block. No compressive stresses were thus directly introduced to the UHPFRC layer, contrary to the RILEM-type PO tests. To counteract the moments caused by the eccentricity and thus avoid uplift of the block, a pair of two horizontal beams parallel to the bar axis tied the block to the base plate. Rupture of the concrete block during testing was avoided by externally reinforcing it with a system of steel beams prestressed together by two high-strength steel rods.

The test set-up was not only designed to carry out the pull-out tests, but also to evaluate the development of prestressing forces in the prestrained Fe-SMA bars installed along the UHPFRC layer. To this end, an additional middle rigid frame, also shown in Fig. 14, was used during the activation procedure; note that this frame is irrelevant for Specimens L1 to L4 discussed in this paper, which were not activated.

The experiments were carried out in displacement control using a hydraulic actuator with a capacity of ± 150 kN and a maximum stroke

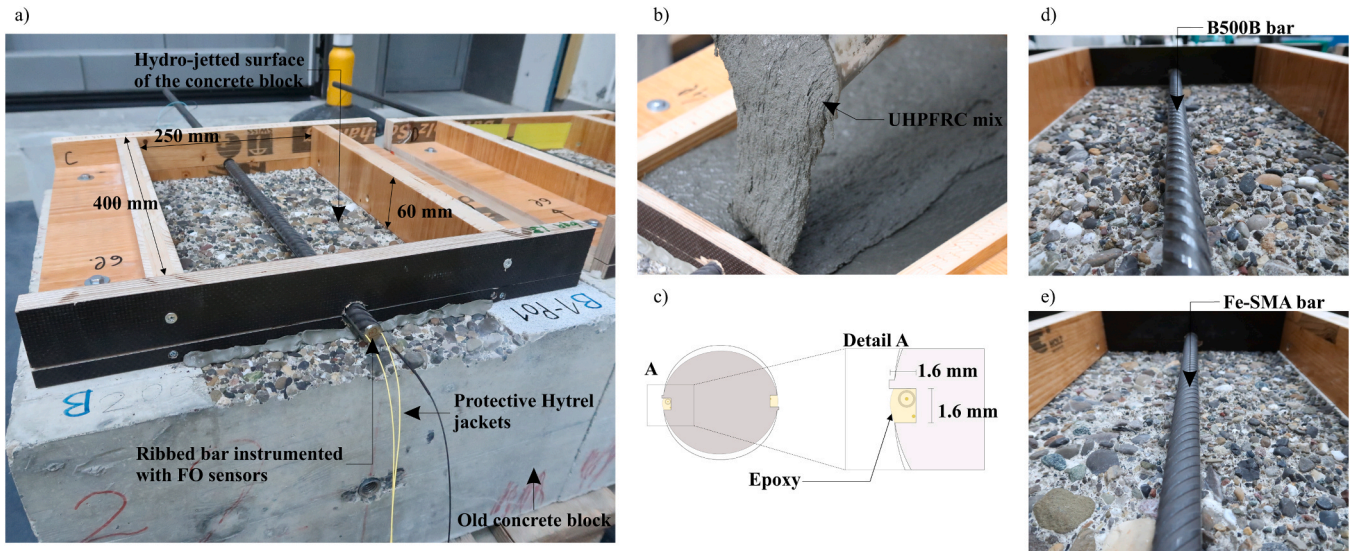


Fig. 12. Preparation works for the long PO test: a) Specimen configuration prior to casting of the UHPFRC layer; b) Casting of the UHPFRC matrix with the steel fibres aligned with the reinforcing bar axis; c) Detail of the instrumentation of the steel bars with FO sensors; d) Zoomed view of the installed B500B bars; e) Zoomed view of the installed Fe-SMA bars.

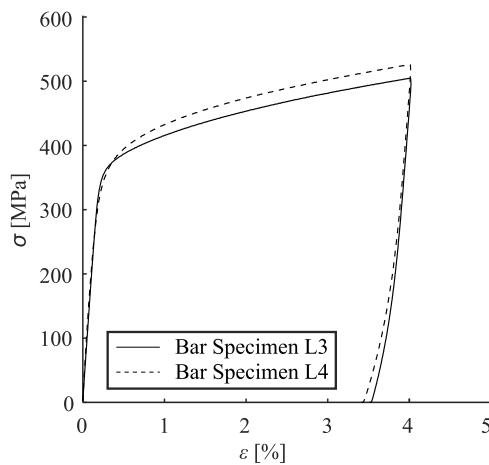


Fig. 13. Prestraining of the Fe-SMA ribbed bars installed in Specimens L3 and L4.

Table 7
Compressive strength of concrete cores.

Specimen	Sample Type	$f_{c, \text{core}}$ [MPa]	ρ [kg/m ³]
L1, L2	Core ϕ 50H50	58.7 (+/- 4.8)	2487 (+/- 41)
L3, L4	Core ϕ 50H50	56.1 (+/- 3.5)	2472 (+/- 37)

of 600 mm. An LVDT connected to the hydraulic actuator allowed measurements up to 500 mm with an accuracy of $\pm 1 \mu\text{m}$. An initial rate of 0.015 mm/s was applied and increased to 0.1 mm/s in the plastic range. A load cell with a capacity of 200 kN and a measurement error of less than $\pm 0.25\%$ recorded the load during testing. A signal amplifier was used for the data acquisition with a sampling frequency of 5 Hz.

Fig. 14c) schematically shows additional instrumentation used during testing. An LVDT with a measuring range of 5 mm and an accuracy of $\pm 0.01 \text{ mm}$ was positioned at the unloaded end of the bar (passive end) to record the end slip. Between the clamping system and the UHPFRC layer, a clip-on extensometer was installed to measure the

elongation of the bar during the test. The extensometer was mounted in a 100 mm gauge length configuration and allowed measuring elongations up to 20 mm. A DIC system was installed to monitor the development of splitting cracks on the top surface of the UHPFRC layer as well as the slip on both the loaded and unloaded ends of the bar.

2.3.4. Results and discussion

2.3.4.1. Failure modes. Depending on the type of steel reinforcing bars, different failure modes were obtained. In Specimens L1 and L2 (with B500B bars), failure occurred in the free length of the bars, whereas for Specimen L4 (Fe-SMA bar), combined pull-out and splitting of the UHPFRC was observed. Specimen L3 could not be loaded up to failure due to slippage of the bar inside the clamping system. Due to these issues, new clamping plates were installed. It is important to note that jaw slippage occurred only once higher load levels were reached, by which time FO measurements had already been lost and the clip-on extensometer had been removed.

Table 8 provides additional information for each specimen such as the maximum recorded load, F_{max} , the corresponding nominal tensile stress, σ_{max} , and the maximum strain measured with DIC, $\epsilon_{\text{max}}^{\text{DIC}}$. The latter does not necessarily correspond to the maximum load as the DIC measurements along the bar were occasionally lost prior to failure.

The inability to achieve bar failure in Specimens L3 and L4 is attributed to the high deformation capacity and pronounced strain hardening of the Fe-SMA reinforcing bars: axial strains greater than 20 % are necessary for bar rupture (in a prestrained condition), causing correspondingly high lateral contraction. In fact, at an axial strain of 10 % in the as-received (non-prestrained) Fe-SMA bars, the DIC measurements already indicated a lateral contraction of roughly 4 %, corresponding to a reduction of the original bar diameter of 16.8 mm by about 0.7 mm. Being of the same order as the rib height of the Fe-SMA bars (see Table 3, $a_{1/2} = 0.61 \text{ mm}$), this contraction doubtlessly impaired the bond strength and could also explain the slippage in the clamping system in Specimen L3.

2.3.4.2. Digital image correlation results. Stereo-DIC (often referred to as 3D-DIC) was used to monitor the development of cracks at the top surface of the UHPFRC layer, the strains in the reinforcing bars as well as the slip at the loaded and unloaded bar ends. Targets were also installed to detect possible rigid body movements of the concrete block. A speckle

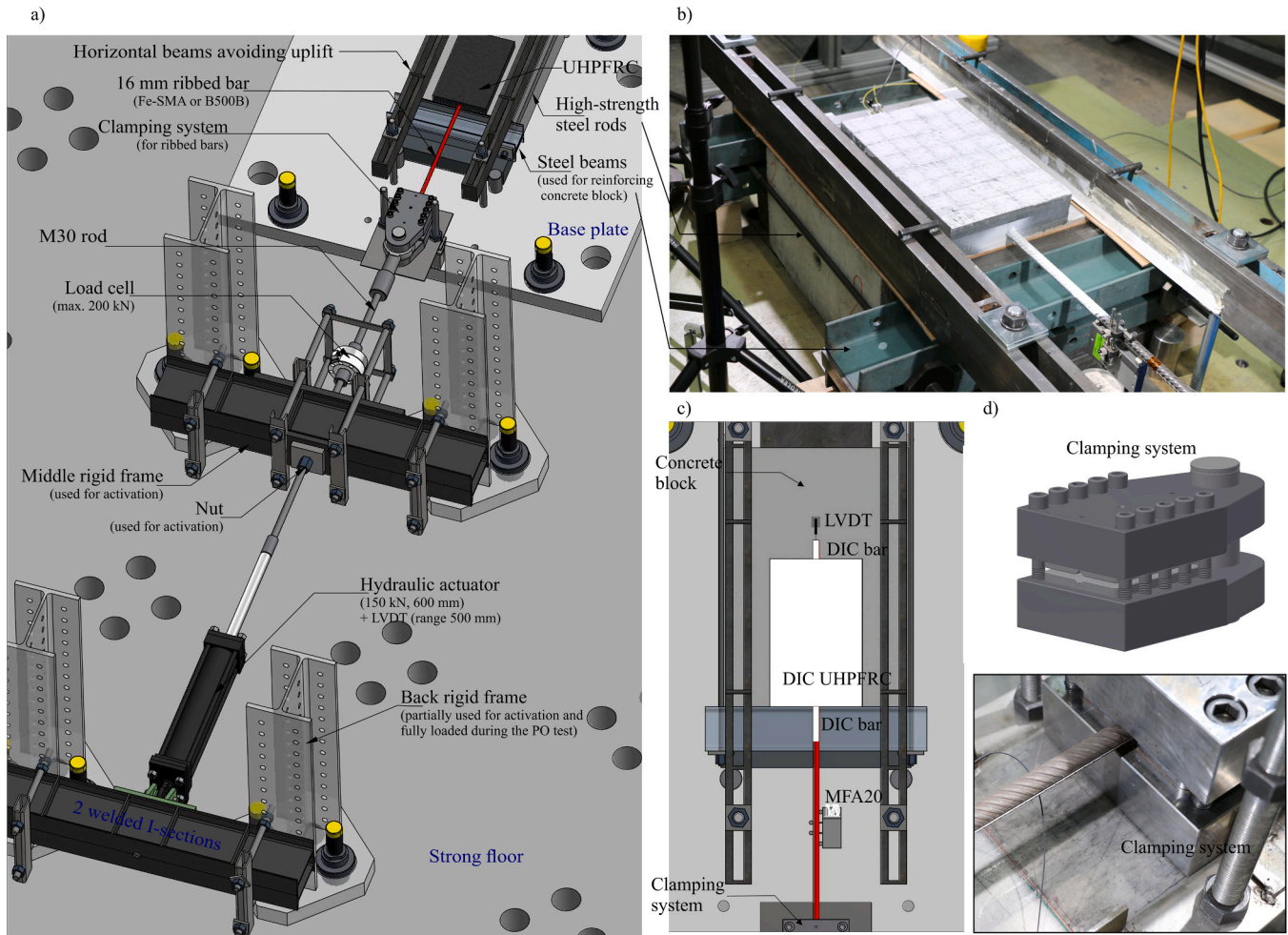


Fig. 14. Description of the test set-up used for the long PO tests ($l_b = 400$ mm): a) Rendering of the test set-up; b) Picture taken prior to testing Specimen L2; c) Schematic plan view of the test set-up; d) Clamping system developed for ribbed bars.

Table 8

Test results – Long PO tests.

Bar material:	B500B		Non-act. Fe-SMA	
Specimen:	L1	L2	L3	L4
F_{max} [kN]	117.9	116.1	141.1	144.3
σ_{max} [MPa]	594	584	637	651
ϵ_{max}^{DIC} [%]	5.6	6.9	10.3	9.9
Failure Type	Bar	Bar	No Failure	Bond

dot size of 0.033 mm was applied to the UHPFRC surface using speckling stamp rollers after spreading a very thin coating of gypsum to homogenise the UHPFRC surface ("elephant skin"). The speckle pattern was applied manually to the ribbed bars. Stickers with the same speckle size were attached to the markers.

Two 12 Mpx cameras were used together with 24 mm focal length lenses. The sensor size used was 14.13 mm by 10.35 mm. A baseline (distance between sensors) of roughly 352 mm, a resolution of 8.2 px/mm and a stereo angle of 22° were used.

Image correlation and post-processing were carried out using the software VIC-3D 9 from Correlated Solutions Inc. [56], and the crack pattern and crack widths were evaluated with the Automated Crack Detection and Measurement (ACDM) software [57]. A subset size of 27 px and a step size of 7 px were chosen for the UHPFRC surface and targets. For the bar surface, a subset of 21 px and a step size of 6 px were used. Fig. 15a) shows the typical DIC set-up and the Field of View (FOV) at the initial state for Specimen L4, and Fig. 15b) depicts the crack

patterns obtained at the UHPFRC surface of Specimens L1 to L4.

Fig. 16a) and b) illustrate the development of the longitudinal and transversal governing cracks up to the maximum load in Specimens L1 to L4. Table 9 compiles the key results for the governing longitudinal (subscript l) and transversal (t) cracks identified in Fig. 16. These include the maximum crack widths ($w_{l,max}$ and $w_{t,max}$), the crack widths at the end of the test at zero force ($w_{l,end}$ and $w_{t,end}$), and the corresponding relative crack recovery, $(w_{max} - w_{end})/w_{max}$.

Longitudinal cracks typically formed at lower load levels, followed by transversal cracks. For Specimens L1 and L2, the crack widths continued to increase up to the onset of bar failure. On the other hand, for specimens L3 and L4, a reduction of the governing cracks widths was observed at imminent bond failure as damage progressed further along the embedment length.

Crack recoveries between 43 % and 75 % were obtained, being more significant for Specimens L1 and L2, which is consistent with the observed bar ruptures. Specimen L1 exhibited the largest crack openings, which developed at lower load levels. This behaviour may be attributed to initial damage at the start of the bonded length, though the exact cause remains uncertain.

2.3.4.3. DFOS data acquisition and post-processing. Strain measurements with a sampling frequency of 20 Hz were taken with Fibre Optical (FO) sensors consisting of fibres installed along the reinforcing bars, using an ODiSI 6000 Series measurement system [47]. The latter uses Rayleigh backscatter for enhanced spatial resolution under static or quasi-static

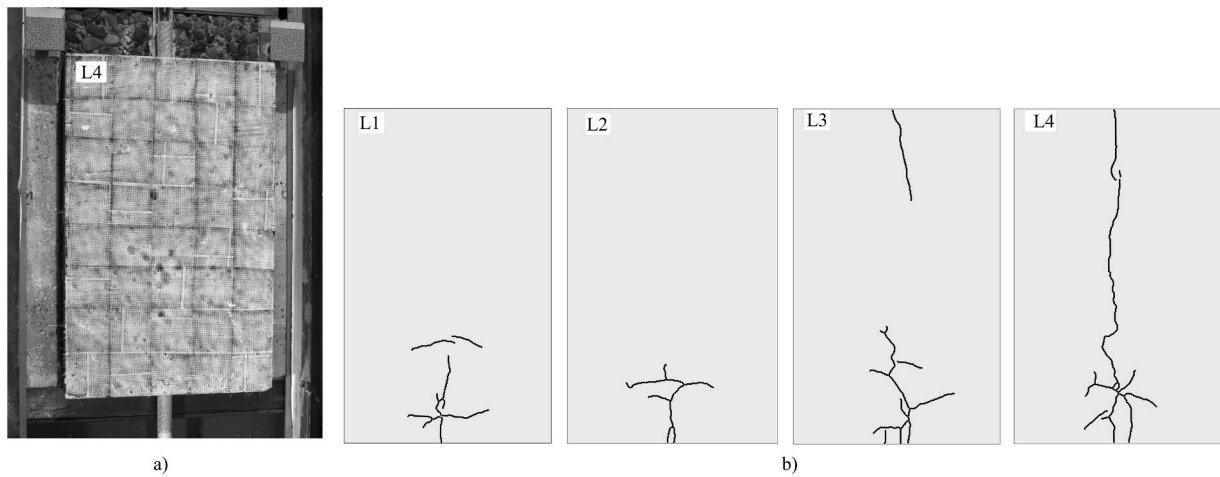


Fig. 15. DIC set-up and crack pattern: a) DIC set-up and defined Field of View (FOV) for Specimen L4; b) crack patterns obtained for Specimens L1 to L4.

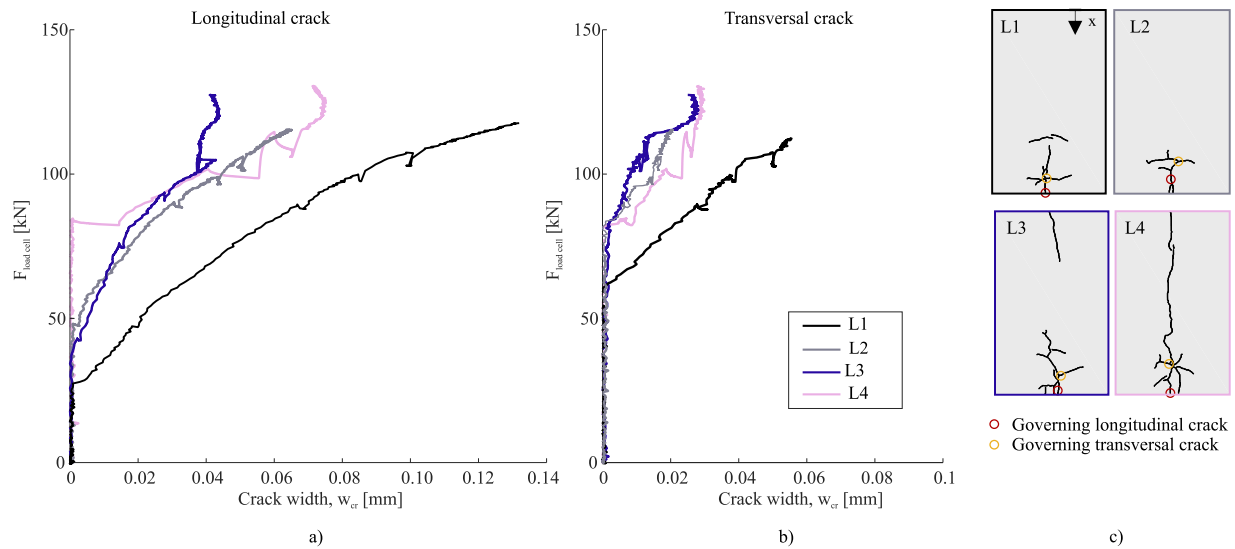


Fig. 16. Crack development for specimens 1–4: a) governing longitudinal crack; b) governing transversal crack; c) identification of governing cracks.

Table 9
Maximum and final crack widths measured with DIC.

Specimen	L1	L2	L3	L4
$w_{l,max}$ [mm]	0.132	0.066	0.044	0.075
$w_{l,end}$ [mm]	0.043	0.026	0.024	0.043
recovery of w_l	67 %	60 %	45 %	43 %
$w_{t,max}$ [mm]	0.058	0.021	0.028	0.030
$w_{t,end}$ [mm]	0.015	0.011	0.015	0.016
recovery of w_t	75 %	47 %	47 %	48 %

applications. The distance between two adjacent virtual gauges along the fibres (gauge spacing), was set to 0.65 mm. To down-sample the resulting considerable amount of data, a space multiplier of 5 and a frequency divider of 2 were used. Furthermore, to neglect the influence of the ribs on the strain profiles, a Gauss filter with a window size that was a multiple of the rib spacing was applied, as recommended in [45].

The workflow provided in Fig. 17 was used to post-process the raw data provided by the measurement system. For clarity, the strain profiles of only two load steps are shown, along the bonded and free lengths of the bar, and separately for both grooves per bar (referred to as G1 and G2), see Fig. 17a). The bars were oriented such that the grooves remained on opposite sides, left and right. Both the raw data, ϵ_{Raw} , and

the down-sampled and filtered signal are represented, $\epsilon_{smoothed,Gi}$, along with the average of both grooves, $\bar{\epsilon}_{smoothed}$. The colour-code used corresponds to the applied load (load-cell measurements). The strains measured by the clip-on extensometer, plotted at its centre position, correspond well with the DFOS strain measurements in the free length

In the flowchart, the different stages of the post-processing are identified as (1) to (4). To convert strain profiles to stress profiles – Step (1) – the stress-strain relationship of the bar σ - ϵ is used, see Fig. 17b). Each strain measurement in time and space, $\epsilon(x_k, t_i)$, can thus be transformed to a stress measurement $\sigma(x_k, t_i)$. The resulting stress profiles for the same two load steps are presented in Fig. 17c), again separately for both grooves and on average. The average results in the free length correspond well with the load cell measurements, confirming the accuracy of the FO measurements

In Step (2), the relative displacements between the bar and the concrete, denoted as slip, at a given position along the bonded length and time step, $\delta(x_k, t_i)$, are determined by integrating the strains between the position $x = 0$, where slip is negligible, and x_k and subtracting the deformation of the concrete over the same length, i.e.:

$$\delta(x_k, t_i) = \int_{x_0}^{x_k} \epsilon_s(x_k, t_i) dx - \int_{x_0}^{x_k} \epsilon_c(x_k, t_i) dx \quad (7)$$

For simplicity, and acknowledging that the UHPFRC deformations

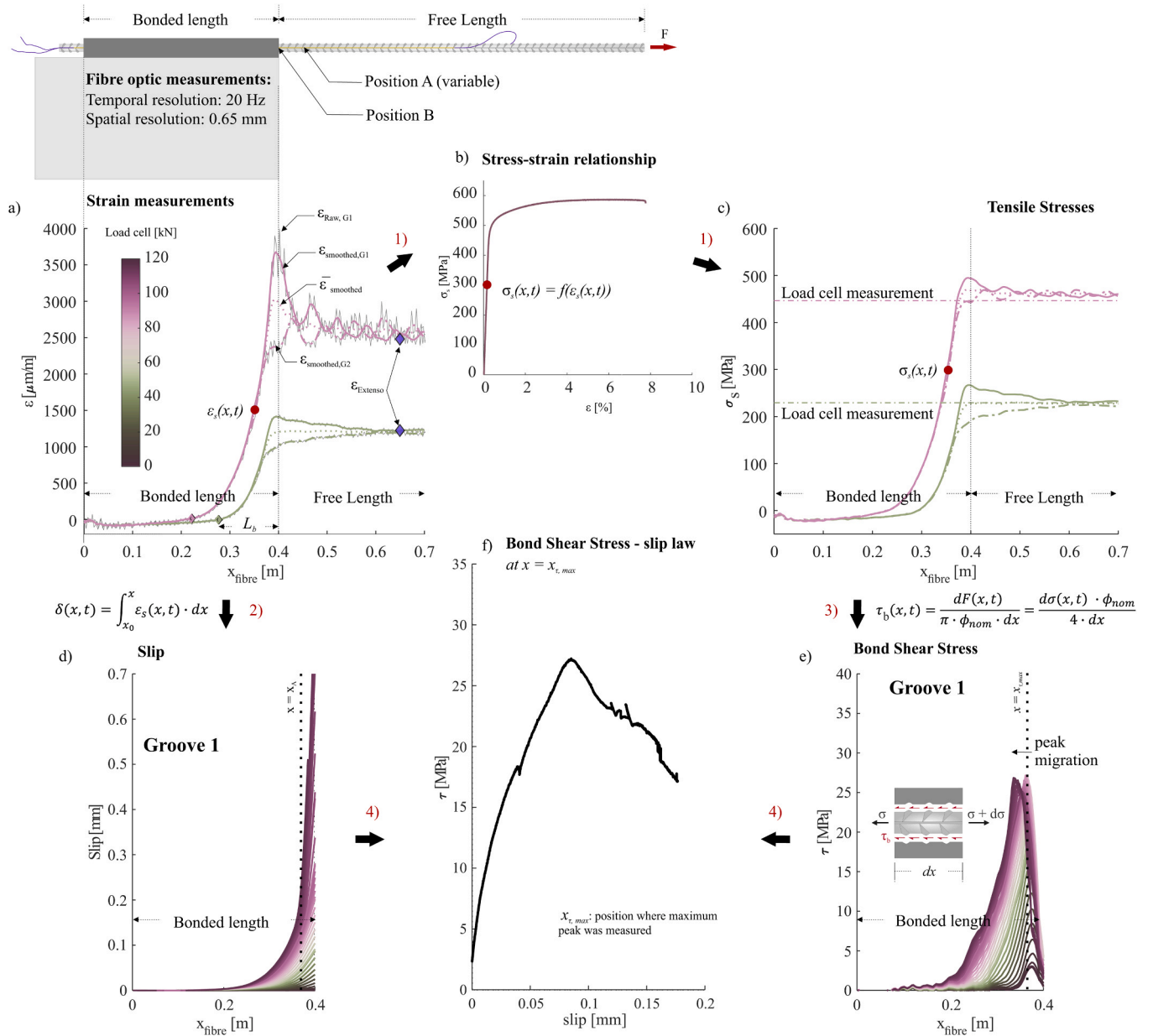


Fig. 17. Typical flowchart used to post-process the FO strain measurements (exemplified for Specimen L2).

are very small in comparison to the steel deformations, the second term of the Eq. (7) is neglected. The slip profiles along groove G1 are depicted in Fig. 17d).

As an additional plausibility check, the slip determined with DFOS according to Eq. (7) is compared to the slip obtained from the DIC measurements at $x = 0.4$ m (position B = transition from embedded to free length) by tracking the displacement of a specific point in the free length of the bar (position A) and subtracting both the displacement of the UHPFRC at $x = 0.4$ m (position B) and the bar elongation between A and B (obtained from DIC). Fig. 18 compares the slips obtained from both measurement techniques in Specimen L2. Noting that the two methods differ in their levels of accuracy and that the DIC-based calculations are subject to the superposition of errors arising from the three measurements required to determine the slip, the correlation is satisfactory and confirms the plausibility of the DFOS results.

In Step (3), the forces transferred from the steel bar to the concrete are determined from the force variation between two adjacent measuring points or virtual gages, $dF(x_k, t_i)$, obtained from the strains

measured by DFOS using the stress-strain relationship of the reinforcing bar. Using the common assumption of bond shear stresses τ_b uniformly distributed along the nominal bar perimeter, the bond shear stresses follow as:

$$\tau_b(x_k, t_i) = \frac{dF(x_k, t_i)}{\pi \cdot d_s \cdot dx} = \frac{d\sigma(x_k, t_i) \cdot d_s}{4 \cdot dx} \quad (8)$$

where d_s is the reinforcing bar nominal diameter and dx the distance between two measuring points. The resulting bond shear stress profiles for Groove G1 are depicted in Fig. 17e).

Finally, in Step (4), by combining the slip and bond shear stress profiles, one obtains the bond shear stress-slip relationships at any position x_k along the embedment length, enhancing the statistical significance of each PO test. For illustration, in Fig. 17f), the bond shear stress-slip relationship is reported at the position where the maximum peak bond shear stress was detected, for Groove G1.

The wavy pattern observed in the raw strain measurements, more pronounced along the free length of the B500B bars, see Fig. 17a), is

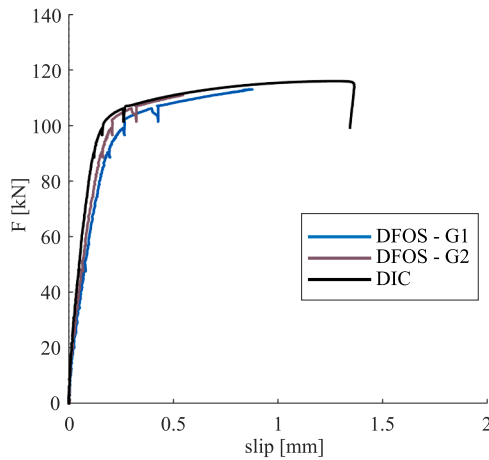


Fig. 18. Plausibility check for slip calculation with DFOS – Specimen L2.

likely related to the manufacturing process of the bars. Potential explanations are plastic deformations caused by the rollers used to form the rib pattern [42] or during de-coiling stages, where rollers are used to straighten the bars subjecting them to reversed bending moments. The machinery process can plastify and harden certain regions of the bar. As a result, upon re-loading, the hardened regions of the bar will remain elastic at higher stress levels while the remaining areas will be already yielding thus exhibiting much higher strains. In accordance with the latter, a wavy strain pattern would be perceptible when strains approach the yielding limit, over $2000 \mu\text{m/m}$, which is indeed only observed for the higher load level measurement presented in Fig. 17a).

This wavy pattern was not observed in Fe-SMA bars, which are produced as straight elements with a length of about 5.5 m (not coiled). The production process comprises an initial hot-rolling of smooth bars with a defined length, a cold pressing of the ribs and eventually a solution treatment (annealing). Fig. 19 compares typical strain profiles for the two types of bars, for Specimen L2 and L3. For simplicity, only one groove is presented.

Simplifications

The following simplifications were adopted for post-processing the results with DFOS:

- The stress-strain relationship used for converting strains into stresses was obtained from the clip-on extensometer installed along the free length of the bar. As the bars were subjected to residual moments resulting from the positioning of the concrete block and clamping of

the steel bar, the stress-strain relationship along the bar length may slightly vary. The latter was disregarded for simplicity;

- As the clip-on extensometer had to be removed prior to failure to prevent its damage, the stress-strain relationship derived from tensile tests on ribbed bars was fitted to the extensometer results up to this point, and this relationship was then used to extrapolate the missing data after the extensometer had been removed;
- In certain cases, the epoxy used along the grooves did not cure as expected and the DFOS measurements were not reliable (e.g. Specimen L1 – Groove G1). The respective results are omitted in the figures.

2.3.4.4. DFOS results

2.3.4.4.1. Bond shear stress-slip relationships. Bond shear stress-slip relations were derived for Specimens L1 to L4 by post-processing the FO strain measurements following the previously described workflow. Fig. 20 and Fig. 21 illustrate the bond shear stress-slip relationships obtained at the bond shear stress peak position per groove (different in G1 and G2), in Specimens L2 and L3, respectively. The steel stress-strain relationship and crack width developments are included to better evaluate their effect on the bond shear strength. Relevant points such as the 0.2 % offset yield point, and the peak bond shear stress, are indicated to facilitate the correlation among the different graphs.

The peak bond shear stress observed in Specimen L2 (Fig. 20) using the FO-sensor installed along Groove G1 (on the right side of the bar), at the position $x = 0.365 \text{ m}$, correlates with the formation of the transversal crack. Conversely, the results along Groove G2 indicate that the bond shear stress continued to increase after this crack had formed. These observations suggest that the crack distribution on the surface of the UHPFRC element adversely affected the bond behaviour, although its impact varied at different locations along the reinforcement.

The results of Specimen L3 (Fig. 21) reveal almost identical peak bond shear stresses for both grooves. The peaks did not coincide with crack formation, but occurred after both transversal and longitudinal cracks had developed. However, they coincided with instances where, despite the continued increase in load, slight reductions in crack widths were measured at the surface. At this point, the Fe-SMA bar already exhibited significant non-linear behaviour, with strains exceeding 0.3 %. The latter indicates that the large nonlinear steel strains affected the bond strength, causing a shift of the location where the maximum bond shear stresses occurred to the interior of the bonded length to enable a further load increase, leading to a reduction in the width of the initial cracks formed at the UHPFRC surface.

The shift of the location of the maximum bond shear stress along the embedment length is also visible in Specimen L2, see Fig. 17e). To

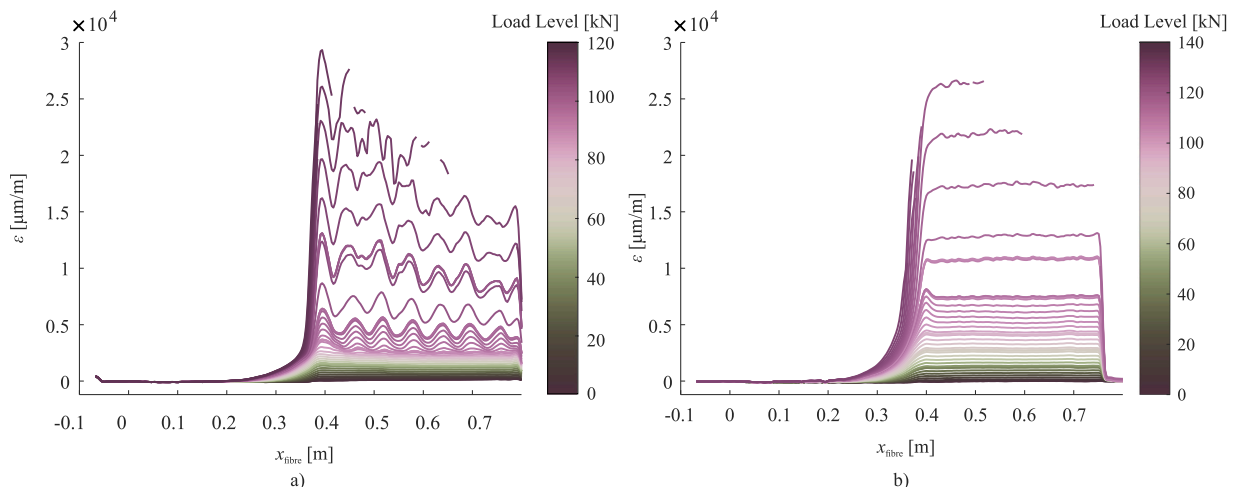


Fig. 19. Strain profiles measured along the bar: a) Specimen L2 (B500B bar); b) Specimen L3 (Fe-SMA bar).

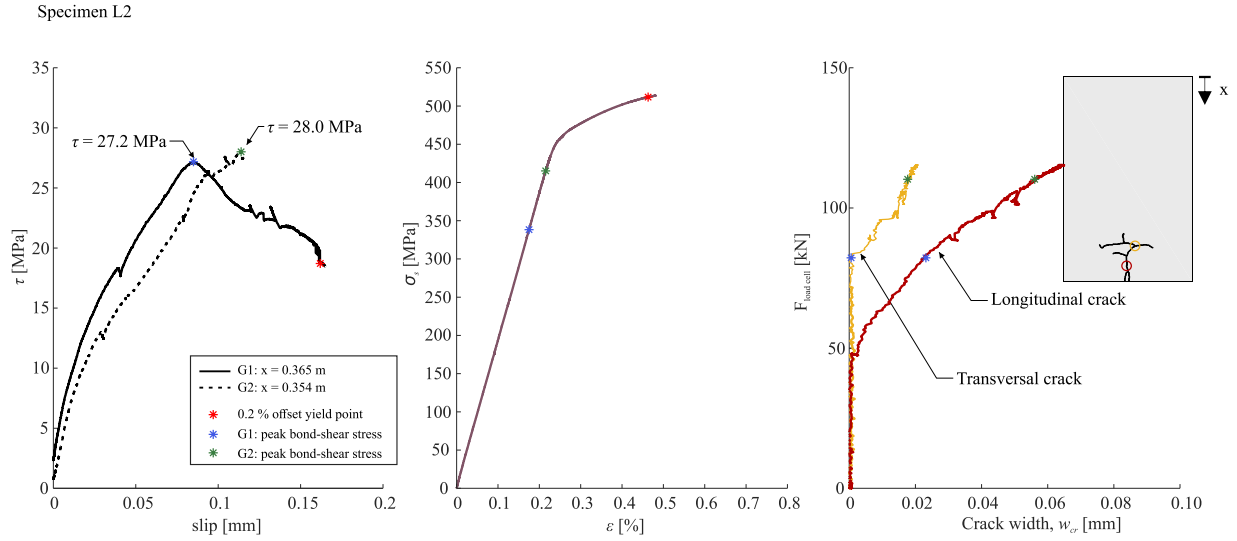


Fig. 20. Bond shear stress-slip relationship, stress-strain relationship for B500B and crack width development – Specimen L2.

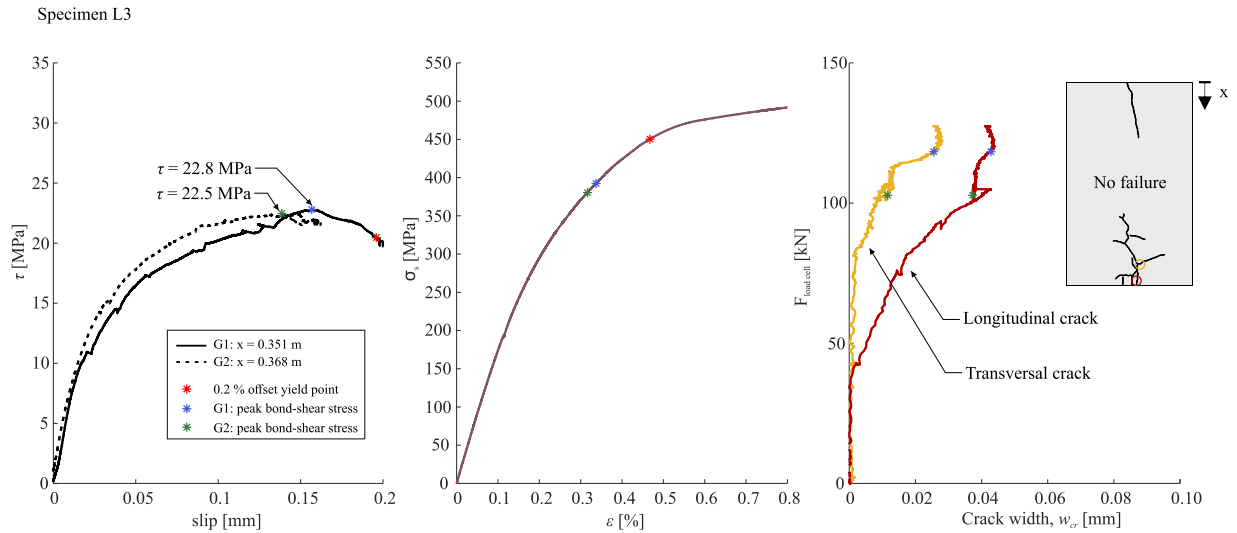


Fig. 21. Bond shear stress-slip relation, stress-strain relationship for Fe-SMA and crack width development – Specimen L3.

investigate the damage progression along the embedment length, the maximum bond shear stress τ_{max} (right ordinate) as well its location $x_{\tau_{max}}$ along the bar axis (left ordinate) are plotted against the bar stress for Specimens 2 and 3 in Fig. 22a) and b). At the beginning of both tests, the maximum bond shear stress was located near the loaded UHPFRC edge. Its initial position, slightly offset from $x = 0.4$ m, was likely affected by shrinkage strains of the UHPFRC and potential strains introduced during the positioning of the concrete block and the clamping of the reinforcing steel bar. As the load increased during the test, the position of the peak bond shear stress progressively shifted inside the embedded length. The bond shear stresses did not start at zero for reasons similar to those affecting the maximum shear stress position. For Specimen L2, along Groove G1, the bond shear stress increased until it reached a maximum, followed by a decrease. The stress subsequently increased again, reaching another local maximum further inside the embedded length. This demonstrates that multiple local maxima can occur during the test, although their values do not appear to change significantly. For Specimen L3, the bond shear stress increased up to roughly 23 MPa for both grooves. A slight decrease can be observed thereafter, although it is not clear whether another maximum could have been reached, as the fibre measurements were subsequently lost.

2.3.4.4.2. Active anchorage length. The active anchorage length (L_b) can be inferred from the strain profiles obtained with DFOS by detecting the position where the spatial strain rate along the bar does not exceed a specified small threshold. A threshold of $1500 \mu\epsilon/m$ provided reasonable results and was therefore adopted for all specimens. In Fig. 17a), the position used for the determination of the active anchorage length is indicated in the strain profiles at both illustrated load levels.

Fig. 23 shows the development of the active anchorage length for Specimens L1 and L2 (a) as well as L3 and L4 (b) as a function of the tensile stress in the reinforcing bar. The active anchorage length starts increasing at roughly 100 MPa, for all specimens, growing nearly linearly with the stress level after this. These results confirm the earlier observation that damage progresses along the embedment length, mobilising a longer anchorage length at higher load levels. Note that the plateaus observed in the graphs correspond to instances where the displacements of the hydraulic actuator were held constant for a short period of time to allow the creation of a new sensor key for the DFOS measurement system, in order to extend its measurement range.

The maximum anchorage length measured with DFOS was approximately 0.21 m for Specimens L1 and L2 and 0.23 m for Specimens L3 and L4. However, longer anchorage lengths were likely mobilised after

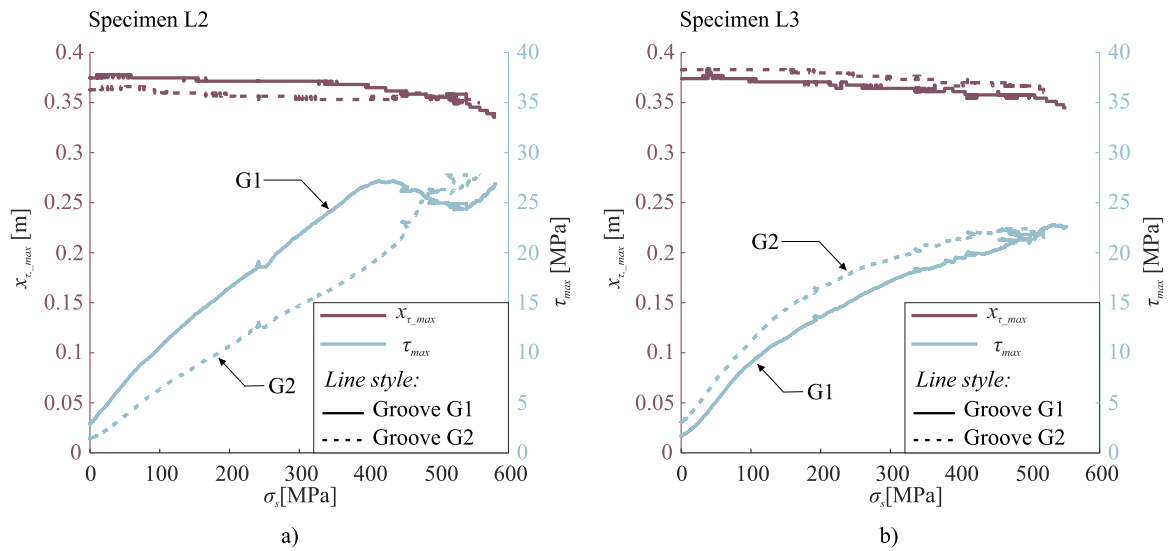


Fig. 22. Variation of the bond shear stress peak position and values with increasing tensile stress level: a) Specimen L2; b) Specimen L3.

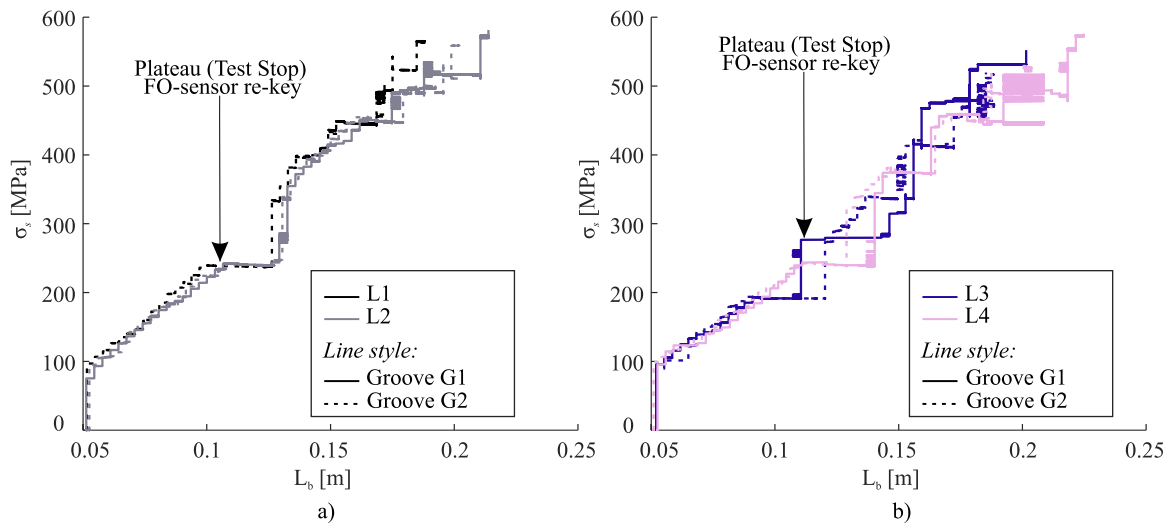


Fig. 23. Active anchorage length: a) Specimens L1 and L2; b) Specimens L3 and L4.

Table 10
FO-sensors measurement range.

Specimen	F_{G1} [kN]	F_{G2} [kN]	F_{max} , Load cell [kN]
L1	*	112.5	117.9
L2	115.5	111.0	116.1
L3	122.4	115.2	141.1
L4	127.7	117.4	144.3

* Measurements not reliable for G1 in Specimen L1

the FO measurements were lost, particularly in Specimens L3 and L4. Table 10 compares the load level at which the FO sensors failed to the maximum load recorded in each test. For Specimens L1 and L2, measurements with fibre optics were possible nearly up to failure (max. 5 % force increase after the signal loss). For Specimens L3 and L4, the load increased by 13–23 % after the loss of the sensor measurements. Nonetheless, although measurements with FO sensors could not be continued up to failure, they successfully captured strains and enabled the analysis of bond shear stresses significantly beyond service load levels.

Regarding ultimate limit states, an anchorage length of 0.21 m was

sufficient to anchor approximately 95 % of the strength of the B500B reinforcing steel bars (with a concrete cover of 39 mm). Increasing the anchorage length significantly beyond this length would thus not result in much higher resistances, as failure typically occurs in the free length of the bars. For Fe-SMA bars, achieving full anchorage up to the maximum resistance is unlikely, as their ultimate tensile strain ranges between 30–40 %, where lateral contraction significantly affects the bond strength (see Section 6.4.1) causing damage to progress further into the embedment length. While a longer anchorage length is thus mobilised with increasing load, the contribution of the initial portion of the bonded length to the overall resistance diminishes. Consequently, further increasing the embedment length would not necessarily cause a significant strength increase. Notably, at the imminence of failure, a longitudinal crack rapidly propagated across the entire surface of the UHPFRC of Specimen L4, indicating a brittle and sudden failure mechanism, which underlines these observations.

An anchorage length of 400 mm, corresponding to the embedment length in the long pull-out tests in this study, thus appears reasonable. However, it is important to acknowledge that the bond stresses are far from being uniformly distributed along the anchorage length.

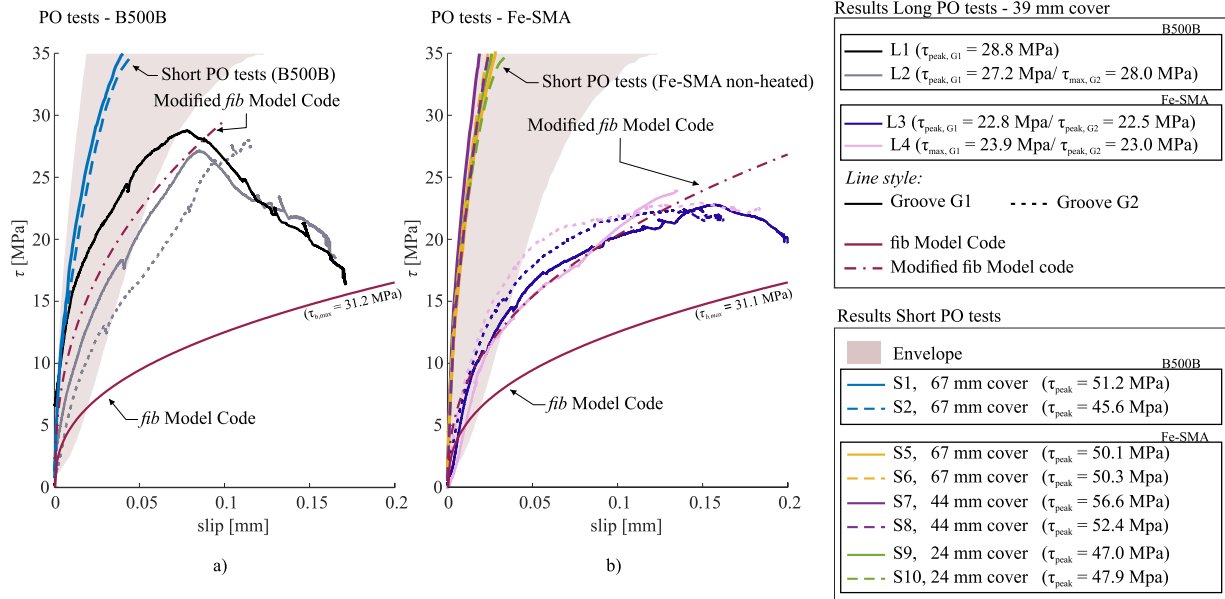


Fig. 24. Bond shear stress-slip relationships: a) PO tests using B500B bars; b) PO tests using Fe-SMA bars.

3. Comparison between testing methods

Fig. 24 compares the bond shear stress-slip relationships for the specimens tested in the long pull-out test campaign, using B500B reinforcing steel (a) and Fe-SMA reinforcing steel (b), respectively. The relationships were derived at the positions where the absolute peak bond shear stress values were measured (example illustrated in Fig. 17.e). The B500B bars with long embedment length exhibited higher peak bond shear stresses at lower slip, indicating higher bond stiffness and strength at the expense of reduced ductility. These results contradict the observations of the short pull-out tests, see e.g. Fig. 8b, thereby corroborating the concerns that the short pull-out tests – whose relevant results are included in Fig. 24 for a direct comparison – were affected by local effects. Furthermore, the much lower bond stresses observed in the long pull-out tests substantiate the observation that the short pull-out tests significantly overestimate the bond strength. In this regard, SIA 2052 also seems to provide non-conservative estimates of the peak bond strength of bars in UHPFRC when the actual strengthening system configuration is considered.

The bond shear stress-slip relationships according to the *fib* MC are also shown in Fig. 24 for $0 < s < s_1$, based on Eq. (3). As in Section 2.2.4, the mean UHPFRC cylinder compressive strength f_{cm} was derived from cube tests using a 0.95 conversion factor. The compressive strength values are compiled in Table 2, for different hardening times, and Table 6 shows the hardening times corresponding to each long PO test. Based on the latter, $f_{cm, cube} = 163.9$ MPa for Specimens L1 and L2, and $f_{cm, cube} = 162.8$ MPa for Specimens L3 and L4.

Originally developed for NC, the *fib* MC relationships fail to realistically predict the bond shear stress-slip relationships of both the short PO tests in UHPFRC (see Fig. 8) as well as the long PO tests. While the maximum bond strengths calculated with the *fib* MC ($\tau_{b, max} \approx 31$ MPa) align reasonably well with those observed in the long PO tests, the slips (s_1) require adjustments.

To improve correlation with the experimentally observed behaviour, modified *fib* MC curves were calculated for the long PO tests. The latter incorporate the *fib* MC recommendations to account for the presence of longitudinal cracks with a width of approximately 0.04 mm for all specimens (L1- L4), and consider the influence of steel yielding in specimens using Fe-SMA bars (Specimens L3- L4). Although PO tests on specimens with B500B bars reached bar failure, Fig. 20 shows that a reduction of the peak bond shear stress occurred prior to the yielding of

the bar, which is likely related to the crack distribution on the UHPFRC surface. The values of s_1 were also adjusted to better match the experimental curves. For B500B bars, s_1 was set to 0.1 mm, while for Fe-SMA bars, s_1 was set to 0.2 mm. The modified *fib* MC curves align well with the experimental results from the long PO tests, unlike the short PO tests. At imminent splitting failure, the peak bond shear stress predicted by the modified stress-strain relationship would not be reached, which applies to the PO tests using Fe-SMA bars.

Although the curves fit well the experimental results, the slip corrections to the *fib* MC model should not be considered as design recommendations. At this stage, only a limited number of tests have been performed.

4. Conclusions

As part of a study of a new strengthening method developed for RC structures, a pilot investigation on the bond behaviour of ribbed Fe-SMA bars embedded in UHPFRC was conducted in which the results were compared to the performance of conventional steel bars. Two series of pull-out tests were conducted: a PO test series with short embedment lengths ($l_b = 25$ mm) and a PO test series with longer embedment lengths ($l_b = 400$ mm). The latter intended to represent the actual strengthening system while addressing the limitations of short PO tests. To enable the investigation of the complex bond behaviour along the longer bonded lengths, fibre optical sensors were installed along the reinforcing bars. The experimental results from the two test series were then compared to the Swiss technical specification SIA 2052 [50] for UHPFRC and to the *fib* Model Code 2020 [55]. The main outcomes are as follows:

- The short PO tests showed peak bond strengths ($\tau_{b, max}$) of 42–57 MPa, exceeding the predictions from *fib* MC, (32 MPa) and SIA 2052 for UHPFRC (35–45 MPa). This confirms that short PO tests tend to overestimate bond strength due to simplified testing conditions (e.g. unrealistically beneficial bond conditions).
- Short PO tests remain valuable because of their simplified test set-up and specimen geometry, which facilitate reproducibility and allow to easily study the influence of several parameters on the bond behaviour. Heating Fe-SMA bars ($\sim 200^\circ\text{C}$) led to a peak bond-shear stress reduction of up to 20 %. For unheated specimens, cover thicknesses above 44 mm had little influence on bond behaviour,

whereas a thinner cover (24 mm) led to combined pull-out and splitting failure mechanisms.

- Due to the short embedment lengths ($\sim 1.5d_s$), local effects limited their applicability for comparing different types of reinforcement, consistent with observations reported in the literature [27].
- Long PO tests showed significantly lower peak bond-shear stresses (22.5–28.8 MPa), thereby confirming the tendency of short PO tests to overestimate the bond strength of bars embedded in the current strengthening system configuration. The B500B bars embedded in UHPFRC exhibited higher peak bond shear stresses at lower slip values, indicating increased bond stiffness and resistance but reduced ductility. These findings, being opposite to those of the short pull-out tests, corroborate that the latter were influenced by local effects.
- The refined strain measurements provided by fibre optical sensors installed along the reinforcing bars allowed the calculation of bond shear stresses and slips with high temporal and spatial resolution. Slip and bond shear stress profiles were combined to obtain bond shear stress-slip relationships at any position along the embedment length, thus providing enhanced statistical relevance. Moreover, strain measurements were used to determine the active anchorage length and provide a better insight into the complex mechanisms occurring along the bonded length, such as bond degradation and damage progression within the embedment length.
- Although measurements with FO sensors could not be carried out up to failure, they successfully captured strain distributions and enabled the analysis of bond shear stresses significantly beyond service load levels. The data is highly useful for refining models that estimate crack spacings, crack widths, and element deflections, as well as for enhancing the understanding of tension stiffening effects. Additionally, these results can help estimate the ultimate strength of end anchorages and predict the overall deformation and rotation capacity of structures.
- The *fib* MC fits well the long PO test results in terms of strength, though the recommended slip values (s_r) may need adjustments for UHPFRC. SIA 2052 appears to overestimate the peak bond strength, indicating that design values and testing procedures could be refined for UHPFRC used in analogous strengthening applications.
- As part of a pilot investigation into the bond behaviour between Fe-SMA bars and UHPFRC, no formulation has yet been proposed. This study is part of a broader research effort that will also consider the influence of the activation procedure on the bond behaviour.

5. Limitations and outlook

The present study should be understood as a preliminary case study, where priority was given to varying parameters rather than achieving a statistically robust number of repetitions. While the results were consistent, a more extensive experimental programme with additional repetitions is required for a full characterisation of the bond behaviour.

CRediT authorship contribution statement

Julien Michels: Writing – review & editing, Resources, Funding acquisition. **Ivan Markovic:** Writing – review & editing, Supervision, Funding acquisition, Conceptualization. **Walter Kaufmann:** Writing – review & editing, Supervision. **Christoph Czaderski:** Writing – review & editing, Supervision, Funding acquisition, Conceptualization. **Angela Lemos:** Writing – original draft, Methodology, Investigation, Conceptualization.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Data availability

Data will be made available on request.

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