

Article

Flexural Performance of Carbon Fiber-Reinforced Polymer Prestressed Spun High-Strength Concrete Pile

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Abstract: Prestressed spun high-strength concrete (PHC) piles are commonly used in various types of structures, including bridges, buildings and marine infrastructures. However, piles installed in aggressive environments are vulnerable to corrosion of the steel, which can lead to rapid degradation of the piles. As a corrosion-resistant material, carbon fiber-reinforced polymer (CFRP) is considered an alternative to steel tendons for durability enhancement. In this study, a new pile system with CFRP was proposed. Experimental tests of three full-scale piles and a numerical analysis of eight piles with various parameters were performed to investigate the flexural performance of CFRP prestressed spun high-strength concrete pile. The proposed piles were loaded under four-point bending after prestressing. The experimental and numerical results verified the feasibility of the proposed system, and the CFRP pile exhibited twice of flexural capacity of that of steel-reinforced piles. The flexural performance of the CFRP PHC pile was significantly affected by the reinforcement ratio, prestressing level and modulus of the CFRP. An analytical approach predicting the flexural capacity of the CFRP PHC pile was proposed based on the parametric study. Ninety percent accuracy was achieved for the proposed analytical approach. The presented study can significantly promote the application of CFRP in pile foundations and improve the durability of PHC piles.

Keywords: CFRP; PHC piles; experimental tests; FE analysis; analytical approach



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1. Introduction

Prestressed spun high-strength concrete (PHC) piles are extensively utilized in foundations worldwide due to their notable advantages such as high axial capacity, standardized manufacturing processes, assurance of quality, short on-site construction period and low permeability [1,2]. These piles not only carry vertical loads from superstructures but also carry lateral forces induced by wind, waves and earthquakes [3]. The high strength steel is prestressed to resist the bending moment exerted by lateral loads. The strength of steel used in PHC piles varies in the range of 1420 MPa–1860 MPa, and the concrete grade ranges from 60 MPa to 100 MPa [3]. Despite the above benefits, the application of PHC piles has encountered various challenges over time. Brittle fracture of the high-strength steel limits the flexural capacity of PHC piles, which has led to a series of accidents. During the Hyougoken–Nanbu Earthquake [4] in 1995 and the Tohoku Pacific Earthquake [5] in 2011, brittle failure on PHC pile heads was observed. In addition, a 13-story building collapsed in Shanghai due to the lack of flexural capacity of the PHC piles [6].

Many attempts have been made to improve the flexural resistance of PHC piles. Many studies on the effect of additional non-prestressed steel have been conducted. Ren et al. [7] found that the flexural performance of PHC plies was significantly improved with a combination of steel strands and additional deformed bars. The flexural and seismic performance of PHC piles with additional deformed bars was also investigated by Yang et al. [2,8] and Zhang et al. [9]. Akiyama et al. [10] proposed a PHC pile system with an un-bonded strand at the center of the pile to reinforce the integrity of the pile. Confinement

of the PHC piles is considered an effective way to improve their flexural capacity and ductility. Muguruma et al. attempted to improve the ductility of PHC piles by confining the pile with high-strength steel hoops [11]. Thusoo et al. [12] investigated the effect of steel jackets on the flexural strengthening of PHC piles. In addition, PHC piles jacketed by various of FRPs were tested and a significant improvement in flexural performance was observed [13–15]. The other alternative for flexural improvement is infilling concrete. Irawan et al. [16] and Bung et al. [17] tested PHC piles with infilled concrete in the core, and the result showed that the flexural ductility was significantly improved.

Although flexural performance could be improved by various methods, corrosion of the reinforcing steel is still a problem limiting the application of PHC piles, especially for marine structures. Corroded piles were reported on many bridges in harsh environments, which resulted in huge maintenance costs [18–20]. In order to avoid the occurrence of corrosion, the non-corrosion material FRP was tested as a substitute for steel reinforcement. Roddenberry et al. [21] tested five square piles prestressed by CFRP strands, and the development length of the CFRP strands and the driven performance were investigated. Belarbi et al. [19] and Sharp [20] investigated the life-cycle cost of CFRP piles. Compared with steel-reinforced piles, the increased initial cost could be recouped within twenty years and there would be a major saving after twenty years.

Spun PHC pile is considered to have a denser micro-structure and lower permeability compared to vibrated concrete; however, there is still a high risk of corrosion due to cracks exerted during its service life [22–25]. Other than ordinary steel, there are various materials used as reinforcement in concrete structures, such as FRP and shape memory alloys [26–28]. Since CFRP has been proven to be effective in the flexural reinforcing of concrete components [29–36], there is a great possibility that CFRP can significantly improve the flexural performance of spun PHC piles and eliminate the risk of corrosion. However, there is very limited research on flexural performance of spun PHC with CFRP.

This paper presents a comprehensive investigation on flexural performance of spun PHC piles prestressed by CFRP rods including experimental tests of 3 full-scaled specimen and a numerical parametric study. The numerical model was verified against the experimental results, and parameters including the prestressing level, reinforcement ratio and CFRP modulus were investigated. In addition, a simplified design formula was proposed based on the parametric study. The presented work can significantly promote the application of prestressed CFRP and help improve the durability of pile foundations.

2. Experimental Tests

In total, three full-scaled spun PHC piles were fabricated and tested under four-point bending. The experimental test verified the feasibility of the fabrication procedure and evaluated the flexural performance of the piles in terms of capacity and ductility. A comparison between CFRP PHC and ordinary steel-reinforced PHC was conducted.

2.1. Materials and Methods

2.1.1. General Configuration Description

Ten-meter long full-scaled spun PHC piles were designed as shown in Figures 1 and 2. A commonly used ring cross-section with a 400 mm outer diameter and 95 mm thickness were adopted for all the piles in this study. The detailing of the pile is specified in Chinese code GB 13476-2023, Pretensioned Spun Concrete Piles [36]. The reference pile specimen (PREF) was designed with seven 9 mm high-strength steel prestressing rods and 4 mm steel spirals along the pile. The concrete cover was 37.5 mm. The longitudinal steel rods were pretensioned with a prestressing level of 994 MPa, which was 70% of the ultimate strength of the steel. This prestressing level is specified in Chinese code GB 13476-2023, Pretensioned Spun Concrete Piles [36]. Meanwhile, for the CFRP piles, seven 9 mm CFRP rods were evenly distributed in the concrete ring, and 3.4 mm glass fiber reinforced polymer (GFRP) spirals were arranged along the pile. The steel plates were used at both ends for pretensioning. The concrete cover was 38 mm. Two different prestressing levels was

designed for the CFRP piles. The specimen with 1000 MPa prestressed CFRP was named “PFRP1000”, while the specimen with 1500 MPa prestressed CFRP was named “PFRP1500”. The ratios between the prestressing level and the ultimate strength were 34.5% and 51.8%, respectively. The 1000 MPa prestressing level was used for comparison with steel and 1500 MPa was close to 55% of the ultimate strength, which is the suggested sustained load limitation in ACI 440.1R-15, Guide for the Design and Construction of Structural Concrete Reinforced with Fiber-Reinforced Polymer (FRP) Bars [37].

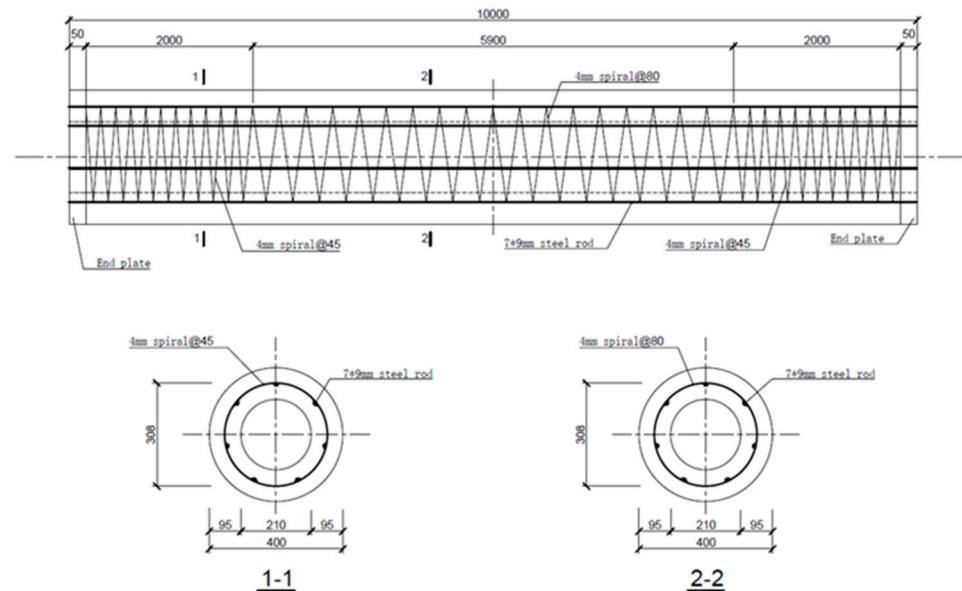


Figure 1. Spun PHC pile with steel reinforcement.

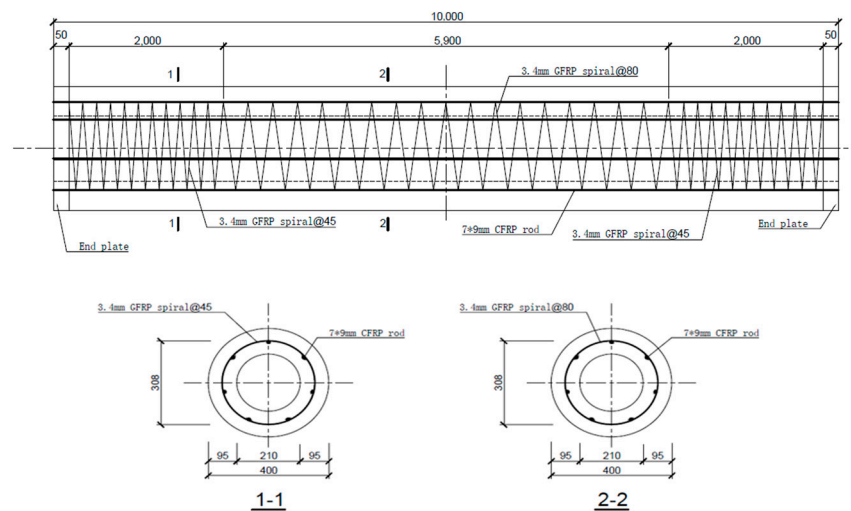


Figure 2. Spun PHC pile with FRP reinforcement.

2.1.2. Material Properties

Commercially available FRP materials manufactured by Zhongfu Carbon Fiber Core Cable Technology Co., Ltd. Lianyungang, China, were used in this research. The properties of FRPs and steel are summarized in Table 1. The properties of steel and GFRP were provided by the manufacturer, and the properties of CFRP were tested by the authors. The steel materials were standard products conforming to the Chinese code GB 13476-2023, Pretensioned Spun Concrete Piles [36]. The FRP materials were tested according to Chinese code GB50608-2020, Technical standard for fiber-reinforced polymer (FRP) in construction [38]. Five samples were tested for each type of FRP. The average strength of

CFRP was 2892 MPa with a standard deviation of 27 MPa, while the average strength of GFRP was 917 MPa with a standard deviation of 29 MPa.

Table 1. Material property of steel and FRPs.

	Steel Rod	Steel Spiral	CFRP	GFRP	End Plate
Tensile Strength (MPa)	1420	500	2892	917	375
Yield Strength (MPa)	785	235	/	/	235
Tensile Modulus (GPa)	210	210	181	52	210
Ultimate Strain (%)	7	23	1.6	1.8	28

We adopted 80 MPa high-strength concrete provided by Zhejiang Feiying Concrete Co., Ltd. (Yongkang, China) in this research. However, the field test showed that the concrete strength reached 90 MPa on the day of the tests. The tests were conducted according to Chinese code GB 13476-2023, Pretensioned Spun Concrete Piles [36]. Ten cubic samples were tested, and the average compressive strength was 90 MPa with a standard deviation of 1.1 MPa.

2.1.3. Fabrication

The fabrication of the FRP reinforced piles including five steps, as shown in Figure 3. The fabrication processes are described below:

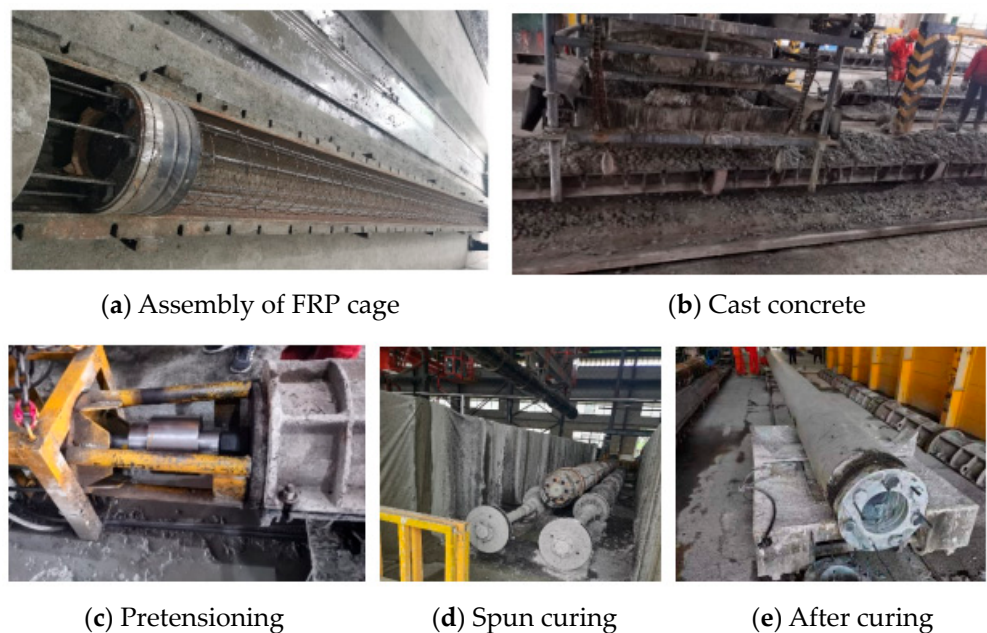


Figure 3. Fabrication process.

(a) Assembly of FRP cage: The CFRP rods were fixed in a metal anchor, which was manufactured by Zhongfu Carbon Fiber Core Cable Technology Co., Ltd. The longitudinal CFRP rod anchor was fixed in the end plate. Then, the GFRPs were spiraled on the longitudinal CFRP with stainless steel wires. The details of the CFRP anchor and the end plates are shown in Figure 4. A reusable anchor and a permanent one were used and fixed in the end plates. The reusable anchor could be removed after pretensioning, and the nut was tightened.

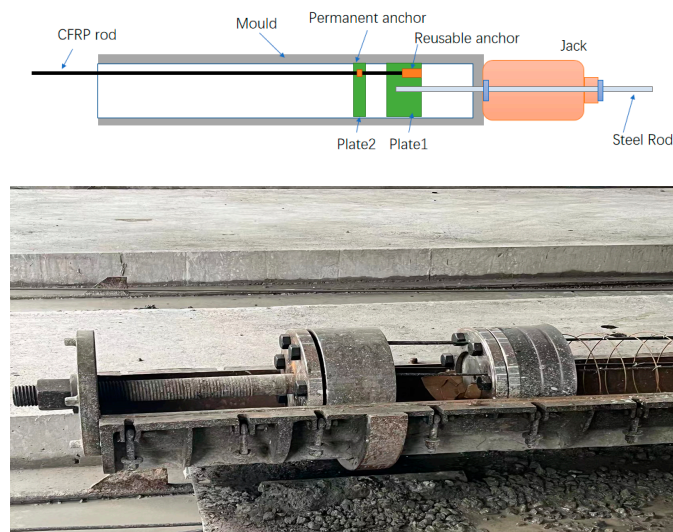


Figure 4. Prestressing and anchor devices.

(b) Concrete pouring: The assembled FRP cage with an anchor plate was placed in the half-circular mold and concrete was poured in the mold by an auto-pouring machine.

(c) Pretensioning: The end plate was connected with a hydraulic jack using a high-strength threaded bar. The CFRP rods were pretensioned through the end plate right after the concrete was poured.

(d) Spinning: The other half the mold was assembled with a high-strength bolt. The specimen was placed on a spinning machine and spun for 15 min.

(e) Curing: After being cured under a temperature of 85–90 °C for 6 h, the mold was removed and the piles were cured under a high temperature and a pressure of 1 MPa.

2.1.4. Test Setup and Instrumentation

The four-point bending test was performed. The load span was 6 m and the two loading points were located at 500 mm from center of the specimen. The theoretical cracking moment and ultimate moment were calculated according to Chinese code GB 13467-2023, Pretensioned Spun Concrete Piles [36]. The theoretical cracking moment was the minimal moment which would cause the bottom concrete to reach cracking strain. The theoretical ultimate moment was the minimal moment which would cause the top concrete to reach crushing strain. Load control with a loading rate of 3 kN/min was adapted in the test and the load was applied as follows:

(a) Preloading and instrument check: The preload in this test was controlled within 60% of the cracking moment. Three steps were applied for preloading, and each load level was kept for 1 min, followed by incremental unloading.

(b) Load to cracking: The load was applied incrementally from zero to 80% of the cracking load with increments of 20% of the cracking load. Each load level was kept for 3 min. If no cracks appeared at 80% of the cracking load, the amount of load increment was lowered to 10% of the cracking load until 100% cracking load was reached. Each load level was maintained for 3 min at this stage. If cracks did not appear at 100% of the cracking load, the specimen was continually loaded in increments of 5% of the cracking load until cracks appeared. The appearance of cracks was monitored, measured and recorded.

(c) Load to failure: The specimen was loaded with an increment of 5% of its theoretical ultimate capacity until failure. It was considered failure when the load dropped below 80% of the highest load or the crack width reached 1.5 mm, which is the limitation specified in GB 13476-2023, Pretensioned Spun Concrete Piles [36].

(d) Unloading: The specimen was unloaded gradually.

As shown in Figure 5, a total of 14 strain gauges were distributed on the concrete surface to monitor the strain distribution of specimen. Unfortunately, the strain gauges on

the concrete surface failed at a very early stage; therefore, the strain data are not presented in this paper. Linear Variable Displacement Transducers (LVDTs) were used to monitor the deformation of the tested specimen. Optical Fiber Bragg Grating (OFBG) sensors were installed on the CFRP rods; however, all the sensors died during the spun and high-temperature high-pressure curing process. In addition, a digital cracking width measuring device was used to accurately record the width of the cracks.

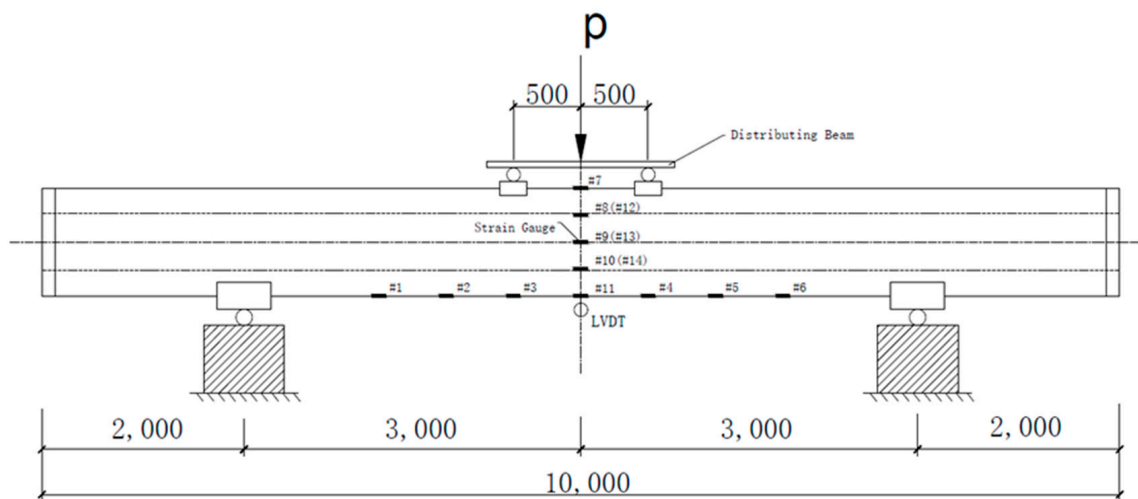


Figure 5. Test setup and instrumentation.

2.2. Results and Discussion

2.2.1. Results

Cracking

The reference pile (PREF) was longitudinally reinforced with 1420 steel rods and spiraled with Q235 steel. The first four cracks were observed on the bottom of the mid-span, at a load of 27.6 kN. As the load increased to 39.2 kN, a few new cracks appeared on the specimen. As the load increased to 42.1 kN, the cracking propagated to the flexural-shear zone. The pile failed at a load of 71 kN with a loud sound. The cracking of the reference pile before and after failure is shown in Figure 6.

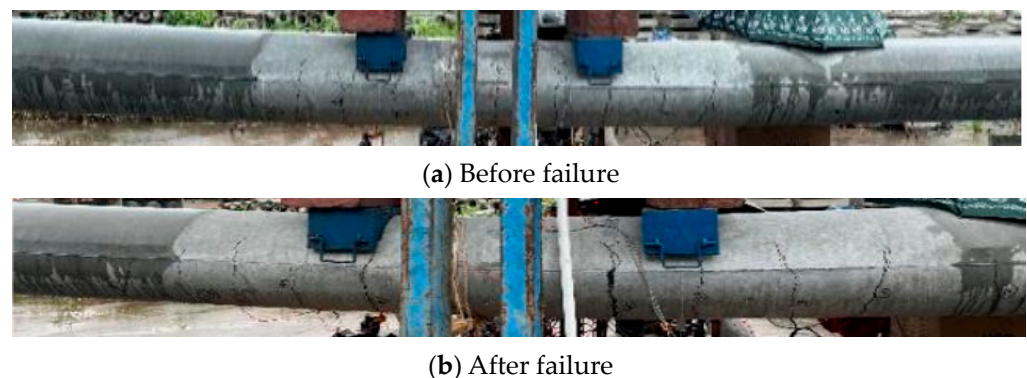


Figure 6. Cracks before and after failure of the reference pile.

For specimen PCFRP1000, which was longitudinally reinforced with CFRP rods with a 1000 MPa prestressing level, the first crack was observed at a load of 57 kN. The first crack was located at 100 mm from the mid-span. As the load increased to 62 kN, a second crack was observed symmetrical to the first one. As the load increased to 71 kN, one more crack was observed near the loading point. The concrete started to spall off at a load of 140 kN,

and a loss of capacity occurred at a load of 145 kN. The cracking of specimen PCFRP1000 before and after failure is shown in Figure 7.



Figure 7. Cracking before and after failure of PCFRP1000.

Specimen PCFRP1500 started to crack at a load of 86.5 kN. The first crack was located at 110 mm from the mid-span. As the load increased to 90.8 kN, the second crack was observed on the other side of the pile, at a location of 300 mm from the mid-span. As the load increased to 107.7 kN, cracks were observed outside the range of the distributing beam. The concrete started to crush at a load of 174 kN, and the pile failed at a load of 180 kN. The cracking of specimen PCFRP1500 before and after failure is shown in Figure 8.

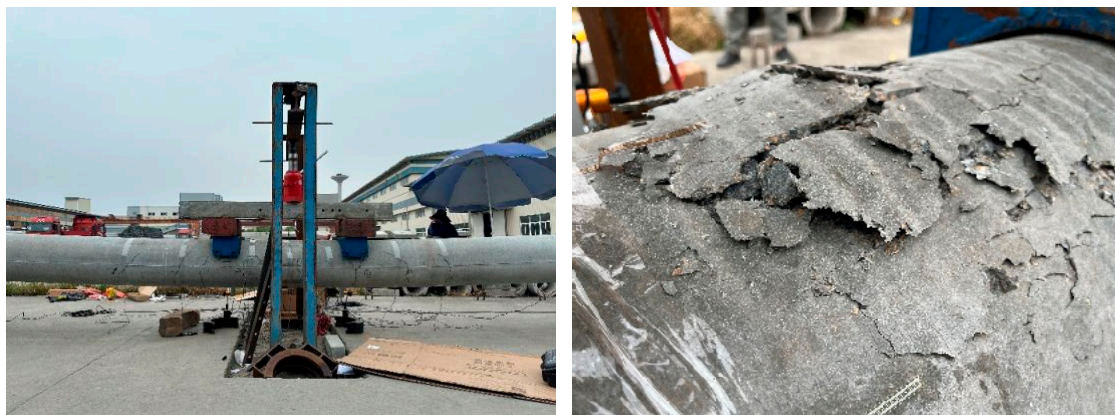


Figure 8. Crack before and after failure of PCFRP1500.

Ultimate Capacity and Failure Mode

The cracking load, ultimate load, deformation at failure and failure mode of the tested specimen are summarized in Table 2. The flexural capacity was calculated using Equation (1), which took the self-weight into consideration. Equation (1) is specified in Chinese code GB 13467-2023, Pretensioned Spun Concrete Piles [36]. The flexural capacity was the bending moment of the mid-span section at failure. As the steel reinforcements were replaced by CFRP rods, significant improvement was observed in terms of the cracking load and the ultimate load. In addition, the failure mode switched from rupture of the longitudinal reinforcement to crushing of the concrete.

$$M = \frac{P}{4} \left(\frac{3L}{5} - 2a \right) + \frac{1}{40} WL \quad (1)$$

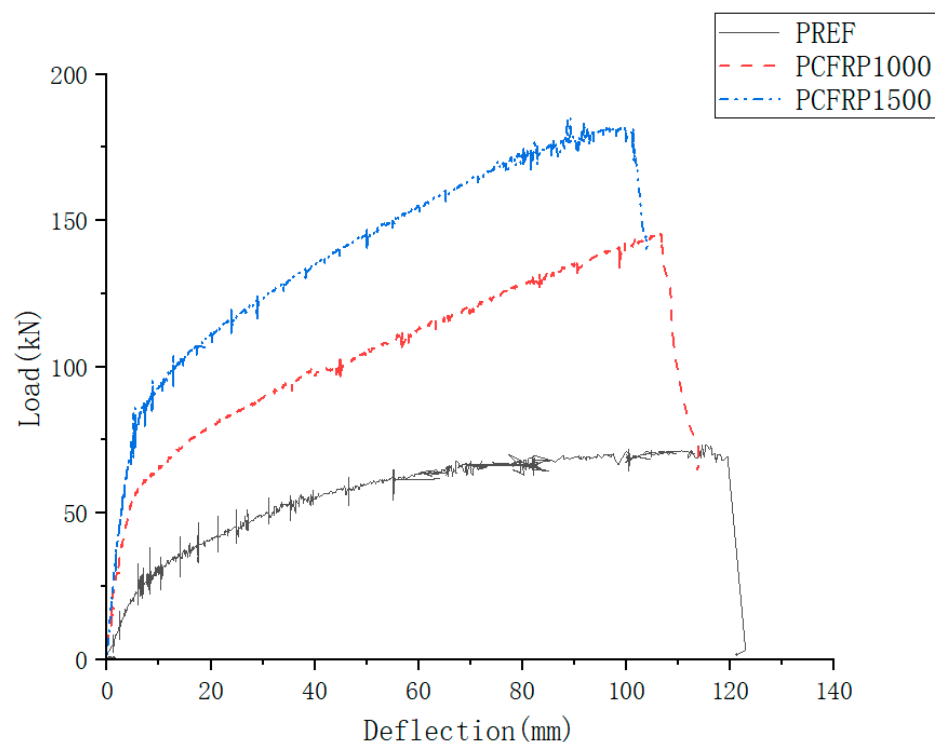
where M is the flexural capacity, P is the applied load, L is the length of the pile, W is the weight of the pile and a is the load span.

Table 2. Characteristic loads and failure mode.

Specimen	Cracking Load (kN)	Ultimate Load (kN)	Deformation at Failure (mm)	Ultimate Flexural Capacity (kN·m)	Failure Mode
PREF	27.6	71	120	95.3	Steel rupture
PCFRP1000	57.0	145	107	188.4	Concrete crushing
PCFRP1500	86.5	180	101	233.4	Concrete crushing

Load–Deflection Response

A load–deflection diagram of the tested piles is shown in Figure 9. It can be seen that the stiffness was significantly improved by replacing the steel reinforcement with CFRP rods, especially after the yielding point. It should be noted that even though the CFRP is an elastic material, the yield of concrete still resulted in a significant stiffness reduction. Before the yielding point, the cracking of concrete led to a slight stiffness reduction as well.

**Figure 9.** Load–deflection diagram.

The three specimens exhibited similar deformation at failure. However, the pile with a higher prestressing level exhibited the highest capacity and smallest deformation at failure. All the specimens experienced a sudden drop in capacity at the final stage, regardless of whether the failure was caused by concrete crushing or rupturing of the reinforcement.

The CFRP improved the stiffness of the pile, especially at a later stage. This was because the steel yielded soon after loading, and no more tensile resistance could be provided by the steel. However, the CFRP continued providing tensile resistance until failure. In addition, the pile with higher prestressing exhibited higher stiffness.

2.2.2. Discussion

Effect of Reinforcing Material

The reference pile (PREF) and the CFRP pile (PCFRP1000) had the same reinforcement ratio and initial prestressing level. However, the CFRP had a much higher tensile strength than the 1420 MPa steel, which resulted in a different failure mode and much higher ultimate flexural capacity. As the load increased, the tensile stress on the bottom

reinforcement increased. For the PREF specimen, as the tensile stress on the steel reached 1420 MPa, very limited strength could be further developed. However, for the specimen PCFRP1000, the CFRP continuously provided increasing tensile resistance until crushing of the top concrete, which indicated that the compressive strength of the concrete was fully developed. There is a still high possibility that the flexural capacity could be further improved if higher-strength concrete was adopted, as the strength of the CFRP was not fully developed. It can be concluded, for PHC piles, that high-strength reinforcing material tends to make more use of the concrete strength and significantly improve the flexural capacity.

Effect of Prestressing Level

The pile PCFRP1000 and the pile PCFRP1500 were reinforced with the same CFRP material, and had the reinforcement ratio. The CFRP rods in these two specimens were pretensioned to prestressing levels of 1000 MPa and 1500 MPa, respectively. The higher prestressing level provided more compressive stress surplus on the bottom concrete, which significantly postponed the occurrence of cracking. As mentioned, the CFRP strength of pile PCFRP1000 was not fully developed, and the increased prestressing level helped develop the tensile strength of the CFRP, which resulted in a higher ultimate capacity on PCFRP1500.

3. Numerical Analysis

3.1. Modeling Methodology

A finite element model was developed using the commercial package ABAQUS 6.21 software. Arc-shaped washers were placed at the supports and the loading point to mitigate the effect of stress concentration. Concrete and steel plates were modeled using C3D8R [39] (a general-purpose linear brick element, with a reduced integration three-dimensional solid element), while the CFRP rods and GFRP spirals were modeled using T3D2 [39] (two-node linear displacement three-dimensional wire element). The material properties listed in Table 1 were adopted in the FE model.

Tie constraints were adopted on the surface between the washers and the concrete. A full bond between the FRP and concrete was assumed and the embedded constraints were adopted. The mesh density gradually decreased from the mid-span to the ends. Concrete was simulated using a damage plasticity model, with constitutive models and material properties input based on material tests. The FRPs were simulated using a linear elastic model. Before applying external loads, prestressing was applied using the temperature method. For the temperature method, a thermal coefficient of CFRP was defined and prestressing force was applied by imposing a temperature decrease. The bending load was applied after the prestressing step. The finite element model is shown in Figure 10.

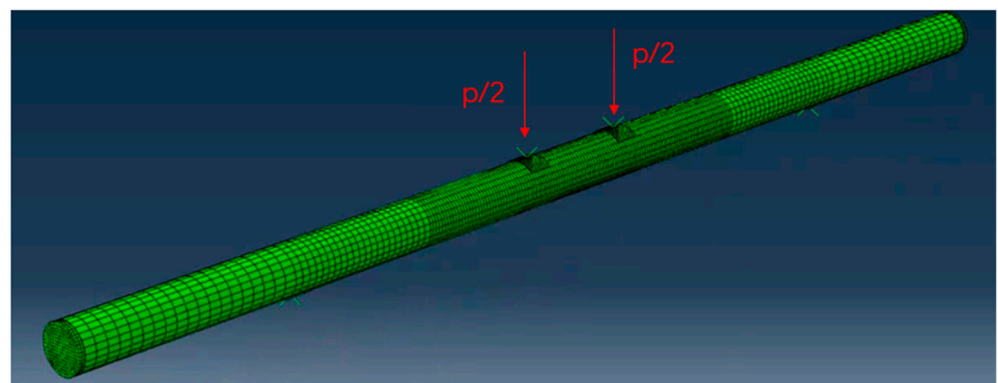


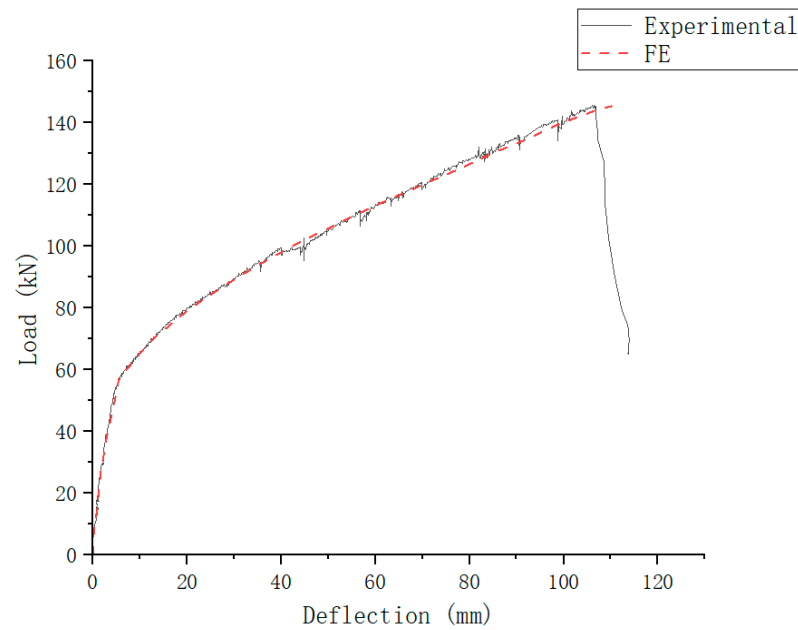
Figure 10. Finite element model.

There were three meshing sizes for the concrete: 30 mm between the loading points; 60 mm between the loading point and the support; and 100 mm for the cantilever part. The meshing size of the CFRP was 50 mm.

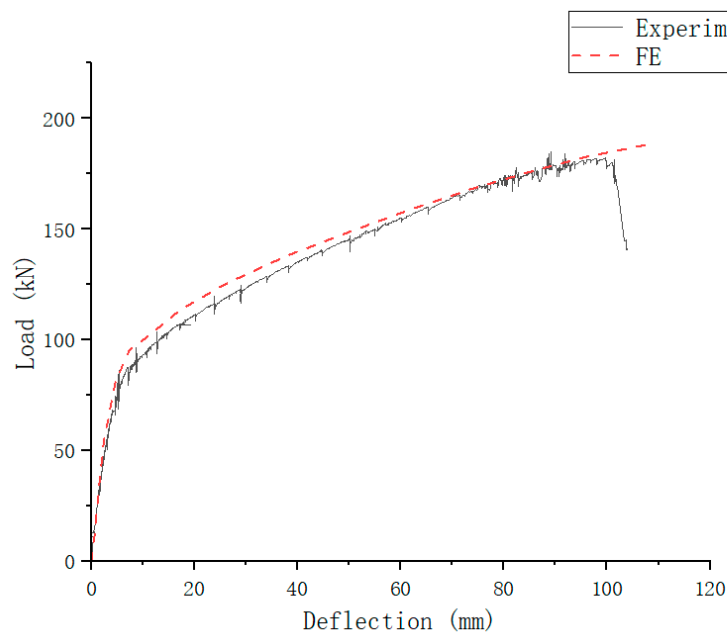
The analysis ended when either the compressive strain on the concrete exceeded 0.003 or the CFRP stress exceeded 2892 MPa. The corresponding load was taken as the ultimate load of the pile.

3.2. Validation of the FE Model

The develop FE model was validated against the test results. As shown in Figures 11 and 12, good agreement was achieved. The stiffness and ultimate capacity of the numerical results were slightly higher than those of the test results, which is attributed to the assumption of a full bond between the FRP and concrete. The similar cracking modes also verified the accuracy of the proposed FE model, as shown in Figure 12.



(a) PCFRP1000



(b) PCFRP1500

Figure 11. Comparison between test and FE results.



Figure 12. Cracking comparison.

3.3. Parametric Study

3.3.1. Investigated Parameters

In order to more comprehensively investigate the effect of the prestressing level, reinforcement ratio and modulus of the prestressing material, a parametric study was performed, as shown in Table 3. For the reinforcement ratio analysis, the location of the CFRP rods remained the same, the diameter of the CFRP rods was changed. The ultimate tensile strength of the CFRP was assumed to be the same in this study; thus, the CFRPs with a higher elastic modulus were modeled with a smaller ultimate strain.

Table 3. Modeling matrix.

Specimen	CFRP Rod Diameter (mm)	Prestressing Level (MPa)	CFRP Modulus (GPa)
P9-1000-180	9	1000	180
P9-1500-180	9	1500	180
P9-2000-180	9	2000	180
P9-1500-140	9	1500	140
P9-1500-200	9	1500	200
P9-1500-220	9	1500	220
P10-1500-180	10	1500	180
P12-1500-180	12	1500	180

3.3.2. Numerical Results

Load–deflection diagrams of piles with various parameters are shown in Figures 13–15. The failure mode, ultimate capacity and stress level on the CFRP are summarized in Table 4. The piles with higher prestressing level exhibited higher capacity and cracking resistance. Specimens P9-1500-200 and P9-1500-220, which had CFRP moduli of 200 GPa and 220 GPa, failed due to rupture of the CFRP with a lower flexural capacity, compared to specimen P9-1500-180. The specimens with higher reinforcement ratios exhibited higher flexural capacity and lower CFRP stress at failure.

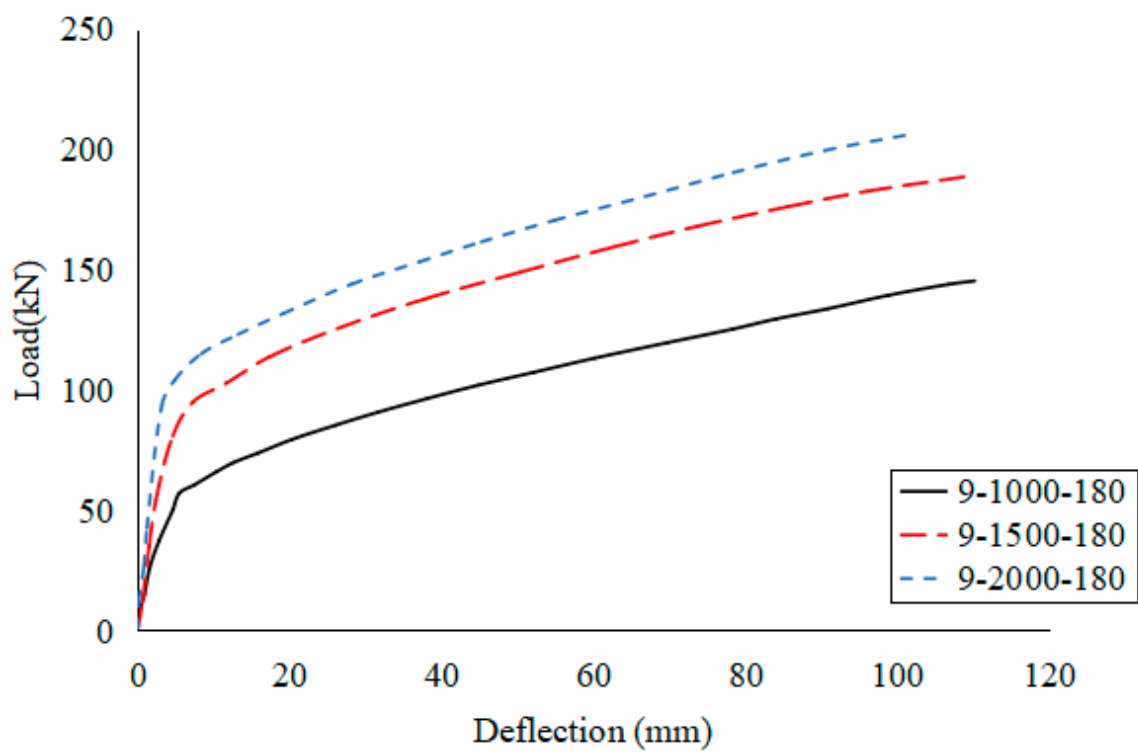


Figure 13. Effect of prestressing level.

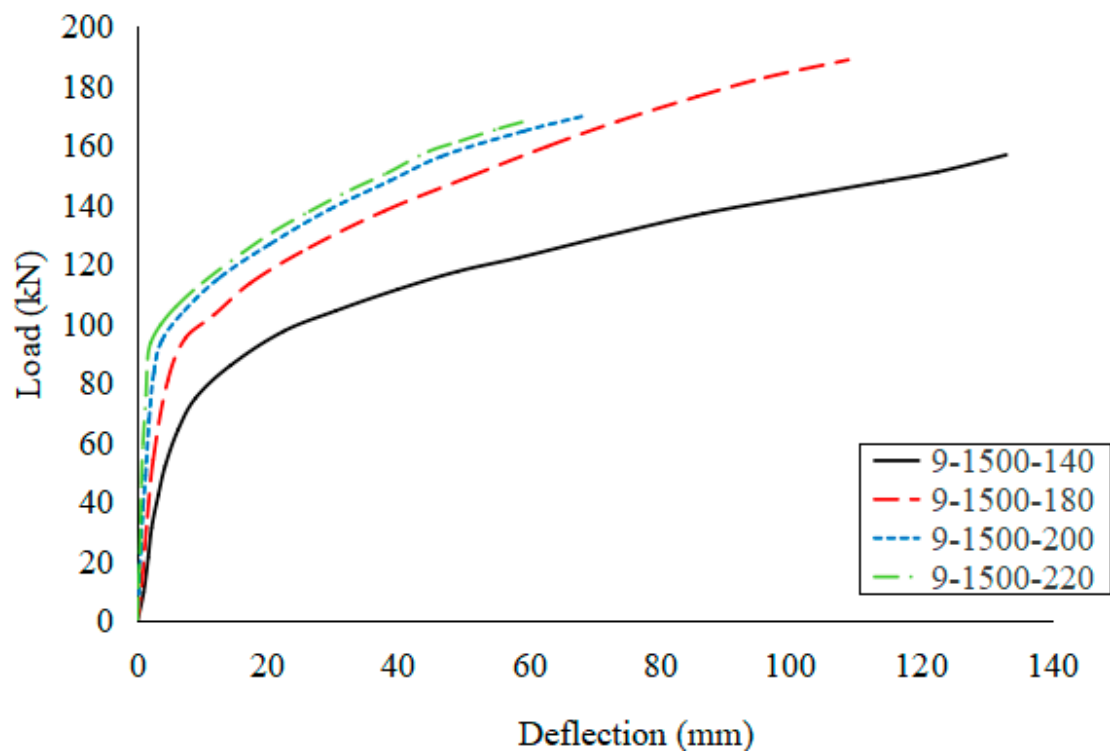


Figure 14. Effect of modulus.

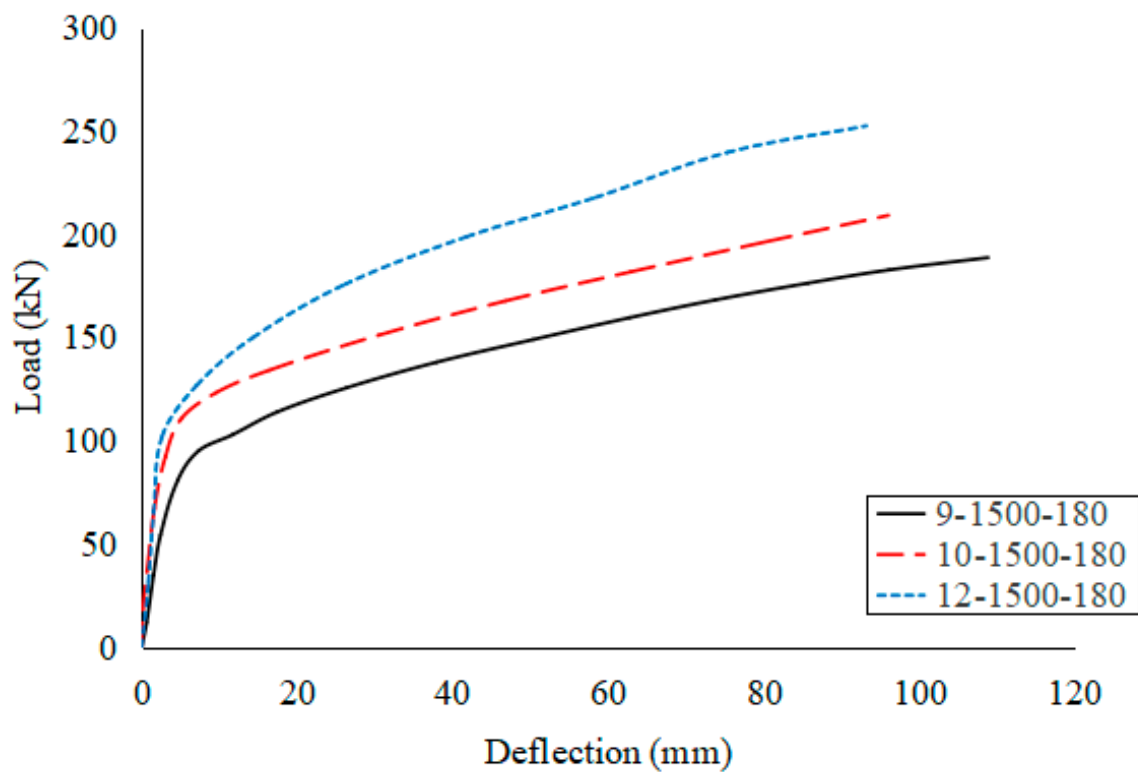


Figure 15. Effect of reinforcement ratio.

Table 4. Numerical results.

Specimen	Ultimate Load (kN)	Maximum CFRP Stress at Failure (MPa)	Failure Mode
P9-1000-180	145	2744	Concrete crushing
P9-1500-180	188	2567	Concrete crushing
P9-2000-180	206	2494	Concrete crushing
P9-1500-140	157	2212	Concrete crushing
P9-1500-200	169	2892	CFRP rupture
P9-1500-220	168	2892	CFRP rupture
P10-1500-180	209	2481	Concrete crushing
P12-1500-180	252	1982	Concrete crushing

3.4. Discussion

3.4.1. Effect of Elastic Modulus

The elastic moduli of most reported CFRP rods range from 140 GPa to 200 GPa [40–43]. As shown in Table 4, the capacity of P9-1500-180 was higher than that of P9-1500-140. However, as the modulus rose to 200 and 220 GPa, premature CFRP rupture failure occurred and a lower capacity was observed. The high-modulus CFRP was not able to develop adequate deformation to crush the concrete before reaching its ultimate tensile stress.

3.4.2. Effect of Prestressing Level

Prestress ranging from 1000 MPa to 2000 MPa was applied to the CFRP rods. An improvement in flexural capacity was observed as the prestressing level increased from 1000 MPa to 2000 MPa, and all the modeled piles failed due to concrete crushing. Limited improvement was observed upon comparing P9-2000-180 to P9-1500-180; this is attributed to the higher prestressing level resulting in larger compressive force on the top concrete, which further limited the development of compressive resistance.

3.4.3. Effect of Reinforcement Ratio

Heavier reinforcement was modeled for piles P10-1500-180 and P12-1500-180 compared to pile P9-1500-180. A higher reinforcement ratio evidently offered higher flexural capacity. Even though the stress exerted on CFRP rods was reduced, the increased CFRP cross-section area offered adequate tensile resistance to develop full strength of the concrete. The reduced stress helped avoid rupture of the CFRP rods as well. In addition, a larger reinforcement ratio provided a more compressive stress margin to the bottom concrete, which enabled cracking control under bending load.

4. Analytical Approach

Based on the parametric study performed in this study, an analytical approach to predict the flexural capacity of CFRP prestressed spun PHC pile was proposed and verified against experimental and numerical results. In the proposed approach, it was assumed that the sections remained plane under bending, and no debonding occurred between the CFRP and concrete. For simplicity, the discrete reinforcement was converted into a layer of annulation.

There are two potential failure modes for CFRP prestressed PHC piles: concrete crushing and CFRP rupture. The proposed analysis approach involved performing flexural capacity calculations for both cases and comparing the calculation results; the lower one would be the ultimate capacity of the pile. The stress and strain distributions of the analyzed section at failure are shown in Figures 16 and 17, in which, N.A. indicates the neutral axis.

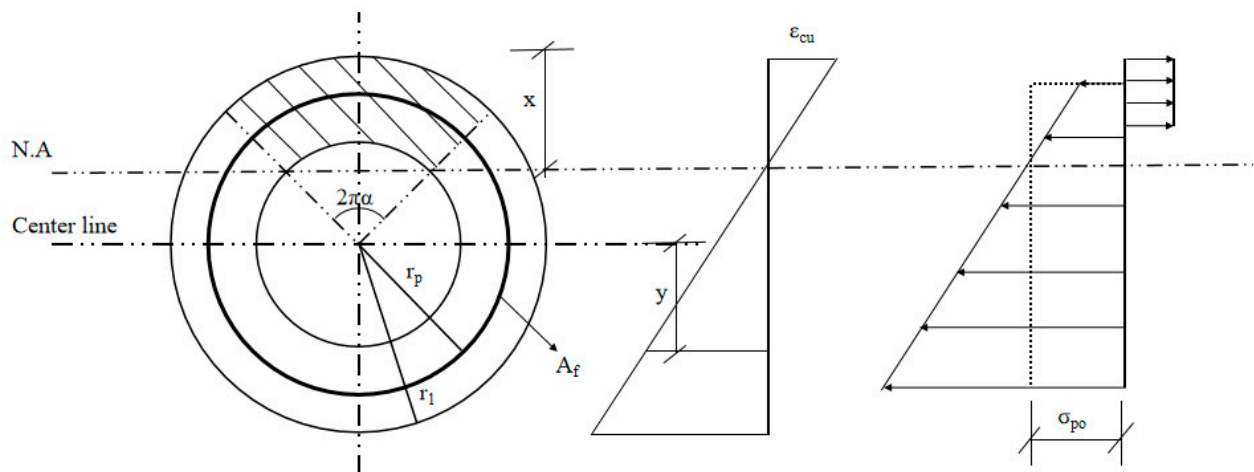


Figure 16. Stress and strain distribution of pile that failed due to concrete crush.

For the concrete crushing failure mode, the force equilibrium can be written as:

$$\alpha_1 f_{cu} A_c = 2r_p t_f \left(\int_{-\frac{\pi}{2}}^0 \left(\sigma_{f0} + \frac{y+r_1-x}{x} \epsilon_{cu} E_f \right) d\theta + \int_0^{(\frac{1}{2}-\alpha)\pi} \left(\sigma_{f0} + \frac{r_1-x-y}{x} \epsilon_{cu} E_f \right) d\theta + \int_{(\frac{1}{2}-\alpha)\pi}^{\frac{\pi}{2}} \left(\sigma_{f0} - \frac{y-r_1+x}{x} \epsilon_{cu} E_f \right) d\theta \right) \quad (2)$$

where α_1 is the ratio of the stress of the equivalent rectangular compression zone to the axial compressive strength of concrete, which was 0.94 in this study; f_{cu} is the compressive strength of the concrete; σ_{f0} is the initial prestressing stress; A_c is the area of concrete in the compression zone; ϵ_{cu} is the ultimate compressive strain of the concrete; E_f is the elastic modulus of the CFRP; r_p is the distance between the center of the whole section and the center of each CFRP rod; t_f is the equivalent thickness of the CFRP; r_1 is the outer radius of the section; r_2 is the inner radius of the section; x is the height of the compression zone; y is the vertical distance between the investigated point and the center line of the section; and θ is the angle between the center line of the section and the line from the investigated point to the center of the section.

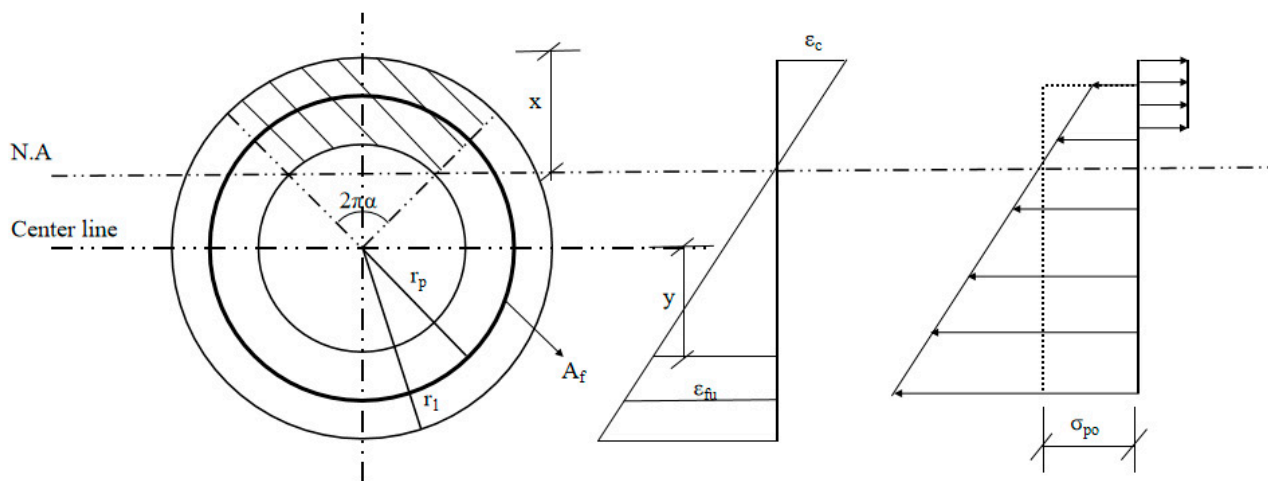


Figure 17. Stress and strain distribution of pile that failed due to CFRP rupture.

The left part of Equation (2) represents the compressive force developed by the concrete. Since a concrete crushing failure mode was assumed, the ultimate strength of the concrete was adapted here. The right part of Equation (2) represents the tensile force developed by the CFRPs, which were divided into three parts: the part below the centerline; the part between centerline and the neutral axis; and the part above the neutral axis.

$$y = r_p \sin \theta \quad (3)$$

$$\beta_1 x = r_1 - r_p \cos \alpha \pi \quad (4)$$

$$t_f = \frac{A_p}{2\pi r_p} \quad (5)$$

The depth of compression zone x and the corresponding angle α can be solved, and the ultimate capacity can be calculated as:

$$M = 2r_p t_f \left(\int_{-\frac{\pi}{2}}^0 \left(\sigma_{f0} + \frac{y+r_1-x}{x} \varepsilon_{cu} E_f \right) (r_1 - y) d\theta + \int_0^{(\frac{1}{2}-\alpha)\pi} \left(\sigma_{f0} + \frac{r_1-x-y}{x} \varepsilon_{cu} E_f \right) (r_1 + y) d\theta + \int_{(\frac{1}{2}-\alpha)\pi}^{\frac{\pi}{2}} \left(\sigma_{f0} - \frac{y-r_1+x}{x} \varepsilon_{cu} E_f \right) (r_1 + y) d\theta \right) - \alpha_1 f_{cu} A_c \left(2r_2 - \frac{x\beta_1}{2} \right) \quad (6)$$

In Equation (6), the moment was calculated to be about the very bottom of the section. The right side of the equation was divided into four parts: the moment of the CFRP below the centerline; the moment of the CFRP between the centerline and the neutral axis; the moment of the CFRP above the neutral axis; and the moment of the concrete in the compression zone.

For the CFRP rupture failure mode, the force equilibrium can be written as:

$$\alpha_1 f_c A_c = 2r_p t_f \left(\int_{-\frac{\pi}{2}}^{(\frac{1}{2}-\alpha)\pi} \left(\sigma_{f0} + \frac{y+r_1-x}{r_1-x+r_p} \varepsilon_{fu} E_f \right) d\theta + \int_{(\frac{1}{2}-\alpha)\pi}^{\frac{\pi}{2}} \left(\sigma_{f0} - \frac{y-r_1+x}{r_1-x+r_p} \varepsilon_{fu} E_f \right) d\theta \right) \quad (7)$$

$$f_c = E_c \varepsilon_c \quad (8)$$

$$\varepsilon_c = \frac{x}{r_1 - x + y} \varepsilon_{fu} \quad (9)$$

The left part of Equation (7) represents the compressive force developed by the concrete. The right part of Equation (7) represents the tensile force developed by the CFRPs, which are divided into three parts: the part below the centerline; the part between the centerline and the neutral axis; and the part above the neutral axis. Since a CFRP rupture failure mode was assumed, the full strength of the CFRP was adapted in Equation (6).

The depth of compression zone x and the corresponding angle α can be solved, and the ultimate capacity can be calculated as:

$$M = 2r_p t_f \left(\int_{-\frac{\pi}{2}}^0 \left(\sigma_{f0} + \frac{y+r_1-x}{r_1-x+r_p} \varepsilon_{fu} E_f \right) (r_1 - y) d\theta + \int_0^{(\frac{1}{2}-\alpha)\pi} \left(\sigma_{f0} + \frac{y+r_1-x}{r_1-x+r_p} \varepsilon_{fu} E_f \right) (r_1 + y) d\theta + \int_{(\frac{1}{2}-\alpha)\pi}^{\frac{\pi}{2}} \left(\sigma_{f0} - \frac{y-r_1+x}{r_1-x+r_p} \varepsilon_{fu} E_f \right) (r_1 + y) d\theta \right) - \alpha_1 f_c A_c (2r_2 - \frac{x\beta_1}{2}) \quad (10)$$

In Equation (10), the moment was calculated to be about the very bottom of the section. The right side of the equation was divided into four parts: the moment of the CFRP below the centerline; the moment of the CFRP between the centerline and the neutral axis; the moment of the CFRP above the neutral axis; and the moment of the concrete in the compression zone.

The flexural capacity of the piles modeled in a previous study was calculated using the proposed approach, and the results are summarized in Table 5. The proposed approach offered a conservative prediction compared to the numerical and finite element results. A deviation rate within 10% was achieved for all the specimens.

Table 5. Analytical results.

Specimen	FE		Proposed Approach		Deviation Rate (%)
	Capacity (kN·m)	Failure Mode	Capacity (kN·m)	Failure Mode	
P9-1000-180	185	CC	167	CC	−10
P9-1500-180	218	CC	197	CC	−9.8
P9-2000-180	231	CC	226	CC	−4.9
P9-1500-140	196	CC	183	CC	−7.6
P9-1500-200	208	R	203	R	−2.4
P9-1500-220	207	R	203	R	−2.9
P10-1500-180	246	CC	229	CC	−7.9
P12-1500-180	291	CC	285	CC	−2.1

“CC” is concrete crushing and “R” is rupture of CFRP.

5. Conclusions

In this study, a comprehensive experimental and numerical study was performed to investigate the flexural performance of CFRP prestressed PHC pile. In addition, a simplified approach to predict flexural capacity was proposed based on a parametric study. Based on the experimental and numerical results, the following conclusions can be drawn:

1. The proposed PHC pile system reinforced with CFRP rods exhibited superior flexural performance compared to PHC pile reinforced with 1420 steel, in terms of flexural capacity and cracking resistance.
2. The flexural capacity of the CFRP prestressed PHC pile was dominated by the failure modes. The concrete crushing and rupture of CFRP are the two typical failure modes of CFRP prestressed PHC piles under bending. The failure mode was affected by the stress level of the CFRP and concrete strain at the ultimate stage.
3. The increment of the reinforcement ratio and prestressing level improved the flexural capacity and the cracking resistance of CFRP prestressed PHC piles.
4. The increment of the elastic modulus improved the flexural capacity of CFRP prestressed PHC piles. However, the high modulus of the CFRP resulted in premature rupture failure of the CFRP rods. It is suggested to perform a stress check according to the analytical approach proposed in this paper.
5. An analytical approach for the flexural capacity of CFRP prestressed PHC pile was proposed, and 90% accuracy was achieved. The proposed analytical approach offers conservative predictions of flexural capacity.

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