

Article

Experimental and Analytical Investigation of the Flexural Performance of UHPC Beams Reinforced with Hybrid GFRP and Steel Bars

Tianyu Shi ^{1,*}, Kangsheng Huang ¹, Ruochen Wang ^{2,*} , Changming Dai ³ and Kun Wang ³

¹ The Third Engineering Co., Ltd. of CCCC Fourth Harbor Engineering Co., Ltd., Zhanjiang 524005, China; huangkangsheng@ccccltd.cn

² Jiangsu Testing Center for Quality of Construction Engineering Co., Ltd., Nanjing 210033, China

³ College of Civil Science and Engineering, Yangzhou University, Yangzhou 225127, China; mx120240624@stu.yzu.edu.cn (C.D.); wangkun@yzu.edu.cn (K.W.)

* Correspondence: shitianyu2@ccccltd.cn (T.S.); wr921927610@gmail.com (R.W.); Tel.: +86-182-5288-7050 (T.S.); +86-150-0527-9701 (R.W.)

Abstract

To investigate the bending response of ultra-high-performance concrete (UHPC) beams reinforced with hybrid glass-fiber-reinforced polymer (GFRP) and steel bars, five specimens were tested in four-point bending in the present experimental study. The effect of varying reinforcement ratios on the flexural behavior was evaluated. It was observed that all tested beams failed due to reinforcement yielding while maintaining satisfactory ductility; the failure mode was characterized by yielding of the bottom tensile reinforcement followed by crushing of the UHPC in the compression zone. When the steel reinforcement ratio increased from 2.03% to 2.42% and 3.08%, the beam load-carrying capacity increased by 6.27% and 14.34%, respectively. When the GFRP reinforcement ratio increased from 0.91% to 1.19% and 1.51%, the peak load-carrying capacity increased by 9.58% and 15.55%, respectively. Based on reasonable assumptions, analytical formulas were proposed to predict the cracking moment and the flexural capacity of the UHPC beams reinforced with hybrid GFRP and steel bars, with errors within $\pm 5\%$. By fully accounting for the bridging effect of steel fibers, modified coefficients were introduced to estimate beam deformation and crack width, along with corresponding calculation methods. The proposed formulas accurately predicted cracking moment, ultimate moment, deflection and crack width for the beam. The findings propose a theoretical basis for the design and application of UHPC beams reinforced with hybrid GFRP and steel bars.

Keywords: UHPC; glass fiber-reinforced polymer; hybrid reinforcement; flexural behavior



Academic Editor: Marco Di Ludovico

Received: 29 December 2025

Revised: 23 January 2026

Accepted: 27 January 2026

Published: 28 January 2026

Copyright: © 2026 by the authors.

Licensee MDPI, Basel, Switzerland.

This article is an open access article distributed under the terms and conditions of the [Creative Commons Attribution \(CC BY\)](https://creativecommons.org/licenses/by/4.0/) license.

1. Introduction

In marine environments, reinforced concrete (RC) structures often face high concentrations of chloride ion erosion and salt mist, leading to corrosion of steel reinforcements and a significant reduction in the load-bearing capacity and service life of the structures [1–4]. Fiber-reinforced polymer (FRP) bars provide notable benefits, including low weight, high tensile strength, good fatigue performance and excellent corrosion resistance [5,6], making them a potential choice for structural reinforcement in marine environments. Many scholars have conducted in-depth research on the mechanical properties of the structures reinforced with FRP bars [7–9]. However, GFRP bars are characterized by a relatively low elastic

modulus and the absence of a yielding plateau, which results in poor ductility in fully GFRP-reinforced members. This may lead to brittle failure modes, wider cracks and excessive deflection [10], thereby limiting their widespread application in structural engineering.

To overcome these limitations, researchers have proposed a hybrid reinforcement strategy. In this approach, GFRP bars replace the corner steel bars that are prone to corrosion, while the corrosion-susceptible steel bars are arranged on the interior side to afford a thicker concrete cover. This approach enhances the ductility and service performance of the members. Ge [11] conducted bending tests on hybrid concrete beams reinforced with basalt-fiber-reinforced plastic (BFRP) and steel bars. He found that the ductility of hybrid reinforced concrete beams was superior to that of purely BFRP-reinforced beams, but lower than that of conventional RC beams. The calculation formula of the flexural bearing capacity of hybrid reinforced concrete beams was established. Similarly, Refar [12] carried out bending tests on hybrid concrete beams reinforced with BFRP and steel bars. Formulas were introduced to predict both deflection and crack width. Maranan [13] performed bending tests on geopolymer concrete beams reinforced with a hybrid reinforcement of steel and GFRP bars, demonstrating improved ductility and crack resistance compared to beams reinforced solely with GFRP.

Although the use of hybrid reinforcement can enhance the durability of structural members to some extent, it possesses significant limitations and fails to completely resolve the corrosion issue in RC structures within marine environments. This is primarily because the wetting–drying cycles of the marine environments cause concrete to shrink due to drying and to expand when wet, leading to the formation and propagation of micro-cracks in the concrete [14]. Moreover, the wetting–drying cycles can significantly exacerbate chloride-induced corrosion. Research by Li [15] revealed that due to the adsorption of chloride ions, the depth and concentration of intrusion even exceeded that of continuous chloride salt immersion in the process of dry–wet alternation.

Consequently, chloride ions will inevitably invade the interior of the RC structure in the marine environment. The chloride salts present in seawater accelerate the corrosion of steel bars, which additionally react with concrete material to form non-cementitious by-products [16]. This degradation process leads to the expansion of concrete cracks and the increase in porosity, reducing the durability and service life of the structure. Therefore, improving the compactness of concrete and preventing or slowing the ingress of aggressive agents like chloride ions, is critical to enhancing the corrosion resistance and long-term durability of structures used in marine engineering, bridges and port facilities.

Ultra-high-performance concrete (UHPC), alternatively referred to as reactive powder concrete (RPC), is an advanced construction material primarily composed of cement, quartz sand, silica fume, slag and steel fibers. It is characterized by ultra-high strength, excellent durability and exceptional impermeability [17–19], giving it superior resistance to erosion and corrosion under marine exposure. Pyo [20] found that under standard curing conditions, the total porosity of UHPC is only 1–3% of the total volume, which is five to ten times lower than that of high-performance concrete (HPC) and ordinary concrete. In a separate study, Pyo [21] reported that the chloride diffusion coefficient in UHPC is merely 0.01–0.1 times that of ordinary concrete, highlighting its effectiveness in blocking external corrosive agents. Additionally, Ghafari [22] revealed that the time to cracking caused by accelerated corrosion in UHPC is twice as long as in HPC.

Despite these advantages, some steel fibers embedded near the surface of UHPC structures remain vulnerable to corrosion. However, research by Pyo [23] indicates that if the steel bars in UHPC members have sufficient protective cover thickness, surface steel fiber corrosion exerts little effect on the structural mechanical performance. For thin

components like slabs, however, the effect of steel fiber corrosion near the surface can become notable and should not be disregarded.

This research investigates UHPC beams incorporating hybrid reinforcement. In this approach, steel bars at the corners of the beam, which are close to the concrete surface and susceptible to corrosion, are replaced with GFRP bars, such that internal steel bars are provided with increased concrete cover. This configuration not only addresses the corrosion issue in marine environments but also leverages the high stiffness and excellent ductility of UHPC in its plastic stage. This compensates for the inherent deficiencies of GFRP bars, such as insufficient ductility and excessive deflection caused by low stiffness [11,12], thus markedly improving both the mechanical behavior and durability of hybrid reinforced beams. In addition, GFRP bars are deliberately selected in this study because they are widely available and cost-effective in practical applications, and their relatively low elastic modulus makes serviceability (deflection and crack-width control) a critical and representative design concern for hybrid reinforced UHPC beams. It should be noted that higher-modulus FRP bars (e.g., AFRP or CFRP) may substantially alter stiffness, crack development and load-sharing mechanisms; therefore, the conclusions drawn herein are primarily applicable to hybrid GFRP–steel-reinforced UHPC beams, and further validation for other FRP types will be pursued in future studies.

Although FRP-reinforced and hybrid FRP–steel-reinforced beams have been widely studied, existing research has mainly focused on normal-strength or geopolymer concrete [24–26], while investigations on UHPC beams with hybrid reinforcement remain limited. Owing to the ultra-high strength, strain-hardening behavior and pronounced fiber-bridging effect of UHPC, together with the low elastic modulus and non-yielding behavior of GFRP bars, the flexural response and serviceability performance of hybrid reinforced UHPC beams may differ significantly from those of conventional concrete members. In particular, the cooperative working mechanism between GFRP bars and steel bars, the applicability of the plane-section assumption and reliable prediction methods for flexural capacity and serviceability behavior (deflection and crack width) have not yet been fully clarified.

Although considerable research has been conducted on steel-reinforced UHPC beams, studies on UHPC beams incorporating hybrid reinforcement remain limited.

To advance the engineering application of hybrid reinforced UHPC beams, several key challenges must be resolved: (1) The influence of the bond performance between GFRP bars and UHPC on the crack resistance and bearing capacity of beams has not been fully clarified. (2) The cooperative working mechanism of steel bars and GFRP bars under flexural loading requires further investigation to verify whether the two types of bars meet the plane-section assumption. (3) The prediction method of flexural bearing capacity and the control theories for deflection and cracking of hybrid reinforced UHPC beams need to be further refined and validated.

To address these challenges, five UHPC beams reinforced with hybrid GFRP and steel bars were designed and fabricated. Four-point bending tests were conducted to systematically investigate the failure modes, crack distribution, mid-span deflection and strain variations. On this basis, analytical formulas to predict the cracking load and flexural capacity were developed. In addition, prediction methods to estimate short-term stiffness and crack width were proposed. The research findings offer both experimental evidence and theoretical foundation for the practical use of hybrid reinforced UHPC beams.

2. Experimental Programs

2.1. Specimen Design and Fabrication

Five UHPC beams reinforced with hybrid GFRP and steel bars were designed and fabricated, labeled B-1 through B-5. As illustrated in Figure 1, each of the five beams used a rectangular section ($b \times h$) of 150 mm $\times$ 250 mm. The overall length was 1800 mm with a clear span of 1500 mm, leaving 150 mm overhangs beyond each support. The constant-moment (pure bending) region measured 500 mm. Compression reinforcement consisted of two 6 mm diameter steel bars, while 12 mm stirrups were installed at 50 mm intervals. To ensure that the beam specimens failed in flexure rather than in flexure–shear, both the flexural capacity and the shear capacity were calculated. The results indicated that providing stirrups at a spacing of 50 mm satisfied the strong-shear–weak-flexure design requirement, thereby ensuring flexural failure of the specimens. Each specimen was designed to fail in a balanced mode.

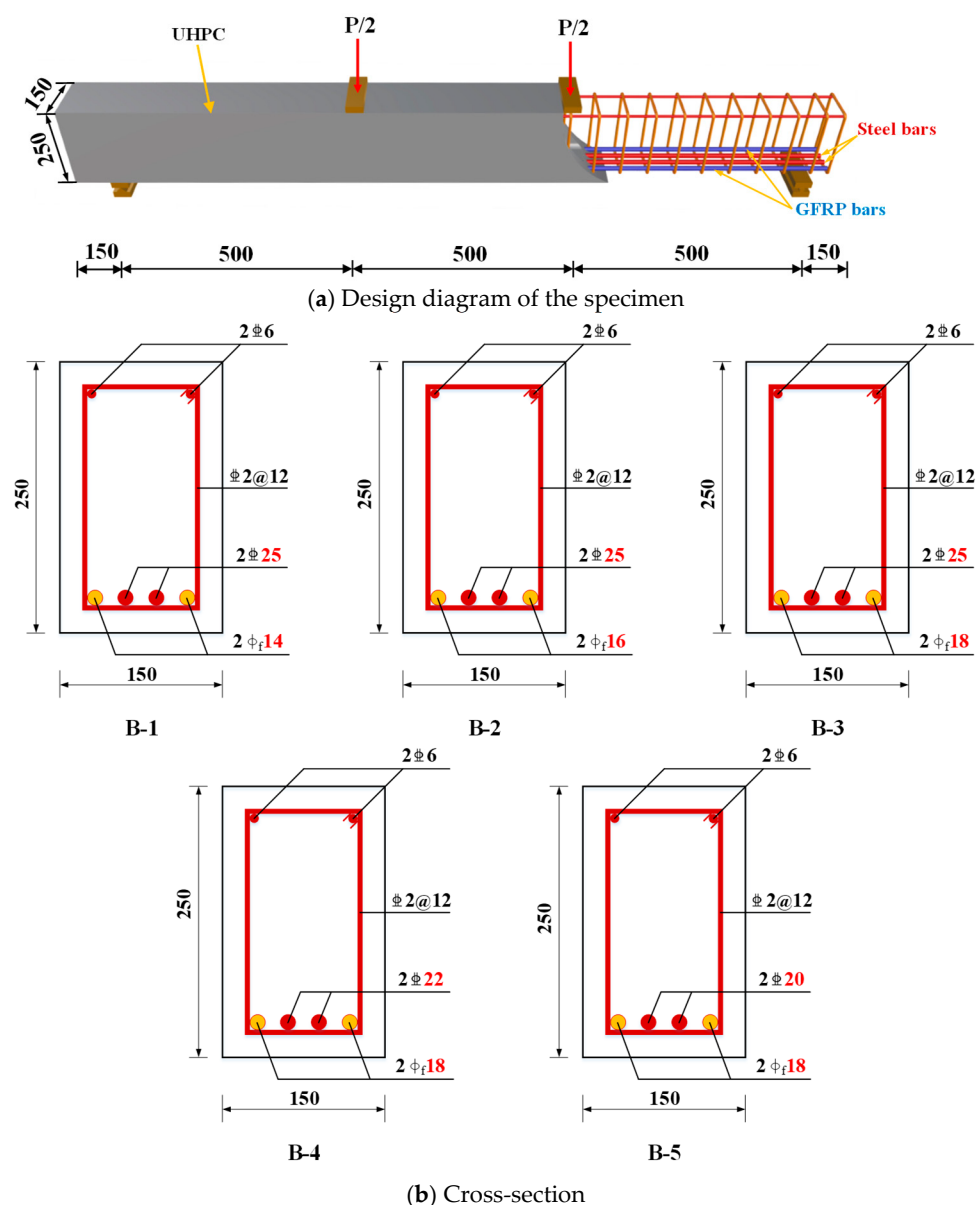


Figure 1. Details of the specimens (Unit: mm).

The reinforcement layouts are summarized in Table 1, where ρ_s denotes the reinforcement ratio of steel bars and ρ_f represents that of GFRP bars.

Table 1. Design parameters of specimens.

No.	Steel Bars	ρ_s (%)	GFRP Bars	ρ_f (%)	ρ (%)	ρ_1 (%)	ρ_2 (%)
B-1	2C25	3.08	2A _f 14	0.91	3.99	4.03	3.30
B-2	2C25	3.08	2A _f 16	1.19	4.27	4.33	3.37
B-3	2C25	3.08	2A _f 18	1.51	4.58	4.67	3.45
B-4	2C22	2.42	2A _f 18	1.51	3.93	4.02	2.80
B-5	2C20	2.03	2A _f 18	1.51	3.54	3.63	2.40

The primary test parameters were ρ_s and ρ_f . To quantify the overall reinforcement ratio of the specimens, the ratios ρ , ρ_1 and ρ_2 were defined as shown in Equations (1)–(3), representing the actual ratio, the strength-converted ratio and the modulus-converted ratio, respectively. The design tensile stress for GFRP reinforcement f_{fd} was specified as a minimum of $0.01 E_f$ and $0.75 f_{fu}$, where f_{fu} is the ultimate tensile strength of the GFRP bars, E_s is the elastic modulus of steel bars and E_f is the elastic modulus of the GFRP bars.

$$\rho = \rho_s + \rho_f \quad (1)$$

$$\rho_1 = \rho_s + \frac{f_{fd}}{f_y} \rho_f \quad (2)$$

$$\rho_2 = \rho_s + \frac{E_f}{E_s} \rho_f \quad (3)$$

The main materials used for preparing UHPC included Portland cement of grade PO42.5 (Jiangsu Helin Cement Co., Ltd., Zhenjiang, China), 40–70 mesh quartz and 70–140 mesh quartz sand (mass ratio of 1:1, Fengyang County Chaoxin Building Materials Co., Ltd., Chuzhou, China), S95 slag powder (Nanjing Nangang Jiahua New Building Materials Co., Ltd., Nanjing, China), silica fume (Shanghai Shengkuo Chemical Technology Co., Ltd., Shanghai, China) and high-efficiency polycarboxylate superplasticizer (Jiangsu China Railway Arit New Materials Co., Ltd., Nanjing, China). The mass ratio of cement, slag powder, silica fume and quartz sand was 1:0.28:0.14:1.2. The content of high-efficiency polycarboxylate superplasticizer was 1.5%, the steel fiber (Jiangsu Bositai Steel Fiber Co., Ltd., Taizhou, China) volume fraction was 2.5% (avg. diameter 0.22 mm length 12 mm) and the water–binder ratio was 0.18.

2.2. Material Properties

During specimen casting, multiple sets of companion specimens were prepared and cured under the same conditions as the UHPC beams. These included cubic specimens with dimensions of 100 mm × 100 mm × 100 mm, prismatic specimens of 100 mm × 100 mm × 300 mm and three dumbbell-shaped specimens with mid-section dimensions of 40 mm × 15 mm. UHPC mechanical properties were determined following DBJ43/T 325-2017 [27]. The mechanical property tests of steel bars were performed following GB/T 228.1-2021 [28], and GFRP bars in accordance with ISO 10406-1:2015 [29].

Table 2 presents the average mechanical properties of UHPC, where the cube compressive strength f_{cu} , axial compressive strength f_c and tensile strength f_t are measured values, while the elastic modulus E_c is calculated according to Ouyang [30]. The stress–strain relationships of UHPC are given in Equations (4) and (5), where σ_c and σ_t denote the axial compressive stress and axial tensile stress of UHPC, respectively; ε_c and ε_t denote the corresponding axial compressive strain and axial tensile strain. ε_{c0} and ε_{t0} are the strains corresponding to the peak compressive stress and peak tensile stress of UHPC, respectively. Table 3 summarizes the average mechanical properties of the GFRP bars, in which the tensile strength f_{fu} is experimentally measured and the elastic modulus E_f is calculated

based on [29]. Table 4 reports the average mechanical properties of the steel bars, where the yield strength f_y and ultimate tensile strength f_u are measured values, and the elastic modulus E_s is calculated according to [28]. The material properties of the steel fibers are presented in Table 5.

Table 2. Mechanical properties of UHPC.

f_{cu} (MPa)	f_c (MPa)	f_t (MPa)	E_c (GPa)
107.24	94.37	9.47	39.7

Table 3. Mechanical properties of GFRP bars.

Diameter (mm)	f_{fu} (MPa)	E_f (GPa)
14	1050	49.3
16	1120	49.4
18	1134	49.8

Table 4. Mechanical properties of steel bars.

Diameter (mm)	f_y (MPa)	f_u (MPa)	E_s (GPa)
16	463.5	625.6	202
20	479.1	638.5	201
22	476.7	659.7	201
25	467.8	619.4	200

Table 5. Mechanical properties of steel fibers.

Steel Fiber Type	Average Length (mm)	Diameter (mm)	Tensile Strength (MPa)
Copper-coated micro steel fiber	13	0.23	2850

It is worth emphasizing that, in contrast to conventional steel bars, GFRP bars lack a distinct yield point and maintain a linear elastic response until failure, typically demonstrating a higher tensile strength but a lower elastic modulus than the steel. By incorporating the experimentally obtained material parameters into Equations (2) and (3), the comprehensive reinforcement ratios ρ_1 and ρ_2 for the specimens can subsequently be determined, as detailed in Table 1.

$$\frac{\sigma_t}{f_t} = \begin{cases} 1.17\varepsilon_t/\varepsilon_{t0} + 0.65(\varepsilon_t/\varepsilon_{t0})^2 - 0.83(\varepsilon_t/\varepsilon_{t0})^3 & 0 \leq \varepsilon_t/\varepsilon_{t0} < 1 \\ \frac{\varepsilon_t/\varepsilon_{t0}}{5.5(\varepsilon_t/\varepsilon_{t0} - 1)^{2.2} + \varepsilon_t/\varepsilon_{t0}} & \varepsilon_t/\varepsilon_{t0} \geq 1 \end{cases} \quad (4)$$

$$\frac{\sigma_c}{f_c} = \begin{cases} 1.55\varepsilon_c/\varepsilon_{c0} - 1.20(\varepsilon_c/\varepsilon_{c0})^4 + 0.65(\varepsilon_c/\varepsilon_{c0})^5 & 0 \leq \varepsilon_c/\varepsilon_{c0} < 1 \\ \frac{\varepsilon_c/\varepsilon_{c0}}{6(\varepsilon_c/\varepsilon_{c0} - 1)^2 + \varepsilon_c/\varepsilon_{c0}} & \varepsilon_c/\varepsilon_{c0} \geq 1 \end{cases} \quad (5)$$

2.3. Loading Scheme and Measurement Point

Figure 2 illustrates the test loading setup and the layout of the measurement points. Each specimen was supported by a fixed hinge at one end and a sliding hinge at the other. Symmetrical two-point loading was applied using two hydraulic jacks. To prevent local compressive failure of the UHPC, 5 mm thick steel plates were placed at each loading point and support.

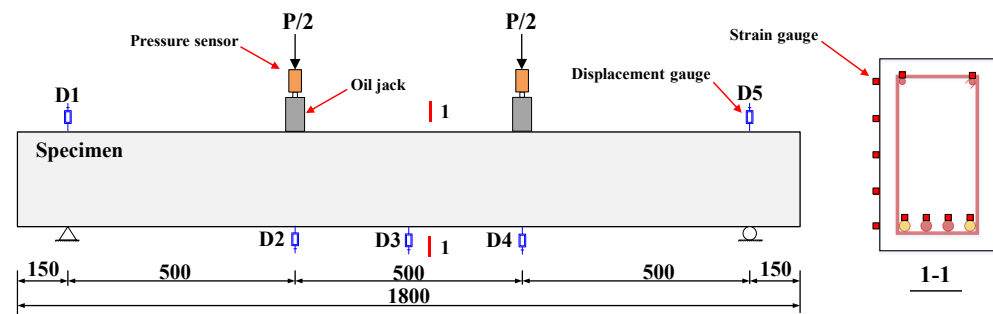


Figure 2. Loading device and measuring point layout.

Prior to formal testing, a preload was applied to check the proper operation of the loading apparatus and measurement devices. During the main loading phase, 5 kN increments were used initially to identify the cracking load, after which the increment was adjusted to 10% of the estimated peak load. Once the applied load reached 80% of the calculated peak load, the load increment at each stage was appropriately reduced to ensure safety and accurate observation. The test was terminated when either a crushing failure occurred in the UHPC compression zone or there was a sharp decline in load, indicating failure.

Measurements covered the load–deflection response of the specimens; the strains of GFRP bars, steel bars and the UHPC side surface within the pure bending span; the cracking P_{cr} , yield P_y and peak load P_u of the specimens; and the crack widths measured at various load levels.

Support settlement and mid-span deflection were determined using five electronic displacement gauges located at the mid-span, the loading points and the supports. Strain gauges were attached to the tensile GFRP and steel bars at mid-span, with additional gauges fixed to the UHPC side face to verify the plane-section assumption.

Strain and pressure data were collected using a TS3862 static resistance strain gauge (Yangzhou Test Electric Co., Ltd., Yangzhou, China). Crack widths were gauged with a crack-width meter. A marker pen was used to trace the crack development patterns.

3. Experimental Results and Analysis

3.1. Experimental Observations and Failure Modes

Figure 3 shows the failure modes and crack patterns of the five tested specimens. In the shear-bending spans, vertical cracks consistently formed in the lower regions and propagated upward toward the loading point after reaching a certain height. Within the pure bending span, cracks primarily developed vertically, initiating from the bottom and extending upward. All specimens exhibited typical flexural failure behavior: the tensile steel bars in the lower region yielded and the UHPC in the upper compression zone was crushed.

Taking specimen B-2 as a representative case, the loading procedure and corresponding phenomena can be summarized as follows: At the onset of loading, the specimen exhibited linear elastic response, with mid-span deflection exhibiting a proportional relationship to the applied load. Upon the vertical load attaining 60.2 kN, the initial vertical crack emerged within the pure bending span of the specimen, measuring approximately 0.010 mm in width and 4.6 cm in length. As the load continued to increase, additional vertical cracks formed almost symmetrically on both sides of the pure bending span, progressively widening and extending upward.

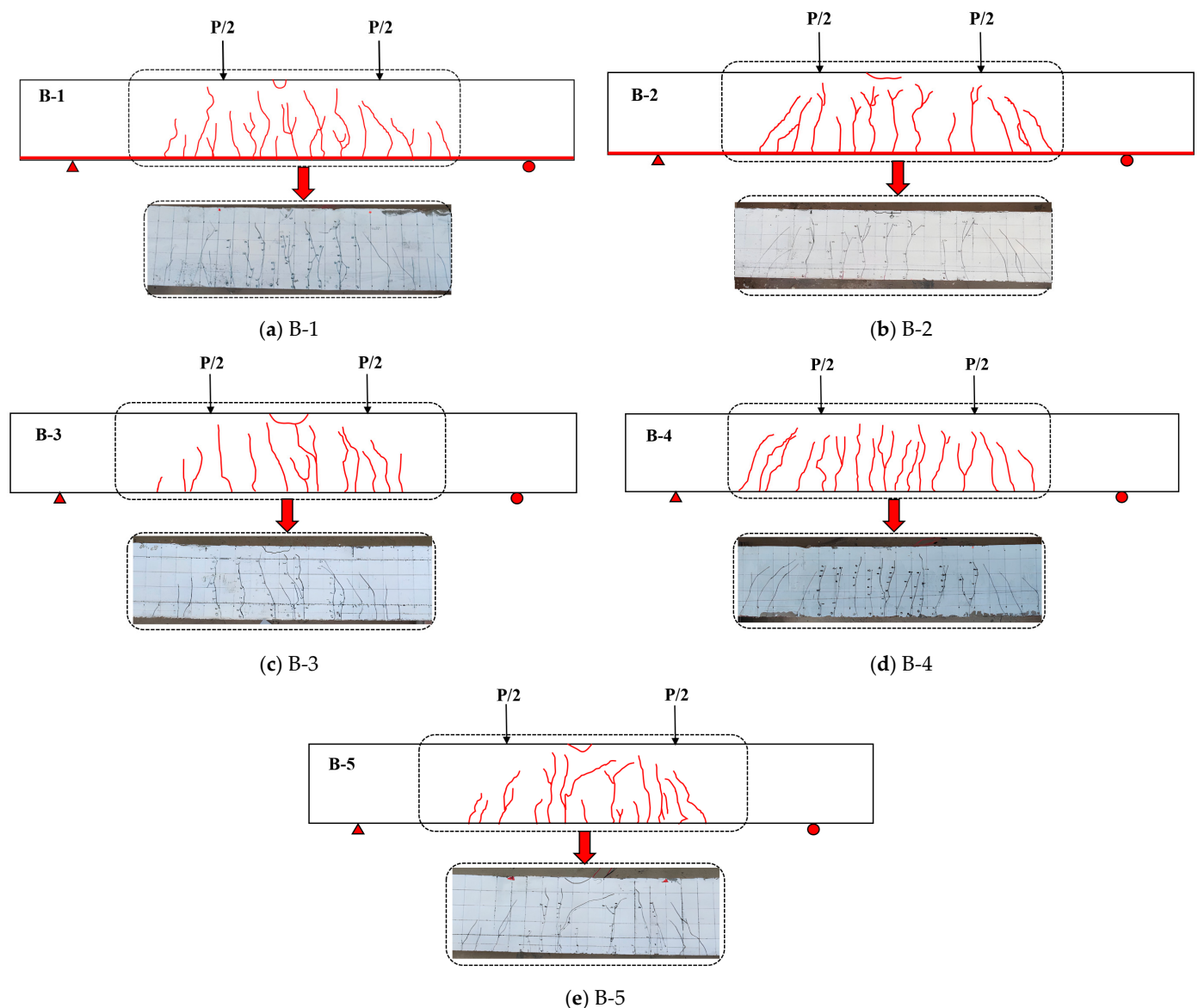


Figure 3. Failure modes and crack distribution.

When the load attained 167.4 kN, no additional cracks were observed in the pure bending region. Upon increasing the load to 216.6 kN, the tensile steel bars yielded and the maximum crack width at mid-span reached 0.203 mm. Subsequent loading produced horizontal longitudinal cracks on the upper UHPC face and a pronounced increase in mid-span deflection.

Ultimately, the UHPC in the upper region was crushed and the specimen achieved its ultimate capacity. Benefiting from the presence of steel fibers, significant spalling was prevented following UHPC crushing, and the specimen maintained good structural integrity, which was consistent with the experimental observations by Lai [31].

A peak load of 265.0 kN was attained, associated with a mid-span deflection of 15.63 mm. The maximum crack within the pure bending span widened to 1.022 mm and extended to a length of 20.8 cm. Upon unloading, the mid-span deflection recovered well, and the fine cracks exhibited effective closure.

3.2. Load–Deflection Curve

Table 6 summarizes the characteristic loads alongside their mid-span deflections. In the table, P_{cr} , P_y and P_u refer to the cracking, yield and peak load, respectively, while Δ_{cr} , Δ_y and Δ_u denote the associated deflections.

Table 6. Characteristic load and deflections.

No.	P_{cr} (kN)	Δ_{cr} (mm)	P_y (kN)	Δ_y (mm)	P_u (kN)	Δ_u (mm)
B-1	59.6	1.12	199.8	5.28	265.0	15.63
B-2	60.2	0.9	216.8	5.48	290.4	15.97
B-3	66.8	1.01	238.8	5.9	306.2	15.68
B-4	56.2	0.98	194.2	5.35	284.6	16.05
B-5	49.2	1.01	183.5	5.49	267.8	15.92

The mid-span load–deflection (P – Δ) responses are shown in Figure 4, which can be partitioned into three stages: (1) from initial loading to UHPC cracking; (2) from UHPC cracking to yielding of the tensile steel bars; and (3) from steel yielding to ultimate failure.

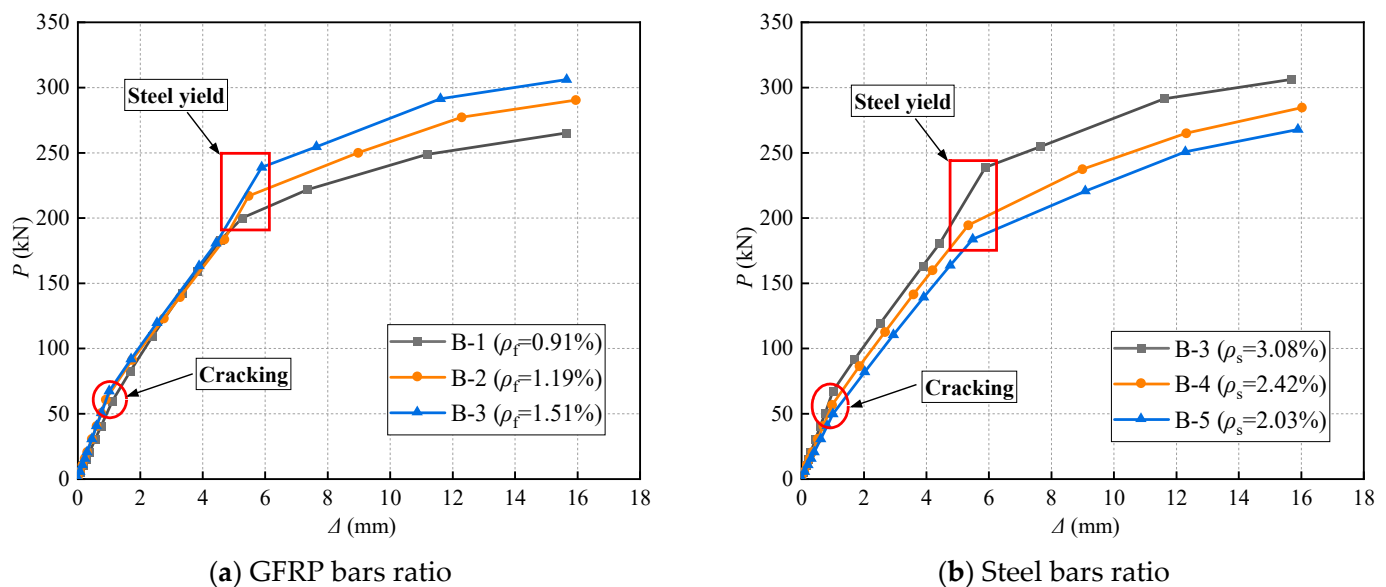


Figure 4. Load–deflection curve at mid-span.

Cracking of the tensile zone UHPC produced a distinct kink on the curve. The load and mid-span deflection at this point were defined as the cracking load P_{cr} and cracking deflection Δ_{cr} . As loading continued, yielding of the tensile steel bars led to a sharp growth in mid-span deflection and notable crack widening. The curve exhibited a second clear inflection point, corresponding to the yield load P_y and yield deflection Δ_y . As the load continued to increase, the specimen entered the hardening stage: mid-span deflection continued to increase while the load increment slowed, until the upper UHPC was crushed and the beam failed. The maximum applied load at this point was defined as the peak load P_u , with the corresponding deflection denoted as the ultimate deflection Δ_u .

A comparative analysis of the curves of B-1, B-2 and B-3 indicated that, with a constant steel bar reinforcement ratio, increasing the reinforcement ratio of GFRP bars raised both the cracking and yield loads, while the overall stiffness remained nearly unchanged. Specifically, the peak load capacity increased from 265.0 kN to 290.4 kN and 306.2 kN, representing increases of 9.58% and 15.55%, respectively. This phenomenon is primarily ascribed to the relatively low elastic modulus of GFRP bars, which limits their contribution to the section's

flexural stiffness, despite their high strength. This behavior is further supported by an analysis of the reinforcement ratios: as the GFRP bars' reinforcement ratio ρ_f was increased from 0.91% to 1.19% and 1.51%, the reinforcement ratio converted by strength ρ_1 rose from 3.99% to 4.27% and 4.58%, representing increases of 7.02% and 14.79%. This trend aligned closely with the observed increase in peak bearing capacity. In contrast, reinforcement ratio converted by elastic modulus ρ_2 increased only marginally from 3.30% to 3.37% and 3.45%, which corresponded to the insignificant change observed in the flexural stiffness of the specimens.

A comparative analysis of the curve of B-3, B-4 and B-5 demonstrates that when the GFRP reinforcement ratio ρ_f was held constant, increasing the steel reinforcement ratio ρ_s resulted in a simultaneous increase in the load at cracking, yield point and overall stiffness. Specifically, the peak load exhibited an increase from 267.8 kN to 284.6 kN and 306.2 kN, marking increases of 6.27% and 14.34%, respectively.

This trend is further explained by the analysis of the reinforcement ratios. As ρ_s was increased from 2.03% to 2.42% and 3.08%, the ρ_1 rose from 3.63% to 4.02% and 4.67% (increases of 10.74% and 28.65%, respectively). Similarly, the ρ_2 rose from 2.40% to 2.80% and 3.45% (increases of 16.67% and 43.75%, respectively). Consequently, both the reinforcement ratios converted by strength and elastic modulus exhibited significant growth with the increase in the reinforcement ratio of steel bars. This directly corresponded to the synchronized enhancements observed in the stiffness and peak bearing capacity of the specimens.

Furthermore, increasing the reinforcement ratio of GFRP bars from 0.87% to 1.14% and 1.44% led to peak bearing capacity enhancements of 9.58% and 15.55%, respectively. In contrast, as the steel reinforcement ratio increased from 1.94% to 2.32% and 2.95%, the peak load rose by only 6.27% and 14.34%, respectively. These results revealed that the peak bearing capacity exhibited greater sensitivity to the reinforcement ratio of GFRP bars. This is because GFRP bars with high ultimate tensile strength could continue to bear the load after the steel bars in the tensile zone reached the yield strength. Moreover, there was no tensile failure in the test in which GFRP bars ruptured and UHPC in the compression zone was not crushed. This demonstrates that the hybrid reinforced UHPC beams can enable more effective utilization of the strength of both steel and GFRP bars.

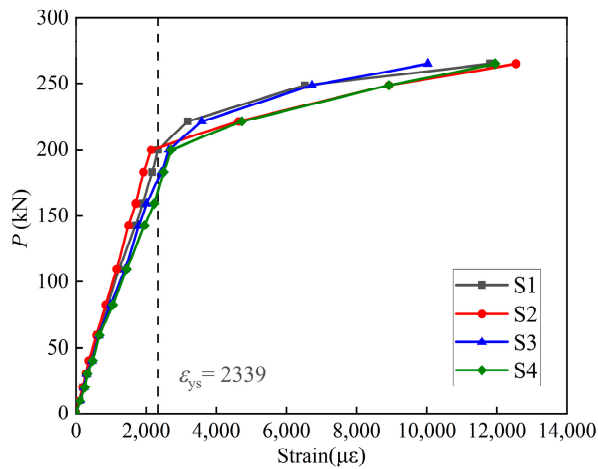
3.3. Load–Strain Response

Figure 5 presents the load–strain response for the tensile steel bars and GFRP bars within the pure bending span of the specimens. Specifically, S1 and S2 represent strain gauges installed on the GFRP bars, while S3 and S4 correspond to those affixed to the steel bars. From the figure, it is evident that a strong bond was maintained between the UHPC and GFRP bars during the loading process, with no significant slip observed. At the point of ultimate bearing capacity of the test beam, the tensile steel bars had already yielded. Moreover, the GFRP and steel bars located at the same height exhibited similar strain values, further confirming the compatibility of deformation and effective load transfer between the materials.

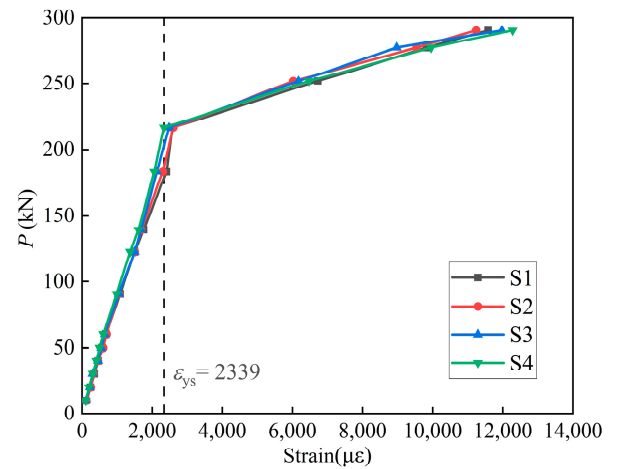
3.4. Strain Distribution Across the Depth of the Mid-Span Cross-Section

Figure 6 illustrates the average strain distribution across the depth of the mid-span cross-section. It is evident that prior to the yield point of the tensile steel bars, the strain across the section exhibited an approximately linear variation with the height, thereby validating the plane-section assumption. After the yielding of the steel bars, although the increased strain gradient in the plastic zone led to localized deflections in the curve, the strain in the concrete compression zone and uncracked regions maintained a linear

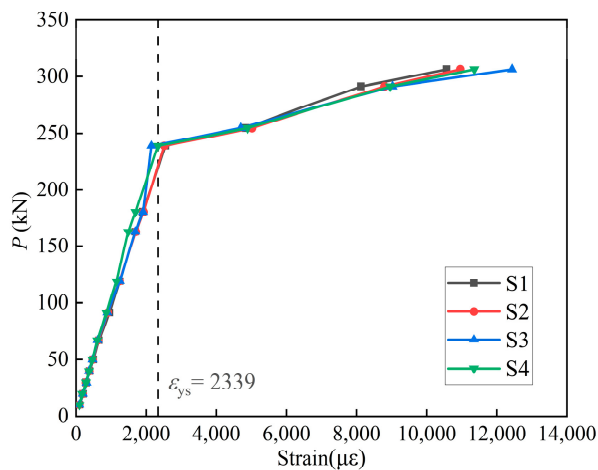
distribution, thus remaining broadly consistent with the assumption of a plane section. This supports the theoretical foundation for deriving flexural capacity formulas.



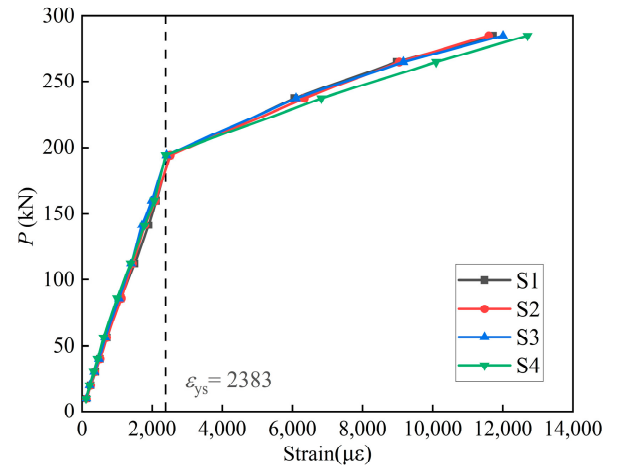
(a) B-1



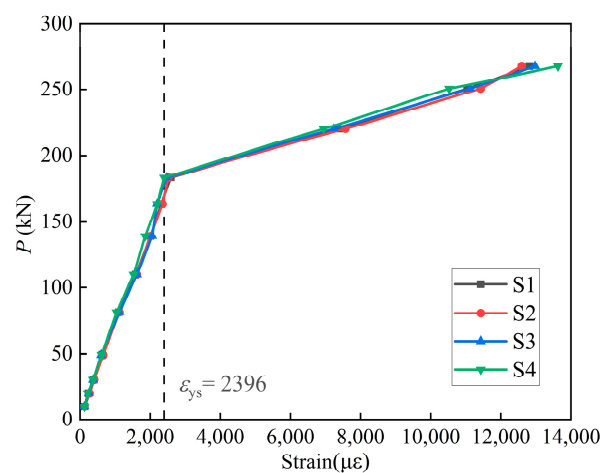
(b) B-2



(c) B-3



(d) B-4



(e) B-5

Figure 5. Load–strain response.

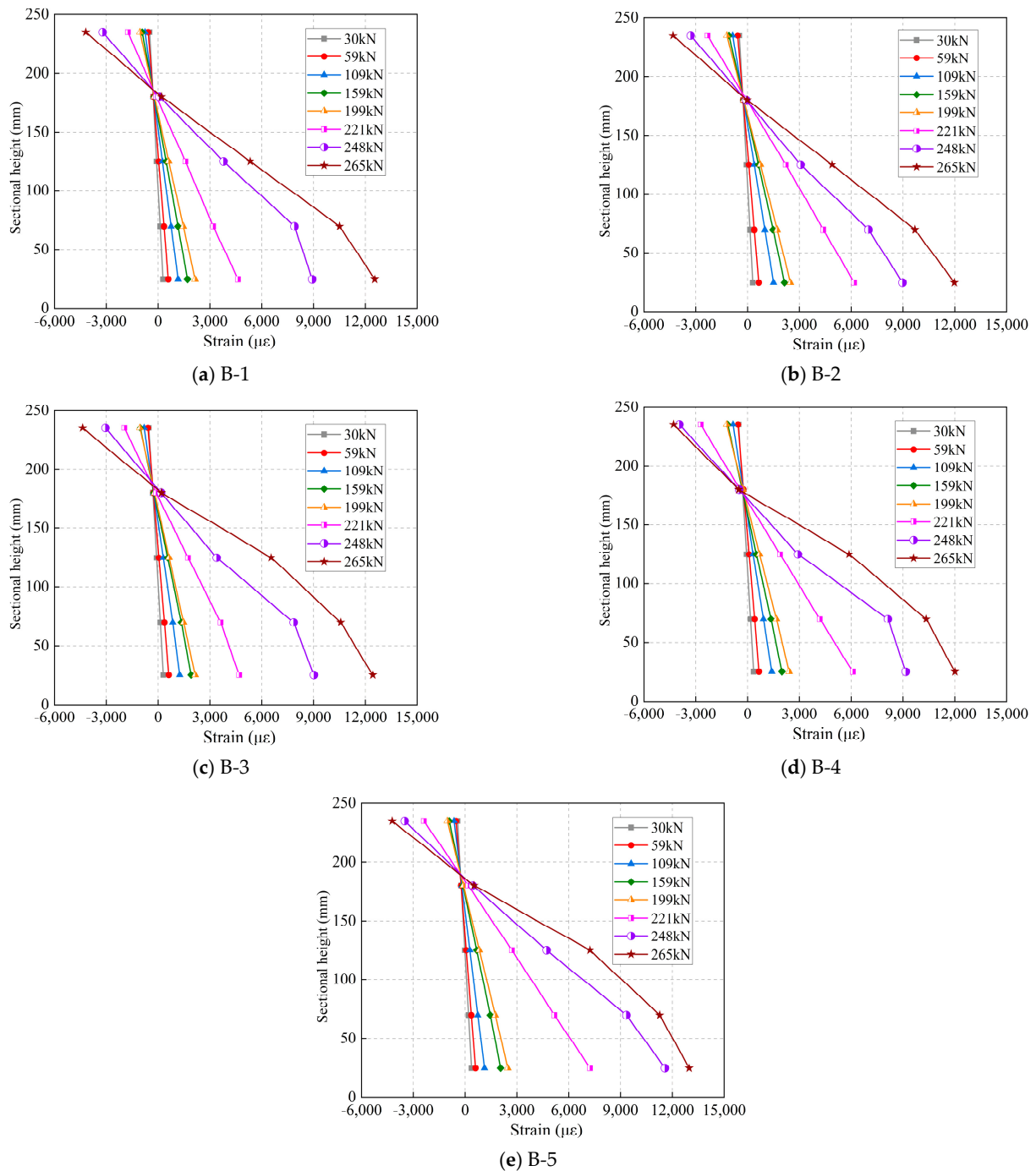


Figure 6. Average strain distribution along height of cross-section at mid-span.

4. Cracking Moment

4.1. Calculation Method

When the applied load reaches the cracking load of the specimen, according to the principle of sectional moment conservation, the cracking moment is given by Equation (6).

$$M_{cr} = M_c + M_f + M_s = f_t + W_s \quad (6)$$

M_c , M_f and M_s denote the balanced moments of the UHPC, the GFRP bars and the tension-side steel bars, respectively; f_t is the ultimate tensile strength of the UHPC; W_s is the section resisting moment accounting for the plasticity of UHPC in the tension zone.

The plasticity influence coefficient γ_m of the section resisting moment is introduced to characterize the elastic–plastic development of the tension zone.

$$M_{cr} = \gamma_m f_t W_0 \quad (7)$$

W_0 denotes the section resisting moment of the beam calculated based on elastic theory. Combining the above equations, Equation (8) can be obtained.

$$I_m = W_s / W_0 \quad (8)$$

The compressive stress in the compression zone of the beam is relatively low; its distribution can be idealized as triangular. In contrast, the tensile zone stress is nonlinearly distributed. To simplify the method, the stress curve of the tension zone is approximated by a combination of triangular and trapezoidal shapes, as depicted in Figure 7. The heights of the elastic and plastic regions in tension are denoted by x_1 and x_2 , respectively. The compression zone depth is x_c . Following Zheng [32], the strain ε_{t0} at the peak tensile stress of UHPC is 0.00025 and the ultimate tensile strain ε_{tu} is 0.00075. Therefore, the value of $\varepsilon_{t0}/\varepsilon_{tu}$ is 1/3. Under the plane-section assumption, $x_1 = 1/2$ and $x_2 = 1/3 (h - x_c)$ can be obtained, where h is the section height of the beam.

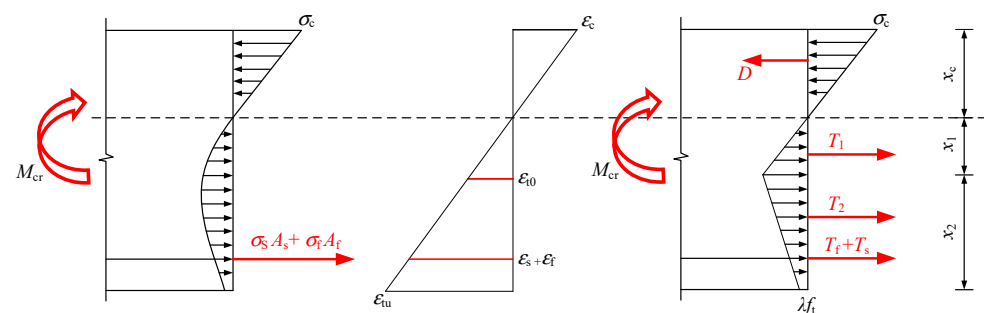


Figure 7. Simplified stress diagram for calculating cracking moment.

The UHPC compressive strain, together with the tensile strains in the steel and GFRP bars, can be determined, where x_c denotes the compression zone depth and h_0 is the effective section depth.

$$\varepsilon_c = \frac{x_c}{h - x_c} \varepsilon_{tu} = 3\varepsilon_{t0} \frac{x_c}{h - x_c} \quad (9)$$

$$\varepsilon_s = \frac{h_0 - x_c}{h - x_c} \varepsilon_{tu} = 3\varepsilon_{t0} \frac{h_0 - x_c}{h - x_c} \quad (10)$$

$$\varepsilon_f = \frac{h_0 - x_c}{h - x_c} \varepsilon_{tu} = 3\varepsilon_{t0} \frac{h_0 - x_c}{h - x_c} \quad (11)$$

The resultant compressive force D and its bending moment about the neutral axis M_c are given by Equations (12) and (13), where σ_c denotes the UHPC compressive stress, b is the section width and E_c is the compressive elastic modulus of UHPC.

$$D = \frac{1}{2} \sigma_c b x_c = \frac{1}{2} E_c \varepsilon_c b x_c = \frac{3}{2} E_c b \frac{x_c^2}{h - x_c} \varepsilon_{t0} \quad (12)$$

$$M_c = \frac{2}{3} x_c D = E_c b \frac{x_c^3}{h - x_c} \varepsilon_{t0} \quad (13)$$

The elastic-tension resultant force T_1 of UHPC and its bending moment about the neutral axis M_1 are given by Equations (14) and (15).

$$T_1 = \frac{1}{2}f_t b x_1 = \frac{1}{6}f_t b (h - x_c) \quad (14)$$

$$M_1 = \frac{2}{3}x_1 T_1 = \frac{1}{27}f_t b (h - x_c)^2 \quad (15)$$

The resultant force T_2 of the plastic part of UHPC in the tension zone and its moment about the neutral axis M_2 are given by Equations (16) and (17).

$$T_2 = \frac{1}{2}(\lambda + 1)f_t b x_2 = \frac{1}{3}(\lambda + 1)f_t b (h - x_c) \quad (16)$$

$$M_2 = \left(\frac{2\lambda + 1}{3\lambda + 3}x_2 + x_1 \right) T_2 = \frac{7\lambda + 5}{27}f_t b (h - x_c)^2 \quad (17)$$

Tensile steel resultant force T_s and its moment about the neutral axis M_s are given by Equations (18) and (19), where σ_s denotes the tensile stress of the steel bar and E_s denotes its tensile elastic modulus.

$$T_s = \sigma_s A_s = E_s \varepsilon_s A_s = 3E_s A_s \varepsilon_{t0} \frac{h_0 - x_c}{h - x_c} \quad (18)$$

$$M_s = T_s (h_0 - x_c) = 3E_s A_s \varepsilon_{t0} \frac{(h_0 - x_c)^2}{h - x_c} \quad (19)$$

The resultant force T_f of the GFRP bar and its bending moment about the neutral axis M_f are given by Equations (20) and (21), where σ_f denotes the tensile stress of the GFRP bar and E_f denotes the tensile elastic modulus of GFRP bars.

$$T_f = \sigma_f A_f = E_f \varepsilon_f A_f = 3E_f A_f \varepsilon_{t0} \frac{h_0 - x_c}{h - x_c} \quad (20)$$

$$M_f = T_f (h_0 - x_c) = 3E_f A_f \varepsilon_{t0} \frac{(h_0 - x_c)^2}{h - x_c} \quad (21)$$

According to the equilibrium condition of the axial force of the section, Equation (22) can be obtained.

$$\left[\frac{3}{2}E_c b \varepsilon_{t0} - \frac{1}{6}f_t b - \frac{1}{3}(\lambda + 1)f_t b \right] x_c^2 + \left[\frac{1}{3}f_t b h + \frac{2}{3}(\lambda + 1)f_t b h + 3E_s A_s \varepsilon_{t0} + 3E_f A_f \varepsilon_{t0} \right] x_c - \frac{1}{6}f_t b h^2 - \frac{1}{3}(\lambda + 1)f_t b h^2 - 3E_s A_s \varepsilon_{t0} h_0 - 3E_f A_f \varepsilon_{t0} h_0 = 0 \quad (22)$$

The value of x_c can be determined from Equation (22). Based on the sectional moment equilibrium, the cracking bending moment is determined by Equation (23).

$$M_{cr} = E_c b \frac{x_c^3}{h - x_c} \varepsilon_{t0} + \frac{1}{27}f_t b (h - x_c)^2 + \frac{7\lambda + 5}{27}f_t b (h - x_c)^2 + 3E_s A_s \varepsilon_{t0} \frac{(h_0 - x_c)^2}{h - x_c} + 3E_f A_f \varepsilon_{t0} \frac{(h_0 - x_c)^2}{h - x_c} \quad (23)$$

4.2. Calculation Results

Table 7 lists the cracking moment values calculated according to Equation (22), M_{cr}^P alongside the test values M_{cr}^t . The average value of M_{cr}^P / M_{cr}^t is 1.04, with a standard deviation of 0.051 and a coefficient of variation of 0.049, indicating close agreement between the analysis and experiment.

Table 7. Comparison of M_{cr}^P and M_{cr}^t .

No.	M_{cr}^P (Mpa)	M_{cr}^t (Mpa)	M_{cr}^P/M_{cr}^t
B-1	31.5	29.8	1.05
B-2	31.8	30.1	1.05
B-3	32.1	33.4	0.96
B-4	29.2	28.1	1.03
B-5	27.3	24.6	1.10

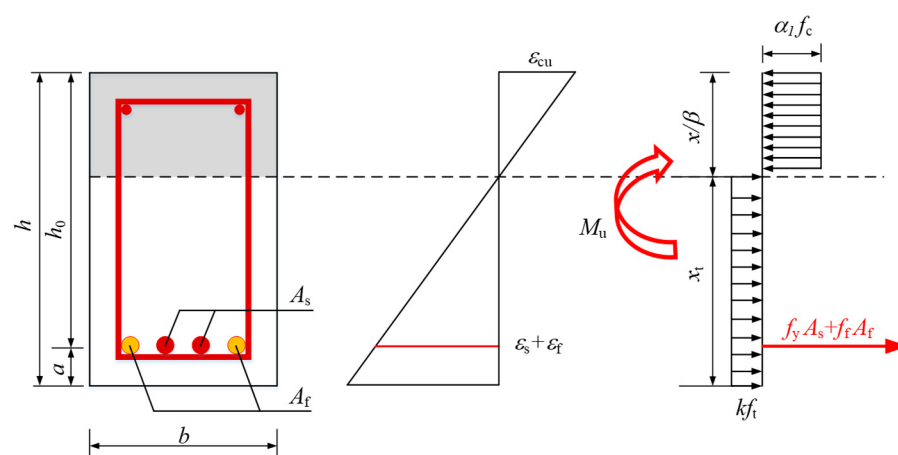
5. Flexural Capacity

5.1. Basic Assumptions

Normal section flexural capacity is determined subject to the following assumptions: (1) The cross-section is assumed to remain plane throughout bending, implying a linear strain distribution with depth from the neutral axis. (2) A perfect bond is maintained between the GFRP bars and the UHPC; the steel and GFRP bars located at the same height have the same strain. (3) The constitutive relationship of UHPC is defined by the model specified in DBJ43/T 325-2017 [27], which incorporates its tensile capacity. (4) The constitutive relationships of steel bars are represented by an ideal elastic–plastic model, while the linear elastic model is used for GFRP bars. The absolute value of the stress of steel bars shall not surpass its yield strength, while the absolute value of the stress of GFRP bars is limited to its design value of tensile stress f_{fd} .

5.2. Calculation Method

Figure 8 illustrates the compressive stress distribution across the normal section of the UHPC beam with hybrid GFRP–steel reinforcement. For the purpose of flexural capacity evaluation, the actual compressive stress is idealized as an equivalent rectangular block. The equivalence is based on two principles: (1) The area of the theoretical stress curve must be equal to that of the equivalent rectangular block, ensuring identical resultant force. (2) The centroid of the block must coincide with that of the actual stress distribution to maintain the position of the resultant compressive force, thereby preserving the internal moment arm.

**Figure 8.** Stress distribution of normal section.

Accounting for the tensile capacity of UHPC, the tensile strain of GFRP bars is given by Equation (24) under the plane-section assumption.

$$\epsilon_f = \epsilon_{cu} \frac{h_0 - x_c}{x_c} = \epsilon_{cu} \frac{\beta h_0 - x}{x} \quad (24)$$

The tensile stress of GFRP bars is given by Equation (25).

$$F_f = E_f \varepsilon_f = E_f \varepsilon_{cu} \frac{\beta h_0 - x}{x} \quad (25)$$

From the force equilibrium condition, Equation (26) is derived.

$$A_1 f_c b x = f_y A_s + f_f A_f + k f_t b x_t \quad (26)$$

In these Equations, UHPC ultimate compressive strain ε_{cu} is taken as 0.0055; b is the section width; h_0 is the effective depth; x denotes the compression depth; and the depth of tensile zone x_t is defined as $h - x/\beta$. α_1 is the ratio of the equivalent compressive strength of UHPC to its axial compressive strength; β represents the coefficient for the equivalent rectangular stress block in the compressive zone. The values of α_1 and β are taken as 0.90 and 0.77 in accordance with DBJ43/T 325-2017 [27].

A_s and A_f refer to the areas of the tensile steel bars and GFRP bars. To account for the contribution of UHPC to the tensile resistance of the specimens, a coefficient k (i.e., the equivalent rectangular stress block coefficient for the UHPC tensile zone) is introduced to quantify the tensile contribution of UHPC. According to Ref. [33], k typically ranges from 0.27 to 0.44; for simplicity and to err on the conservative side, k is taken as 0.25 in this study. a denotes the distance from the resultant tensile force to the UHPC tensile edge. According to the above formula, the height of equivalent compression zone can be calculated, and the load-bearing capacity is given by Equation (27).

$$M_u = (f_y A_s + f_f A_f) \left(h_0 - \frac{1}{2} x \right) + k f_t b x_t \left(\frac{1}{2} x_t + \frac{x}{\beta} - \frac{1}{2} x \right) \quad (27)$$

5.3. Calculation Results

Table 8 compares the calculated flexural capacity M_u^P and the experimentally measured values M_u^t for five specimens. The mean ratio of M_u^P / M_u^t is 0.975, with a standard deviation of 0.022 and a coefficient of variation of 0.023. Close agreement between the calculated and experimental results demonstrates the reliability of the formula for predicting the load-bearing capacity of hybrid reinforced beams.

Table 8. Comparison of M_u^P and M_u^t .

No.	M_u^P (Mpa)	M_u^t (Mpa)	M_u^P / M_u^t
B-1	134.5	132.5	1.02
B-2	140.9	145.2	0.97
B-3	147.4	153.1	0.96
B-4	136.9	142.3	0.96
B-5	130.5	133.9	0.97

6. Stiffness and Deflection

6.1. Stiffness Before the Cracking of UHPC

Prior to UHPC cracking, the bending moment varies approximately linearly with the curvature, and the slope of the bending moment–curvature M - ϕ curve is defined as the bending stiffness. According to the code GB50010-2010 [34], stiffness can be calculated according to Equation (28), where I_0 denotes the moment of inertia of the transformed section of the entire cross-section and the value is calculated according to Equation (29). In the formula, x represents the compression zone depth; A_0 is the area of the transformed section of the whole section; α_{Es} is the elastic modulus ratio, defined E_s / E_c ; A_s and A'_s

represent the areas of the tensile and compressive steel bars, respectively; and a'_s is the thickness of the UHPC cover of the compressive reinforcement.

Considering the plasticity of UHPC in the tensile zone, its elastic modulus is reduced by 20%. Additionally, the contribution of GFRP bars to the flexural stiffness is neglected. This is justified by the relatively low elastic modulus of GFRP bars, which is only approximately 1.2 times that of UHPC, resulting in a minimal influence on the overall section stiffness. Furthermore, this omission is exactly offset by the area that is doubly counted when calculating the transformed area of the steel bars, thereby significantly simplifying the calculation formula.

$$B = 0.8E_c I_0 \quad (28)$$

$$I_0 = \frac{1}{3}bx^3 + \frac{1}{3}b(h_0 - x)^3 + \alpha_{Es}[A_s(h_0 - x)^2 + A'_s(x - a'_s)^2] \quad (29)$$

$$x = \frac{\frac{1}{2}bh^2 + \alpha_{Es}(A_s h_0 + A'_s a'_s)}{A_0} \quad (30)$$

$$A_0 = bh + \alpha_{Es}(A_s + A'_s) \quad (31)$$

6.2. Short-Term Stiffness After the Cracking of UHPC

Short-term stiffness B_{sf} in the service stage is evaluated based on JGJT 465-2019 [35], as given by Equation (32). B_s denotes the flexural stiffness of the hybrid reinforced concrete beam converted to the UHPC strength grade and β_B is the steel fiber influence coefficient determined experimentally. λ_f is the characteristic fiber parameter defined as $V_f l_f / d_f$, where V_f is the fiber volume fraction, l_f is the average length of steel fiber and d_f is the average diameter of steel fiber.

$$B_{sf} = B_s(1 + \beta_B \lambda_f) \quad (32)$$

6.2.1. Calculation Formula of B_s

Based on the code GB50010-2010 [34], flexural stiffness B_s is determined as follows: During the service stage, the flexural stiffness of the cross-section decreases as the bending moment increases, making it a variable quantity. For the convenience of calculation, the stiffness at this stage is defined as the slope of the secant line connecting any point on the moment–curvature M – ϕ curve to the origin.

In this phase, the UHPC in the tensile zone is assumed to be fully cracked and is therefore disregarded, with only the steel and GFRP bars resisting the tensile forces. Consequently, as the contribution of tensile UHPC is neglected, the role of the GFRP bars becomes significant and must be considered. The contribution of the GFRP bars is equivalent to the section of steel bars by the area according to the elastic modulus ratio.

Furthermore, experimental results indicate that the distribution of the tensile strain of the steel bars and the compressive strain in the UHPC edge are not uniform. The height of the compressive zone varies continuously, causing the neutral axis to exhibit an undulating profile along the length direction of the beam. Therefore, average strain is employed to determine the average curvature and the average stiffness of the cross-section, as detailed below.

$$B_s = \frac{M}{\phi} \quad (33)$$

$$\phi = \frac{\varepsilon_{cm} + \varepsilon_{sm}}{h_0} \quad (34)$$

$$\varepsilon_{cm} = \frac{M}{\zeta b h_0^2 E_c} \quad (35)$$

$$\varepsilon_{sm} = \frac{\psi M}{(E_s A_s + E_f A_f) \eta_s h_0} \quad (36)$$

$$B_s = \frac{(E_s A_s + E_f A_f) h_0^2}{\frac{\psi}{\eta} + \frac{\alpha_{Es} \rho_s + \alpha_{Ef} \rho_f}{\zeta}} \quad (37)$$

$$\frac{\alpha_{Es} \rho_s + \alpha_{Ef} \rho_f}{\zeta} = 0.2 + 6(\alpha_{Es} \rho_s + \alpha_{Ef} \rho_f) \quad (38)$$

ε_{cm} is the average strain of UHPC at the edge of the compression zone, ε_{sm} is the average tensile strain of the longitudinal reinforcement. The parameter ζ represents a comprehensive average strain coefficient for the UHPC at the edge of the compression zone, which can be determined using an empirical formula similar to the standard, as given in Equation (38). H denotes the internal force lever arm coefficient. Generally, η is not significantly influenced by factors such as concrete type, cross-sectional dimensions or reinforcement configuration [36]. Therefore, a value of 0.87 is adopted for η . ψ is the non-uniformity coefficient of longitudinal tensile reinforcement strain between cracks.

6.2.2. Non-Uniformity Coefficient of Longitudinal Tensile Reinforcement Strain

Due to the existence of steel fiber in UHPC, the strain distribution of tensile steel bars is more uniform than that observed in ordinary concrete. However, since the enhancement effect of steel fiber has been considered by the factor $\beta_B \lambda_f$ in the previous analysis, their additional influence is disregarded hereafter. Therefore, the study of the coefficient ψ is simplified by considering it within ordinary concrete which has the same strength as UHPC.

Following the approach of Liu [37], ψ is calculated using Equation (39), where ρ_{te} represents the longitudinal tensile reinforcement ratio which is obtained from the effective tensile area. A_{te} is the effective tensile UHPC section area, which is assumed to be $0.5bh$ for rectangular sections. σ_{sk} represents the tensile reinforcement stress at the cracked section and is determined by Equation (41).

$$\psi = 1.12 - 0.38 \frac{f_t h}{\rho_{te} \sigma_{sq} h_0} \quad (39)$$

$$\rho_{te} = \frac{A_s + E_f A_f / E_s}{A_{te}} \quad (40)$$

$$\sigma_{sk} = \frac{M}{0.87 h_0 (A_s + A_f E_f / E_s)} \quad (41)$$

By substituting both the calculated stiffness of an equivalent-strength ordinary concrete B_s and the experimentally measured stiffness B_{sf} of the specimens into Equation (32), the value for β_B is determined to be 0.2 through a fitting process.

6.3. Calculation Formula of Deflection

According to mechanical theory, the deflection f of a simply supported beam under symmetric two-point loading can be calculated according to Equation (42), where the load at each point is $P/2$, L_0 denotes the effective span of the beam and a represents the length of the bending-shear span.

$$F = Pa \left(\frac{L_0^2}{8} - \frac{a^2}{6} \right) / B_s \quad (42)$$

By substituting the results from Equations (28) and (32) into Equation (42), the mid-span deflections of the specimens both before cracking and during the service stage are obtained. Figure 9 shows a comparison of predicted deflection f^p and measured deflection f^t . The mean ratio of f^p / f^t is 0.92, with a standard deviation of 0.08 and a coefficient of variation of 0.09. The results demonstrate that the flexural stiffness predicted by the

proposed formula closely aligns with the experimental data, confirming its applicability for practical engineering use.

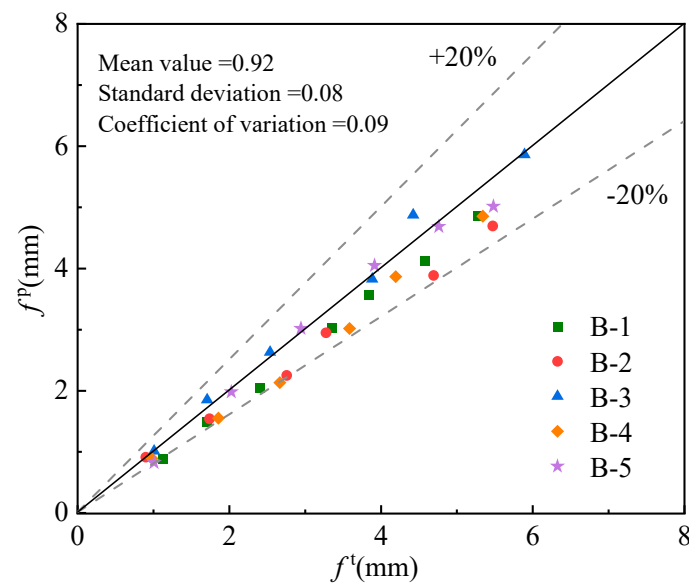


Figure 9. Comparison of f^P and f^t .

7. Calculation of the Crack

The addition of steel fiber to UHPC can effectively inhibit crack propagation after the initiation of cracking in hybrid reinforced beams. Therefore, the bridging effect of steel fiber at the crack should be adequately considered. In addition, the bonding performance between UHPC and GFRP bars differs from that of steel bars, and the interface characteristics significantly influence the formation and development of cracks. Consequently, further investigation is required to develop accurate methods for predicting average crack spacing and width in hybrid reinforced UHPC beams under applied loads.

7.1. Average Crack Spacing

During the service stage, the bond anchorage between steel bars and concrete is governed by both chemical cementation and mechanical interlock. Near the vicinity of cracks, mechanical interlock plays a more dominant role, whereas chemical adhesion is more significant in regions farther from the cracks. In the case of UHPC, which lacks coarse aggregates, the bond anchorage depends primarily on chemical cementation. However, fine aggregates such as fine sand and mineral powder effectively fill the grooves of the reinforcement ribs, densifying the bonding layer. As a result, the interfacial friction coefficient is not compromised by the absence of coarse aggregate. Therefore, the relative bonding characteristic coefficient v_i between steel bars and UHPC can be assumed equivalent to that of conventional concrete, as referenced in GB 50010-2010 [34]. The average crack spacing l_m of hybrid reinforced UHPC beams is calculated according to Equation (43).

$$L_m = \lambda_c 1.9c + 0.08 \frac{d_{eq}}{\rho_{te}} \quad (43)$$

$$d_{eq} = \frac{\sum n_i d_i^2}{\sum n_i v_i d_i} \quad (44)$$

The parameter c denotes the protective layer thickness for the tensile reinforcement; d_{eq} represents its equivalent diameter, determined from Equation (44), where n_i is the number of longitudinal tensile reinforcements; and v_i indicates the bond coefficient associated with

the longitudinal reinforcement. For ribbed and round steel bars, v_i is assigned values of 1.0 and 0.7, respectively. For GFRP bars, since the bond between UHPC and GFRP bars is good, v_i is taken as 0.9. The value of ρ_{te} is determined from Equation (40). Because UHPC exhibits both high strength and low porosity, its required protective layer thickness is reduced relative to that of conventional concrete [27]. Therefore, a protective layer thickness influence coefficient λ_c is introduced to account for the differences in protective layer thickness, calculated as $c/(c - 5)$.

Table 9 presents a comparison between the predicted average crack width l_m^p and the measured average crack width l_m^t for five specimens. The mean ratio l_m^p/l_m^t equals 1.07, with a mean square deviation of 0.062 and a coefficient of variation of 0.058. These results demonstrate that the calculation formula closely matches the experimental data and can accurately predict the average crack width of the beam.

Table 9. Comparison of l_m^p and l_m^t .

No.	l_m^p (mm)	l_m^t (mm)	l_m^p/l_m^t
B-1	68.68	67.99	1.01
B-2	69.01	67.31	1.03
B-3	69.35	65.48	1.06
B-4	70.12	60.31	1.16
B-5	70.77	64.26	1.10

7.2. Crack Width

The average crack width ω_m is defined as the difference between the mean elongation of the reinforcement within the cracked section and the mean elongation of the UHPC on the side surface at the same level. Based on this definition, Equation (45) is derived, where α_c denotes the influence coefficient of UHPC's own elongation between cracks on the crack width, which is taken as 0.85 [34]. ψ is determined from Equation (39). σ_{sk} is taken from Equation (41). When UHPC cracks, steel fibers bridging the crack connect the UHPC matrix on both sides, restraining further crack propagation [38]. Therefore, the steel fiber influence coefficient β_c is introduced to account for this crack-width reduction effect.

$$\omega_m = \beta_c \alpha_c \psi \frac{\sigma_{sk}}{E_s} l_m \quad (45)$$

Using the experimental data, the calculated values of β_c are illustrated in Figure 10. The mean value of β_c is 0.66. It is suggested that the value of β_c should be 0.7, which tends to be conservative.

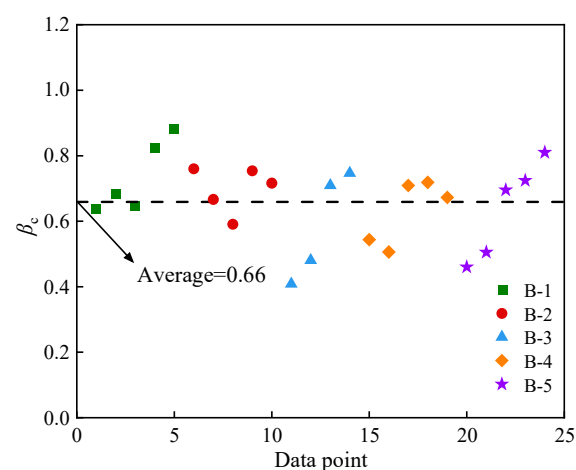


Figure 10. The value of coefficient β_c .

The maximum crack width $\omega_{\max}$ under short-term loading is calculated as the product of the average crack width and the crack width expansion coefficient τ .

$$\omega_{\max} = \tau \omega_m \quad (46)$$

It is worth noting that $\omega_{\max}$ does not represent the absolute maximum crack width, but rather a relative maximum value corresponding to a 95% confidence level [36]. Therefore, a probabilistic distribution approach is used to determine the value of τ . Statistical analysis indicates that, for the five specimens under various load levels, the ratios of individual crack widths to the average crack width approximately follow a normal distribution. At a 95% confidence level, τ is determined to be 1.4. Therefore, it is suggested that the crack width expansion coefficient of the hybrid reinforced UHPC beam is 1.4.

As shown in Figure 11, the maximum crack widths of five specimens under short-term loading are compared. The average value of $\omega_{\max}^p / \omega_{\max}^t$ is 1.01, accompanied by a standard deviation of 0.20 and a coefficient of variation of 0.19. The results confirm that the calculated crack widths correspond closely to the measured values, supporting the reliability of the proposed prediction method.

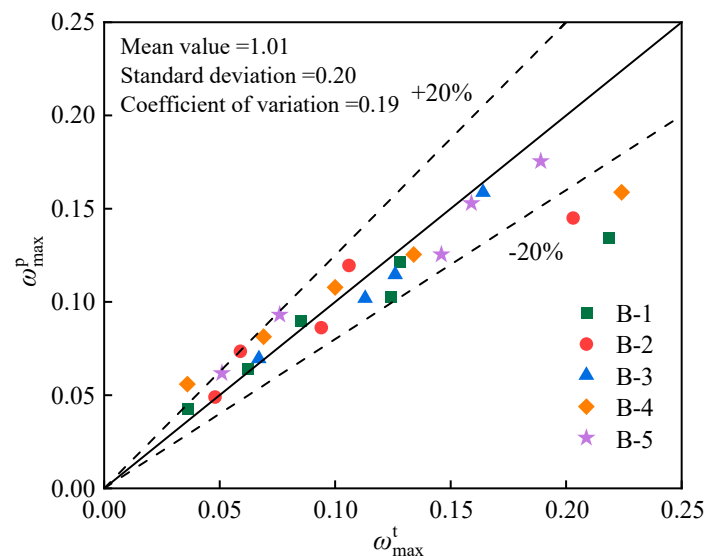


Figure 11. Comparison of $\omega_{\max}^p$ and $\omega_{\max}^t$.

8. Conclusions

Four-point bending tests were performed on UHPC beams reinforced with hybrid GFRP and steel bars. The effect of the reinforcement ratios of steel bars and GFRP bars on the flexural performance of the specimens was investigated. The formulas for calculating the cracking moment, load-bearing capacity, deformation capacity and crack width were proposed. Drawing on both experimental findings and analytical investigations, the following conclusions are obtained:

- (1) All specimens experienced typical flexural failure: tensile bars were yielded followed by UHPC crushing. Throughout the loading, GFRP bars remained well bonded to the UHPC with no significant slip observed. The high stiffness and ductility of UHPC effectively compensated for the large deflections and poor ductility of GFRP-reinforced beams.
- (2) Increasing steel reinforcement notably enhanced stiffness and ultimate load capacity. In contrast, increasing the GFRP reinforcement ratio had little influence on stiffness but contributed more significantly to improving ultimate load capacity than to an equivalent increase in steel reinforcement.

(3) The strain distributions in both GFRP and steel bars were generally consistent, confirming cooperative action between the two materials. A flexural capacity formula accounting for the tensile effect of UHPC was derived. The predicted outcomes showed close agreement with experimental measurements, validating the predictive capability of the model.

(4) By converting the GFRP area into an equivalent steel section based on the elastic modulus ratio and incorporating the effect of steel fibers, a stiffness calculation formula was established. The deflection predicted by this formula showed close agreement with the experimental results.

(5) Steel fibers effectively bridged cracks, restricting crack propagation. Based on this mechanism, the calculation methods for average crack spacing and crack width in current concrete standards were modified. The outcomes derived from the modified method exhibited close agreement with the experimental data, thereby confirming the validity of the proposed calculation approach.

9. Discussion

This study investigated the flexural behavior of UHPC beams reinforced with hybrid GFRP and steel bars through bending tests on five specimens and proposed analytical models for predicting their flexural capacity and serviceability performance. Nevertheless, several limitations should be acknowledged.

First, the experimental program was limited to five laboratory-scale beam specimens. Although the results were sufficient to reveal the fundamental flexural behavior and performance trends of hybrid reinforced UHPC beams, the limited number of specimens restricted the statistical robustness of the conclusions, and potential size effects associated with full-scale members were not addressed.

Second, the present study was primarily based on experimental observations and analytical formulations. Finite element analysis was not conducted to further investigate the internal stress–strain distribution, crack evolution and parameter sensitivity of hybrid reinforced UHPC beams. The incorporation of numerical simulations would allow a more comprehensive assessment of the proposed analytical models over a wider parameter range.

In addition, only GFRP bars were considered in this study. Since FRP bars with higher elastic modulus (e.g., CFRP bars) may significantly influence stiffness, deflection and crack development, the conclusions of this study are mainly applicable to UHPC beams reinforced with hybrid GFRP and steel bars, and their extension to other FRP types requires further validation.

Future research will focus on conducting additional tests with a larger number of specimens, including full-scale UHPC beams, developing finite element models to systematically study key parameters and investigating the flexural behavior of UHPC beams reinforced with different types of FRP bars combined with steel bars.

Author Contributions: Conceptualization, K.W.; Methodology, K.W.; Software, T.S., R.W. and C.D.; Validation, T.S., K.H. and R.W.; Formal analysis, C.D.; Investigation, T.S., R.W. and C.D.; Resources, K.H.; Data curation, K.H., R.W. and C.D.; Writing—original draft, T.S.; Writing—review & editing, K.W.; Visualization, T.S., K.H., R.W. and C.D.; Supervision, K.W.; Project administration, K.H.; Funding acquisition, K.W. All authors have read and agreed to the published version of the manuscript.

Funding: The authors gratefully acknowledge the support provided by Yangzhou University graduate research and practice innovation program (KYCX25-3984) and Science and Technology Planning Project of Yangzhou City (YZ2024223).

Data Availability Statement: The original contributions presented in this study are included in the article. Further inquiries can be directed to the corresponding authors.

Conflicts of Interest: Authors Tianyu Shi and Kangsheng Huang were employed by The Third Engineering Co., Ltd. of CCCC Fourth Harbor Engineering Co., Ltd. Author Ruochen Wang was employed by the company Jiangsu Testing Center for Quality of Construction Engineering Co., Ltd. The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

References

- Rodrigues, R.; Gaboreau, S.; Gance, J.; Ignatiadis, I.; Betelu, S. Reinforced Concrete Structures: A Review of Corrosion Mechanisms and Advances in Electrical Methods for Corrosion Monitoring. *Constr. Build. Mater.* **2021**, *269*, 121240. [\[CrossRef\]](#)
- Robles, K.P.V.; Gucunski, N.; Kee, S.-H. Evaluation of Steel Corrosion-Induced Concrete Damage Using Electrical Resistivity Measurements. *Constr. Build. Mater.* **2024**, *411*, 134512. [\[CrossRef\]](#)
- Zheng, Y.; Zheng, S.-S.; Yang, L.; Dong, L.-G.; Ruan, S. Experimental Study and Numerical Model of the Seismic Behavior of Reinforced Concrete Beams in an Artificial Corrosion Environment. *J. Build. Eng.* **2022**, *46*, 103705. [\[CrossRef\]](#)
- Van Steen, C.; Van Beirendonck, T.; Vrijdaghs, R.; Hendriks, M.A.; Verstrynge, E. A Two-Phased Modelling Approach for Corrosion-Induced Concrete Cracking and Bond Deterioration in Reinforced Concrete. *Eng. Struct.* **2023**, *294*, 116624. [\[CrossRef\]](#)
- Escórcio, P.; França, P.M. Experimental Study of a Rehabilitation Solution That Uses GFRP Bars to Replace the Steel Bars of Reinforced Concrete Beams. *Eng. Struct.* **2016**, *128*, 166–183. [\[CrossRef\]](#)
- Ali, A.H.; Mohamed, H.M.; Benmokrane, B.; ElSafty, A.; Chaallal, O. Durability Performance and Long-Term Prediction Models of Sand-Coated Basalt FRP Bars. *Compos. Part B* **2019**, *157*, 248–258. [\[CrossRef\]](#)
- Mias, C.; Torres, L.; Turon, A.; Barris, C. Experimental Study of Immediate and Time-Dependent Deflections of GFRP Reinforced Concrete Beams. *Compos. Struct.* **2013**, *96*, 279–285. [\[CrossRef\]](#)
- Tran, D.T.; Pham, T.M.; Hao, H.; San Ha, N.; Vo, N.H.; Chen, W. Precast Segmental Beams Made of Fibre-Reinforced Geopolymer Concrete and FRP Tendons against Impact Loads. *Eng. Struct.* **2023**, *295*, 116862. [\[CrossRef\]](#)
- Rosa, I.C.; Firmo, J.P.; Correia, J.R.; Mazzuca, P. Influence of elevated temperatures on the bond behaviour of GFRP bars to concrete—Pull-out tests. In *IABSE Symposium, Guimaraes 2019: Towards a Resilient Built Environment Risk and Asset Management—Report*; Universidade de Lisboa: Lisboa, Portugal, 2019; pp. 861–868.
- Xue, W.C.; Zheng, Q.W.; Yang, Y. Design Recommendations on Flexural Capacity of FRP Reinforced Concrete Beams. *Eng. Mech.* **2009**, *26*, 79–85.
- Ge, W.; Zhang, J.; Cao, D.; Tu, Y. Flexural Behaviors of Hybrid Concrete Beams Reinforced with BFRP Bars and Steel Bars. *Constr. Build. Mater.* **2015**, *87*, 28–37. [\[CrossRef\]](#)
- El Refai, A.; Abed, F.; Al-Rahmani, A. Structural Performance and Serviceability of Concrete Beams Reinforced with Hybrid (GFRP and Steel) Bars. *Constr. Build. Mater.* **2015**, *96*, 518–529. [\[CrossRef\]](#)
- Maranan, G.B.; Manalo, A.C.; Benmokrane, B.; Karunasena, W.; Mendis, P.; Nguyen, T.Q. Flexural Behavior of Geopolymer-Concrete Beams Longitudinally Reinforced with GFRP and Steel Hybrid Reinforcements. *Eng. Struct.* **2019**, *182*, 141–152. [\[CrossRef\]](#)
- Liang, C.; Cai, Z.; Wu, H.; Xiao, J.; Zhang, Y.; Ma, Z. Chloride Transport and Induced Steel Corrosion in Recycled Aggregate Concrete: A Review. *Constr. Build. Mater.* **2021**, *282*, 122547. [\[CrossRef\]](#)
- Li, C.; Li, K. Chloride Ion Transport in Cover Concrete under Drying-Wetting Cycles: Theory, Experiment and Modeling. *Kuei Suan Jen Hsueh Pao* **2010**, *38*, 581–589.
- Jones, C.; Suraneni, P.; Hale, W.M. Investigating Concrete Deterioration Due to Calcium Oxychloride Formation. *Constr. Build. Mater.* **2022**, *320*, 125600. [\[CrossRef\]](#)
- Du, J.; Meng, W.; Khayat, K.H.; Bao, Y.; Guo, P.; Lyu, Z.; Abu-Obeidah, A.; Nassif, H.; Wang, H. New Development of Ultra-High-Performance Concrete (UHPC). *Compos. Part B* **2021**, *224*, 109220. [\[CrossRef\]](#)
- Yu, R.; Zhang, X.; Hu, Y.; Li, J.; Zhou, F.; Liu, K.; Zhang, J.; Wang, J.; Shui, Z. Development of a Rapid Hardening Ultra-High Performance Concrete (R-UHPC): From Macro Properties to Micro Structure. *Constr. Build. Mater.* **2022**, *329*, 127188. [\[CrossRef\]](#)
- Zhao, S.; Liu, R.; Liu, J.; Yang, L. Comparative Study on the Effect of Steel and Plastic Synthetic Fibers on the Dynamic Compression Properties and Microstructure of Ultra-High-Performance Concrete (UHPC). *Compos. Struct.* **2023**, *324*, 117570. [\[CrossRef\]](#)
- Pyo, S.; Kim, H.-K. Fresh and Hardened Properties of Ultra-High Performance Concrete Incorporating Coal Bottom Ash and Slag Powder. *Constr. Build. Mater.* **2017**, *131*, 459–466. [\[CrossRef\]](#)
- Pyo, S.; Tafesse, M.; Kim, H.; Kim, H.-K. Effect of Chloride Content on Mechanical Properties of Ultra High Performance Concrete. *Cem. Concr. Compos.* **2017**, *84*, 175–187. [\[CrossRef\]](#)

22. Ghafari, E.; Arezoumandi, M.; Costa, H.; Júlio, E. Influence of Nano-Silica Addition on Durability of UHPC. *Constr. Build. Mater.* **2015**, *94*, 181–188. [[CrossRef](#)]
23. Pyo, S.; Koh, T.; Tafesse, M.; Kim, H.-K. Chloride-Induced Corrosion of Steel Fiber near the Surface of Ultra-High Performance Concrete and Its Effect on Flexural Behavior with Various Thickness. *Constr. Build. Mater.* **2019**, *224*, 206–213. [[CrossRef](#)]
24. Hasgul, U.; Turker, K.; Birol, T.; Yavas, A. Flexural Behavior of Ultra-High-Performance Fiber Reinforced Concrete Beams with Low and High Reinforcement Ratios. *Struct. Concr.* **2018**, *19*, 1577–1590. [[CrossRef](#)]
25. Feng, Z.; Li, C.; Yoo, D.-Y.; Pan, R.; He, J.; Ke, L. Flexural and Cracking Behaviors of Reinforced UHPC Beams with Various Reinforcement Ratios and Fiber Contents. *Eng. Struct.* **2021**, *248*, 113266. [[CrossRef](#)]
26. Cao, X.; Ren, Y.-C.; Qian, K.; Fu, F.; Deng, X.-F.; Zhang, W.-J. Size Effect on Flexural Behavior of Ultra-High-Performance Concrete Beams with Different Reinforcement. In *Proceedings of the Structures*; Elsevier: Amsterdam, The Netherlands, 2022; Volume 41, pp. 969–981.
27. DBJ43/T 325-2017; Technical Specification for Reactive Powder Concrete Structures. China Architecture and Building Press: Beijing, China, 2017.
28. GB/T 228.1-2021; Metallic Materials-Tensile Testing-Part 1: Method of Text at Room Temperature. China Architecture and Building Press: Beijing, China, 2021.
29. ISO 10406-1:2015; Fibre-Reinforced Polymer (FRP) Reinforcement of Concrete—Test Methods—Part 1: FRP Bars and Grids. ISO: Geneva, Switzerland, 2015.
30. Ouyang, X.; Shi, C.J.; Shi, J.H.; Shan, B.; Wu, Z.M.; Huang, Y.; Xing, Y. Compressive Mechanical Properties and Prediction for Elastic Modulus of Ultra-High Performance Concrete. *J. Chin. Ceram. Soc.* **2021**, *49*, 296–304.
31. Lai, B.; Liew, J.R.; Xiong, M. Experimental Study on High Strength Concrete Encased Steel Composite Short Columns. *Constr. Build. Mater.* **2019**, *228*, 116640. [[CrossRef](#)]
32. Zheng, W.; Lu, S.; Li, L. Experimental Research on Mechanical Performance of Reactive Powder Concrete Beams Reinforced with GFRP Bars. *J. Build. Struct.* **2011**, *32*, 115–124. (In Chinese)
33. Wenzhong, Z.; Li, L.I.; Shanshan, L.U. Experimental Research on Mechanical Performance of Normal Section of Reinforced Reactive Powder Concrete Beam. *J. Build. Struct.* **2011**, *32*, 125. (In Chinese)
34. GB 50010-2010; Code for Design of Concrete Structures. China Architecture and Building Press: Beijing, China, 2010.
35. JGJT 465-2019; Standard for Design of Steel Fiber Reinforced Concrete Structures. China Architecture and Building Press: Beijing, China, 2019.
36. Jin, W.; Lu, C.; Wang, H.; Yan, Y. Experiment and Calculation of Crack Width of Reinforced Concrete Beams with 500 MPa Steel Bars. *China Civ. Eng. J.* **2011**, *44*, 16–23.
37. Liu, S.; Wang, X.; Wu, Z. Calculation Method of Short-Term Stiffness for Flexural Hybrid-RC Members with FRP and Steel Bars. *Eng. Mech.* **2023**, *41*, 1–11. (In Chinese)
38. Liu, X.; Ding, Y. Experimental Research on Crack Width of Steel Fibers Reinforced Self-Consolidating Concrete Beams. *Eng. Mech.* **2017**, *34*, 116–124. [[CrossRef](#)]

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.