

Flexural behavior of steel and polypropylene fiber-reinforced concrete beams with low longitudinal reinforcement ratios

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ABSTRACT

The use of fiber-reinforced concrete in structural applications is currently under development. It has been proven that high steel fibers and low reinforcing steel lower the structure's ductility since localized cracks are generated. It is hypothesized that this can be solved by using less rigid fibers such as polypropylene. In this research, a comparative study of the influence of synthetic and metallic fibers in normal resistance reinforced concrete with a low reinforcement ratio is carried out. This study focuses on the structural behavior and development of cracking at the service limit state (ELS) and the ultimate limit state (ELU). To this end, an extensive experimental campaign was conducted, comprising, first, the characterization of the material and, second, the testing of reinforced concrete beams. The variables used are (1) the material of the fibers: polypropylene (PP) and steel (ST), (2) the volume of fibers: 0.33 %, 0.66 % and 1.00 %, and (3) the size of the beams: 1.6 and 3.2 m long. The results show that concrete with 1 % steel fibers exhibited higher post-cracking stiffness than with PP fibers, increasing by 104 % and 71 % in 4.00 m beams, respectively, due to its greater residual strength. A higher fiber volume (1 %) increased the yield moment by up to 50 % and the maximum load by 22 %–25 %. However, it reduced ductility, especially with 1 % steel fibers, where it decreased by up to 63 % in 1.60 m beams due to crack localization. In 4.00 m beams, fiber-reinforced concrete showed better flexural performance, with similar maximum load increases as in 1.60 m beams, but with a smaller ductility reduction (24 % with steel fibers and 11 % with PP fibers), indicating that the greater span promotes better strain redistribution.

1. Introduction

Fiber-reinforced concrete has demonstrated greater toughness and the ability to exhibit residual strengths, which unreinforced concrete does not possess [1]. Consequently, it is widely used in the construction of slabs and tunnels. Nevertheless, its use is still limited in other structural elements due to the lack of standard development. The publication of the Model Code 2010 [2] was a significant step in its application to structural elements. However, recent research has shown occurrences not covered by the standard. Several authors [3–5] have reported that when low reinforcement ratios and a high volume of metallic fibers are used, localized and distinct cracks of greater width than the rest of the cracks appear in fiber-reinforced concrete (FRC) beams subjected to bending. The localization of the cracks is influenced by the quantity of fibers, which restricts the formation of cracks, and their high stiffness, which does not allow the cracks to widen. Therefore, investigating the flexural behavior of FRC beams with lower stiffness may be a solution to eliminate this problem. Synthetic polypropylene fibers, PP, are presented as the ideal material because they have a low modulus of elasticity, ranging between 3.5 and 9.5 GPa [6]. In addition, recent research has shown that PP fibers improve tensile strength and toughness and

restrict crack width in concrete [7,8].

Deflection and cracking during the bending of reinforced concrete beams are the two primary phenomena considered in the serviceability limit state (SLS). The stiffness of both the uncracked and cracked sections influences these properties. Steel fibers in reinforced concrete enhance its axial stiffness in tension, in both cracked and uncracked states, as the fibers resist the tensile forces before the reinforcing steel yields [9]. This increase in stiffness is useful for reducing the deflection of an element subjected to bending in the SLS [10]. Swamy et al. [11] concluded that beam stiffness under service loads increases by 50–70 % with the incorporation of steel fibers, resulting in substantial reductions in deformation, section curvature, and steel strain. According to Conforti et al. [3], a noticeable reduction in deflection occurs when the longitudinal reinforcement varies between 0.9 % and 1.2 %, and the residual strength, f_{r1} , according to the EN 14651 test [12], exceeds 3 MPa. For lower reinforcement ratios (0.5 %) and lower residual strength values, the reduction in deflection is less, though not negligible. Although there is an influence between the concrete's toughness and crack spacing, that higher toughness leads to a smaller crack spacing, this effect is not highly significant [10]. The increase in axial stiffness in tension within a reinforced concrete element leads to reduced crack spacing and

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narrower crack widths [13]. Yoo & Moon [5] studied the cracking behavior of Fiber-Reinforced Concrete (FRC) and reported that the number of cracks decreases when the reinforcement ratio is reduced and increases with an increase in fiber volume. The adherence between the steel bar and fiber-reinforced concrete is superior compared to concrete without fibers [9]. This is because, in fiber-reinforced concrete, transverse cracks act as load transfer bridges that minimize reinforcement slippage, thereby reducing the formation of longitudinal splitting cracks [14]. The cracking behavior of reinforced concrete elements in tension is strongly influenced by the diameter of the reinforcing steel bar; an increase in diameter leads to an increase in crack spacing and wider crack widths [15]. Generally, an increase in the reinforcement ratio reduces the average crack spacing in fiber-reinforced and non-fiber-reinforced concrete elements [16]. Experimental results show that fiber-reinforced concrete (FRC) and fiber-reinforced self-compacting concrete exhibit significant load increases after first cracking, with recorded increases of 72 % and 81 %, respectively, compared to high-strength concrete, where fiber efficiency is lower [17].

The ultimate limit state (ULS) in the behavior of reinforced concrete beams under bending is determined by the ultimate load capacity, ductility, and failure mechanisms. According to Conforti et al. [3], the increase in load capacity due to the incorporation of steel fibers is significant only when the section has relatively low reinforcement ratios (0.5 %). Swamy & Sa'ad [18] reported that the maximum increase in ultimate flexural load capacity was 10.5 % when using 1 % of fibers and working with reinforcement ratios between 0.94 % and 1.78 %. Meanwhile, Bentur & Mindness [19] reported an increase of up to 55 % with 1.5 % of fibers and a reinforcement ratio of 0.5 %. For post-yield displacements, steel bar deformation and crack openings generally concentrate in a single location, forming a localized crack. The confining effect of the fibers prevents the reinforcing steel from developing its deformation capacity along the constant moment region, resulting in premature localized failure. There is a discussion among various authors about the influence of the appearance of this localized crack on the ductility of the beam. Conforti et al. [3] mention that the appearance of localized cracks does not influence the ductility of the element with typical reinforcement ratios of 0.5–1.2 %. On the other hand, Yoo and

Moon [5] indicate that this localized crack reduces the ductility of the beam. These conclusions were drawn from tests on reinforced concrete beams with very low reinforcement ratios, ranging from 0.2 % to 0.4 % and fiber volumes between 0.5 % and 1.0 %. Dancygier and Berkover [20] also reported a pronounced reduction in ductility with reinforcement ratios less than 0.5 % and using fiber volumes of 0.5–0.8 %.

Research on the flexural behavior under monotonic loads of reinforced concrete beams with polypropylene fibers is limited, as shown in Table 1. Akin et al. [21] conducted experiments with macro and micro polypropylene fibers and concluded that adding macrofibers positively affects the ductility and the concrete's cracking behavior. They also found that the load capacity increased by 13 % when working with polypropylene fibers and a 0.5 % reinforcement ratio. One of the few studies on the tensile stiffening of concrete sections with polypropylene fibers concluded that adding PP fibers enhances the tensile stiffening. However, this improvement is not proportional to the fiber volume, as the most significant result was achieved with a low fiber content [22]. According to Sahoo et al. [21], the influence of both polypropylene and steel fibers on the beam's load capacity is limited. The author worked with a reinforcement ratio of 1.35 % and a fiber volume of 0.5–1 % [23]. On the other hand, the same author mentions that the section's ductility increases by 120 % compared to pattern concrete. Another important point to highlight from Table 1 is that previous research works always used a single beam size ranging from 0.90 m to 3.60 m. To the best of the authors' knowledge, a comparison between the flexural behavior of medium-sized 1.60 m beams and large 4.00 m reinforced concrete beams with polypropylene and steel fibers with low steel ratios has not been reported yet. It is crucial to expand our knowledge about the comparison of the contribution of polypropylene fibers in the serviceability limit state (SLS) and ultimate limit state (ULS) concerning steel fibers in beams of different dimensions.

This research conducts a comparative study on the influence of incorporating polypropylene fibers in normal-strength reinforced concrete with low reinforcement ratios, in contrast to the addition of steel fibers in reinforced concrete. The present study focuses on the structural behavior through the analysis of the serviceability limit state (SLS) and the ultimate limit state (ULS) of steel and PP fiber-reinforced RC Beams.

Table 1

Review of previous studies and the current research characteristics.

Ref	Mat	lxd (mm)	V _f (%)	L _{beam} (m)	f _c (MPa)	ρ _s (%)	ELS		ULS		P- δ	M- φ
							Cr	Y	P _{max}	δ _{max}		
[18]	ST	50x0.50	0.5, 1.0	2.5	36.9–40.9	0.94, 0.99, 1.78						
[19]	ST	50x0.50	1.0, 1.5	1.0	-	0.56						
[24]	ST	40x0.70	1.0, 2.0	2.0	43.0–48.0	0.38, 0.61, 0.85						
[25]	ST	30x0.50	0.5, 1.0, 1.5, 2.0	2.0	40.8–42.3	1.60						
[26]	ST	35x0.64	1.5	2.0	68.9	0.57						
[27]	ST	18x0.3518x0.4525x0.50	1.0	2.0	64.1–82.6	2.22						
[28]	ST	60x0.80	0.5, 1.0	3.1	48.6–111.4	1.18, 1.77, 2.37						
[29]	ST	35x0.54	0.4, 0.6	3.6	-	1.03						
[30]	ST	53x1.00	0.5, 1.0, 1.5	3.0	48.0–55.6	0.30						
[31]	ST	30x0.55	1.0	3.5	22.3–43.9	0.20, 0.30, 0.53, 0.81, 1.06, 1.60, 2.02, 2.13, 2.50						
[23]	ST, PP	ST: 50x0.50PP: 40x0.86	ST: 0.5, 1.0PP: 0.5, 1.0	2.0	35.0	1.35						
[20]	ST	35x0.55	0.5, 0.8	3.2	26.9–37.5	0.15, 0.40, 0.63, 0.94, 1.26, 3.27						
[32]	ST	40x0.50	0.5	1.5	44.0–85.0	1.22, 1.77, 2.42						
[5]	ST	35x0.53	0.3, 0.5, 0.8, 1.0	3.0	43.4	0.18, 0.27, 0.32, 0.41						
[4]	ST	30x0.6660x0.75	0.5, 1.0, 2.0	1.2	76.3–95.2	0.28, 0.44, 0.70						
[3]	ST, PP, GF	ST: 50x1.00PP: 58x0.67GF: 36x0.54	ST: 0.3, 0.6PP: 0.5, 1.0GF: 0.7, 1.3	0.9	45.2–54.6	0.50, 1.20						
[21]	PP	54x0.65	0.5	3.0	39.5–43.7	0.29						
PR	ST, PP	ST: 35x0.53PP: 40x0.86	ST: 0.3, 0.6, 1.0PP: 0.3, 0.6, 1.0	1.6, 4.0	43.9 – 47.2	0.29, 0.32						

Ref: Reference, Mat: material, lxd: length and diameter, V_f: fiber dosage, L_{beam}: Beam length, f_c: compressive strength, ρ_s: Steel Reinforcement, Cr: cracking, Y: yielding, P_{max}: maximum load, δ_{max}: maximum deflection, P-δ: load deflection curve, M-φ: curvature moment relationship, PR: present research.

A comprehensive experimental campaign was carried out to achieve this, consisting of material characterization and testing of reinforced concrete beams. The variables used include (1) the fiber material, polypropylene and steel; (2) the fiber volume, high, medium, and low; and (3) the size of the beams, medium-sized at 1.60 m and large-sized at 4.00 m.

2. Experimental program

The experimental program was carried out in the structural laboratory at the University of Lima. Below, we provide the material details and mix design. Additionally, we detail the setup and instrumentation of the reinforced concrete beam flexural test, along with specimen details.

2.1. Materials and mix design

The concrete was elaborated using the following conventional materials: water, cement, coarse aggregate, fine aggregate, superplasticizer admixture, steel fibers, and polypropylene fibers. The water used was potable water with a pH of 7. The cement used was the Portland cement type I, according to ASTM C150 [33], with a density of 3.13 g/ml, consisting of 12 % dicalcium silicate (C₂S), 55 % tricalcium silicate (C₃S), 10 % tricalcium aluminate (C₃A), and 10 % tetracalcium aluminoferrite (C₄AF), see Table 2. The coarse aggregate used was standardized crushed stone according to HUSO 5 and ASTM C33 [34], with a maximum nominal size of 19 mm and a fineness modulus of 7.49. The fine aggregate used was river sand, also complying with the particle size distribution of ASTM C33, with a fineness modulus of 2.59; furthermore, the percentage of material passing through the No. 200 sieve was 7.6 %. The physical properties of the aggregates are presented in Table 3. The superplasticizer admixture used was manufactured in accordance with ASTM C494 [35], classified as type D, plasticizer, and type G, superplasticizer, with a density of 1.20 g/ml. Two types of fibers were used: steel fibers and polypropylene fibers, see Fig. 1 and Table 4. The steel fibers used had hooks at the ends, with a length of 35 mm, a diameter of 0.53 mm, and a slenderness ratio of 65. They had a tensile strength of 1100 MPa, a modulus of elasticity of 200 GPa, and a density of 7850 kg/m³. The polypropylene fibers used were straight and had a textured surface, with a length of 40 mm, a diameter of 0.86 mm, and a slenderness ratio of 46. They had a tensile strength of 540 MPa, a modulus of elasticity of 4.7 GPa, and a density of 910 kg/m³.

The mix design was prepared based on the ACI method [36], with seven different mixes designed, as detailed in Table 5. The reference concrete without fibers was designed for a target strength of 40 MPa. A water-to-cement ratio of 0.50 was chosen to balance workability and compressive strength, as a lower ratio improves strength and durability. The paste-to-aggregate ratio of 0.49 was selected to optimize cohesion and density, enhancing the mix's consistency [22]. Steel and polypropylene fibers were incorporated to improve crack resistance and toughness, as they significantly enhance the post-cracking behavior and overall structural integrity. Additionally, a high-range water-reducing admixture was incorporated at a rate of 1.5 % by cement weight to improve workability without compromising strength. These parameters were carefully selected to optimize the concrete's structural performance and durability.

The fibers were added at volumes of 0.33 %, 0.66 %, and 1.00 %; for polypropylene fibers, this equates to 3.0, 6.0, and 9.1 kg/m³, and for steel fibers, 25.9, 51.8, and 78.5 kg/m³. Each parameter in the mix design was tailored to assess the impact of these fibers on the concrete's

Table 2

Properties and chemical composition of the Portland cement.

Type	ρ (g/ml)	C ₂ S (%)	C ₃ S (%)	C ₃ A (%)	C ₄ AF (%)
Portland Type I	3.13	12	55	10	10

ρ : density

Table 3

Properties of the coarse aggregate and fine aggregate.

Aggregate type	Graduation	NMS (mm)	FM	% Passing No. 200
Coarse aggregate	HUSO 5	19	7.49	-
Fine aggregate	-	-	2.59	7.6 %

NMS: Nominal Maximum Size; FM: Fineness modulus

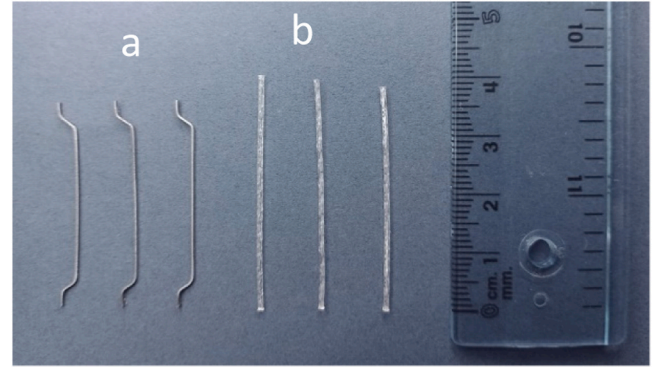


Fig. 1. Fibers used: (a) steel fibers and (b) polypropylene fibers.

Table 4

Geometric and mechanical properties of the fibers.

Fiber	l (mm)	d (mm)	l/d	σ_{ult} (MPa)	E (GPa)	ρ (kg/m ³)
Steel	35	0.53	65	1100	200	7850
Polypropylene	40	0.86	46	540	4.7	910

l = Fiber length; d: Fiber diameter; σ_{ult} : Maximum strength; E: Elastic modulus; ρ : density

mechanical properties, ensuring a comprehensive evaluation of their effects on structural performance.

2.2. Characterization tests and specimen preparation

The following tests were conducted on the fresh concrete: slump test according to ASTM C143 [37], unit weight, and air content according to ASTM C138 [38]. A sample was taken for each batch of mix. To characterize the concrete in its hardened state, the following tests were conducted: compressive strength, ASTM C39 [39], and modulus of elasticity, ASTM C469 [40], three-point bending test of fiber-reinforced concrete beams [12]. Three specimens were prepared for each test. These mechanical characterization tests were conducted using specialized equipment for characterizing post-cracking concrete, with two load frames: one for flexural testing with a capacity of 350 kN and the other for compression testing with a capacity of 3000 kN (CONTROLS, Autotest Multitest). For the compressive strength test, cylinders with a diameter of 100 mm and a height of 200 mm were prepared, and for the modulus of elasticity test, cylinders with a diameter of 150 mm and a height of 300 mm were used. The concrete specimens were prepared and cured according to ASTM C192 [41] standards. A diagonal-axis mixer with a capacity of 311 liters was used. The sequence of pouring was as follows: first, the mixer was cleaned and moistened, then 30 % of the water was added, followed by 100 % of the coarse aggregate. At this stage, it was allowed to mix for one minute to ensure that the aggregates were superficially moistened. Next, all the fine aggregate was added and mixed for another minute. After that, all the cement was added. Immediately after, the superplasticizer additive was mixed with the remaining 50 % of the water and allowed to mix for 150 s. The fibers were added progressively during this interval. Finally, the remaining 20 % of the water was added, and mixing continued for another 150 s.

Table 5Design of concrete mixtures using ACI method (kg/m^3).

Type	Water	Cement	Fineaggregate	Coarseaggregate	Waterreducer	PPFibers	STFibers
B_Reference	260.0	520.0	750.0	850.0	7.8	-	-
B_PP_0.33 %	260.0	520.0	750.0	850.0	7.8	3.0	-
B_PP_0.66 %	260.0	520.0	750.0	850.0	7.8	6.0	-
B_PP_1.00 %	260.0	520.0	750.0	850.0	7.8	9.1	-
B_ST_0.33 %	260.0	520.0	750.0	850.0	7.8	-	25.9
B_ST_0.66 %	260.0	520.0	750.0	850.0	7.8	-	51.8
B_ST_1.00 %	260.0	520.0	750.0	850.0	7.8	-	78.5

2.3. Beam flexural test setup and specimen details

The geometric properties of the tested beams are presented in Table 6. Two beam sizes were tested: medium-sized beams measuring 1.60 m in length, 0.15 m in width, and 0.40 m in height, and large beams measuring 4.00 m in length, 0.20 m in width, and 0.30 m in height. A four-point bending test configuration was set up to ensure a constant moment region. Therefore, in both small and large beams, the loads were applied at the central thirds, and the distance between supports was 1.50 m for the small beams and 3.90 m for the large beams. The beam supports were of the roller type. The load was applied using a hydraulic actuator with a capacity of 1000 kN and a piston travel of 500 mm (MTS, Serie 201). This load was distributed as two-point forces on the beam through a rigid metal frame, as shown in Fig. 2. The displacement rate was 2 mm/min for the 1.60 m beams and 5 mm/min for the 4.00 m beams, programmed through a controller (MTS, Flextest 60).

Due to economic constraints associated with the fabrication and testing of large-scale specimens, only one beam was manufactured and tested for each beam type. While the authors acknowledge the importance of statistical repeatability for enhancing the credibility of experimental results, this approach is consistent with several structural experimental studies in the literature, which also relied on single large-scale specimens because of high production and testing costs [5,21]. Despite this limitation, every effort was made to ensure consistency in the mixing, casting, curing, and testing procedures, thus supporting the validity of the observed structural behavior.

The longitudinal reinforcement in the beams was designed to meet the minimum reinforcement ratio calculated according to the ACI 318 code [42] for a concrete compressive strength of 45 MPa and a steel yield strength of 420 MPa. Consequently, the longitudinal reinforcement for the 1.60 m beams consisted of two 8 mm deformed steel bars, resulting in a reinforcement ratio of 0.32 %. In the case of the 4.00 m beams, three 8 mm steel bars were used, resulting in a reinforcement ratio of 0.29 %. In both cases, transverse reinforcement was used to prevent shear failure. This transverse reinforcement consisted of placing steel stirrups at 100 mm intervals from the support points to the load application points, leaving the central third without stirrups. The stirrups were made with 135° hooks. The beam details are shown in Fig. 3a and Fig. 3b. The reinforcing steel used complies with ASTM A615

standards, with a minimum yield strength of 450 MPa and a minimum ultimate strength of 680 MPa.

The instrumentation used to measure the beam's behavior included: (1) LVDTs were used to measure the beam's deflection at 0.25 L, 0.50 L, and 0.75 L, and one LVDT was placed at each support. In total, 5 LVDTs were used; (2) three strain gauges (extensometers) were used at the midpoint of the beam: one at the top of the section and two on the longitudinal steel bars. This was done to assess the change in the neutral axis position at different loading stages, and (3) a thermal camera was used to assess the change in temperature within the section due to the applied loads and for digital image correlation. Two reflectors were installed to illuminate the beam adequately. The test setup and instrumentation configuration can be seen in Fig. 3.

The selection of mix parameters and testing conditions was based on previous studies that emphasize their relevance to the structural behavior of fiber-reinforced concrete. A water-to-cement ratio of 0.50 and a target compressive strength of 40 MPa were selected following ACI guidelines [36], ensuring a balance between workability and mechanical performance. The fiber volume fractions used (0.33 %, 0.66 %, and 1.00 %) fall within the most commonly studied range, as observed in the majority of previous works summarized in Table 1, and have proven effective for evaluating post-cracking stiffness and ductility [3,5,20]. Two beam lengths (1.60 m and 4.00 m) were selected to investigate the influence of span on stress redistribution and crack localization, an area with limited coverage in the literature, also identified in the literature review presented in Table 1. The longitudinal reinforcement ratios were deliberately kept at minimum values calculated according to ACI 318 [42] to simulate critical reinforcement conditions where the contribution of fibers becomes more evident. This approach is supported by the findings of previous studies that demonstrated fiber effects are more pronounced in beams with low reinforcement ratios [3–5]. Additionally, earlier research confirmed that low reinforcement levels combined with fibers can significantly influence ductility, failure mode, and ultimate strength [18,19]. Therefore, the experimental program was carefully designed to assess the influence of fiber type, dosage, and beam size on the serviceability and ultimate limit states.

3. Experimental results and discussion

3.1. Mechanical properties of materials

Table 7 summarizes the results of the characterization of the concrete mixes used. The reference concrete had a slump of 40 mm, while the fiber-reinforced mixes showed a typical trend of decreasing workability as the fiber dosage increased. The compressive strength, f'_c , of the reference mix was 45 MPa, comparable to studies [3,5,24], which reported ranges between 43 MPa and 48 MPa for similar mixes. Normal variations in the 43–47 MPa range were observed for the fiber-reinforced mixes. The modulus of elasticity resulted in values ranging from 32 to 40 GPa, slightly exceeding the upper limits reported by previous studies [19,28], where typical values were between 30 and 38 GPa for similar fiber dosages. Additionally, the table presents the calculated tensile mechanical properties, where f_L is the flexural tensile strength, and f_{R1} , f_{R2} , f_{R3} , f_{R4} represent the residual strengths at

Table 6

Detail of the reinforced concrete beams.

Beam identification	Batch	Steel area (mm^2)	Reinforcement ratio (%)
Reference_1.60	B_Reference	100.53	0.32
PP_0.33_1.60	B_PP_0.33	100.53	0.32
PP_0.66_1.60	B_PP_0.66	100.53	0.32
PP_1.00_1.60	B_PP_1.00	100.53	0.32
ST_0.33_1.60	B_ST_0.33	100.53	0.32
ST_0.66_1.60	B_ST_0.66	100.53	0.32
ST_1.00_1.60	B_ST_1.00	100.53	0.32
Reference_4.00	B_Reference	150.80	0.29
PP_1.00_4.00	B_PP_1.00	150.80	0.29
ST_1.00_4.00	B_ST_1.00	150.80	0.29

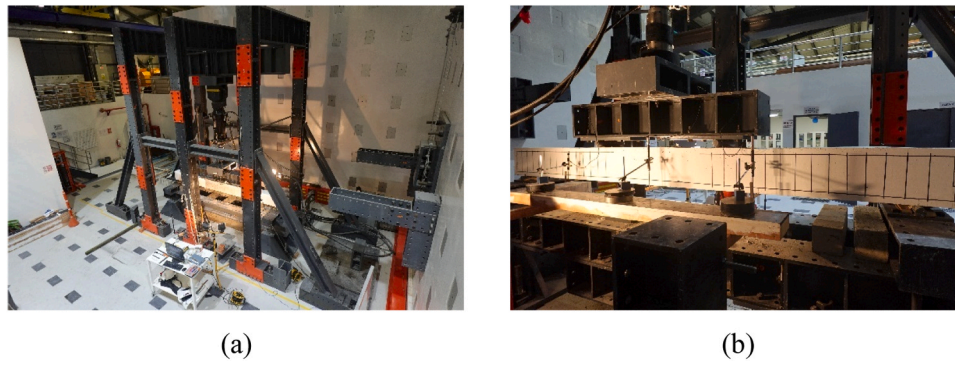


Fig. 2. View (a) of the reaction frame and (b) 4 m beam.

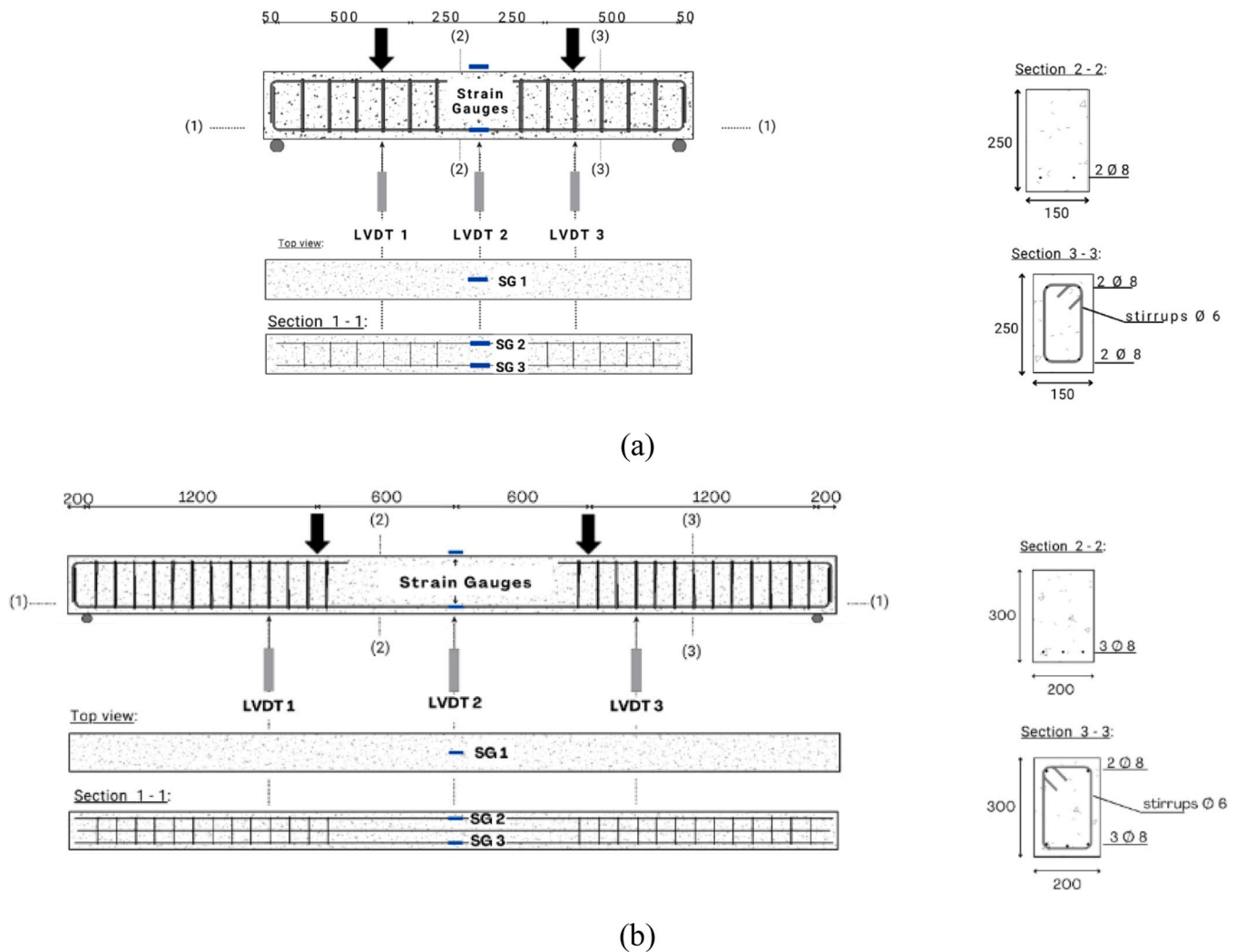


Fig. 3. Test setup and instrumentation of the reinforced concrete beams of (a) 1.60 m and (b) 4.00 m.

crack openings, CMOD, of 0.5, 1.5, 2.5, and 3.5, respectively. The current study observed higher residual strengths at CMOD of 3.5 mm compared to studies such as [25, 29], emphasizing the impact of optimized fiber dosage and distribution. Fig. 4a shows the results of the 3-point bending test of the mixtures with steel fibers, while Fig. 4b shows the results for the mixtures with polypropylene fibers. The results show that steel fiber-reinforced concrete exhibited a post-cracking hardening behavior. After cracking and the corresponding drop in strength, the fibers contributed to an increase in residual strength. In

contrast, polypropylene fiber-reinforced concrete displayed a softening behavior, meaning that while the fibers provided residual strength through crack bridging, this strength gradually decreased over time.

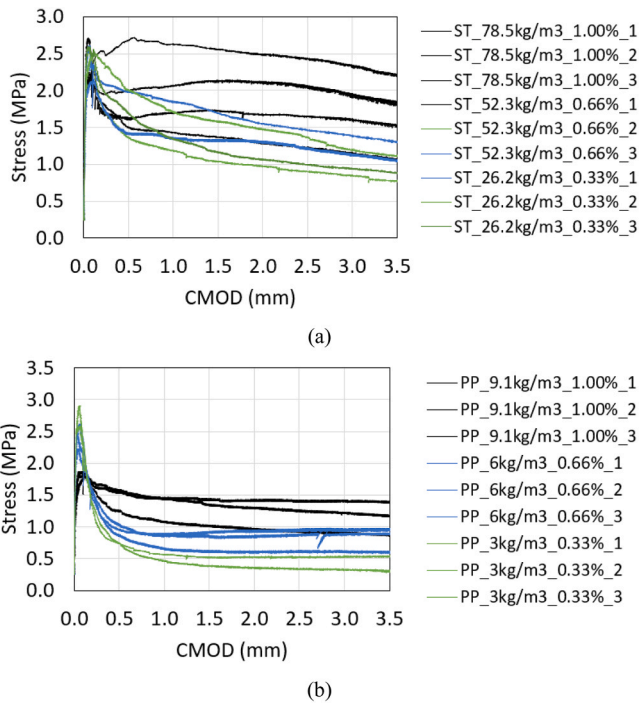
3.2. Flexural behavior of reinforced concrete beams

Table 8 summarizes the results of the four-point bending test on reinforced concrete beams. 3.2.1 compares the load-displacement curves with fiber volume, fiber type, and beam size variations. 3.2.2

Table 7

Slump, compression properties, and tensile properties.

Batch	Slump (mm)	f'_c (MPa)	COV (%)	E (GPa)	COV (%)	f_L (MPa)	f_{R1} (MPa)	f_{R2} (MPa)	f_{R3} (MPa)	f_{R4} (MPa)
Reference	40.6	45.10	2.75	35.40	5.90	2.42	-	-	-	-
PP_0.33 %	30.0	43.66	2.28	36.69	0.91	2.74	0.75	0.48	0.44	0.42
PP_0.66 %	27.5	45.94	2.03	35.94	8.64	2.44	0.95	0.80	0.80	0.81
PP_1.00 %	10.0	47.19	6.41	32.54	3.41	1.85	1.46	1.26	1.20	1.14
ST_0.33 %	25.0	43.92	0.92	40.78	8.68	2.44	1.70	1.35	1.19	1.01
ST_0.66 %	10.0	46.81	1.14	34.16	1.22	2.63	1.62	1.37	1.20	1.05
ST_1.00 %	5.00	47.11	0.79	33.81	1.40	2.48	2.34	2.35	2.25	2.03

**Fig. 4.** Stress-CMOD plots: (a) RFC w/ST fibers, and (b) RFC w/PP fibers.

discusses the behavior of cracking, yielding, and maximum loads with the variation of these variables. 3.2.3 addresses the drift-based deflection capacity. Finally, 3.2.4 delves into the beams' damage evolution and cracking behavior.

3.2.1. Load-deflection curves

Fig. 5 compares the load-drift responses in the monotonic flexural test of reinforced concrete beams with fiber volume, fiber material, and beam length variations. To ensure a consistent comparison across beams

of different lengths, the displacement data has been normalized by dividing the central deflection by the distance from the beam's midpoint to the outer support, i.e., $L/2$. This value is also known as drift. Fig. 5a shows that for the 1.60 m beams, the addition of PP fibers does not increase the load-carrying capacity and, on average, only records a 3.28 % variation with slightly lower values. When normalized by the shear span, the results indicate that the PP fibers have minimal impact on the relative performance of the beams in terms of stiffness and load-carrying capacity. Regarding the flexural stiffness before yielding, it is reported that the beam with 0.33 % PP fibers (3 kg/m^3) showed a 26.8 % reduction, and the beam with 1.00 % PP fibers (9.1 kg/m^3) showed a 3.45 % increase compared to the reference beam. Fig. 5b illustrates the behavior of FRC beams with the addition of ST fibers (STFRC). Similar to polypropylene fiber reinforced concrete (PPFRC), the influence of STFRC on the load-carrying capacity is negligible, as is stiffness in the 1.60 m beams. However, a 52.4 % reduction in drift capacity is observed compared to the reference beam, which will be discussed in 3.2.3.

The effect of beam length is shown in Fig. 5c. It is observed that the 4.00 m beams showed an increase in load-carrying capacity by 25 % for mixes with ST fibers and 22 % for mixes with PP fibers. This result for the 4.00 m beams contrasts with the results for the 1.60 m beams, showing that the incorporation of fibers does not affect the maximum load-carrying capacity. This result can be explained by the fact that, despite the 4.00 m beams developing less curvature, as discussed in 4.3, the number of localized cracks increases, allowing for higher stresses due to strain-hardening in the reinforcing steel. Additionally, the contribution of fibers is greater when the crack width is larger.

3.2.2. Cracking, yielding, and maximum Load

Three distinct parts in the load-curvature behavior are identified: (1) before cracking, (2) after cracking and before yielding, and (3) yielding of the reinforcement steel. The load-curvature curves before cracking are shown in Fig. 6. These curves are necessary to identify the cracking loads. The deflection was calculated from the measurements of the strain gauges placed on the longitudinal steel bars and the upper face of the concrete. Fig. 7 shows the curves focusing on the cracking and yielding points. The maximum load can be identified from Fig. 5.

Table 8

Main flexural behavior parameters of reinforced concrete beams.

Beam	P _{cr} (kN)	$\epsilon_{c,cr}$ (μ)	$\epsilon_{s,cr}$ (μ)	δ_{cr} (mm)	P _y (kN)	$\epsilon_{c,y}$ (μ)	$\epsilon_{s,y}$ (μ)	δ_y (mm)	P _{max} (kN)	P _u (kN)	δ_{max} (mm)
Reference_1.6m	18.78	-148.70	135.10	2.95	41.62	-913.40	2846.95	6.88	60.10	49.46	62.54
PP_0.33 % 1.6m	18.62	-102.10	166.80	4.91	41.10	-583.30	3008.50	9.28	57.14	45.16	58.95
PP_0.66 % 1.6m	14.10	-90.20	112.20	2.65	46.43	-1397.10	2774.80	7.58	58.40	45.43	57.90
PP_1.00 % 1.6m	21.51	-154.90	128.10	2.23	53.04	-1196.80	3287.25	8.48	58.84	51.40	55.50
ST_0.33 % 1.6m	16.01	-96.70	99.10	2.12	45.39	-883.50	2993.20	5.98	53.32	47.90	29.46
ST_0.66 % 1.6m	20.72	-	-	3.88	47.73	-	-	8.56	56.85	52.25	27.57
ST_1.00 % 1.6m	14.88	-94.40	95.10	3.20	47.06	-1007.50	3267.7	8.74	56.03	52.82	32.10
Reference_4.0m	7.66	-116.00	61.70	2.77	26.54	-1076.80	3067.55	17.24	36.42	33.05	180.01
PP_1.00 % 4.0m	6.91	-40.90	65.55	2.49	33.50	-749.30	2823.4	17.13	44.56	36.60	207.96
ST_1.00 % 4.0m	9.07	-62.00	60.95	3.70	39.86	-901.70	2590.05	21.37	45.77	39.83	148.43

P_{cr}: beam cracking load, $\epsilon_{c,cr}$, $\epsilon_{s,cr}$: unit strain of steel in concrete cracking, δ_{cr} : beam deflection in concrete cracking, P_y: beam load that produces steel yielding, $\epsilon_{c,y}$: unit strain of concrete in steel yielding, $\epsilon_{s,y}$: unit strain of steel in steel yielding, δ_y : beam deflection in steel yielding, P_{max}: maximum beam load, P_u: ultimate load of the beam, δ_{max} : maximum deflection of the beam

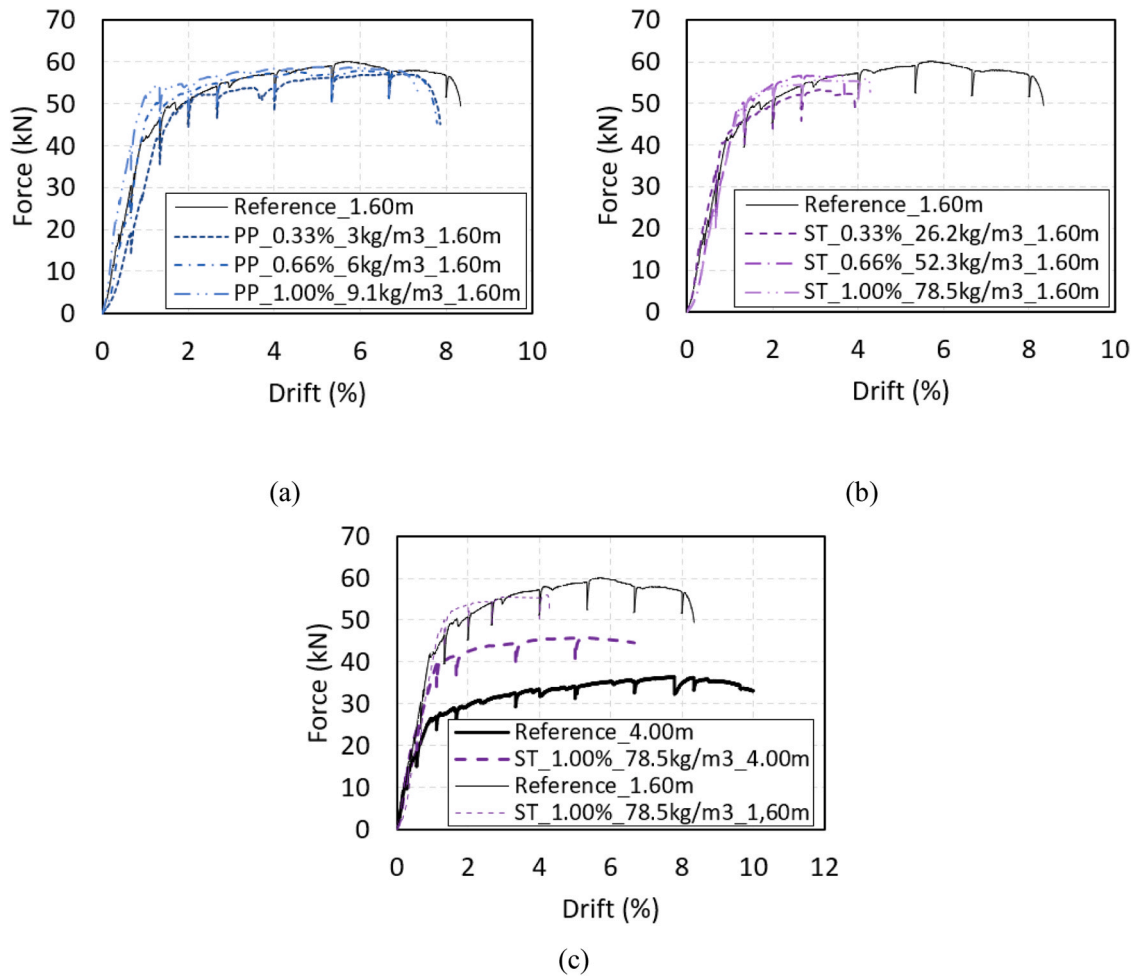


Fig. 5. Load-drift curves, influence of (a) addition of PP fibers, (b) addition of ST fibers, and (c) beam length.

In the first stage, when the section behaves elastically, the flexural stiffness remains high until the first crack appears [15]. In the 1.60 m beams, the flexural stiffness of fiber-reinforced concrete (FRC) increased by 14.8 % with 1 % PPFRF (Fig. 7a) and by 18.7 % with 1 % STFRF (Fig. 7b). In the 4.00 m beams, the increase was more significant, reaching 31.9 % and 49.8 % for PPFRF and STFRF, respectively (Fig. 7c). Akın et al. [21] reported a 2.8 % increase in initial stiffness for 3.00 m beams. These results confirm that the tensile properties of the FRC mixture enhance flexural stiffness, with a more pronounced effect in longer beams.

Fig. 8 shows the behavior after the first crack. In the reference concrete without fibers, a sudden load drop occurs immediately after cracking due to the reduction in the effective cross-sectional area and the corresponding decrease in the moment of inertia [43]. This effect is mitigated in FRC mixtures as the tensile properties of the concrete improve due to the crack-bridging effect of fibers, increasing post-cracking resistance.

Comparing the curves in Fig. 7a and Fig. 7b, two parallel branches are generated, and the vertical distance between them represents the increased load-carrying capacity of FRC at this stage due to improved tensile strength and post-cracking behavior. The ACI 318–19 standard [42] determines the maximum allowable deflection for live loads, given by $L/240$. This results in a maximum permissible service deflection of 6.25 mm for the 1.60 m beams, corresponding to a maximum curvature of $26000 \mu/m$. For a curvature of $10000 \mu/m$, which is approximately 40 % of the service deflection, the load increase due to adding 1 % ST fibers before yielding is approximately 30 %. In contrast, the contribution of PP fibers under the same conditions is about 16 %. This effect is

more pronounced in the larger 4.00 m beams. In this case, the maximum service deflection is 15 mm, corresponding to a maximum curvature of $10800 \mu/m$. The load increase for a curvature of $5000 \mu/m$, which is 46 % of the maximum service deflection due to ST fibers, is 119 %, and for PP fibers, it is 49 %, as shown in Fig. 7c.

The flexural stiffness of the cracked section is reduced by 90 % compared to the uncracked section for all tested beams. In the 1.60 m beams, PPFRF increased the flexural stiffness of the cracked section by 13 % compared to the reference beam, while STFRF resulted in a 42 % increase. In the 4.00 m beams, this effect was more pronounced, with PPFRF increasing stiffness by 71 % and STFRF by 104 % compared to the reference beam. This behavior is attributed to the higher residual tensile strength of STFRF, which results in lower deformations under the same load conditions, thus enhancing stiffness. This is because ST fibers, having a higher modulus of elasticity, exhibit lower deformations for the same load level than PP fibers, leading to greater overall stiffness in the cracked section.

The yielding loads are observed in Fig. 8 and Fig. 9. The influence of fiber-reinforced concrete (FRC) is more significant in yielding than in cracking, as the improved tensile properties of the matrix delay plastic deformations [5]. The increase in residual tensile strength due to fiber reinforcement enhances the ability of the section to sustain higher loads before yielding, particularly at higher fiber volumes [44]. In the 1.60 m beams, FRC with higher tensile strength led to increases in the yielding load of 11.6 % and 27.4 % for fiber dosages of 0.66 % and 1.00 % (6 and 9 kg/m³) in PPFRF, whereas in STFRF, the increases were 9.1 %, 14.7 %, and 13.1 % for dosages of 0.33 % (26.2 kg/m³), 0.66 % (52.3 kg/m³), and 1.00 % (78.5 kg/m³), respectively (Fig. 8a). In the

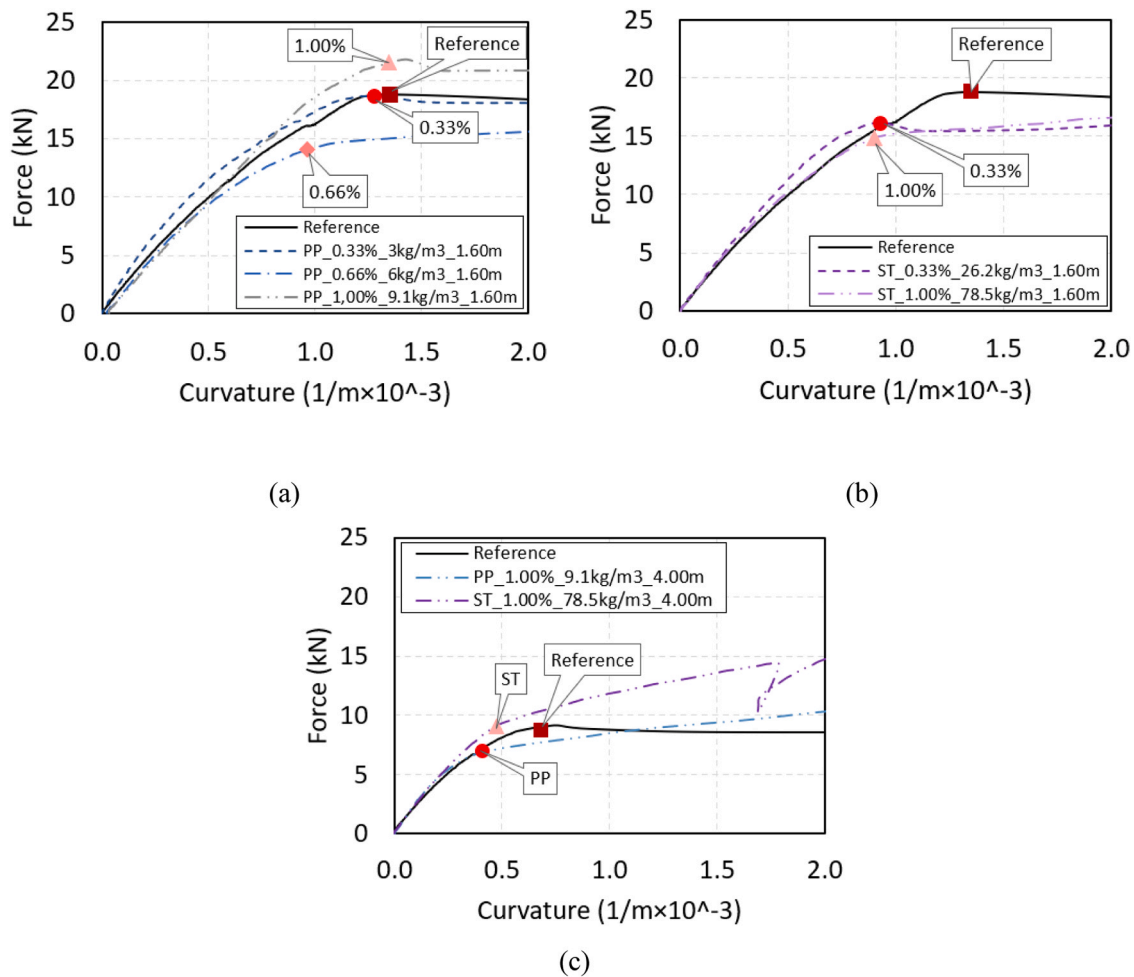


Fig. 6. Moment-curvature diagrams of reinforced concrete beams with and without fibers before cracking.

4.00 m beams, the effect was more pronounced, with increases of 26 % and 50 % in yielding loads for PPFRF and STFRF, respectively (Fig. 8b). Previous studies have reported increases of up to 79 % in yielding load using 1 % STFRF [5] and up to 13 % with 0.5 % PPFRF [21], supporting the correlation between enhanced residual tensile strength and improved yielding capacity.

Fig. 5 shows the maximum loads. After yielding, the curves exhibit a parabolic trend in all cases, reaching the peak load due to the enhanced post-cracking tensile resistance of fiber-reinforced concrete (FRC), which improves load redistribution. This behavior contrasts with the results of Cardoso et al. [4], whose findings show a decrease in load-carrying capacity immediately after the yield point. However, their study was conducted on high-strength concrete (80 MPa) with reinforcement ratios ranging from 0.28 % to 0.70 %. Similarly, Dancygier and Berkover [20] observed comparable results for 30 MPa concrete with a reinforcement ratio of 0.39 % but also reported reductions in the post-yield curve when using very low (0.15 %) or very high (3.27 %) reinforcement ratios.

Regarding the influence of the mechanical properties of FRC, its effect on maximum load is minimal in the 1.60 m beams, where slight reductions were observed at a fiber volume of 0.33 %. These results align with previous studies that concluded that the flexural strength of reinforced concrete members remains largely unaffected by fiber addition at a conventional reinforcement ratio of 1.35 % [23]. However, in the 4.00 m beams, the enhanced tensile strength and crack-bridging effects of FRC led to significant increases in maximum load, reaching 22 % and 25 % compared to the reference beam without fibers. Prior studies have also demonstrated that fiber-reinforced concrete,

particularly at low steel reinforcement ratios, improves load-carrying capacity [3,4,21,44].

3.2.3. Drift capacity

The drift capacity is defined as the ratio of the central deflection to the span length in a four-point bending test, but with the central loads applied together [45]. This research measured drift by dividing the central deflection by half the span length because the central loads were applied at the third point. As mentioned in 3.2.2, according to the ACI, the maximum allowable deflection for a reinforced concrete beam is $L/240$; this corresponds to a maximum drift of 0.8 % for service loads, values similar to those reported in studies on composite materials such as R/UPHC [45]. This corresponds to curvatures of $26000 \mu/m$ and $10800 \mu/m$ for the 1.60 m and 4.00 m beams, respectively. Fig. 8a and Fig. 8b show that for these curvature levels, the reinforcement steel in the 1.60 m beams has already yielded, while the reinforcement steel in the 4.00 m beam is still in the cracking stage. These results suggest that the limits the code allows need to be adjusted for smaller beams. It is suggested to use a more conservative limit of $L/480$.

Regarding the maximum drift capacity, the comparative column chart in Fig. 9 shows the performance of each tested beam. The addition of ST fibers reduced the drift capacity by 52.4 % for 1.60 m beams, a behavior consistent with the crack localization phenomenon reported in other studies on high-bond materials [20]. In contrast, PP fibers showed a significantly smaller reduction of 8 %, highlighting their ability to distribute deformation better. In the case of the 4.00 m beams, the addition of PP fibers increased the drift capacity by 15 %, while the beams with ST fibers reduced their capacity by 17 %. This variation

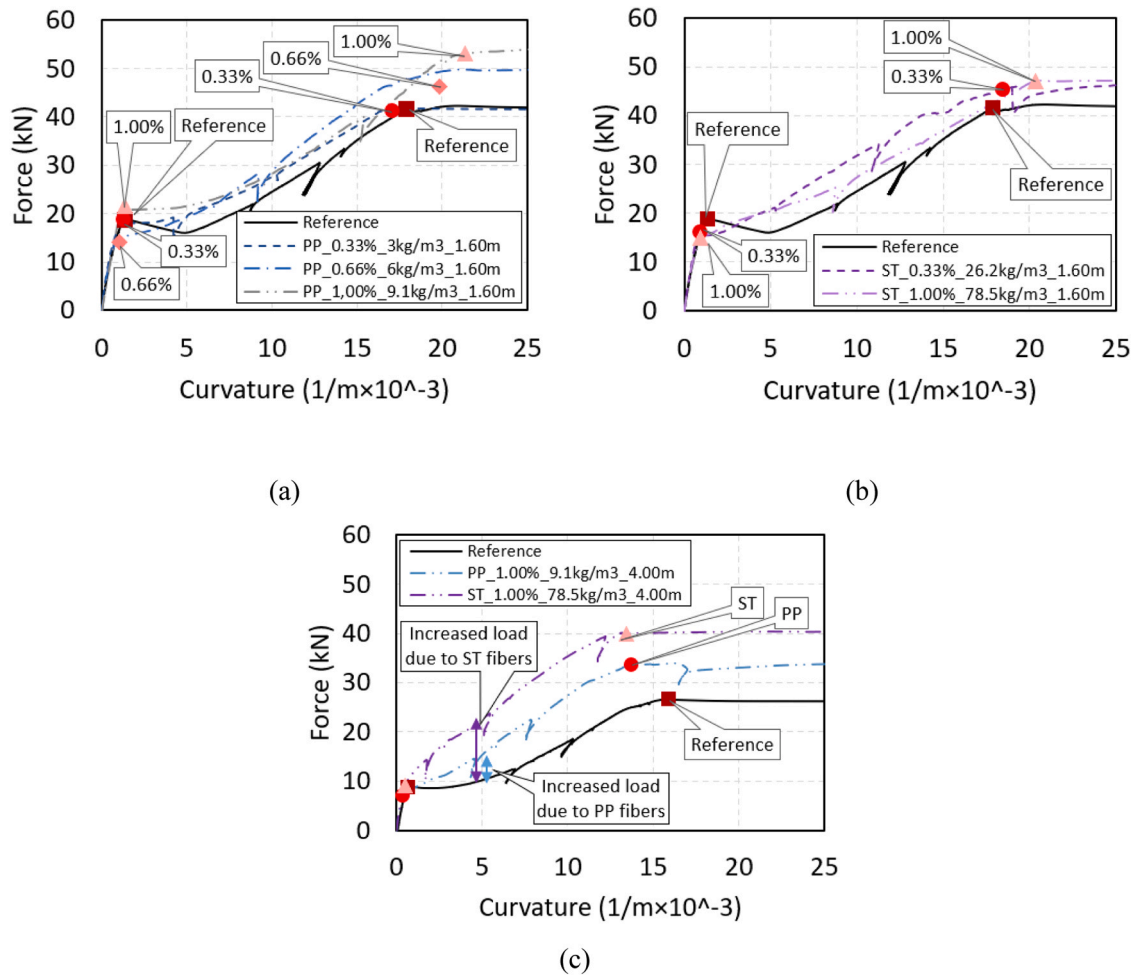


Fig. 7. Moment-curvature diagrams of reinforced concrete beams with and without fibers at the steel yielding stage.

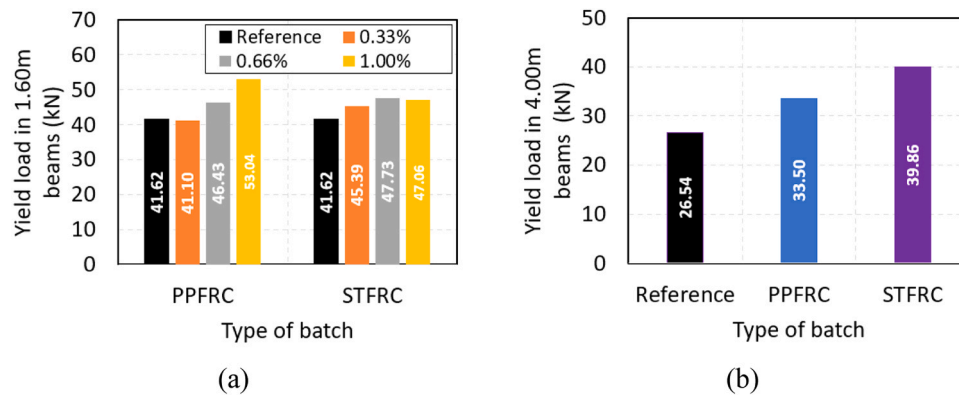


Fig. 8. Effect of (a) volume and type of fibers in 1.60 m beams, and (b) type of fibers in 4.00 m beams in the yield load of reinforced concrete sections.

between medium and large beams is because the 4.00 m beams, when subjected to the same curvature as the medium beams, develop more cracks along the span, leading to increased ductility because deformation is not concentrated in a single crack.

3.2.4. Cracking behavior

Fig. 10 shows the evolution of cracking in the 1.60 m beams for the following cases: (a) reference beam, (b) beam with 9.1 kg/m³ of PP fibers, and (c) beam with 78.5 kg/m³ of steel fibers. In all cases, flexural failures occurred, and each case had a different cracking pattern. The

reference beam had 7 cracks with an average spacing of 16.2 mm (Fig. 11a); the beam with 1 % PP fibers had 10 cracks with an average spacing of 13.7 mm (Fig. 11b), while the beams with 1 % ST fibers had 7 cracks with an average spacing of 14.3 mm (Fig. 11c). In both cases, the crack spacing was reduced. PP fibers were more effective than ST fibers in reducing crack spacing in the 1.60 m and 4.00 m beams. This reduction was approximately 11 % with ST fibers and 24 % with PP fibers. Furthermore, the addition of PP fibers led to the formation of more cracks compared to the ST fiber-reinforced beams for both beam sizes. Therefore, it can be concluded that PP fiber-reinforced beams generated

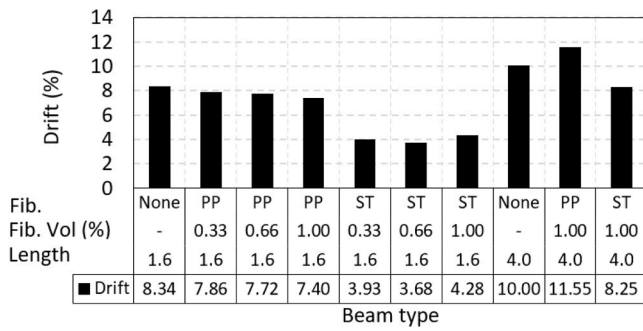


Fig. 9. Drift capacity of the beams.

more cracks with smaller spacings than ST fiber-reinforced beams. This is explained by the fact that fibers help to redistribute stresses in fiber-reinforced concrete due to better bridging action [46]. Moreover, the low elastic modulus of PP fibers helps develop more cracks in contrast to ST fibers, as their high elastic modulus restricts crack opening and formation more intensely. Additionally, it was observed that the average crack spacing decreased with an increase in fiber volume, as reported in previous studies [5].

In the 1.60 m beam, the widest crack width in the reference concrete was 24.5 mm before failure. The crack width was reduced with the addition of fibers by 31 % with PP fibers and 7 % with ST fibers, resulting in widths of 16.9 mm and 22.8 mm, respectively. Therefore, the fibers were effective in reducing crack widths. Crack localization occurred in the steel fiber-reinforced beams because a single crack concentrated 90 % of the total cracking. For reference, the largest cracks in the reference beam and PP fiber-reinforced beams concentrated 41 % and 33 % of the cracking, respectively. The same phenomenon occurred in the 4 m beam, although the concentration occurred in two cracks. The two largest cracks in the ST fiber-reinforced concrete concentrated 64 % of the total cracking. At the same time, the reference beam had evenly distributed cracks with almost uniform widths, and its two widest cracks concentrated only 26 % of the total cracking. Similarly, the two largest cracks in the PP fiber-reinforced beam concentrated 32 % of the total cracking. This phenomenon has already been reported in previous studies [3,4,20,43]. The probability of crack localization is very high in beams with lower quantities of longitudinal steel reinforcement and higher volumes of fibers [3,4]. Crack localization occurs when one or two of the multiple cracks grow more than the others, leading to increased curvature at those points. This localization results in localized tensile strain in the steel and leads to premature failure, reducing the ductility of the beam [5].

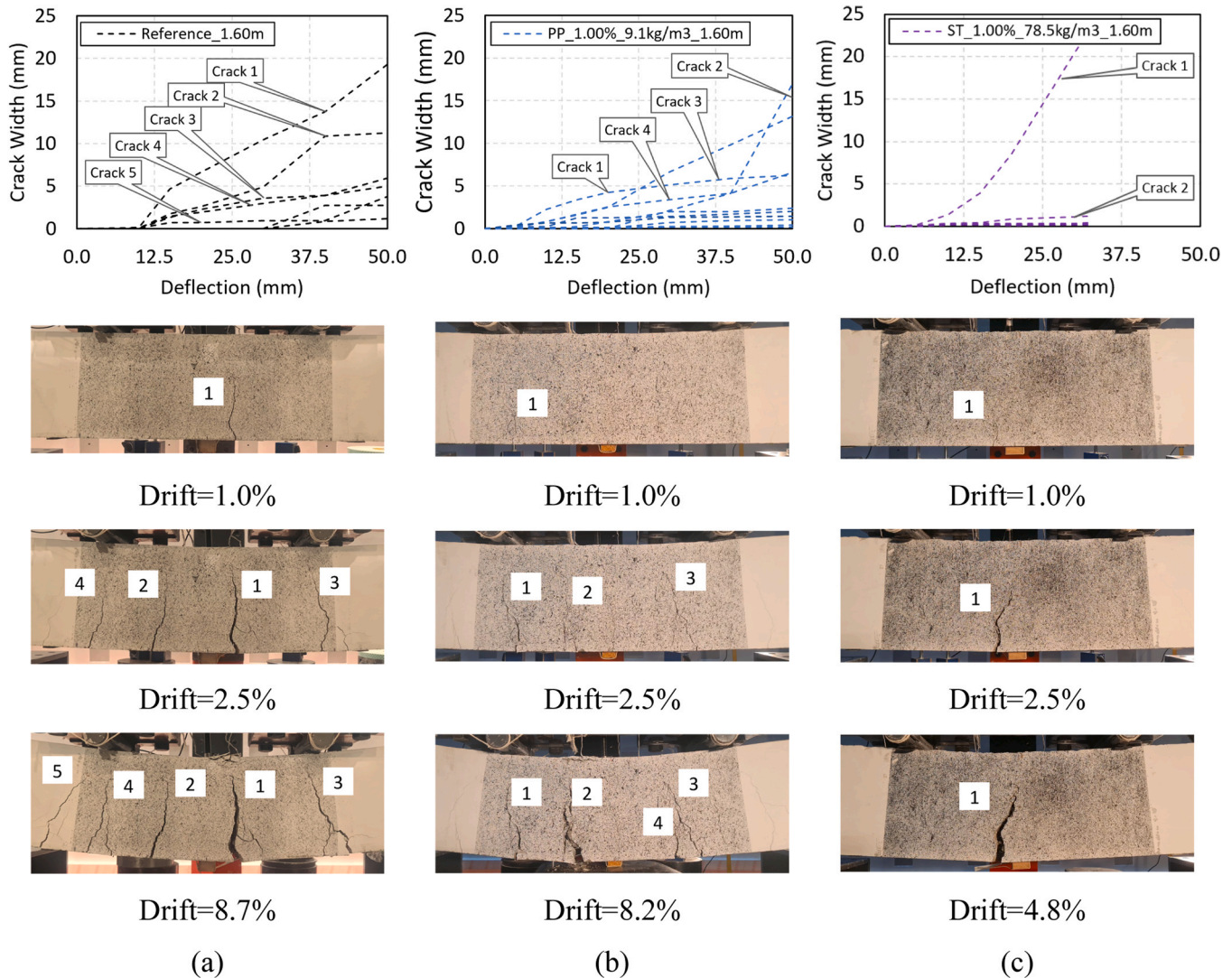


Fig. 10. Crack width as a function of deflection and damage evolution at different drifts for (a) the reference beam without fibers, (b) with 1 % polypropylene fibers, and (c) with 1 % steel fibers.

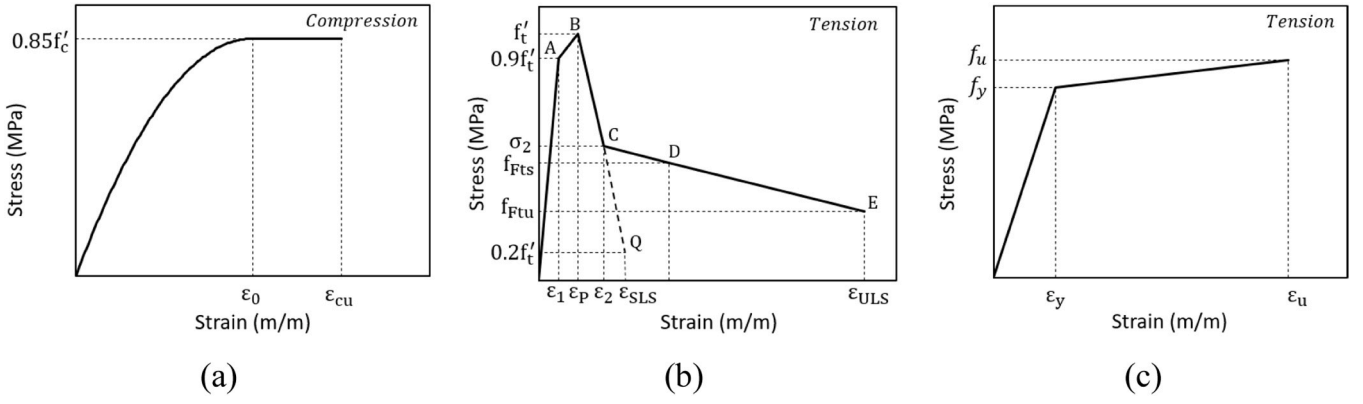


Fig. 11. Constitutive models (a) of concrete in compression, (b) of concrete in tension and (c) of reinforcing steel in tension.

4. Moment-curvature analysis

4.1. Constitutive relations

The bending moment diagram predicts the flexural behavior of reinforced concrete beams with and without fibers. The constitutive relationships for concrete in compression, concrete in tension, and reinforcing steel in tension are detailed below. Fig. 11a shows the model recommended by RILEM [47] and used in this investigation for the stress-strain diagram of concrete in compression. Previous studies have already used this model to idealize concrete with fibers in compression [5]. This diagram is characterized by having a first section of parabolic shape that grows from the origin to a point given by a unit strain ϵ_0 of 0.002 and 85 % of the specified strength. From this point, the relationship continues horizontally up to an ultimate unit strain ϵ_{cu} of 0.0035. The compressive strength can be calculated from Eqs. 1 and 2.

$$\text{Si } 0 \leq \epsilon_c \leq \epsilon_0, f_c = -0.85 \times f'_c \times \left[\frac{2\epsilon_c}{\epsilon_0} - \left(\frac{\epsilon_c}{\epsilon_0} \right)^2 \right] \quad (1)$$

$$\text{Si } \epsilon_0 \leq \epsilon_c \leq \epsilon_{cu}, f_c = -0.85 \times f'_c \quad (2)$$

The model recommended by the Fib Model Code for FRC in tension is used for the analytical model. This model is shown in Fig. 11b. Four sections are identified. The first span shows the linear elastic behavior of the material where Hooke's law is applied up to a unit strain defined by $0.9 \times f'_c / E_c$. The second span shows a slight stiffening up to the maximum tensile strength f'_c and a strain of ϵ_p defined in the code with a value of 0.00015. The third section represents the stress drop after concrete cracking; the slope is defined by the point of maximum strength B, and point Q is defined as strength corresponding to 20 % of the maximum and a strain of ϵ_Q , defined by Eq. 3.

$$\epsilon_Q = \frac{G_f}{f'_c \times l_{cs}} + \left[\epsilon_p - \left(\frac{0.8 \times f'_c}{E_c} \right) \right] \quad (3)$$

where G_f represents the fracture energy of concrete and l_{cs} is the characteristic length of the structural crack, both defined by Eqs. 4 and 5.

$$G_f = 73 \times (f'_c)^{0.18} \quad (4)$$

$$l_{cs} = \min(s_{rm}; y) \leq 2.5 \text{ mm} \quad (5)$$

where s_{rm} is the average distance between cracks and y is the distance between the neutral axis and the tensile face of the cracked section evaluated for a load corresponding to the serviceability state. Finally, the fourth section represents the development of the residual strengths of concrete with fibers whose slope is defined by the deformations corresponding to the serviceability limit state, ϵ_{SLS} , and their respective residual strengths ϵ_{ULT} , and their respective residual strengths f_{PIS} and f_{PU}

which are defined in Eqs. 6 to 9.

$$\epsilon_{SLS} = \frac{CMOD_1}{l_{cs}} \quad (6)$$

$$\epsilon_{ULT} = \frac{w_u}{l_{cs}} = \min \left(\epsilon_{Pu}; \frac{2.5}{l_{cs}} \right) \quad (7)$$

$$f_{PIS} = 0.45 \times f_{R1} \quad (8)$$

$$f_{PU} = f_{PIS} - \frac{w_u}{CMOD_3} \times (f_{PIS} - 0.5 \times f_{R3} + 0.2 \times f_{R1}) \geq 0 \quad (9)$$

where $CMOD_1$ and $CMOD_3$ are the crack openings of the 3-point bending test defined by EN 14651 standard equal to 0.5 and 3.5 mm respectively, f_{R1} and f_{R3} are the residual strengths corresponding to $CMOD_1$ and $CMOD_3$, w_u is the ultimate crack width and ϵ_{Pu} is the ultimate strain whose value is considered to be 2 % for a variable strain distribution. Finally, the constitutive relationships for each span are shown in Eqs. 10 to 13.

$$0 \leq \epsilon_c \leq \epsilon_1 E_c \times \epsilon_c \quad (10)$$

$$\epsilon_1 \leq \epsilon_c \leq \epsilon_p f'_c \times \left[1 - 0.1 \times \frac{\epsilon_p - \epsilon_c}{\epsilon_p - \epsilon_1} \right] \quad (11)$$

$$\epsilon_p \leq \epsilon_c \leq \epsilon_Q f'_c \times \left[1 - 0.8 \times \frac{\epsilon_c - \epsilon_p}{\epsilon_Q - \epsilon_p} \right] \quad (12)$$

$$\epsilon_Q \leq \epsilon_c \leq \epsilon_{ULT} f_{PU} + \frac{\epsilon_{ULT} - \epsilon_c}{\epsilon_{ULT} - \epsilon_{SLS}} \times (f_{PIS} - f_{PU}) \quad (13)$$

A bilinear model was adopted for the constitutive relationship of the reinforcing steel, see Fig. 11c. The yield and ultimate stress are 450 and 680 MPa, respectively, while the modulus of elasticity is 200 GPa. The yield and ultimate strengths were considered 0.00225 and 0.11, respectively.

4.2. Analytical model

The curvature moment diagrams were generated through sectional analysis, whose calculation procedure is as follows: first, it is assumed that the flat sections remain flat after bending; consequently, the deformation profile has a defined curvature for each state. Then, the value for the curvature is assumed from zero to the maximum curvature, which is when failure occurs. The stress profile is drawn according to the constitutive diagrams described in 4.1 for each curvature value. From the stress profile, the diagram of forces acting on the section is drawn with their respective locations. Then, the equilibrium equations are formulated to calculate the position of the neutral axis. Then, the resistance moment of the section is calculated for each state. Finally, the

procedure is repeated for curvature increments until the unit strain values of the concrete or steel are exceeded, i.e., when the section failure occurs. Fig. 12 shows the sectional analysis of the three remarkable points of the bending moment diagram: cracking, yielding, and the maximum capacity of the section.

4.3. Comparison between analytical and experimental results

Fig. 13 compares the experimental and theoretical results of the curvature moment diagram for the serviceability limit state. The experimental results of the curvature were calculated through the measurement of strain gauges placed on the steel and the top face of the concrete through Eq. 14:

$$\phi = \frac{|\epsilon_c| + |\epsilon_s|}{d_s} \quad (14)$$

where ϕ is the curvature, ϵ_c is the unit strain of the concrete on the most compressed face, ϵ_s is the average unit strain of the reinforcing steels in tension, and d_s is the vertical distance between the reinforcing steels in tension and the top face of the concrete in compression. The moment is calculated through the structural analysis of the beam supported at the ends and with loads in the thirds, according to Eq. 15.

$$M = \frac{P \times L}{6} \quad (15)$$

where M is the analytical bending moment and L is the free span between beam supports. Finally, the analytical bending moment diagram was computed, as explained in 4.1.

Fig. 13a, Fig. 13b and Fig. 13c compare the 1.60 m reference beams without fibers, with 1 % PP and 1 % ST fibers. The curvature of the

beams before the occurrence of the first crack is uniform if subjected to a constant moment [20]. Because of this, no major difference in cracking moment was observed in all experimental results. Concerning the comparison with the analytical model, the analytical cracking moment underestimates the experimental value in the beam without fibers by 30 %. The analytical models with fibers do show an increase in the cracking moment, overestimating the experimental values, 15 % for the beams with 1 % PP fibers and 90 % for the beams with 1 % ST fibers.

The yield moment has a very good fit with an error of 12 %. In addition, the steel yielding curvature is presented for a value 48 % higher than that preceded, like what occurred in the reference beam. For the beam with 1 % ST fibers, the moment overestimation is larger both in the cracked section and in the yielding, being 13 % lower for the latter case. Meanwhile, the curvature is 53 % higher than the calculated one. Similar results were obtained for the 4.00 m beam; see Fig. 13d, Fig. 13e and Fig. 13f. It is observed that the yield moment increases with the addition of PP fibers by 28 % concerning the reference beam, while the increase with the addition of steel fibers is 50 %. It should be noted that the 1.60 m beams with PP fibers experienced an increase in yield moment of 25 %, and with steel fibers, only 13 %. This leads to the conclusion that the fibers contribute more significantly the larger the beam size is. Fig. 14 shows the comparison of the yield moments of the beams. The beams with steel fibers reached higher yield moments than those with PP fibers, while the latter resulted in higher values than the reference beams. Therefore, it can be concluded that adding fibers increases the yield moment, with steel fibers being more advantageous, see Fig. 14a. In addition, it is also observed that the greater the number of fibers, the higher the yield moment, see Fig. 14b.

Fig. 15 shows the complete moment-bending diagrams of the tested beams. The curvature was estimated from the central load-deflection

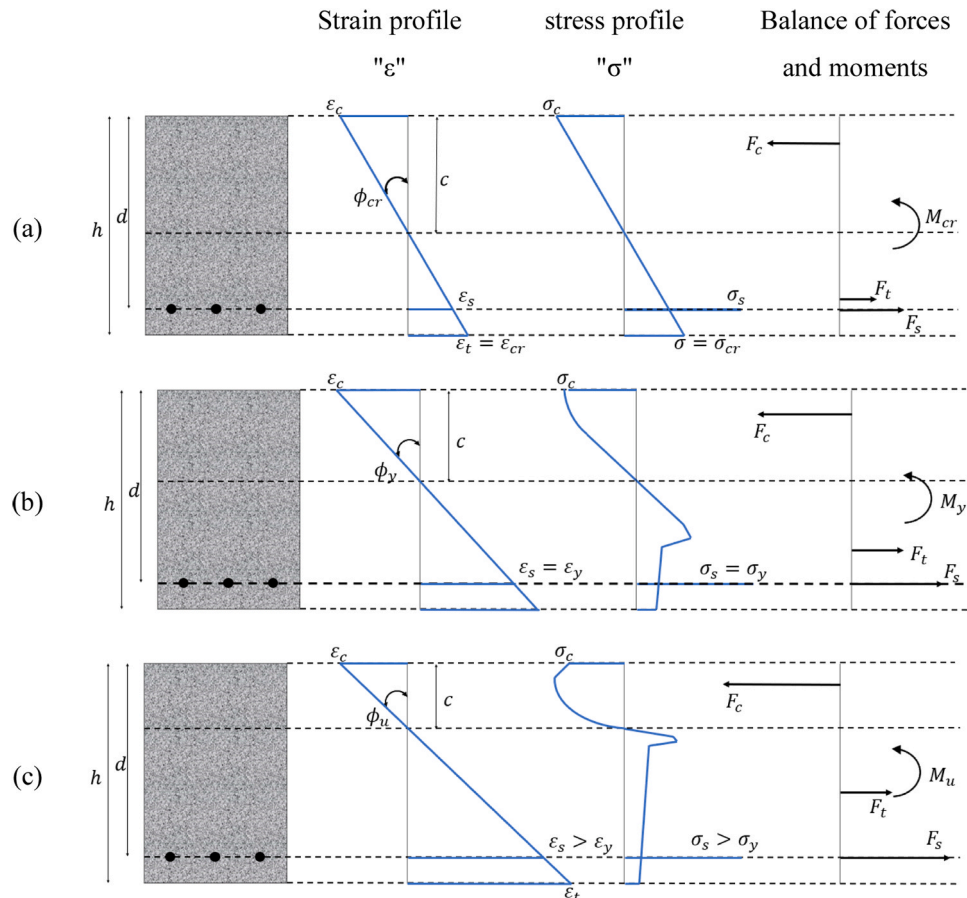


Fig. 12. Deformation profile, stresses, and forces for the remarkable points of (a) cracking, (b) yielding and (c) maximum capacity of the section.

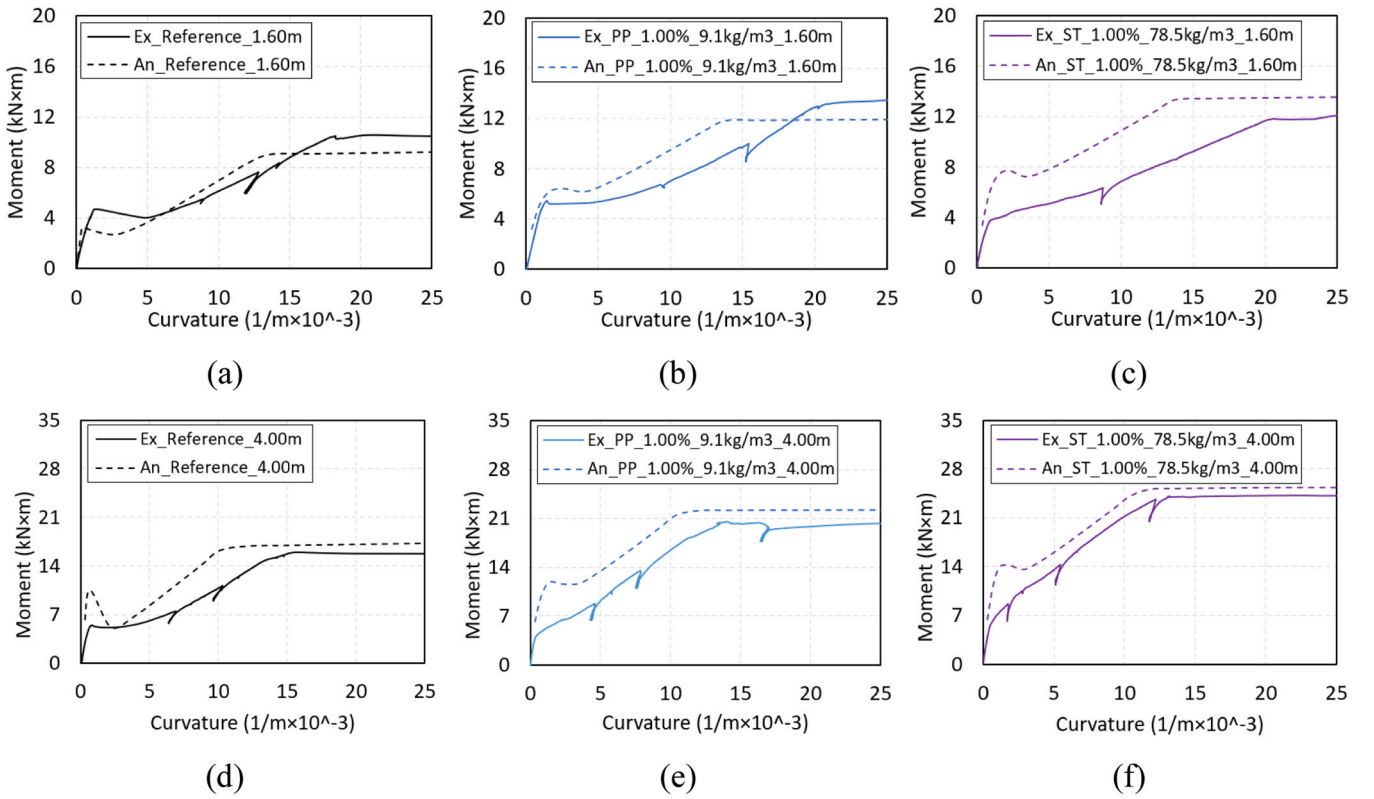


Fig. 13. Experimental and calculated moment-bending diagrams of reinforced concrete beams in cracking and yielding zones: (a) 1.60 m reference beam, (b) 1.60 m beam with 1 % PP fibers, (c) 1.60 m beam with 1 % steel fibers, (d) 4.00 m reference beam, (e) 4.00 m beam with 1 % PP fibers and (f) 4.00 m beam with 1 % steel fibers.

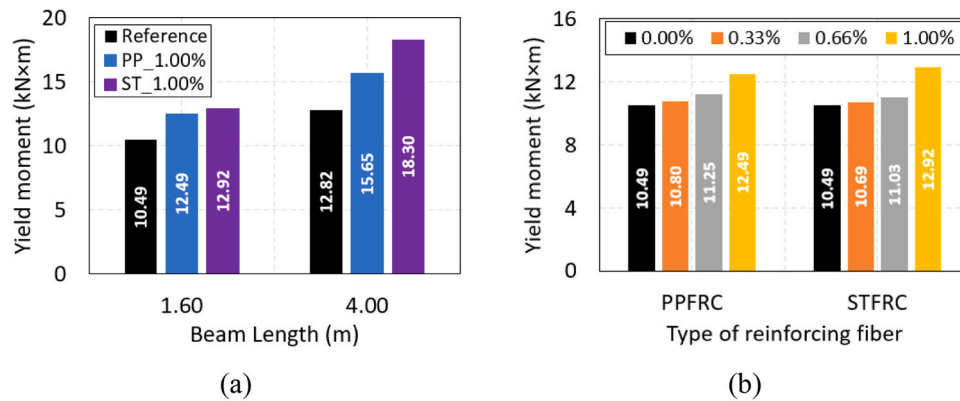


Fig. 14. Comparative column diagram of (a) yield moment for 1.60 m and 4.00 m beams, (b) yield moment for beams with steel and polypropylene fibers.

diagram through Eq. 16, which comes from linear elastic theory and was previously used in other studies [5]:

$$\varphi = \frac{24\delta}{(3L^2 - 4L_1^2)} \quad (16)$$

where δ is the central deflection of the beam and L_1 is the distance between the support and the load located at the third of the beam.

The results show that the analytical model underestimates the experimental results for the reinforced concrete beam. This has already been reported in previous studies [5]. The explanation is that the contribution of the interlocking aggregates in the tensile zone was not considered. The experimental results of the 1.60 m beams shown in Fig. 15a, Fig. 15b and Fig. 15c indicate no variation in the maximum moment due to the addition of fibers in both the steel and PP beams.

However, the analytical models do show that there should be an increase in this property with the addition of fibers. This increase does occur in the 4.00 m beams, see Fig. 15d, Fig. 15e, and Fig. 15f, although with less extent than the analytical model.

The curvature ductility is defined as the ratio of the maximum curvature to the yield curvature; see Eq. 17.

$$\mu_\varphi = \frac{\varphi_u}{\varphi_y} \quad (17)$$

In general, the ductility of the specimens varied depending on the type of fiber-reinforced concrete used, as shown in Fig. 16. For the 1.60 m beams, the ductility of steel fiber-reinforced concrete (STFRC) was considerably lower due to the occurrence of crack localization, decreasing by 63.4 % compared to the reference beam. This suggests that the mechanical interaction between steel fibers and the concrete

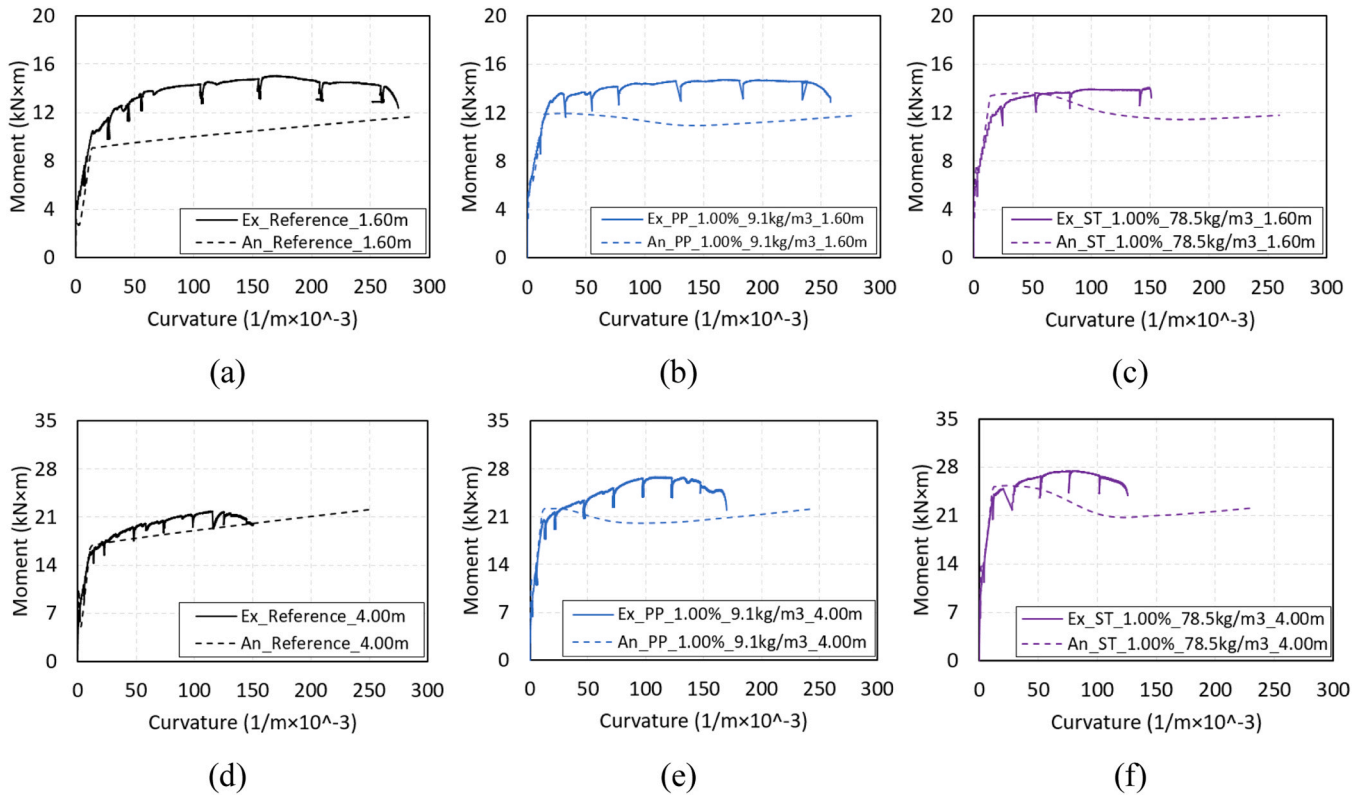


Fig. 15. Experimental and calculated moment-bending diagrams of reinforced concrete beams up to the maximum capacity of the section: (a) 1.60 m reference beam, (b) 1.60 m beam with 1 % PP fibers, (c) 1.60 m beam with 1 % steel fibers, (d) 4.00 m reference beam, (e) 4.00 m beam with 1 % PP fibers and (f) 4.00 m beam with 1 % steel fibers.

matrix led to a concentration of deformation in a single location, rather than a more distributed cracking pattern. In contrast, the bending ductility of polypropylene fiber-reinforced concrete (PPFRC) was only slightly lower than that of the reference beam, showing a reduction of 14.7 %. For the 4.00 m beams, although the general trend of curvature differences was maintained, the gap between STFRC and PPFRC was smaller. PPFRC exhibited an 11.3 % reduction in ductility, while STFRC showed a 24 % reduction. Notably, both the reference beams and the beams with 1 % PPFRC exhibited lower ductility in the 4.00 m specimens, with reductions of 42.6 % and 40.3 %, respectively. However, in STFRC beams, ductility increased by 19 %, indicating a more favorable performance in larger specimens. This suggests that the crack localization effect observed in STFRC was more pronounced in the 1.60 m beams, where the limited size restricted the development of multiple cracks, concentrating deformation in a single zone and leading to

premature failure. Finally, no significant changes were observed with an increase in fiber content, reinforcing the idea that the interaction between the fiber type and the concrete matrix plays a crucial role in determining the ductility behavior rather than the mere presence of fibers.

5. Conclusions

This study investigated the flexural behavior of RC beams of different spans reinforced with polypropylene and steel fibers. The results were evaluated in terms of ELS and ULS with special emphasis on the occurrence of localized cracking. The properties evaluated in the ELS are cracking and yielding, while the properties studied in the ULS are maximum capacity and ductility. The following conclusions be drawn:

1. The incorporation of fibers improved the initial stiffness of fiber-reinforced concrete, with a more pronounced effect in longer beams. In 4.00 m beams, stiffness increased by 32 % with PPFRC and 50 % with STFRC, while in 1.60 m beams, the increases were 15 % and 19 %, respectively. This behavior is attributed to the ability of fiber-reinforced concrete to enhance stress transfer within the cementitious matrix, improving the structural response before cracking.
2. The cracking moment showed erratic behavior with fiber variations. In general, it can be concluded that the fibers have a negligible effect on the cracking moment. This behavior can be attributed to the fact that the fibers primarily influence post-cracking behavior by bridging cracks and enhancing stiffness after cracking rather than affecting the initial resistance of the concrete matrix to cracking.
3. After cracking, fiber-reinforced concrete retained higher stiffness compared to plain concrete despite an approximate 90 % reduction in flexural stiffness in all cases. However, the presence of fibers

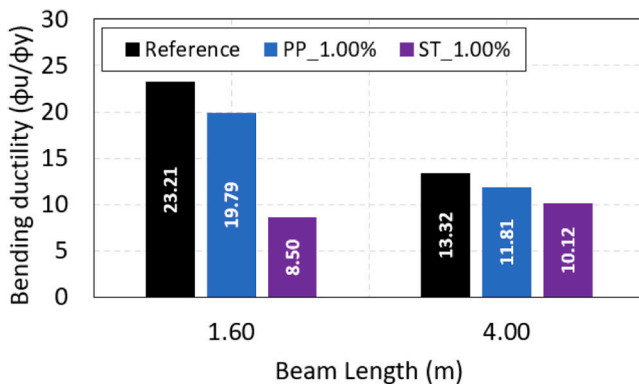


Fig. 16. Comparative column diagram of the bending ductility for the 1.60 m and 4.00 m beams.

mitigated this reduction, leading to significant increases in the stiffness of the cracked section. In 4.00 m beams, stiffness increased by 71 % with PPFRF and 104 % with STFRC, while in 1.60 m beams, the increases were 13 % and 21 %, respectively. This behavior is attributed to the residual tensile strength provided by the fibers, which enhances stress redistribution across the cracked section. The greater increase in stiffness observed in the 4.00 m beams suggests that longer spans benefit more from the reinforcing effect of fibers, likely due to improved crack distribution and stress transfer within the section.

4. The use of fibers contributed to an increase of up to 50 % in the yield moment in both 1.60 m and 4.00 m beams, particularly with 1 % STFRC. This increase is attributed to the fibers' ability to control crack propagation and enhance the bond between the cementitious matrix and longitudinal reinforcement, delaying steel yielding.
5. The 4.00 m beams with steel and PP fibers showed increases in maximum load capacity by 22 % and 25 %, respectively, due to an enhanced ability to redistribute stresses across the section, reducing stress concentration. However, this effect was less significant in the 1.60 m beams, where the shorter span limits the formation of multiple cracks.
6. Ductility decreased with fiber incorporation, with a more pronounced reduction in STFRC due to crack localization. In 1.60 m beams, ductility was 63 % lower in STFRC and 14.7 % lower in PPFRF compared to the reference beams, while in 4.00 m beams, the reductions were 24 % and 11.3 %, respectively. This behavior indicates that STFRC tends to concentrate deformations in fewer cracks, leading to premature failure, whereas PPFRF promotes a more uniform crack distribution, mitigating this effect. The analytical results show that there should be a decrease in ductility with increasing residual strength of the concrete.

This study evaluates the flexural behavior of reinforced concrete beams with low longitudinal reinforcement ratios and different types and dosages of fibers. Due to economic constraints, only one specimen per configuration was tested, which limits the generalization of the results. Future research should include multiple specimens per case and test with moderate and high reinforcement ratios to assess their impact on section ductility. In addition, semi-cyclic and cyclic loading tests are recommended to investigate crack propagation, energy dissipation, and long-term durability. These approaches would enhance the understanding of fiber-reinforced concrete behavior, especially in seismic and fatigue-prone applications.

CRedit authorship contribution statement

Darwin La Torre Esquivel: Writing – original draft, Validation, Methodology, Investigation, Formal analysis, Data curation, Conceptualization. **Flávio de Andrade Silva:** Writing – review & editing, Validation, Methodology, Conceptualization. **Alexandre Almeida Del Savio:** Writing – review & editing, Supervision, Resources, Project administration, Funding acquisition.

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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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