

Structural behaviour of 3D printed concrete beams with various reinforcement strategies

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ABSTRACT

3D concrete printing (3DCP) offers many new possibilities. This technology could increase the productivity of the construction industry and reduce its environmental impact by producing optimised structures more efficiently. Despite significant developments in materials science, little effort has been put in developing reinforcement strategies compatible with 3DCP and on the characterisation of their structural behaviour. Consequently, 3DCP still lacks compliance with structural integrity requirements. This study presents an experimental investigation consisting of nine four-point bending tests on extrusion 3DCP beams reinforced with various types of reinforcement. As interlayer shear reinforcement, aligned end-hook fibres (0.3 and 0.6%) or steel cables (0.1%) placed between the layers of printed concrete were used. As longitudinal reinforcement, unbonded post-tensioning and conventional bonded passive reinforcement were explored. The crack patterns and their associated kinematics were tracked using digital image correlation. The results show that the post-tensioned beams failed in a brittle manner due to the crushing of concrete in bending, with deformations localised in a few bending cracks. In the beams with conventional bonded longitudinal reinforcement, both bending as well as shear cracks were generated, and the brittle failure of the interlayer shear reinforcement limited the ultimate load. Estimations based on the measured crack kinematics show that the interlayer shear reinforcement carried most of the applied shear force. Based on these results, a simple mechanical model is developed to understand the mechanical behaviour and to pre-design the required amount of interlayer shear reinforcement.

1. Introduction

The building sector, including the construction industry, is one of the primary consumers of energy and emitters of CO₂ [1]. It needs to reduce its negative impact on the environment drastically to tackle the challenges of climate change that humanity is facing. A significant part of the negative impact of the construction industry can be attributed to the massive consumption of cement and concrete and the inherently high CO₂ emissions caused by the production of clinker as the main component of cement. Multiple factors need to be adjusted to change that impact. In a recent study, Favier et al. [2] pointed out different scenarios and actions. While most actions concern aspects covered by materials science (reduction of CO₂ emissions in cement production; using less clinker in cement and less cement in concrete, use of recycled materials), the need for structurally more efficient buildings consuming less material, which needs to be addressed by structural engineers, is highlighted.

However, the production of structurally optimised building elements with conventional methods is highly challenging due to the high cost and waste of formwork when producing non-standard shapes [3,4]. Here, new technologies such as digital fabrication with concrete (DFC) could offer the possibility to produce optimised structures using less material at little extra cost [3]. The most widely applied technology in this area is the so-called concrete extrusion 3D printing (3DCP) [5,6], sometimes also referred to as contour crafting. This printing process recreates an object by extruding consecutive layers of concrete on top of each other. Currently, this technology is being researched and developed all around the world [3,7]. Due to these efforts, technical and material constraints have been overcome to a large extent. However, little effort has been put so far in the development of suitable reinforcement strategies. Consequently, 3DCP still lacks compliance with structural integrity requirements, which hinders its applicability on a structural level.

So far, only a few structures have been built with 3DCP, mostly

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Nomenclature		$\varnothing_i$	steel diameter
<i>Capital latin letters</i>			
A_i	steel cross-section area	<i>Lower case greek letters</i>	
$CMOD$	crack mouth opening displacement	$\varepsilon_{i,u}$	strain at peak stress
E_i	axial tensile modulus of elasticity	$\varepsilon_{i,y}$	yield strain
F	total applied load	ε_{wr}	steel wire strain at the crack
F_{max}	maximum applied load	δ_h	horizontal crack opening
K_P	correction factor for the assumption of a perfectly plastic behaviour	δ_v	vertical crack opening
$P_{0,i}$	initial prestressing force	$\delta_{v,max}$	vertical crack opening at maximum load
$P_{u,i}$	prestressing force at ultimate load	$\delta_{v,60\%}$	vertical crack opening at 60% of maximum load
V	shear force	ρ_{eff}	effective reinforcement content
V_{max}	maximum shear force	ρ_{web}	effective reinforcement content in the web
$V_{s,i}$	shear transfer contribution of the reinforcement	ρ_{nom}	nominal reinforcement content
<i>Lower case latin letters</i>		σ_{cf}	effective fibre tensile stresses in the concrete
b_{web}	web width	σ_{cp}	average prestressing stress
d	static depth	σ_{cs}	equivalent tensile strength of the concrete due to the reinforcement
h_{layer}	layer height	σ_{wr}	stress of the cable at the crack
f_c	concrete compressive strength	τ_b	bond shear stress
f_{ct}	concrete tensile strength	τ_{b0}	bond shear stress before yielding
$f_{i,u}$	ultimate strength	<i>Subscripts</i>	
$f_{i,y}$	yield strength	<i>bot</i>	lower prestressing rod
f_R	residual flexural tensile strength	<i>s</i>	conventional reinforcing steel
f_{Ri}	residual flexural tensile strength at $CMOD_i$ (for $i = 1, 2, 3, 4$)	<i>top</i>	upper prestressing rod
f_u	tensile strength	<i>p</i>	prestressing steel
l_b	bond length	<i>w</i>	steel wire
w	crack opening	<i>Abbreviations</i>	
x	direction of the longitudinal reinforcement	3DCP	3D concrete printing
z	inner lever arm	B-CC	concrete crushing in bending
<i>Capital greek letters</i>		FRC	fibre reinforced concrete
Δ	mid-span deflection	S-DT	shear failure due to diagonal tension
Δ_{max}	maximum mid-span deflection	SLS	serviceability limit state
		ISR	interlayer shear reinforcement type
		PS	Printing session

having very low structural demands. Their load-bearing capacity resembles that of masonry rather than structural concrete [8,9], and they, therefore, cannot replace load-bearing structural concrete elements. Therefore, to have a significant positive impact on the sustainability of the construction industry, 3DCP needs to be able to address a broader range of structural elements, including more structurally demanding structures. The development of reinforcement strategies compatible with the extrusion concrete processing enables the realisation of safe and reliable concrete structures with non-standard shapes [10]. At the same time, these strategies need to be able to provide sufficient reinforcement quantities to fulfil all structural integrity requirements related to the load-bearing capacity as well as the serviceability behaviour [11].

Deformed steel reinforcing bars placed in the formwork before casting are the most common way to reinforce conventionally cast structures. Some DFC technologies, such as Digital Casting Systems [12,13] or Shotcrete 3D Printing [10,14], incorporate conventional reinforcement by casting or spraying the concrete around a preplaced reinforcement cage. The resulting structures provide continuous reinforcement and can be designed according to existing standards for structural concrete. Unfortunately, this approach is not directly applicable in combination with 3DCP because of a high risk of collision between the extruder and the rebar. However, various approaches exist to combine 3DCP with conventional reinforcing bars. Instead of having a single nozzle for the deposition of concrete, the concrete can be placed on both sides of a reinforcement mesh [15,16]. Since the height of the

reinforcement is limited in this approach, Classen et al. [17] additionally propose to extend the reinforcement by welding subsequent reinforcing bar segments. Reinforcement can also be added after the printing process, either externally [18] or inside a 3DCP element used as a lost formwork [19,20]. The latter approach can be further refined by printing a structure with voids, and placing reinforcement and grouting the voids after printing. This approach allows activating the printed concrete, acting together with the reinforcement resisting the tensile forces, as a structural component equivalent to conventionally cast structural concrete. However, the use of conventional deformed reinforcing bars for 3DCP poses two main challenges: Stiff bars limit geometric flexibility while the increased porosity at the interface between layers can accelerate corrosion of the rebar, thus reducing the durability of the structure [21].

Another conventional reinforcement approach for long-span structures is prestressing reinforcement, which can either be tensioned before casting (pre-tensioning) or after producing the elements (post-tensioning). To the authors' knowledge, only post-tensioning has been applied so far to 3DCP [22,23]. The post-tensioning reinforcement is added in voids after the production and then prestressed to pre-compress the concrete; the post-tensioned reinforcement may be bonded by grouting the voids or left unbonded. This approach is quite promising for 3DCP as it provides geometric flexibility since the tendons can adapt to non-straight voids without excessive curvatures.

A larger degree of geometric freedom can be achieved when using fibre reinforced concrete (FRC). In FRC, the fibres, typically made from

steel or polyolefins, are intermixed with the concrete and cast together. Research in fibre reinforced concrete (FRC) dates back decades [24], but its use is still limited to specific applications such as facades, industrial floors or tunnel linings primarily carrying compression. The reasons for the limited use of FRC has mainly two origins. Firstly, fibres are dispersed in the matrix with an unknown location and orientation, which introduces a high uncertainty in the design phase. Secondly, common FRC displays strain-softening behaviour for practical fibre dosages. This softening results in failures with damage localisation that do not provide sufficient ductility to ensure a safe design for the primary load transfer [25], unless other elements of a structure can compensate for the softening behaviour by resisting more load (e.g. may be used in the case of FRC to replace conventional shear reinforcement [26]). The addition of fibres to 3DCP mixes has shown to align the fibres in the direction of printing and, consequently, to improve the mechanical performance in this direction [27–30]. However, the fibres are not able to bridge subsequent printed layers, and additional reinforcement in the perpendicular direction is needed. Furthermore, adding fibres to the concrete mix has a direct implication on the material processing. Most 3DCP processes rely on the use of pumps to transport the material to the extruder. Therefore, the material needs to be adjusted to be pumpable and only short, flexible fibres, which are typically more expensive and structurally less efficient, can be used.

Beyond these reinforcement concepts that are closely linked to conventional methods, approaches specifically developed for 3DCP exist. One of these approaches is adding a thin reinforcing cable during printing, which can be easily bent and adapted to almost any printing path. Previous studies used high strength smooth steel wires or impregnated carbon strands and showed promising structural behaviour in the direction of printing [31,32], but the anchorage of this reinforcement is challenging [33]. Other studies also employ short reinforcing bars inserted perpendicular to the printing direction [34–36]. For these approaches, the main challenge lies in the discontinuity of the reinforcement and the long lap splicing length required to provide continuity of the different segments.

While many of these reinforcement strategies have shown promising mechanical performance on a material scale, only a few have been studied on a structurally relevant scale [18,23,37,38]. It should be noted that all these structural tests were performed on complex geometries using only a single structure. Hence, the results are neither transferable to other structures nor easily usable for the development of mechanical models. Therefore, further research in the field of structural testing is needed to advance the knowledge of the structural performance of reinforced 3DCP elements.

2. Research significance

Reinforcement strategies are needed for 3DCP to have a lasting impact on the construction industry. These strategies should (i) warrant a ductile response of the structure; (ii) not hinder the geometric freedom of 3DPC; (iii) be economical; (iv) have a low environmental impact; and (v) be compatible with an automated fabrication process. So far, the existing reinforcement strategies for 3DCP do not, or only partially fulfil these requirements. Moreover, knowledge about their structural

behaviour is scarce. This paper investigates how a structure produced by 3DCP can be reinforced efficiently and how such a member behaves under structural loading. The proposed reinforcement concept (illustrated in Fig. 1 for an arbitrary structure with complex geometry) consists of adding interlayer shear reinforcement during the printing process (aligned in the printing direction), and post-installing longitudinal continuous reinforcement in printed voids (perpendicular to the printed layers). Both unbonded post-tensioning bars and bonded conventional reinforcement are tested as longitudinal reinforcement. For the interlayer shear reinforcement, high strength steel cable or aligned fibre reinforcement added between the layers of concrete are used. The behaviour without interlayer shear reinforcement is also investigated. The structural performance of these reinforcement approaches is evaluated in a series of nine four-point bending tests on extrusion 3DCP beams with simple geometry, facilitating a direct comparison with conventional concrete structures and a more profound understanding of the structural behaviour than complex geometries. The results, including crack patterns and their associated kinematics tracked with digital image correlation, are discussed and used to estimate the load carried by the interlayer shear reinforcement. Based on the results, a simple mechanical model is developed to understand the mechanical behaviour and allow for simple design recommendations.

3. Proposed reinforcement approaches for 3DCP

3.1. Longitudinal reinforcement

The longitudinal reinforcement is responsible for carrying the main loads of a structure. Therefore, reinforcement approaches with proven performance in conventionally cast concrete structures are preferred. The main reinforcement should be continuous, have a ductile response and warrant a structural behaviour that can be verified according to structural design code provisions, typically based on the lower bound theorem of plasticity theory. A well-suited solution to guarantee reinforcement continuity in 3DCP is to add the reinforcement – conventional passive reinforcing bars, actively prestressed bars, or tendons – in voids. For simple geometries, the use of straight and stiff bars is possible. For more complex geometries, tendons adapting to non-straight voids can be used; note that curved prestressing tendons generate deviation forces that need to be considered in the design. After adding the reinforcement, the void can either be left empty or grouted to achieve bond between reinforcement and surrounding concrete. For simplicity, this study only investigates straight longitudinal reinforcement. One set of beams is prepared with unbonded post-tensioning, while a second set uses bonded passive reinforcing bars. Other authors already applied both of these approaches (see Section 1), but the structural behaviour was not well documented.

3.2. Interlayer shear reinforcement

The structural requirements for the shear reinforcement are lower than for the longitudinal reinforcement. This lower requirement is reflected, e.g., by the fact that design codes typically require much lower minimum reinforcement ratios for stirrups when compared to other

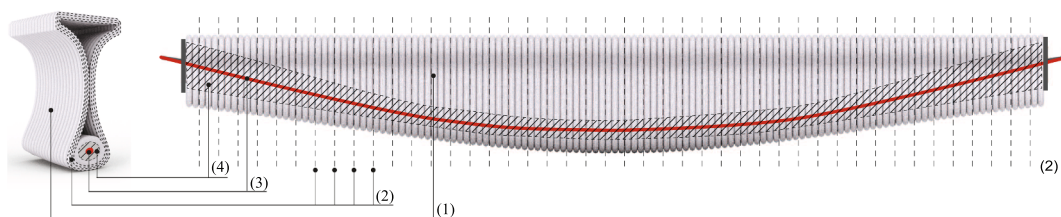


Fig. 1. Concept of the reinforcement strategy illustrated for a beam of complex geometry. (1) 3D concrete printing filament; (2) interlayer shear reinforcement placed in the interlayer during printing; (3) longitudinal reinforcement; (4) grouted void.

loadings. In fact, a strain hardening behaviour of the overall member can be achieved even with strain softening shear reinforcement [26], provided that the longitudinal reinforcement can resist additional load. This behaviour opens the way for innovative solutions, where the shear reinforcement is added during concrete printing to produce structurally active printed elements and not just lost formworks. This reinforcement should (i) allow high geometric flexibility to enable the production of complex structures and (ii) be easily added to the printing path, either in the matrix itself (i.e. printing of FRC) or added separately during printing to ensure an efficient overall process. This study focuses on the mechanical performance of reinforcement added separately during printing and does not address automation aspects. Therefore, the reinforcement was placed manually in all cases, as described in Section 4.3.2. One interlayer shear reinforcement approach used herein is the addition of a steel cable during printing, which has already been proven to be a viable solution for automatically placing continuous reinforcement [31]. Due to the concerns regarding the difficulty of anchoring high strength smooth cables [33], another strategy is also studied. In this strategy, steel fibres instead of a cable are added between the layers of concrete. This strategy is referred to as *aligned interlayer fibre reinforcement* and is described in the following paragraph.

The aligned interlayer fibre reinforcement has been proposed by the authors [39,40] and also investigated by Ahmed et al. [41] to profit from the geometric freedom provided with the placement of short discontinuous fibres and to overcome several processing challenges that arise when the fibres are added into 3DCP mixes (i.e. rheological changes, impossibility to pump long fibres...). Therefore, the authors conceived a reinforcement approach (see Fig. 2) where the fibres can be placed independently of the concrete printing. Right after each concrete layer, a robotic fibre placer adds the fibres in the desired orientation and amount on top of the existing layer. Once the nozzle returns to print the consecutive layer, the fibres are covered with concrete. This process allows the length of the fibres to no longer be limited by processing constraints of the concrete matrix, but merely by the overall geometry of the printed object. Additionally, this process enables the alignment of fibres in the direction of occurring tensile stresses and grading the fibre dosage according to the structural requirements.

The mechanical performance of the aligned interlayer fibre reinforcement using 60 mm-long end-hooked fibres in dosages of 0.7 and 1.4% was characterised through bending tests [42]. These tests showed that the fibres are more efficient when aligned and/or graded where they are needed. However, for high fibre dosages, the ultimate load was limited by a severe and sudden softening due to delamination of layers. Therefore, it was concluded that this approach (i) was not suitable as

main longitudinal reinforcement, due to its difficulty to obtain strain hardening and sufficient deformation capacity, but (ii) applicable as transverse (i.e., shear) reinforcement in combination with an alternative longitudinal reinforcement, as addressed in the experimental campaign described in the following chapter.

4. Experimental programme

4.1. Specimens and test setup

The conducted experimental campaign comprised nine four-point bending tests on beam specimens with and without interlayer reinforcement. Four beams had unbonded longitudinal post-tensioning reinforcement, subsequently referred to as Series 1. The other five beams used passive bonded longitudinal reinforcement (Series 2). Fig. 3 depicts the different test preparation steps, Fig. 4 shows the geometry of the beams and the test setup, and Table 1 gives an overview of all samples. All specimens were printed vertically and rotated by 90° for testing as conventional horizontal beams.

The four beams of Series 1 were printed as a hollow cross-section with a nominal width of 150 mm and a height of 300 mm. The cross-section was produced with a single filament with a printing width of approximately 45 mm and a layer height (h_{layer}) of 5 mm. For this series, the printing width was equal to the web width. The exact value of the web width (b_{web}) is reported in Table 1. For each printing session of Series 1, four half beams were produced. Two of these half-beams were reinforced with aligned interlayer fibres in dosages of around 0.3% (B-1xx-F03) only in the webs (see exact values of ρ_{web} in Table 1), and two half-beams were produced without reinforcement (B-1xx-NR). For all beams, the reinforcement was added manually between every other layer. Despite the fibres being concentrated in the interlayers, the reported fibre dosages refer to the average dosage per concrete volume. More details on the interlayer reinforcement are provided in Section 4.3.2. Once printed, both ends of the beam segments were cut to the desired length, ensuring a level and smooth surface for connecting the halves and attaching the steel anchorage plates. The two half beams were assembled by glueing to form one beam of 1.5 m length. Mortar beds were cast in the areas of the load application and the supports to ensure a proper load application on the rough printed surface. Furthermore, a smooth surface was created with gypsum and painted white on one side of the beam to allow speckling for the digital image

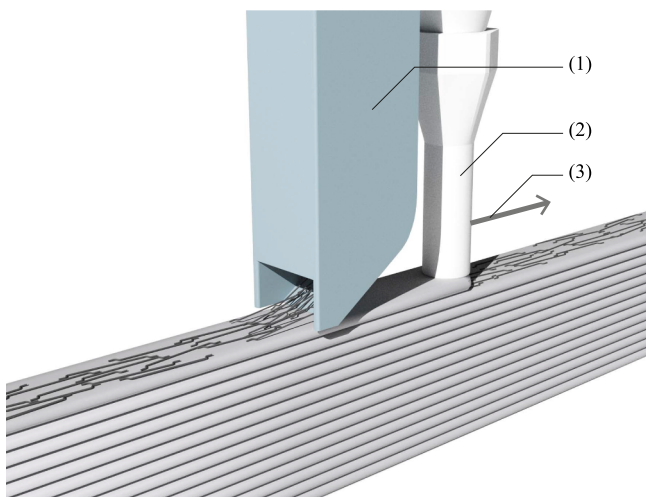


Fig. 2. Schematic sketch of the interlayer fibre reinforcement concept for 3DCP: (1) fibre placer, (2) extruder nozzle, (3) printing direction.

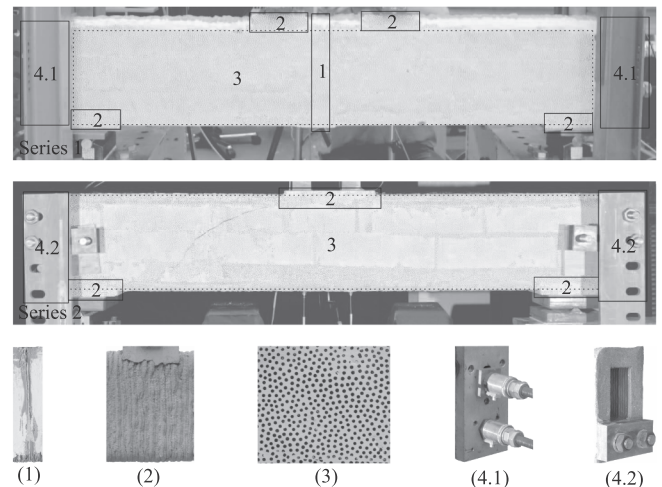


Fig. 3. Test preparation steps for Series 1 and 2: (1) Glueing of two segments (Series 1 and B-231-F06); (2) Mortar beds; (3) Gypsum surface painted white and speckled; (4.1) Post-tensioning reinforcement with anchorage plate and force measurement (Series 1); (4.2) Passive reinforcement with anchorage plate (Series 2).

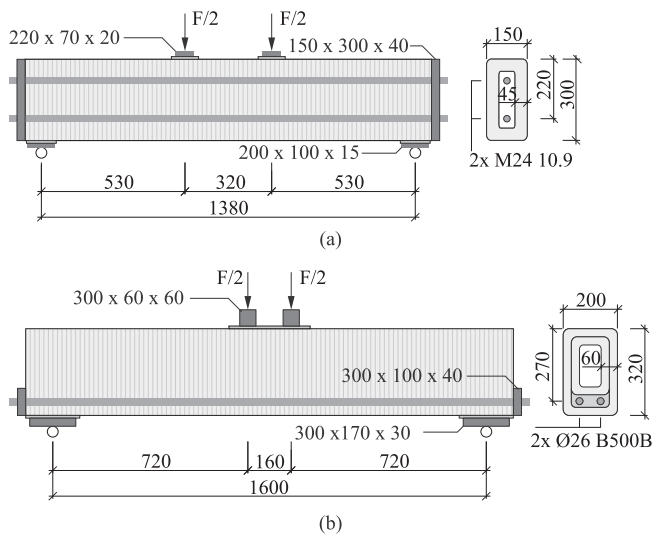


Fig. 4. Test setup, geometry and longitudinal reinforcement of the specimens (dimensions in [mm]): (a) Series 1; (b) Series 2.

correlation measurements. The longitudinal reinforcement consisted of two threaded rods of Type M24 10.9, which were inserted in the void of the cross-section and anchored behind steel plates covering the entire cross-section. Each prestressing rod consisted of two one-meter elements connected with a tailor-made connector. For the first two beams of Series 1 (Printing Session 11), a constant prestress of -9.3 MPa compression was applied. For the other two beams (Printing Session 12), the prestressing force was adjusted to achieve around -3 MPa on the top and -12 MPa on the bottom (average concrete compressive stress $\sigma_{cp} = -7.5$ MPa). The exact values of the applied forces are provided in Table 4.

For Series 2, five beams were produced. The cross-section was different from the one chosen for Series 1. It had a height of 320 mm and a width of 200 mm. The cross-section was printed by two adjoining filaments with a height of 5 mm and a width of approximately 30 mm, resulting in a web width of around 60 mm. The printing of two filaments allowed the cross-section to consist of two voids, one used for material saving and the other for the placement of longitudinal reinforcement. The beams were produced in pairs, one with interlayer reinforcement and one without (B-2xx-NR) per printing session. Among the beams with interlayer reinforcement, one contained a steel wire all around the cross-section (B-212-C01, Printing Session 21). The cable was composed of 19 high strength wires, type 1x19 [43] with a steel area of 0.6 mm^2 . The remaining two beams with interlayer reinforcement (Printing Sessions 22 and 23) were reinforced with a nominal aligned fibre dosage (ρ_{nom}) of 0.3% (B-222-F03) and 0.6% (B-231-F06), respectively, all around the cross-section. The fibres were prepared for a nominal web width of 50 mm. However, the actual web width had to be increased during printing to achieve a proper connection between the two adjoining filaments. The reinforcement was again placed manually between every other layer

of concrete. In contrast to Series 1, the beams of Series 2 were produced in one go, without the need of glueing. During Printing Session 23, the production could not be completed, and only two half-beams, one with and one without reinforcement, were printed. These two half-beams were glued together as in Series 1, and the side without interlayer reinforcement was externally reinforced and prestressed to ensure the failure in the side with interlayer reinforcement. Mortar beds, as well as a thin gypsum layer, were again provided before testing. As longitudinal reinforcement, two conventional reinforcing bars (B500B) with a diameter of 26 mm and threads at both ends (i.e. Bartec technology [44]) were then placed in the lower void and anchored with two steel plates. These bars were not prestressed, but the void was grouted 14 days before testing to achieve bond between the reinforcement and the surrounding concrete. The grout was cast into the void through a hole in one of the anchorage plates, in an upright position (as during printing).

4.2. Materials

The 3DCP process used in this study is based on the set on-demand method, in which an accelerator activates an Ordinary Portland Cement (OPC) mortar through active mixing performed inside the extruder tool. As a result, the extruded filament starts hardening after being expelled from the tool and placed along the print path. For DFC, the first set on-demand process was reported in the Smart Dynamic Casting project at ETH Zurich [45] and later implemented for 3DCP by Xtree [46]. The chemical principles and strategies for set on-demand processes can be found elsewhere [47,48].

The retarded mortar formulation developed at ETH Zurich for 3DCP has an open time of 6 h. It contains calcareous crushed sand with a grain size distribution between 0 and 2 mm, OPC at a water to cement ratio of 0.4, and additives for the desired rheological behaviour (Table 2). The accelerator consists of a Calcium Aluminate Cement (CAC) paste, retarded with 0.1% sodium gluconate, stabilised with 0.1% commercial polyethylene glycol, and with a water to cement ratio of 0.27 [49]. The quantity of accelerator determines the initial yield stress and the hardening rate of the filament, making the accelerator dosage depending on the printing time for one layer.

All beams were printed with the same concrete composition. However, the amount of accelerator was varied to ensure optimal printing for the different lengths of one layer. The accelerator dosage was 4.0% CAC

Table 2
Composition of the concrete used for printing.

	Quantity [kg/m ³]
Sand (0–2 mm)	1246.6
OPC (CEM I 52.5R)	539.2
Microsilica (MLife MS 100U)	56.0
Limestone (Nekafill 15 Filler)	105.0
Thickener (supplied by Akzo Nobel AG)	2.8
Water	277.7
Superplasticiser (BASF Glenium ACE 30)	6.5
Retarder (30% sucrose solution)	2.1

Table 1
Main properties of tested specimens. PS: printing session, ISR: interlayer shear reinforcement type.

	Beam	Codification	PS	Glued	b_{web} [mm]	σ_{cP} [MPa]	ISR	ρ_{nom} [%]	ρ_{web} [%]	Age [days]	f_c [MPa]	f_{ct} [MPa]
Series 1	111	B-111-NR	11	Yes	42	−9.3	–	–	–	28	73.2	2.4
	112	B-112-F03		Yes	43	−9.3	Fibres	0.30	0.38	31		
	121	B-121-NR	12	Yes	50	−7.5	–	–	–	29	70.4	2.5
	122	B-122-F03		Yes	52	−7.5	Fibres	0.30	0.31	29		
Series 2	211	B-211-NR	21	No	62	–	–	–	–	28	66.9	2.2
	212	B-212-C01		No	60	–	Cables	0.10	0.10	28		
	221	B-221-NR	22	No	56	–	–	–	–	28	71.9	2.3
	222	B-222-F03		No	58	–	Fibres	0.30	0.26	28		
	231	B-231-F06	23	Yes	62	–	Fibres	0.60	0.49	50	63.8	2.2

to OPC ratio for the beams in Series 1 and 4.6% CAC to OPC ratio for the beams in Series 2, respectively. The concrete strength was measured at the age of testing. The compressive strength (f_c) was determined on standard compression cylinders with a diameter of 150 mm and a height of 300 mm. The tensile strength (f_{ct}) was determined based on double punch tests on cylinders with a diameter of 150 mm and a height of 150 mm [50]. The cylinders were filled with accelerated printed material without any compaction. Per printing session (two beams), two compression tests and two double punch tests were performed. The concrete properties were determined after both beams of one printing session were tested. The average strengths can be found in Table 1. For the grouting of the voids of Series 2, the same concrete composition was used but without the addition of an accelerator. The strength of the grout at 14 days did not differ significantly from the printed concrete strength (the compressive strength was 7 to 18% lower, while the tensile strength was 4% lower than the strength in Printing Session 21 and 3% or 40% higher for Printing Sessions 22 and 23, respectively). Where needed, the beam segments were glued together with a two-component epoxy (Aradlit AW 2101/HW 2951). For the mortar beds, high strength fast hardening repair mortar (Sika FastFix-4 [51]) was used.

Fig. 5 shows the stress-strain characteristics of the different reinforcements (post-tensioning reinforcement is denoted as p , conventional steel reinforcement as s , and cable reinforcement as w). For each reinforcement, at least three specimens were tested in direct tension until failure. The average measured properties (axial tensile modulus of elasticity (E_t), yield strength (f_{iy}), yield strain (ϵ_{iy}), ultimate strength (f_{iu}), and strain at peak stress (ϵ_{iu}) are reported in Fig. 5. The yield strength was determined as the point with plastic strains of 0.002. The post-tensioning reinforcement (M24 10.9) had a yield stress of 990 MPa without a well-defined yield plateau. The conventional steel bars (B500B $\varnothing 26$ mm) had a well-defined yield plateau at 500 MPa. Both types of steel used for the longitudinal reinforcement showed pronounced ductility. The steel cable, on the other hand, failed with minimal plastic deformations. For further considerations, the cable is modelled as a perfectly round steel cable with a diameter corresponding to its steel cross-section.

The fibres used in this study were commercial end-hooked fibres (Dramix 3D 65/35 [52]) with a length of 35 mm and slenderness of 65. The mechanical performance of the aligned fibre reinforcement was characterised following the procedure proposed by EN 14651 [53] on printed samples that were produced identically as the beams. Therefore, the beams were tested in three-point bending with a span of 500 mm and a 25 mm notch at midspan. The printed samples did not match the dimensions specified in EN 14651 [53] exactly (around 15 mm wider than the nominal 150 mm value). The effective fibre concentration (ρ_{eff}) therefore again slightly varied since the fibres were prepared for the nominal geometry. The real geometry was considered in the evaluation of the results. For each fibre dosage (0.3 and 0.6%), three specimens were tested after 28 days to ensure similar curing times as for the beams. The results of residual flexural tensile stresses (f_R) and crack mouth

opening displacements ($CMOD$) obtained from the three-point bending tests are shown in Fig. 6.b and reported in Table 3. The residual flexural tensile stresses f_{R1} , f_{R2} , f_{R3} and f_{R4} correspond to a $CMOD$ of 0.5, 1.5, 2.5 and 3.5 mm, respectively. Both fibre dosages showed deflection hardening behaviour. The higher fibre dosage led to higher resistance. The samples with the higher fibre dosage exhibited a larger scatter (see the grey area in Fig. 6). The coefficient of variation was 5% for the lower and 11% for the higher fibre content for f_{R1} , and increased to 11 and 16%, respectively, for f_{R3} . It should be noted that even the scatter of the higher aligned fibre content is in the lower range of the scatter typically obtained in similar tests for conventional fibre reinforcement [54]. The direct tensile behaviour was back-calculated by applying the linear post-cracking model proposed in the *fib* Model Code 2010 [55]. The results of this inverse analysis are shown in Fig. 6.c.

4.3. Production

4.3.1. Concrete printing

The beams were printed in the Robotic Fabrication Laboratory of ETH Zurich. The printing setup consists out of a 9-axes kinematic system: 3- axes Güdel gantry and a 6- axes ABB IRB 4600 robot. Two continuous cavity pumps and a custom extruder tool processed the

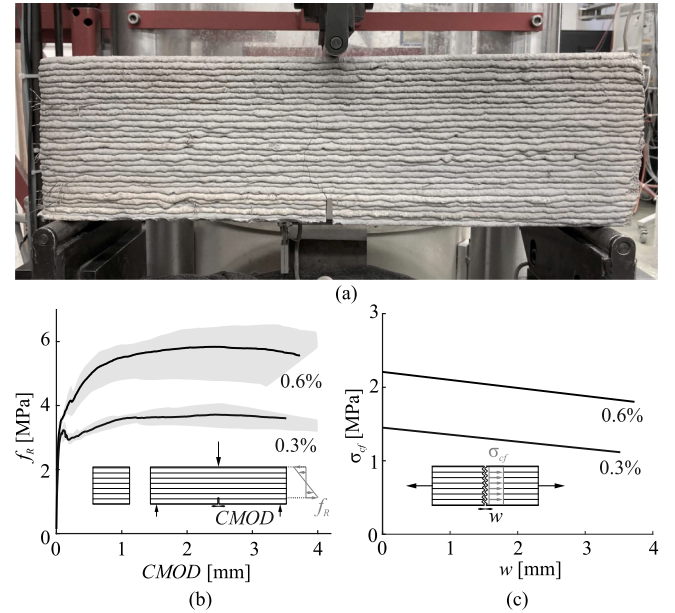


Fig. 6. Mechanical characterisation of the interlayer fibre reinforced 3D printed concrete according to EN 14651: (a) Image from the test setup; (b) Residual flexural tensile strength (f_R) versus crack mouth opening displacement ($CMOD$); (c) Linear post-cracking model.

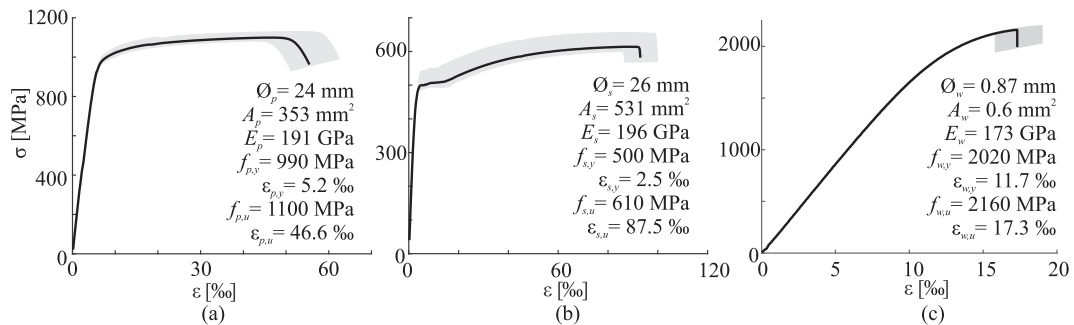


Fig. 5. Stress-strain relationship of the used reinforcements: (a) Post-tensioning reinforcement M24 10.9 used for Series 1 (p); (b) Passive reinforcement $\varnothing 26$ B500B used in Series 2 (s); (c) Cable reinforcement for Series 2 (w). Black lines show average results, and grey fillings indicate the range of test results.

Table 3

Overview of the results of the material characterisation of the interlayer fibre reinforced 3D printed concrete.

	ρ_{eff}	f_{R1}	f_{R2}	f_{R3}	f_{R4}	f_c	f_{ct}
	[%]	[MPa]	[MPa]	[MPa]	[MPa]	[MPa]	[MPa]
Beams 0.3%	0.27	3.2	3.6	3.7	3.6	72.2	2.2
Beams 0.6%	0.53	4.9	5.7	5.8	5.7	68.7	2.2

material at a constant flow rate within the system. The retarded concrete and the CAC were individually pumped towards the extruder tool, where an active mixer ensured homogenisation. The extruder tool is mounted on the 6th axis of the ABB robot, and its tip is the reference point for the print path [56]. The print path was uploaded into an industrial robot controller (type IRC 5), and executed by an operator. In the present study, motion commands were run in manual mode as a safety procedure owing to manually adding the reinforcement during printing, with an upper speed limit of 250 mm/s.

The print paths design for the beams started from one layer, and the target geometry was achieved by repeating similar layer trajectories at 5 mm layer height. In Series 1, the print path connected the four independent beam sections into one contour, as shown in Fig. 7.a. The ratio between contour length and the typical 150 mm/s print speed used in this series gives a period (interlayer time) of 23 s between two successive layers. In Series 2, the robot printed two layers of one beam, after which it transitioned to the second beam to print the corresponding two layers, following the path shown in Fig. 7.b. The print speed of 160 mm/s used in this series resulted in an alternating period of 13 s and 28 s between consecutive layers. The interlayer time of 28 s was used to place the interlayer shear reinforcement manually. The layer height was controlled by the position of the nozzle and did therefore not vary from its nominal value. However, the filament width was the result of the flow rate, the layer height, and the robot's print speed. Even if the pumping system should dose constant volumes, the actual material quantity may deviate from the expected, depending on the actual viscosity. This material variability is tackled by slightly adjusting the print speed during printing [57] to keep the filament width as close as possible to the intended value.

4.3.2. Placement of interlayer shear reinforcement

During printing, the interlayer shear reinforcement was placed manually on top of every second layer of concrete. The initially highly fluid state of the extruded material allows adding reinforcement between the layers, while still ensuring adequate bond between the reinforcement and the printed material. Before production, the cables were first cut to the correct length and pre-bent. After placing a cable on the

concrete layer, slight pressure was applied to prevent it from sticking out. The fibres were packed as glued sets of fibres. Therefore, to place them manually, they had first to be washed to dismantle the sets. The weight of fibres was calculated for the entire cross-section over a height of two layers (i.e. the distance between the interlayers with fibres) and afterwards subdivided for the webs and flanges. The required amount of fibres was then placed on a plastic foil that was lying on top of a magnetic strip. These strips were prepared in advance to allow fast fibre placement during the production of the beams. The magnetic strip held the fibres in position while placing them on top of the extruded concrete layer, and the plastic foil allowed removing the strip while keeping the fibres in place. After removal of the magnetic strip, slight pressure was applied to the foil by hand to avoid fibres sticking out and being dragged along by the nozzle. The fibre placing process and the resulting cross-sections can be seen in Fig. 8. For Series 1 only the webs and for Series 2 the webs, as well as the flanges, were reinforced. Due to the particularities of 3DCP discussed in Section 4.3.1, the cross-section of the printed specimens did not match the nominal dimensions precisely. Therefore, the reinforcement content varied from the nominal one (ρ_{nom}). The actual contents of the webs (ρ_{web}) are reported in Table 1.

4.4. Measurement instrumentation

For Series 1, the prestressing force (P_i) was applied right before testing and controlled by one load cell on each rod. The vertical loads applied (F) to the beams were controlled by load cells installed on hydraulic cylinders. The displacement field on the surface of the beams was tracked quasi-continuously with stereo digital image correlation [58].

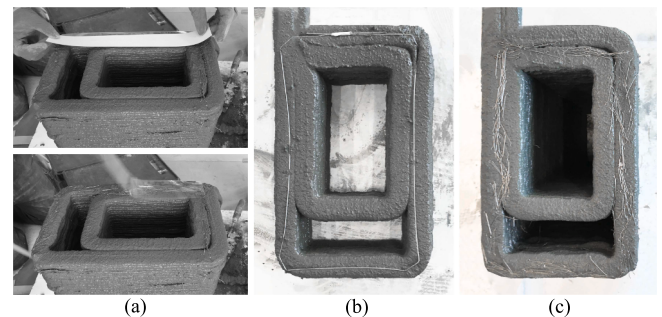


Fig. 8. Fibre placing and resulting cross-section for Series 2: (a) Placement of the fibres with a magnetic strip and a plastic foil in the web of the beam with 0.3% of fibres; (b) Cross-section with cable reinforcement; (c) Cross-section with 0.3% of fibre reinforcement.

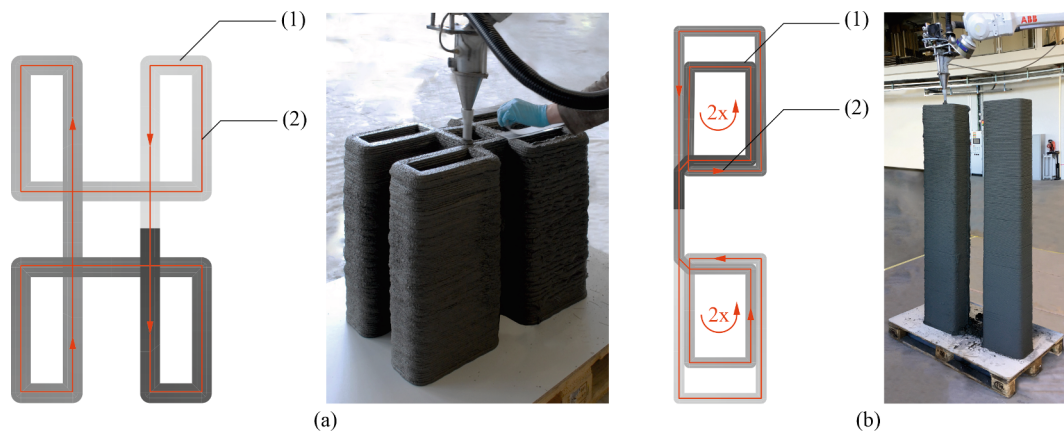


Fig. 7. Print path and prototype during printing: (a) Series 1 with a filament width of 50 mm; (b) Series 2 with a filament width of 30 mm. The grey gradient along the print path corresponds to the age of the concrete filament (1), and the red line shows the print path trajectory (2). (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

The speckle pattern consisted of black circular speckles of approximately 3 mm in diameter. The images were recorded at a frequency of 1 Hz using two monochrome cameras (type FLIR Grasshopper3 12.3 MP) with 25 mm focal length Quioptic MeVis-C lenses. The correlation was carried out with the commercial software VIC-3D (Correlated Solutions Inc. [59]) using a subset size of 15 pixels, a step size of 2 and a strain filter size of 7. From the results, the crack patterns and their associated kinematics were computed with the Automated Crack Detection and Measurement procedure [60]. The continuous evolution of local crack openings was directly measured in the VIC-3D software through manually placed virtual extensometers.

4.5. Test procedure

The loads were applied using a manual hydraulic pump, at a mid-span displacement (Δ) rate of approximately 0.4 mm/min for Series 1 and 0.5 mm/min for Series 2. For Series 1 two hydraulically coupled cylinders (same pressure line) were used, while for Series 2 only one cylinder applied the load. LVDT's below the load application points were used to monitor the deflections during the test. Both ends of the beams were supported transversally (out-of-plane) by two steel profiles to avoid out of plane movements. The profiles were provided with wheels (Series 1) or Teflon (Series 2) to avoid longitudinal restraint. B-111-NR was only supported in the transverse direction at one end, resulting in considerable out of plane movement (i.e. torsion).

5. Test results

5.1. Series 1: Beams with unbonded post-tensioning reinforcement

Fig. 9 shows the load-deformation behaviour of the Series 1 beams, and their crack patterns and associated crack openings. The applied shear force (V) was normalised with the static depth (d) and web width b_{web} . The results are summarised in Table 4. All beams behaved similarly, with a quasi-linear initial uncracked phase, followed by the formation of one or two bending cracks and a gradual decrease of the stiffness until the beams failed in bending due to crushing of concrete (B-CC) in the flexural compression zone. After the formation of one or two bending cracks, all deformation localised in these cracks and no new cracks formed, which lead to high strains in the compression chord and

brittle failure. Similar behaviour was already reported by Leonhardt and Walther [61] in 1962. While the failure of the beams without interlayer shear reinforcement was explosive, leading to a sudden total collapse of the beam, the overall structure of the specimens containing fibre reinforcement stayed intact. Although the fibres could not be activated due to the absence of shear cracks, they were still able to control concrete microcracks [24], which is probably the cause for the observed increase of the shear strength of around 15% with respect to the beams without interlayer shear reinforcement.

The crack patterns showed that the bending cracks developed in some areas independently of the layering, while in other cases the layers triggered and guided the cracks. The particular crack pattern in B-111-NR, where the bending cracks transitioned into longitudinal cracks, was caused by the undesired torsion (as discussed in Section 4.5) due to lacking out-of-plane support. This torsion did not seem to affect the maximum load that could be reached. However, the maximum deflections at midspan were reduced.

5.2. Series 2: Beams with bonded conventional reinforcement

5.2.1. Load deformation behaviour

The results of Series 2 are summarised in Table 5, while the load-deformation behaviour is shown in Fig. 10. The differences between Series 1 and 2 cannot be attributed only to the different longitudinal reinforcement, but also to their different cross-sections. The nominal shear stress (i.e. the shear load normalised by the web width and the static depth) is considered to analyse the load-deformation behaviour to allow for a more direct comparison. The beams showed a significantly different behaviour depending on the used interlayer shear reinforcement. All beams developed multiple bending cracks before the first shear crack was formed. The first bending crack formed between a load F of 13 to 29 kN and the first shear crack developed between 86 kN (B-221-NR) and 115 kN (B-231-F06). In the beams without interlayer shear reinforcement, the load could be further increased after the formation of shear cracks for beam B-211-NR, but not for B-221-NR that failed at shear cracking. For all beams with interlayer shear reinforcement, the load and deformation could be increased significantly until a brittle shear failure in diagonal tension (S-DT) was reached. The beam with 0.3% of fibres reached the lowest load-bearing capacity. After forming shear cracks, the fibres bridged the crack, and the force could be

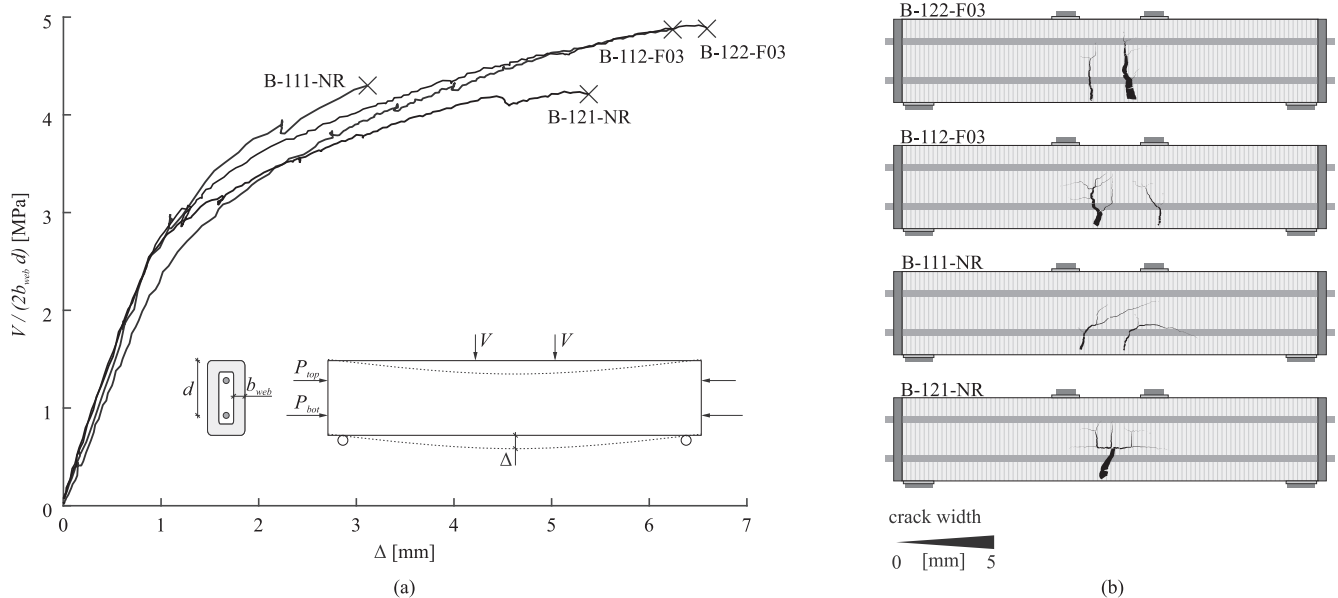


Fig. 9. Results of the unbonded post-tensioned beams (Series 1): (a) Load deformation response; (b) Cracking patterns immediately before failure (99% F_{max}) with line thickness equivalent to crack width.

Table 4

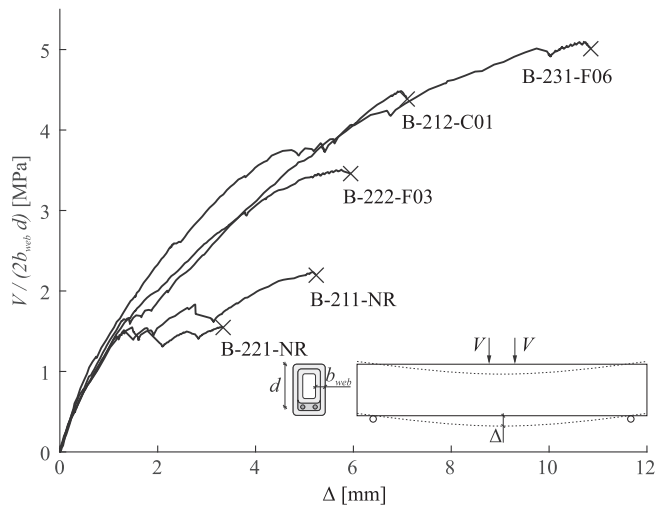
Overview of results of beams with unbonded post-tensioned longitudinal reinforcement (Series 1).

Codification	F_{max} [kN]	V_{max} [kN]	$\frac{V_{max}}{2b_{web}d}$ [MPa]	Δ_{max} [mm]	$P_{0,top}$ [kN]	$P_{0,bot}$ [kN]	$P_{u,top}$ [kN]	$P_{u,bot}$ [kN]	Failure mode
B-111-NR	160.4	80.2	4.3	3.2	149.5	149.9	137.5	200.1	B-CC
B-112-F03	184.8	92.4	4.9	6.1	149.1	148.9	134.0	252.6	B-CC
B-121-NR	186.6	93.3	4.2	5.1	80.3	178.8	75.8	262.0	B-CC
B-122-F03	225.3	112.7	4.9	6.6	82.5	180.6	83.8	292.0	B-CC

Table 5

Overview of results of bonded passive reinforced beams (Series 2).

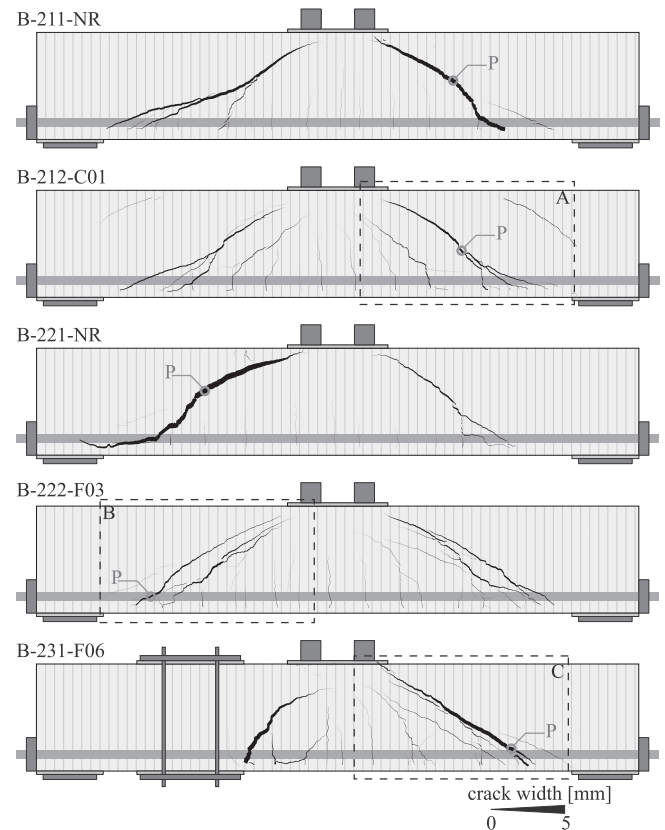
Codification	F_{max} [kN]	V_{max} [kN]	$\frac{V_{max}}{2b_{web}d}$ [MPa]	Δ_{max} [mm]	$\delta_{v,max}$ [mm]	$\delta_{v,60\%}$ [mm]	Failure mode	$V_{s,i}$ [kN]	$\frac{V_{s,i}}{V_{max}}$ [%]
B-211-NR	149.7	74.8	2.2	5.2	–	–	S-DT	–	–
B-212-C01	290.6	145.3	4.5	7.0	1.7	0.85	S-DT	73.7	51%
B-221-NR	94.1	47.1	1.6	3.3	–	–	S-DT	–	–
B-222-F03	219.8	109.9	3.5	5.8	1.3	0.13	S-DT	73.3	67%
B-231-F06	341.1	170.5	5.1	10.6	2.3	0.27	S-DT	117.7	69%

**Fig. 10.** Load deformation response of beams with bonded conventional longitudinal reinforcement (Series 2).

increased further. The stiffness decreased progressively due to fibre pull-out until failure. The beam with double the fibre dosage (B-231-F06) behaved similarly, but only reached a 55% higher maximum load. This increase is similar to the one observed in the material characterisation tests shown in Section 4.2, which indicated that for the aligned inter-layer fibres, the bond between fibre and matrix might decrease with increasing fibre density at the interface, making higher fibre dosages less effective. No fibre rupture was observed at the critical cracks, confirming that failure was due to fibre pull-out. The cable reinforced beam (B-212-C01) also failed in a brittle manner, since all cables in the critical shear crack ruptured simultaneously. In this case, the stiffness remained fairly constant from the formation of the first shear crack until failure, as typically observed in similar tests with conventional steel reinforcement. Despite the lower reinforcement content used for the cable reinforced specimen (0.1%), its strength was between those of the specimens with fibres (0.3–0.6%). This result can be explained by the continuity of the cable reinforcement and its higher tensile strength.

5.2.2. Crack patterns and kinematics of Series 2

Fig. 11 shows the crack patterns and crack openings at 99% of the maximum load for all beams of Series 2. As already observed in Series 1 (see Section 5.1), the influence of the printing layers on the crack pattern

**Fig. 11.** Crack patterns for beams with bonded conventional longitudinal reinforcement (Series 2) right before failure (99% F_{max}). Line thicknesses are proportional to crack width (evolution of crack kinematics at point P are displayed in Fig. 12; crack kinematics for areas A, B and C can be found in Fig. 13).

seemed negligible, i.e., the behaviour matched that of conventionally cast monolithic beams. The bending cracks initiated between layers at the bottom of the beam. However, they further progressed in the beam independently of the layering. Part of this observation might be attributed to a bias in the crack pattern caused by the thin gypsum layer used to apply the speckle pattern. However, this behaviour might also indicate that the potential reduction in the concrete tensile strength between printed layers is close to, or even lower than, the inherent scatter of the tensile strength for this printing setup. The formation of the shear cracks

seemed to be entirely independent of the layering. The two samples without interlayer shear reinforcement developed single diagonal shear cracks with large openings. These shear cracks developed suddenly, leading to a brittle failure without significant previous deformations. In the specimens with interlayer shear reinforcement, diagonal cracks formed as well, but the opening of the shear cracks was controlled. While both fibre reinforced beams developed multiple diagonal cracks, the cable reinforced beam exhibited only one diagonal crack that branched out into three cracks in the lower half of the beam. The cable reinforced beam had wider shear cracks than the fibre reinforced samples at the same applied load. These observations indicate that the fibre reinforcement results in closer spaced and narrower cracks than cable reinforcement.

In Fig. 12, the evolution of the vertical component of the crack opening (δ_v) for the point of the critical crack with the largest opening (i. e. point P) is presented. The kinematics of the critical crack of the reinforced samples were analysed in more detail in Fig. 13 for two load steps close to failure (in critical cracks branching out, only the branch with the largest opening was considered). The results confirm the unstable development of the critical crack for the two unreinforced samples. The cracks in the two fibre reinforced beams behaved similarly, with decreasing stiffness at increasing crack opening. This softening response may be interpreted as a consequence of two phenomena. With increasing crack opening, more and more fibres were activated. At the same time, however, the fibres started to transition from activation to the pull-out phase and started to soften locally. This behaviour resembles the response of the material characterisation tests of the FRC material (see Fig. 6). On the other hand, the shear crack of the beam with cable reinforcement opened very significantly at cracking, after which it opened linearly with increasing load almost until failure. The observed crack opening under service loads (SLS) (Fig. 12), assuming the serviceability range as 60% of the ultimate load, of the beam with cable reinforcement would not comply with typical durability requirements. It seems that the higher efficiency of the cables in terms of ultimate load could not be exploited in design. On the other hand, both beams with fibre reinforcement showed much better serviceability behaviour, with much smaller crack openings at service loads (about 25% of cables), since the fibres, in contrast to the cables, are activated at small crack openings. Given the inherently large scatter of the cracking behaviour, further research is required to confirm these tendencies observed on a single specimen.

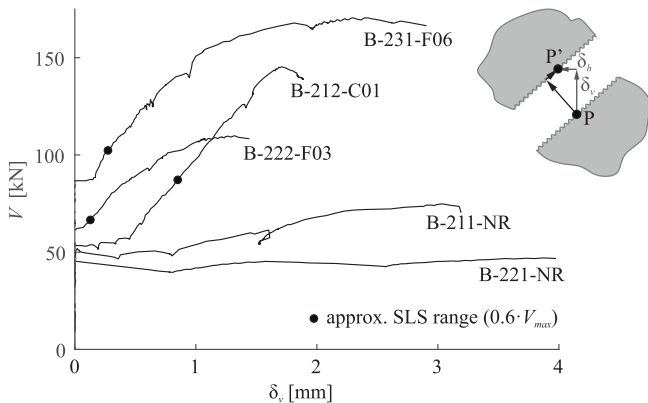


Fig. 12. Evolution of the vertical component of the crack opening for the point with the largest opening (marked as Point P in Figs. 11 and 13).

6. Analysis of the shear transferred by the interlayer shear reinforcement in Series 2

6.1. Quantification based on the measured crack kinematic

The contribution of the interlayer shear reinforcement to the total shear capacity can be calculated based on the measured kinematics of the critical shear crack (Fig. 13) and the particular material and bond properties of the reinforcement. The calculation steps are depicted in Fig. 14. Based on the total crack displacement (resultant of crack opening and crack slip) and the crack opening direction measured along the crack (Fig. 14.a), the component of the crack opening in the direction of the reinforcement is computed (Fig. 14.b). By using a suitable constitutive model for the reinforcement stress transfer across a crack (stress-crack opening in the direction of reinforcement, i.e. vertically), the reinforcement stress for any given vertical opening can be calculated (Fig. 14.c). The load carried by the reinforcement results from the integration of the contribution of the reinforcement along the crack. For the fibre reinforcement, the fibre bridging stresses are typically expressed as effective fibre tensile stresses in the concrete (σ_{cf}), for which the stress-crack opening relationship derived from the material characterisation tests (Fig. 6.c) is adopted here. The shear transfer contribution of the fibres can thus be calculated as follows:

$$V_{sf} = 2b_{web} \int \sigma_{cf}(\delta_v) dx \quad (1)$$

where b_{web} is the width of the web and x is the direction of the longitudinal reinforcement. Note that due to the vertical orientation of interlayer fibres, there is no increase in fibre efficiency due to skew crack opening (activation of more fibres per unit length of the crack), contrary to conventional fibre reinforced concrete with randomly oriented fibres [62].

For cable reinforcement, the shear force transferred by the reinforcement is:

$$V_{sw} = 2b_{web} \int \rho_{web} \sigma_{wr}(\delta_v) dx \quad (2)$$

where ρ_{web} is the cable reinforcement ratio in the web and σ_{wr} is the stress of the cable at the crack.

In the following, a model linking the cable stress at the crack to the opening of the crack in the direction of the cable (i.e. $\sigma_{wr}(\delta_v)$) is proposed. The cable is modelled as a linear elastic brittle material (see Fig. 15.a). In this model, the tensile strength of the cable is taken equal to its experimental yield strength $f_{y,w}$. A pull-out behaviour near the critical shear crack, neglecting any mechanical interaction with other cracks or specimen edges, is assumed (Fig. 15.b). Hence, the stress of the reinforcement at the crack (σ_{wr}) is activated exclusively by bond-shear stresses (τ_b), which are determined using a simplified, rigid-perfectly plastic bond shear stress-slip relationship with $\tau_b = \tau_{b0}$ (as proposed in the Tension Chord Model [63] for conventional reinforcement until the onset of yielding). With these assumptions, formulating equilibrium at the crack, one gets the bond length (l_b):

$$l_b = \frac{\sigma_{wr} \varnothing_w}{4\tau_{b0}} = \frac{\varepsilon_{wr} E_w \varnothing_w}{4\tau_{b0}} \quad (3)$$

where ε_{wr} is the reinforcement strain at the crack, E_w is the modulus of elasticity of the cables and $\varnothing_w$ is the diameter of the cables. The crack opening (w) related to a specific stress level results from the integration of the cable strains near the crack:

$$w = \int \varepsilon_w \cdot dl = \varepsilon_{wr} l_b = \frac{\sigma_{wr}^2 \varnothing_w}{4E_w \tau_{b0}} \quad (4)$$

where the much smaller concrete strains have been neglected. While for conventional deformed steel reinforcement (ribbed surface) $\tau_{b0} = 2f_{ct}$ is

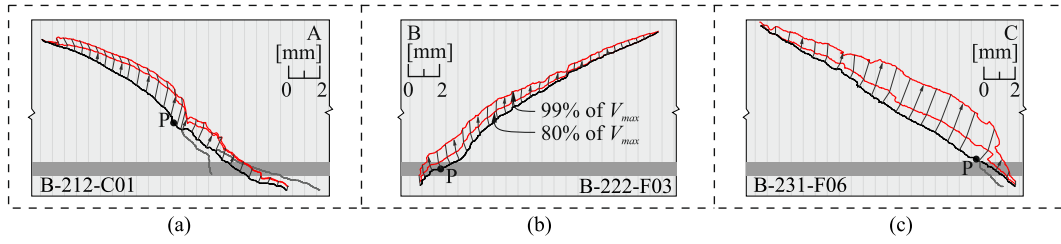


Fig. 13. Kinematics of the critical shear crack (crack width and slip) of the beams of Series 2 with shear reinforced samples at 80 and 99% of the ultimate load: (a) Cable reinforcement (B-212-C01); (b) 0.3% fibre reinforcement (B-222-F03); (c) 0.6% fibre reinforcement (B-231-F06).

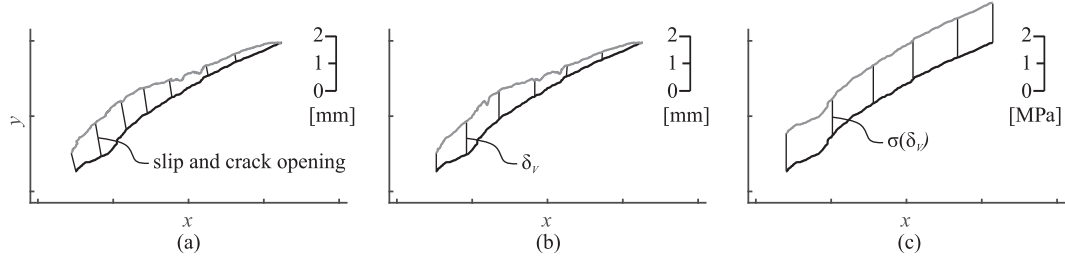


Fig. 14. Calculation of the reinforcement stresses along the critical crack for beam B-222-F03: (a) Slip and crack opening; (b) Vertical component of the crack kinematics; (c) Resulting fibre bridging stresses along the crack.

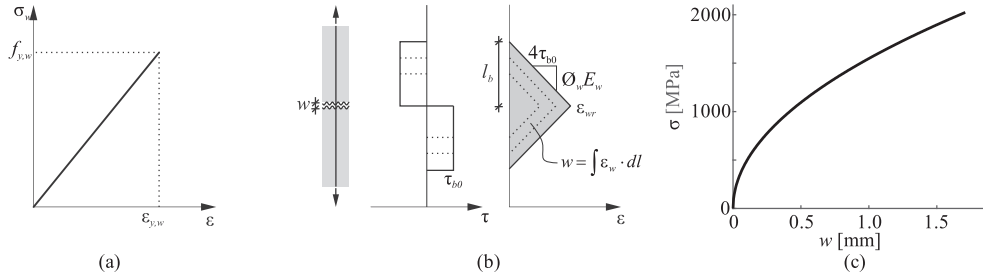


Fig. 15. Idealisation of the response of a bonded cable reinforcement near a crack: (a) Linear elastic-brittle idealisation of the cable; (b) Local strains near crack with assumed rigid-perfectly plastic bond behaviour; (c) Resulting stress-crack opening relationship of the cable reinforcement used in this study.

typically adopted [63], there are no clear recommendations for smooth bars or cables, where the bond is highly dependent on the casting conditions. Therefore, the bond shear stress τ_{b0} is estimated based on the experimental observations, particularly cable ruptures and corresponding crack openings: If a cable failed at a crack opening $\delta_{v,max}$, the bond shear stress can be determined by rearranging Eq. (3), i.e.:

$$\tau_{b0} = \frac{f_{y,w}^2 \phi_w}{4E_w \delta_{v,max}} \quad (5)$$

In specimen B-212-C01, the cable ruptured at a vertical crack opening of $\delta_{v,max} = 1.7$ mm. The corresponding bond shear stress of 3.0 MPa ($1.4f_{ct}$) is much higher than the values measured recently by Bos et al. [33] for cables also inserted in 3DCP. The higher bond stresses measured in the present study might be related to (i) the higher fluidity of the used concrete, (ii) the fact that the cables were slightly pushed into the matrix and (iii) a potential anchorage of the cable in the flanges (the activation length of the cable at failure obtained from Eq. (3), i.e. 292 mm, was similar to the beam depth). Further studies are required to determine the influence of each factor. Once the bond shear strength has been determined, the stress-crack opening behaviour of the cables shown in Fig. 15.c is obtained by applying Eq. (4).

The results of the proposed procedure to estimate the shear force resisted by the interlayer shear reinforcement ($V_{s,i}$) are summarised in Table 4. The shear transfer provided by the interlayer shear reinforcement is the main shear transfer mechanism, carrying more than half the

applied shear force (around 50% for the cable reinforcement and 70% for the fibre reinforcement). The remaining part of the applied shear force was transferred by different mechanisms not quantified in this study, such as dowel action, shear transfer in the compression zone and aggregate interlock. The contribution of the latter mechanism and its interaction with fibre reinforcement has recently been studied in detail [62], identifying a large scatter of aggregate interlock. Furthermore, the contribution of the interlayer shear reinforcement determined above is subject to considerable uncertainty: For the fibre reinforcement, the used linear model of the *fib* Model Code 2010 [55] is a coarse approximation of the actual direct tension behaviour, and for the cable reinforcement, any inelastic branch of the material tensile response was neglected, which tends to underestimate the shear transfer contribution. Moreover, the approach shown in this section cannot be applied for design purposes, in which the crack kinematics are unknown. This issue is not limited to the particular case of reinforcement considered in the present study, but common to all models considering fixed crack inclinations and attributing the shear strength of concrete beams to various mechanisms. It can be avoided by considering fictitious, stress-free cracks in an arbitrary direction, which is equivalent to neglecting the contribution of the tensile strength of concrete to the shear resistance. Such a model is discussed in the following section.

6.2. Plastic estimation of shear strength for preliminary design purposes

In the following, a simple stress field model for the design of interlayer shear reinforcement in 3DCP beams is presented. The model essentially corresponds to the variable angle truss model known from shear design of conventionally reinforced beams. It assumes (i) fictitious, stress-free cracks in arbitrary direction; (ii) perfectly plastic stress-strain relationships of the interlayer shear reinforcement; (iii) sufficient capacity of the longitudinal reinforcement; and (iv) an infinite concrete compressive strength. Note that the latter assumptions are merely adopted for simplicity; checks of concrete compressive stresses and provisions for additional longitudinal reinforcement due to shear (tension shift) could be implemented just like in the variable angle truss model.

Assuming a perfectly plastic behaviour of the interlayer shear reinforcement requires special considerations, given the real response of both materials used in this study as interlayer shear reinforcement: FRC exhibits a softening response, and cables fail with minimal plastic deformations. To cope with these differences, an equivalent plastic behaviour leading to a similar capacity as the real response is proposed. For the fibre reinforcement, the plastic model of the *fib* Model Code 2010 [55], using a constant residual strength of the fibres, is adopted. More specifically, the plastic capacity of FRC in tension is assumed as $f_{R3}/3$, where f_{R3} is the tensile bending stress at $CMOD_3$ (2.5 mm) [55]. For the cable reinforcement, it is observed that for a crack opening linearly along its length, considering the stress-crack opening behaviour derived in Section 6.1, the average cable stress along the crack is equal to two-thirds of the peak stress at the point with the largest opening. Therefore, a plastic capacity of two-thirds of the cable tensile strength, which would lead to the same ultimate load of the cable, is used here as “yield stress” of the cable. In addition to the aforementioned factors to design based on a plastic behaviour, a material safety factor on the effective fibre tensile stresses in the concrete should be considered. The reduced scatter observed in the material tests of the aligned fibre reinforcement indicates that a safety factor as used for common FRC might be appropriate. In the *fib* Model Code 2010 [55], a safety factor of 1.5 is advised. However, while the separate placement of the fibres might have a

beneficial effect on the scatter, the printing process might have a negative influence. Therefore, further research is required to study the interaction of these two processes and to determine the suitable material safety factor for aligned shear fibre reinforcement when applied together to 3DCP.

The stress field yielding the highest shear strength (excluding a direct strut with infinitely small dimensions) is illustrated in Fig. 16.b and c. Formulating equilibrium along the diagonal AB, the shear strength $V_{R,i}$ is limited by the capacity of the interlayer shear reinforcement:

$$V_{R,i} = K_P \cdot \sigma_{cs} \cdot z \cdot \cot(\alpha) \cdot 2 \cdot b_{web} \quad (6)$$

where K_P is a correction factor for the assumption of perfectly plastic behaviour, σ_{cs} is the equivalent tensile strength of the concrete due to the reinforcement, b_{web} is the width of one web and z is the inner lever arm.

As already discussed, for the fibre reinforcement, K_P is 1/3 and σ_{cs} is f_{R3} . For the cable reinforcement, K_P is 2/3 and σ_{cs} is $\rho_w f_{y,w}$ with ρ_w being the reinforcement content of the cable (i.e. 0.1%). In the stress field shown in Fig. 16.b, the interlayer shear reinforcement can be activated over a length of $a = 720$ mm, resulting in shear capacities of 113.4 kN for the cable reinforcement and 103.3 kN and 176.2 kN for the fibre dosage of 0.3% and 0.6%, respectively. Comparing these shear strengths to the experimentally observed values (145.3 kN, 109.9 kN, and 170.5 kN), it is seen that they are in excellent agreement for the fibre reinforced beams (conservative by 6% and unconservative by 3%, respectively), but conservative (22%) for the cable reinforcement. The latter can be attributed to the conservative assumption of K_P ; assuming a uniform crack opening, i.e. $K_P = 1$, for the cable reinforcement, an unconservative value of 170.1 kN would be obtained.

The results above are based on the assumption that very flat inclinations of the compression fields in the webs (18.6°) are possible without causing concrete crushing nor yielding of the longitudinal reinforcement. For design purposes, the inclination should be limited to a minimum value of about 30° , similar to the provisions for conventional structural concrete. Such a stress field is illustrated in Fig. 16.e and f, where the angle of the parallel compression field is set to 34° with the assumption of $z = 0.9d$. The resulting shear resistances amount to 56.7 kN for the cable reinforcement, 51.6 kN for a fibre dosage of 0.3% and

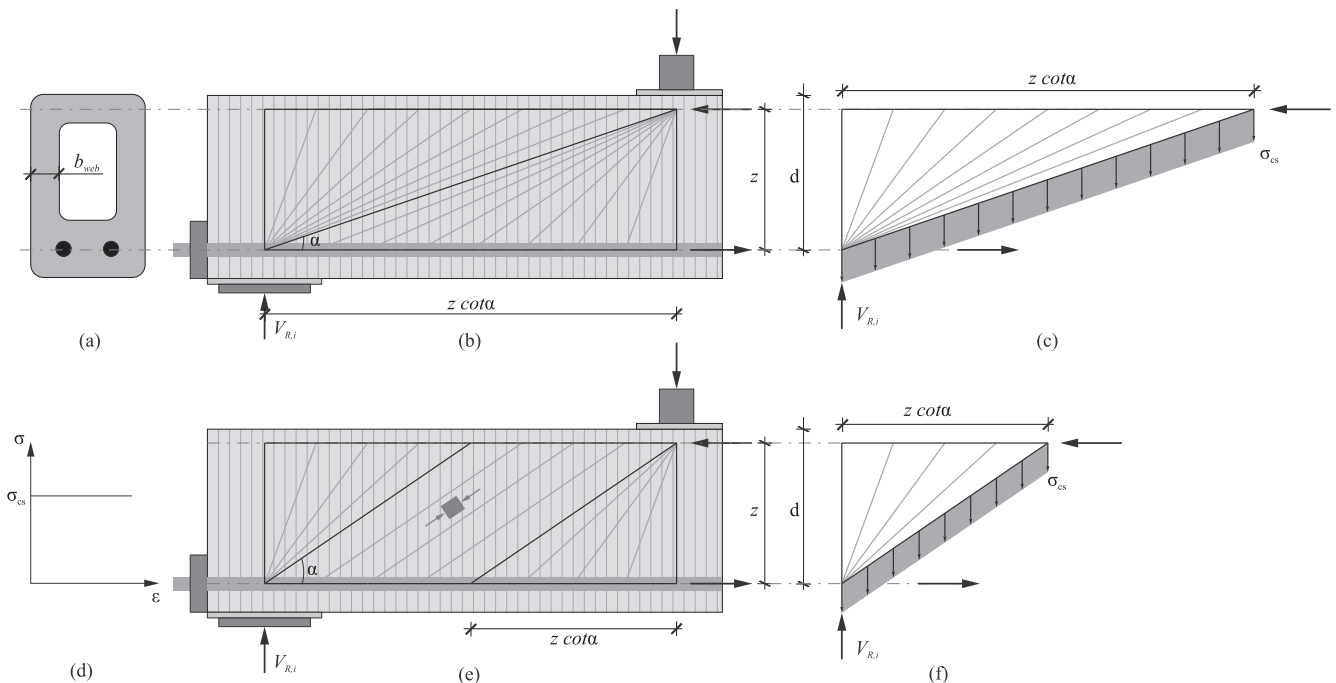


Fig. 16. Simplified model for pre-design of interlayer shear reinforcement in 3DCP beams: (a) Cross-section; (b) and (e) stress field; (c) and (f) stress fan with the resulting stresses from the interlayer shear reinforcement; (d) ideally plastic material behaviour.

88.1 kN for a fibre dosage of 0.6%. These estimations are significantly on the safe side, which would also be the case when considering a compression field at 30°, and appear suitable for preliminary design. Still, the model should be applied with care. Most importantly, 3DCP beams should be designed such that failure is governed by the bending reinforcement to ensure a ductile failure. Additional shear verifications not covered by this model, such as the failure of concrete, should also be addressed. To this end, refined models, including kinematic considerations [64,65], should be extended to 3DCP beams.

7. Conclusions

Due to its automation possibilities, digital fabrication with concrete (DFC) offers the potential to build complex structures with similar effort as standard geometries, but with enhanced structural efficiency and reduced material consumption. However, reinforcing approaches for DFC are not yet developed to ensure compliance with structural integrity requirements, which hinders the implementation of the technologies. A study of requirements for reinforcing for 3D printed concrete (3DCP) indicated that:

- A promising solution to resist the main tensile forces in a 3D printed concrete (3DCP) structure – and, to the authors' opinion, currently the best-suited one – is the addition of the reinforcement in printed voids. This approach allows using inexpensive standard reinforcement such as conventional passive reinforcing bars or prestressing tendons, ensuring a ductile response.
- In the transverse direction, a hardening response of the reinforcement is not necessarily required, which facilitates the addition of reinforcement during printing to ensure a higher degree of geometric freedom.

Based on these findings, this study investigated the structural response, and crack pattern and kinematics of various reinforcement approaches for 3DCP beams. The main findings of the investigation are:

- The influence of the printing layers on the crack pattern was negligible in all tests. Hence, the used printing setup does not introduce a significant reduction of the concrete tensile strength between printed layers.
- The beams with unbonded post-tensioning failed in a brittle manner due to the crushing of concrete in bending, and with the deformations localised in few bending cracks. The presence of fibres between the layers increased the ultimate load slightly, presumably due to better control of micro-cracking.
- The beams with bonded conventional reinforcement showed multiple bending and shear cracks, which allowed the interlayer shear reinforcement to be activated. In the specimens without interlayer shear reinforcement, unstable shear crack development led to brittle failure. For all beams with interlayer shear reinforcement, the load and deformation could be increased significantly after the formation of shear cracks until a shear failure was reached (still brittle, due to cable rupture and fibre pull-out, respectively).
- Due to its continuity and higher tensile strength, the cable reinforcement was more efficient at failure than the aligned fibre reinforcement. However, the higher efficiency of the cables cannot be exploited fully in design due to unfavourable serviceability behaviour.
- The estimation of the bond shear stresses of the cable reinforcement based on the measured crack kinematics and the assumption of a pull-out model leads to a lower value than for deformed reinforcing bars but significantly higher than reported by other authors.
- The shear transfer provided by the interlayer shear reinforcement was estimated based on the measured crack kinematics, showing that this is the main shear transfer mechanism if a critical crack is considered.

- A simple stress field model yields excellent predictions of the ultimate load, particularly for the fibre reinforced beams. Combined with a conservative limitation of the stress field inclination, it appears suitable for the pre-design of interlayer shear reinforcement. Still, the model should be applied with care. Moreover, 3DCP beams should be designed such that the failure is limited by the bending reinforcement to ensure a ductile failure.

CRediT authorship contribution statement

Lukas Gebhard: Conceptualization, Methodology, Visualization, Writing - original draft, Project administration. **Jaime Mata-Falcón:** Conceptualization, Writing - review & editing, Supervision. **Ana Anton:** Conceptualization, Methodology, Visualization, Writing - review & editing. **Benjamin Dillenburger:** Writing - review & editing. **Walter Kaufmann:** Conceptualization, Writing - review & editing.

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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