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Flexural performance of pretensioned centrifugal spun concrete piles with combined steel strands and reinforcing bars

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Abstract: Pretensioned spun high-strength concrete (PHC) piles have been widely used in building foundations in many parts of the world. However, PHC piles are also known to have poor lateral deformation and ductility capacity and hence are not suitable for seismic regions. To improve the horizontal load bearing and deformation capacities, pretensioned centrifugal spun concrete piles with steel strands alone (hereafter referred to as PSC piles) or a combination of steel strands and mild steel deformed rebars (referred to as PSRC piles) have been developed. PSRC piles are expected to benefit on the one hand from steel strands with high deformation capacity, and on the other hand from deformed steel bars with good bond with concrete and hence better control of cracks. This paper presents a comprehensive study on the comparative flexural performance of PSC and PSRC piles. The bending tests of PSC piles and PSRC piles were firstly conducted on six full-scale pile specimens with three different pile diameters, respectively. The flexural performances are evaluated in terms of crack resistance, flexural and deformation...
capacities, as well as crack distribution. The results show that the cracking bending moment of PSRC piles is similar to that of PSC piles, but the ultimate bending moment is about 55% higher due to the addition of deformed rebars.

The general cracking behavior of PSRC piles exhibit remarkable improvement comparing to PSC piles, with an extended crack region, denser distribution of cracks, and much reduced maximum crack width. Following the experimental study, a theoretical method is proposed for the calculation of the flexural strength and flexural stiffness of prestressed concrete piles considering the presence of both steel strands and deformed rebars. Furthermore, a finite element (FE) model has been developed to simulate the flexural performance of the piles. The accuracy of the FE model is validated through comparing with the experimental results, and subsequently parametric analyses are conducted to explore the influences of three parameters, including the prestressing level of the prestressing strands, prestressing strand ratio and deformed steel bar ratio, on the flexural performance of PSRC piles.

**Author keywords:** Pretensioned spun concrete pile; Steel strand; Deformed steel bar; High-strength concrete; Flexural performance; Parametric analysis.

1. Introduction

Pretensioned spun high-strength concrete (PHC) piles have been widely used in building foundations in many parts of the world owing to the advantages of high axial bearing capacity, standardized manufacturing process, well-controlled product quality and outstanding economic benefit. As foundations of buildings, PHC piles not only support vertical loads from superstructures but also resist lateral loads resulting from wind and earthquake loads. However, a range of problems gradually emerged in the application of PHC piles. Studies on the damage to building pile foundations in the 1995 Hyogoken-Nanbu Earthquake [1] and the 2011 Tohoku Pacific Earthquake [2] in Japan showed that PHC piles typically suffered brittle failure at the pile heads under bending and pull-out forces, and generally exhibited poor seismic performance during the earthquakes. The collapse of a 13-story high-rise
residential building in June 2009 was partly attributable to the low flexural capacity of PHC piles [3]. In fact, the
application of PHC piles in seismic regions is generally restricted, for example in China PHC piles can only be used
in low seismic intensity regions and for multi-story buildings and in Japan PHC piles are restricted to middle and
bottom of the whole pile length, with steel encased piles at the top [4]. Some new types of piles have been developed
and studied in recent years, such as steel pipe piles and composite piles, which exhibit good structural performance.
But compared to these new types of piles, precast prestressed concrete piles (PPCP) are still a practical, economical,
and durable option for pile foundations [5]. Thus, more specific research on the structural performance of PHC piles,
which are an important part of PPCPs, is necessary to meet the requirements of engineering applications.
Extensive research has been carried out in the past in attempt to improve the flexural strength and ductility
capacities of PHC and hollow reinforced concrete (RC) piles and make them better suited for application in seismic
areas. The effect of improving the lateral confinement of piles has particularly been studied. Muguruma et al. [6]
experimentally studied the flexural ductility of PHC piles with various amounts of lateral confining hoop
reinforcement of high yield strength. Kishida et al. [7] investigated the influence of increasing the amount of spiral
reinforcement and filling concrete into the hollow part on the ultimate shear strength and deformation behavior of
PHC piles, using an antisymmetric bending-shear experiment setup. Through theoretical analysis and experimental
study, Lignola et al. [8,9] found that the fiber reinforced polymer (FRP) jacketing could significantly enhance the
flexural strength and ductility of hollow RC members. Murugan et al. [10] also carried out an experimental study
on RC piles wrapped with carbon and glass fiber reinforced polymers (CFRP and GFRP) under lateral loads, and
pointed out that the CFRP- and GFRP-confined RC piles possessed higher lateral load bearing capacity than
unconfined piles. Akiyama et al. [11] conducted bending tests on a new type of prestressed hollow RC pile and
demonstrated that a sufficient lateral confinement of high-strength concrete provided by high-strength spirals and
carbon-fiber sheets, combined with concrete infilling could substantially increase the flexural capacity of hollow
RC piles. Thusoo et al. [12] investigated the flexural performance of hollow spun steel-encased high-strength concrete piles and found that core-filled piles had a better ductility because the infilling material provided confinement to the high-strength concrete layer and prevented the spalling of inner concrete. Irawan et al. [13] studied the effect of infilling concrete on the flexural performance of PHC piles and concluded that the ductility of PHC piles was increased by infilling concrete into the pile core, while the failure of PHC piles was triggered by the fracture of prestressing steel bars.

Besides lateral confinement, the effect of additional non-prestressing tendons has also been studied by some researchers. Ikeda et al. [14] experimentally demonstrated that the addition of non-prestressing longitudinal reinforcement could enhance the ductile performance of PHC piles under lateral loads even after the tensile rupturing of prestressing tendons. Yang et al. [15,16] studied the effect of deformed steel bars on the flexural performance and seismic performance of PHC piles through load test and finite element analysis, and pointed out that deformed steel bars could improve the bearing capacity as well as ductility of PHC piles. Furthermore, Wu et al. [17] carried out a full-scale experimental study on the flexural strength and behavior of PHC piles reinforced with GFRP bars and deformed steel bars, respectively. The results showed that piles reinforced with GFRP bars had much higher flexural capacity than those reinforced with deformed steel bars, and increasing the hybrid reinforcing bar ratios could significantly enhance the post-cracking flexural strength and stiffness of PHC piles. Zhang et al. [18] conducted the cyclic loading tests on PHC piles with partial normal-strength deformed bars and demonstrated that increasing the deformed bar ratios could improve the ultimate flexural capacity of PHC piles, but did not have much effect on the ductility due to the poor deformability of prestressing steel bars.

Although strengthening the lateral confinement and incorporating non-prestressing tendons can effectively improve the flexural strength and ductility capacity of PHC piles, these measures cannot eliminate the bottleneck issue associated with the limited deformability of the prestressing steel bars. As a matter of fact, PHC piles are
inherently prone to brittle failure under lateral load owing to the premature rupture of prestressing steel bars in
tension zone. Considering that the prestressing steel bars which are now widely used in PHC piles have relatively
low elongation capacity, the present study has been motivated to look at completely replacing the prestressing steel
bars with higher elongation and higher strength prestressing steel strands, and thereby develop new pretensioned
centrifugal spun concrete piles with steel strands. The flexural performances of PHC pile and PSC pile have been
compared by full-scale bending tests [19], and the results showed that the PSC piles with a smaller reinforcement
ratio had similar anti-cracking performance, but higher flexural strength and better deformation capacity. Because
a higher tensile control stress with the use of prestressing strands, a smaller longitudinal reinforcement ratio can
achieve the same effective compressive pre-stress of concrete. However, the cracking pattern of PSC piles was less
desirable; the number of cracks in the critical region was small, and the maximum crack width was large.

On the other hand, the amount of the prestressing strands needs to be capped so that a desired control prestressing
level can be imposed for the targeted desirable prestress in the concrete. Otherwise, the control stress for the
prestressing strands would have to be reduced to achieve the target prestress in concrete, consequently the strength
of the prestressing strands may not be fully utilized at failure of the piles, rendering the design uneconomical. This
means that the scope of increasing the flexural strength of the piles would be limited when prestressing strands are
used alone.

Therefore, in order to improve the cracking performances and at the same time facilitate further increase of the
flexural strength, the idea of adding non-prestressing rebar to PSC piles has been envisaged to make the piles more
robust and suitable for application in high intensity seismic areas. This paper presents a comprehensive study on the
flexural performance of pretensioned centrifugal spun concrete piles with combined steel strands and deformed steel
bars by means of full-scale bending tests and the associated theoretical analysis. A finite element analysis of the
tested piles has also been carried out to assist in further comparison of flexural performance between PSC and PRSC
2. Experimental program

2.1. Test specimens

PHC piles used in practical projects commonly have an external diameter of 400 mm, 500 mm or 600 mm with a length of 8 m, 12 m or 15 m to meet different engineering demands. In accordance with the dimensions of PHC piles, full-scale pile specimens of external diameters of 400 mm, 500 mm and 600 mm, respectively, and a length of 8 m, were fabricated for the experimental study. All specimens were pretensioned centrifugal spun high-strength concrete piles. For the sake of comparison, each diameter group consisted of two piles, one reinforced with steel strands only (PSC pile) and another with a combination of steel strands and reinforcing bars (PSRC pile). For convenience, the three PSC pile specimens are labeled as PSC400, PSC500, and PSC600, and their counterpart PSRC pile specimens as PSRC400, PSRC500, and PSRC600, respectively. Compared with PSC piles, the PSRC piles retain the same prestressing steel strands but with addition of the same number of HRB400 deformed steel bars of 16 mm in diameter as non-prestressing reinforcement. Fig. 1 and Table 1 show the reinforcement details of the two types of piles.

The steel strands used in all specimens are of 7-wire steel strand with a nominal diameter of 11.1 mm and a net cross-section area of 74.2 mm$^2$. The steel strands had a nominal tensile strength of 1860 MPa and the control stress ($\sigma_{\text{con}}$) for the prestressing of each steel strand was maintained at 70% of the nominal tensile strength. As can be seen, the initial effective compressive pre-stress of concrete ($\sigma_{\text{ce}}$) between PSC and PSRC piles were similar, with those in PSRC piles being about 4% lower on average.
Fig. 1. Schematic diagram of reinforcements in pile test specimens: (a) PSC pile; and (b) PSRC pile. (Unit: mm)

Table 1 Geometric dimensions and reinforcement specifications of test specimens.

<table>
<thead>
<tr>
<th>Pile type</th>
<th>$D_a$ (mm)</th>
<th>$D_b$ (mm)</th>
<th>$t$ (mm)</th>
<th>Longitudinal reinforcement</th>
<th>$\rho_s$ (%)</th>
<th>Stirrup</th>
<th>$\sigma_{con}$ (MPa)</th>
<th>$\sigma_{ce}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PSC400</td>
<td>400</td>
<td>308</td>
<td>95</td>
<td>$7\Phi^511.1$</td>
<td>0.57</td>
<td>$\Phi^4@45/80$</td>
<td>1302</td>
<td>5.57</td>
</tr>
<tr>
<td>PSRC400</td>
<td>400</td>
<td>308</td>
<td>95</td>
<td>$7\Phi^511.1+7\Phi^416$</td>
<td>0.57+1.55</td>
<td>$\Phi^4@45/80$</td>
<td>1302</td>
<td>5.39</td>
</tr>
<tr>
<td>PSC500</td>
<td>500</td>
<td>406</td>
<td>100</td>
<td>$11\Phi^511.1$</td>
<td>0.65</td>
<td>$\Phi^5@45/80$</td>
<td>1302</td>
<td>6.31</td>
</tr>
<tr>
<td>PSRC500</td>
<td>500</td>
<td>406</td>
<td>100</td>
<td>$11\Phi^511.1+11\Phi^416$</td>
<td>0.65+1.76</td>
<td>$\Phi^5@45/80$</td>
<td>1302</td>
<td>6.08</td>
</tr>
<tr>
<td>PSC600</td>
<td>600</td>
<td>506</td>
<td>110</td>
<td>$14\Phi^511.1$</td>
<td>0.61</td>
<td>$\Phi^5@45/80$</td>
<td>1302</td>
<td>5.98</td>
</tr>
<tr>
<td>PSRC600</td>
<td>600</td>
<td>506</td>
<td>110</td>
<td>$14\Phi^511.1+14\Phi^416$</td>
<td>0.61+1.66</td>
<td>$\Phi^5@45/80$</td>
<td>1302</td>
<td>5.76</td>
</tr>
</tbody>
</table>

$^a$ Diameter of pile.

$^b$ Diameter of distribution circle of longitudinal reinforcement.

$^c$ Wall thickness of pile.

2.2. Material mechanical properties

The concrete for pile test specimens had a design strength grade of C105. The mix design of the concrete had a mass ratio of water, cement, sand and gravel as 0.26 : 1.00 : 1.12 : 2.60. When casting the piles, nine concrete cubes of 100 mm side length were prepared, and they were kept in the curing condition similar to the pile specimens. The concrete cubes were tested just before the pile experiments. The average value of cubic compressive strength $f_{cu,10}$ for these test blocks was 116.1 MPa. According to the empirical formulas [20,21], the standard cubic compressive strength $f_{cu}$, axial compressive strength $f_c$ and tensile strength $f_t$ may then be calculated as follows:
\[ f_{cu} = 0.91 f_{cu,10} + 1, \]

\[ f_c = 0.818 f_{cu}, \]

\[ f_t = 0.26 f_{cu}^{2/3}. \]

The steel strands used as the prestressing tendons in the pile specimens are low relaxation strands. For the non-prestressing reinforcement in the PSRC piles, HRB400 deformed steel bars were used. Grade-A cold drawn low carbon steel wires were used as the spiral stirrups in all pile specimens. Three samples for each type of reinforcing bars, namely Φ11.1 steel strand, Φ16 deformed steel bar and Φ6 steel wire, were taken from the same batch as the pile specimens, and they were tested to measure the mechanical properties, respectively. It should be noted that because of some difficulty for the tensile testing machine to hold small diameter bars like Φ4 and Φ5 steel wires, three samples of Φ6 steel wire from the same batch of wires were tested instead. Fig. 2 shows the measured stress-strain responses for the three types of reinforcing bars, and Table 2 presents the key values of material mechanical parameters. It can be seen that the deformed steel bars have a long yield plateau and a much larger ultimate deformation capacity as compared with the prestressing strands.

**Table 2** Material parameters of reinforcing bars.

<table>
<thead>
<tr>
<th>Reinforcing bar</th>
<th>$E_s$ (GPa)</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$\delta$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Φ11.1</td>
<td>195</td>
<td>1725</td>
<td>1920</td>
<td>6.4</td>
</tr>
<tr>
<td>Φ16</td>
<td>191</td>
<td>503</td>
<td>614</td>
<td>11.5</td>
</tr>
<tr>
<td>Φ6</td>
<td>199</td>
<td>477</td>
<td>549</td>
<td>2.6</td>
</tr>
</tbody>
</table>
2.3. Test setup and loading schedule

The standard 4-point loading scheme was adopted to conduct the bending test of pile specimens, as shown in Fig. 3. The load was applied using an electro-hydraulic servo multi-functional testing machine (YAW-10000F) controlled by a computer. Five digital displacement meters (LVDTs) were installed to measure the deflections of the tested piles at the middle and quarter-span points, as well as at two support locations to monitor any support settlements. Nine electronic resistance strain gauges with a gauge length of 100 mm were installed on the concrete surface to measure the concrete strain over the depth of the midspan section and along the bottom tensile edge at locations close to the midspan. The strain readings were automatically recorded by a distributed static strain acquisition system (DH3816N). Each pile test specimen has a total length of 8.0 m and a 1.0 m pure bending segment in the midspan. According to the configuration and dimensions shown in Fig. 3(b), the bending moment of the midspan section of the pile test specimens can be calculated by:

\[
M_e = \frac{P}{4} \left( \frac{3}{5} L - 1 \right) + \frac{1}{40} WL, \tag{4}
\]

where \( P \) is the load applied by the testing machine; \( L \) and \( W \) are the length and the self-weight of pile specimens, respectively.
The cracking moment $M_{cr}$ and the ultimate bending moment $M_u$ of pile test specimens were estimated in accordance with the relevant standard [22]. The loading scheme of bending test was then implemented as follows. First, the tested piles were loaded with a 20% increment of $M_{cr}$ until the bending moment reached 80% of $M_{cr}$ in the midspan. Subsequently, a 10% increment of $M_{cr}$ was applied until the appearance of cracks. When the vertical cracks were observed on the concrete surface of the tested piles, the load was recorded and loading was paused to allow measurements of crack location and width. Loading was then continued with a 5% increment of $M_u$ until the bending moment reached $M_u$ in the midspan. In the final stage, the loading was shifted to a displacement-controlled mode with a displacement increment of 2 mm, until the test specimens were considered to have failed, which was usually marked by a rapid decrease in the applied load. The load duration for each loading step was kept about 3 minutes for the stabilization of measurement readings.

3. Experimental results and analysis

3.1. Load-deflection curves

Fig. 4 depicts the midspan load-deflection curves of six pile test specimens. The cracking and maximum load
points are marked with specific coordinate values. Based on the load-deflection curves and the corresponding observations during the experiment, the whole loading process can generally be described as consisting of three distinctive stages: (1) Pre-cracking stage; from initial loading to the occurrence of the first crack, each pile test specimen exhibited approximately a linear elastic behavior and the load increased linearly with the midspan deflection. The flexural stiffness of PSRC piles was slightly higher than that of PSC piles of the same diameter, which is consistent with the increase in the total longitudinal reinforcement in PSRC piles. (2) Non-linear stage; after first crack appeared in the midspan, the flexural stiffness of the pile specimens decreased significantly, and the response entered into a clear non-linear stage. The PSRC piles exhibited markedly larger flexural stiffness as compared to PSC piles, due to the contribution of the non-prestressing steel bars. (3) Ultimate stage; the concrete in the compression zone began to peel off with the increase of midspan deflection, and the bearing capacity of test specimens reached the maximum level and then turned to decreasing while the concrete in the compression zone crushed completely.

As shown in Fig. 5, the failure modes of PSC and PSRC piles were both the concrete crushing in the compression zone but without the rupture of the yielding steel strands and deformed steel bars, exhibiting the characteristics of ductile failure.
3.2. Flexural performance

Table 3 shows a comparison of the key flexural performance parameters between PSC and PSRC tested piles.

Compared with PSC piles, the effective compressive pre-stress of concrete for PSRC piles was smaller but the converted section moment of inertia was larger, thus the cracking moment $M_{cr,e}$ of both piles were generally similar.

As mentioned in the details of the test specimens, the absolute amount of the longitudinal reinforcement of PSRC piles was significantly increased thanks to the addition of non-prestressing deformed steel bars. As a result, the ultimate bending moments $M_{ue}$ of PSRC piles were substantially increased, which were 61%, 55% and 50% greater than that of corresponding PSC piles, respectively. At the same time, the PSRC piles maintained a good deformation capacity, and the maximum midspan deflection $f_{ue}$ was generally comparable to PSC piles. The above results indicate that adding the non-prestressing steel rebar can significantly enhance the flexural strength from the steel-
strands only piles, without negatively affecting the deformation capacity, making PSRC piles more suitable for high intensity seismic regions.

### Table 3 Comparison of flexural performance of test specimens.

<table>
<thead>
<tr>
<th>Pile type</th>
<th>(M_{e,c}) (kN·m)</th>
<th>(M_{e,c,PSRC}/M_{e,c,PSC})</th>
<th>(M_{u,e}) (kN·m)</th>
<th>(M_{u,e,PSRC}/M_{u,e,PSC})</th>
<th>(f_{u,e}) (mm)</th>
<th>(f_{u,e,PSRC}/f_{u,e,PSC})</th>
</tr>
</thead>
<tbody>
<tr>
<td>PSC400</td>
<td>73.8</td>
<td>1.17</td>
<td>166.5</td>
<td>1.61</td>
<td>63.22</td>
<td>0.97</td>
</tr>
<tr>
<td>PSRC400</td>
<td>86.3</td>
<td>1.01</td>
<td>267.8</td>
<td>1.61</td>
<td>61.28</td>
<td>0.91</td>
</tr>
<tr>
<td>PSC500</td>
<td>170.1</td>
<td>1.01</td>
<td>349.8</td>
<td>1.55</td>
<td>52.02</td>
<td>0.91</td>
</tr>
<tr>
<td>PSRC500</td>
<td>172.3</td>
<td>1.01</td>
<td>543.8</td>
<td>1.55</td>
<td>47.33</td>
<td>0.91</td>
</tr>
<tr>
<td>PSC600</td>
<td>294.5</td>
<td>1.01</td>
<td>540.0</td>
<td>1.50</td>
<td>38.63</td>
<td>1.10</td>
</tr>
<tr>
<td>PSRC600</td>
<td>297.6</td>
<td>1.01</td>
<td>812.5</td>
<td>1.50</td>
<td>42.51</td>
<td>1.10</td>
</tr>
</tbody>
</table>

### 3.3 Crack distributions

Fig. 6 shows the crack distributions of the six tested piles before complete failure. The two types of piles exhibit striking differences in the crack patterns. For PSC piles, the cracks were mainly distributed in a narrower region extending about 1.0 m on both sides of the midspan, and the number of cracks was 8, 12 and 14 for the three piles, respectively. The corresponding average crack spacing was 241.3 mm, 169.2 mm and 135.7 mm and the maximum crack width was 1.54 mm, 1.38 mm and 1.26 mm. On the other hand, for PSRC piles, the cracks appeared in a noticeably wider region of about 1.25 m on both sides of the midspan, with a markedly denser distribution of the cracks; the number of cracks was 20, 22 and 25, respectively. The corresponding average crack spacing was 102.3 mm, 109.8 mm and 104.4 mm. As a result of denser cracks, the maximum crack widths in the PSRC piles were approximately halved from those in the PSC piles, measuring at 0.68 mm, 0.56 mm and 0.70 mm for the three PSRC piles, respectively.

Fig. 7 presents the load vs. maximum crack width curves of test specimens until the loads reach the ultimate values. It can be seen that the cracking loads of both piles with the same diameter are generally similar. After cracking, the maximum crack width increases rapidly for PSC piles with the increasing load, while PSRC piles have a reduced rate of crack development with the increase of load.
3.4. Strain development

The strain gauges were arranged to measure the concrete strain of pile test specimens at selected critical locations, as shown in Fig. 3(b), mainly for the purpose to monitor the occurrence of initial cracking, as well as the strain distribution over the depth of the midspan section. Fig. 8 shows the relationship between the measured strains and the applied load. In the figures, the recorded tensile strain data above 1500 με are omitted as such data could be
susceptible to errors due to concrete cracks. Two vertical straight lines represent the observed visible cracking and maximum load points, respectively.

It can be seen that before the first visible cracking, the strains from all strain gauges increased linearly with the load. Near the initiation of cracking, some abrupt increase of the strains occurred in some tensile strain gauges. After the occurrence of first crack, the strains in the tension zone increased rapidly leading to the failure of some of these strain gauges. In the meantime, the compressive strain at the upper side of the midspan section increased at a slow rate before cracking, and after cracking the compressive strain also exhibited a faster rate of increase with the load.
Fig. 8. Concrete strain development of test specimens: (a) PSC400; (b) PSRC400; (c) PSC500; (d) PSRC500; (e) PSC600; and (f) PSRC600.

Fig. 9 presents the strain distribution over the depth of the midspan section at different loading steps. It can be seen from the diagrams that before cracking the strain of concrete at the midspan cross-section maintained a linear distribution along the depth of the cross-section. This confirms that in the current case with hollow circular section and under a prestressing condition, the assumption of plane section remaining plane still holds under external loading. It should be noted that after cracking the strain readings from the strain gauges in the tension zone were no longer reliable in representing an average strain across cracked and uncracked sections, consequently it is not possible to plot the strain distribution over the cross-section after cracking. However, from the compressive strains it can be observed that the neutral axis markedly shifted upwards with the increase of load.
The initial compressive strain of concrete can be approximately estimated through the effective compressive pre-stress divided by the elastic modulus of concrete. Adding the initial strain value of concrete to the maximum reading from strain gauge #1, the compressive strain of concrete at failure in the compression zone for each pile specimen can be estimated. The average values for PSC and PSRC pile test specimens were similar at about 2800 $\mu$ε, which is higher than the empirical value of peak compressive strain of high strength concrete (2500 $\mu$ε) [20]. This indicated that the compressive performance of high strength concrete was fully mobilised in both types of piles.

4. Theoretical analysis on flexural strength and stiffness

In this section, a theoretical analysis is conducted on the flexural strength and stiffness at a cross-section level for
the PSC and PSRC piles. The key factors considered include the prestressing level and the circular arrangement of longitudinal reinforcements. The analysis is based on strain compatibility, force equilibrium, and material constitutive relationships. The accuracy of theoretical predictions of the flexural strength and stiffness is then verified by the experimental results from the above full-scale PSC and PSRC pile tests.

4.1. General calculation method for flexural strength

Similar to the theoretical calculation of flexural bearing capacity for ordinary reinforced concrete members, it is assumed that on average the pile cross-section remains plane after bending and a perfect bond exists between reinforcing bars and concrete. For prestressed spun concrete hollow piles, the analysis of an annular section is more complicated than that of rectangular section, and moreover the longitudinal reinforcements are uniformly distributed along the circumference, so it is not possible to directly calculate the cross-section bending moment. As adopted in the literature [23,24], herein the cross-section of a pile is discretized into a series of parallel strips, as shown in Fig. 10, and each strip is approximated as a rectangle, and the strain and stress are both assumed to be constant within each strip.

![Pile cross-section and Strain distribution](image)

Fig. 10. Strain distribution of pile cross section for general bending analysis.

In the calculation, the maximum concrete strain in the outermost layer of compression zone can be taken as the peak compressive strain $\varepsilon_{cm}$, which in the present piles was about 2800 $\mu\varepsilon$ according to the experimental results. The detailed calculation procedure can be summarized in the following steps [25] with consideration of the
combined reinforcement:

1) Select appropriate stress-strain models for concrete, prestressing strands and deformed steel bars, and determine the initial strain level in concrete and prestressing strands. Because the initial compressive stress of deformed steel bars is relatively small, this influence of the initial stress is neglected in the calculation.

2) Suppose an initial value of the neutral axis depth $h$ of cross section.

3) According to the plane section assumption and the difference between maximum compressive strain and initial strain of concrete in the upper edge strip, the strain increment of concrete can be calculated in the other strips, then the strain and stress are obtained. The compressive force of concrete $F_c$ can be calculated as:

$$F_c = \sum_{i=1}^{n} f_{ci} A_{ci}, \quad (5)$$

where $f_{ci}$ is the stress in $i$-th concrete strip (positive if compression); $A_{ci}$ is the area of $i$-th concrete strip.

4) According to the assumption that perfect bond between reinforcing bars and concrete exists, the strain of prestressing strands consists of initial strain and strain increment which is deemed to be identical with that of the surrounding concrete, and then the stress of prestressing strands is calculated from the constitutive model. Similarly, the strain and stress of deformed steel bars can be obtained. The tensile forces of prestressing strands $F_p$ and deformed steel bars $F_s$ can be calculated as:

$$F_p = \sum_{j=1}^{n} f_{pj} A_{pj}, \quad (6)$$

$$F_s = \sum_{k=1}^{n} f_{sk} A_{sk}, \quad (7)$$

where $f_{pj}$ is the stress of $j$-th prestressing strand (positive if compression); $A_{pj}$ is the area of $j$-th prestressing strand; $f_{sk}$ is the stress of $k$-th deformed steel bar (positive if compression); $A_{sk}$ is the area of $k$-th deformed steel bar.

5) Check the force equilibrium of section by satisfying that the absolute value of the sum of $F_c$, $F_p$ and $F_s$ is less than a certain allowable tolerance $\zeta$. 


(8) \[ F_i + F_p + F_r \leq \zeta. \]

6) If the force equilibrium is not satisfied, an iterative procedure would be used to determine the neutral axis depth \( h \) by repeating from step 2 to step 5.

7) When the force equilibrium is satisfied, the bending moment \( M \) of the cross section can be calculated as:

\[
M = \sum_{i=1}^{n_c} f_{ci} A_{ci} y_{ci} + \sum_{j=1}^{n_p} f_{pj} A_{pj} y_{pj} + \sum_{k=1}^{n_s} f_{sk} A_{sk} y_{sk},
\]

where \( y_{ci} \) is the distance from the centroid of \( i \)-th concrete strip to the central axis of pile section; \( y_{pj} \) is the distance from the centroid of \( j \)-th prestressing strand to the central axis of pile section; \( y_{sk} \) is the distance from the centroid of \( k \)-th deformed steel bar to the central axis of pile section.

4.2. Constitutive relationships

In order to perform the aforementioned theoretical calculation, it is necessary to define suitable constitutive relationships of concrete and reinforcing bars. There are many stress-strain models for concrete in the literature, and the model proposed in [26] is used here for high-strength concrete. The monotonic compressive stress-strain curve of concrete consists of three parts: initial elastic segment, damage-based plastic rising segment and damage-based plastic declining segment, as shown in Fig. 11(a), and the three parts can be expressed as:

\[
\sigma_c = \begin{cases} 
E_c \varepsilon_c & \varepsilon_c \in [0, \varepsilon_{c0}] \\
\sigma_{cm} \left[ 1 - \left( \frac{\varepsilon_c}{\varepsilon_{cm}} \right)^{\eta_c} \right] & \varepsilon_c \in (\varepsilon_{c0}, \varepsilon_{cm}] \\
\sigma_{cm} \left[ 1 - \left( \frac{\varepsilon_{cm} - \varepsilon_{cm}}{\varepsilon_{c0} - \varepsilon_{cm}} \right)^{\eta_c} \right] & \varepsilon_c \in (\varepsilon_{cm}, \varepsilon_{c0}] 
\end{cases},
\]

where the elastic modulus \( E_c \) is calculated by \( E_c = 10200 f_c^{0.3} \) [27] for high-strength concrete; the peak compressive stress \( \sigma_{cm} \) is actually the axial compressive strength of concrete; the value of the elastic compressive strain \( \varepsilon_{c0} \) is taken as 0.001; the peak compressive strain \( \varepsilon_{cm} \) is calculated by \( \varepsilon_{cm} = 0.0078 f_c^{0.3} \) [27]; the value of final
compressive strain $\varepsilon_{cf}$ is assumed to be 0.0045 according to the compression test; the empirical parameters $\eta_1$ and $\eta_2$ are related to the smoothness of the stress-strain curve, and their values are taken as 1.434 and 1.650, respectively.

As shown in Fig. 11(b), a piece-wise linear line is used to describe the monotonic tensile stress-strain curve of concrete, which is composed of four line segments. The rising part of the curve is a straight line and the descending part of curve consists of three segments, passing through three key points $(\varepsilon_{t1}, \sigma_{t1})$, $(\varepsilon_{t2}, \sigma_{t2})$, $(\varepsilon_{tf}, 0)$ in turn, and the coordinates of these key points can be determined as follows: $\sigma_{t1} = b_1 \sigma_{tm}$, $\sigma_{t2} = b_2 \sigma_{tm}$, $\varepsilon_{t1} = (\varepsilon_{tf} - \varepsilon_{tm}) / \epsilon_1$, $\varepsilon_{t2} = (\varepsilon_{tf} - \varepsilon_{tm}) / \epsilon_2$. Thus, the monotonic tensile stress-strain relationship can be expressed as:

$$\sigma_t = \begin{cases} E_c \varepsilon_t, & \varepsilon_t \in [0, \varepsilon_{tm}] \\ \frac{\sigma_{tm} - \sigma_{t1}}{\varepsilon_{tm} - \varepsilon_{t1}} (\varepsilon_t - \varepsilon_{tm}) + \sigma_{tm}, & \varepsilon_t \in (\varepsilon_{tm}, \varepsilon_{t1}] \\ \frac{\sigma_{t1} - \sigma_{t2}}{\varepsilon_{t1} - \varepsilon_{t2}} (\varepsilon_t - \varepsilon_{t1}) + \sigma_{t1}, & \varepsilon_t \in (\varepsilon_{t1}, \varepsilon_{t2}] \\ \frac{\sigma_{t2}}{\varepsilon_{t2} - \varepsilon_{tf}} (\varepsilon_t - \varepsilon_{t2}) + \sigma_{t2}, & \varepsilon_t \in (\varepsilon_{t2}, \varepsilon_{tf}] \end{cases}$$

(11)

where the peak tensile stress $\sigma_{tm}$ is the tensile strength of concrete; $\varepsilon_{tm}$ and $\varepsilon_{tf}$ are the peak tensile strain and final tensile strain, respectively; the empirical parameters $b_1$ and $b_2$ are mainly used to describe the softening behavior of concrete under tension, and their values can be taken as 0.33 and 0.1, respectively; $c_1$ and $c_2$ are the constants, and their values are taken as 10 and 1.5, respectively.

![Fig. 11. Concrete stress-strain curves: (a) uniaxial compression; and (b) uniaxial tension.](image)
For the reinforcement, there is no obvious yield plateau for prestressing strands, while there is commonly a long yield plateau for deformed steel bars. In order to describe stress-strain relationship in a unified way, the model proposed in [28] is employed here, as shown in Fig. 12, and it can be expressed as follows:

\[
\sigma = \begin{cases} 
E_s \varepsilon & \varepsilon \in [0, \varepsilon_y] \\
fy & \varepsilon \in (\varepsilon_y, k_1 \varepsilon_y] \\
k_4fy + \frac{E_s (1-k_4)}{\varepsilon_y (k_2-k_1)^2} (\varepsilon - k_2 \varepsilon_y)^2 & \varepsilon \in (k_1 \varepsilon_y, k_3 \varepsilon_y] 
\end{cases}
\]

(12)

where \(E_s, f_y, \) and \(\varepsilon_y\) are the modulus of elasticity, the yield strength and the yield strain, respectively; \(k_1\) is the ratio of the strain at the commencement of strain hardening to the yield strain, and for prestressing strands this value is taken as 1.0; \(k_2\) is the ratio of the peak strain to the yield strain; \(k_3\) is the ratio of the ultimate strain to the yield strain; \(k_4\) is the ratio of the peak stress to the yield stress.

Fig. 12. Stress-strain curve of reinforcing bars under uniaxial tension.

4.3. Simplified calculation method for flexural strength

The general calculation method presented in Section 4.1 usually needs to be solved by computer programming and iterative calculation. In this section, using an arc analogy with a polar coordinate system, a simplified formulation is proposed to directly calculate the bending moment of a pile cross-section.
In this simplified calculation method, the concrete area in the compression zone is assumed to be an annular segment, as shown in Fig. 13, and the equivalent rectangular compression zone depth is taken as the average height of the inner arc and the outer arc. Additionally, two thin steel circles are used to replace the discrete prestressing strands and deformed steel bars, accordingly the wall thickness of the two circles can be calculated as 

\[ \delta_p = \frac{A_p}{2\pi r_p} \]

and 

\[ \delta_s = \frac{A_s}{2\pi r_s} \]

observing the total steel area equivalence, respectively.

To simplify the calculation of the stress in the longitudinal reinforcement, the following assumptions are made in accordance with the results from the finite element analysis which will be described in Section 5.3. The stresses of the prestressing strands and deformed steel bars above and below the central axis of the pile cross-section both follow a linear distribution at the ultimate state, as shown in Fig. 13. The stress of the prestressing steel strand at the central axis of the pile cross-section just reaches the yield strength, and the stress of the bottom prestressing steel strand reaches the ultimate tensile strength. Correspondingly, the stress of the deformed steel bars below the central axis are assumed to have all reached the yield strength.

Based on the geometric relationship in Fig. 13, the depth of equivalent concrete compression zone \( h_e \) and the depth of neutral axis \( h \) can be determined as follows:

\[ h_e = r_i - r_c \cos \pi \alpha, \]  
(13)

\[ h = h_e / \beta_i, \]  
(14)
where the average radius $r_c$ of the cross-section is usually very close to the radius $r_p$ of the distributed circle of the prestressing strands, and for simplicity take $r_c = r_p$. $\beta_1$ is the ratio of depth of equivalent rectangular compression zone to the depth of neutral axis, and its value can be taken as 0.65 [29].

Above the central axis, the stress of prestressing strands $\sigma_p$ and deformed steel bars $\sigma_s$ at any height $y = r_p \sin \theta$ can be determined as follows:

$$\sigma_p = \frac{r_i - h - y}{r_i - h} \left( f_{py} - \sigma_{p0} \right) + \sigma_{p0}, \quad (15)$$

$$\sigma_s = \frac{r_i - h - y}{r_i - h} f_{sy}, \quad (16)$$

where $f_{py}$ and $\sigma_{p0}$ are the yield strength and the initial stress of prestressing strands, respectively; $f_{sy}$ is the yield strength of deformed steel bars.

Below the central axis, the stress of prestressing strands and deformed steel bars can be determined as follows:

$$\sigma_p = f_{py} - \frac{y}{r_p} \left( f_{py} - f_{py} \right), \quad (17)$$

$$\sigma_s = f_{sy}, \quad (18)$$

where $f_{py}$ is the tensile strength of prestressing strands.

The tensile forces of prestressing strands $F_p$ and deformed steel bars $F_s$ can then be calculated as:

$$F_p = 2 \int_{-\theta/2}^{\theta/2} \sigma_p r_p d\theta = f_{py} A_p + \frac{f_{py} - f_{py}}{\pi} A_p - \frac{\beta r_p (f_{py} - \sigma_{p0})}{\pi \left( (\beta - 1) r_i + r_p \cos \pi \alpha \right)} A_p, \quad (19)$$

$$F_s = 2 \int_{-\theta/2}^{\theta/2} \sigma_s r d\theta = f_{sy} A_s - \frac{\beta r_p f_{sy}}{\pi \left( (\beta - 1) r_i + r_p \cos \pi \alpha \right)} A_s. \quad (20)$$

On the other hand, the compressive force of concrete $F_c$ can be determined as follows:

$$F_c = a_1 \alpha A f_c, \quad (21)$$

where $\alpha_1$ is the ratio of the stress of the equivalent rectangular compression zone to the axial compressive strength of concrete, and its value is taken as 0.85 [29]; $A$ is the cross-section area of the pile. The value of $\alpha$ can be solved
according to the following force equilibrium equation by an iterative calculation,

\[
\alpha \alpha f_c = f_{py} A_p + \frac{f_{ps} - f_{py}}{\pi} A_p - \beta \frac{r_p}{\pi} \left( f_{py} - \sigma_{p0} \right) A_p + f_{sy} A - \frac{\beta r_p f_{sy}}{\pi} \left( r_p - 1 \right) A . \tag{22}
\]

The total bending moment of the pile cross-section is the sum of the moments resulting from the longitudinal reinforcements and the compression zone concrete, which are calculated by multiplying their axial forces \( F_p \), \( F_s \) and \( F_c \) by the respective distances to the central axis \( x \). The resultant moment of the cross-section can be determined as:

\[
M = \alpha f_c \int_{Y/2}^{Y/2} r \sin \theta \frac{A_p}{2\pi} d\theta + 2 \left[ \int_{-\pi/2}^{\pi/2} -\left( \sigma_p - \sigma_c \right) r_p d\theta \right] \sin \theta
\]

\[
= \frac{\alpha f_c \sin \pi \alpha}{4} + \frac{\left( f_{py} - f_{ps} \right) A_p}{4} + \frac{\beta r_p^2 \left[ f_{py} - \sigma_{p0} \right] A_p + f_{sy} A_j}{4 \left( r_p - 1 \right) A} \tag{23}
\]

### 4.4. Simplified calculation method for flexural stiffness

Flexural stiffness is an important design parameter for different structural members [30] and the initial tangent stiffness and the secant stiffness at the ultimate load are two key points. Before cracking of concrete, the whole section of a pile works like an elastic body. For prestressed concrete members, the initial internal stresses remain in balance between prestressing tendons and concrete. Thus, when calculating the initial tangent stiffness of a prestressed concrete member, it can be considered as a normal non-prestressed member, so the initial tangent stiffness of piles can be calculated by the following equation:

\[
E I_i = E_c I_c + E_p I_p + E_s I_s , \tag{24}
\]

where \( E_c \), \( E_p \) and \( E_s \) are modulus of elasticity of concrete, prestressing strands and deformed steel bars, respectively; \( I_c \) is moment of inertia of gross concrete section, neglecting prestressing strands and deformed steel bars; \( I_p \) and \( I_s \) are moments of inertia of prestressing strands and deformed steel bars about the neutral axis of the section.

Herein the gross concrete section is an annular section with a relatively large thickness-to-diameter ratio. For the reinforcing steel, as mentioned earlier, the discrete prestressing strands and deformed steel bars are replaced by two thin steel circles. Thus, \( I_c \), \( I_p \) and \( I_s \) can be calculated as:
The flexural stiffness of the piles decreases after the concrete cracks, and the secant stiffness is usually used to
describe the stiffness. The calculation method for the secant stiffness at the ultimate load is proposed as follows.

According to the simplified calculation method for flexural strength presented Section 4.3, the depth of neutral axis
$h$ is obtained, from which the curvature of the section can be determined as:

\[ \varphi = \frac{f_{py}}{E_p} \left( \frac{1}{h} - \frac{1}{h_g} \right), \tag{28} \]

where $f_{py}$ and $\varepsilon_{p0}$ are the yield strength and the initial strain of prestressing strands, respectively. The secant stiffness
can then be determined by:

\[ EI_{sec} = \frac{M}{\varphi}. \tag{29} \]

4.5. Theoretical calculations and comparison with experimental results

The above described general and simplified methods are applied to calculate the ultimate bending moments of
the tested piles. Table 4 presents a comparison of the calculated results using the two theoretical methods with the
experimental results, in which $h_g$ and $h_s$ represent the depth of the neutral axis of the cross-section calculated by two
methods. It can be seen that the depth of neutral axis and the ultimate bending moments obtained by the general and
simplified calculation methods match well with each other, indicating that the simplified calculation method using
the arc analogy is acceptable.

Compared with the experimental results, the calculated results from two theoretical methods are always lower
and the deviation is within 15%. In a word, the general and simplified calculation methods give the conservative
predictions on the flexural strength of tested piles and may be used for quick evaluation of the flexural strength of prestressed concrete piles such as at the stage of preliminary design.

Table 4 Comparison between theoretical and experimental results.

<table>
<thead>
<tr>
<th>Pile type</th>
<th>Experiment</th>
<th>General calculation method</th>
<th>Simplified calculation method</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$M_{u,e}$ (kN·m)</td>
<td>$h_e$ (mm)</td>
<td>$M_{u,g}$ (kN·m)</td>
</tr>
<tr>
<td>PSC400</td>
<td>166.5</td>
<td>74.0</td>
<td>147.5</td>
</tr>
<tr>
<td>PSRC400</td>
<td>267.8</td>
<td>96.0</td>
<td>234.5</td>
</tr>
<tr>
<td>PSC500</td>
<td>349.8</td>
<td>91.0</td>
<td>291.1</td>
</tr>
<tr>
<td>PSRC500</td>
<td>543.8</td>
<td>121.0</td>
<td>465.0</td>
</tr>
<tr>
<td>PSC600</td>
<td>540.0</td>
<td>101.4</td>
<td>457.0</td>
</tr>
<tr>
<td>PSRC600</td>
<td>812.5</td>
<td>135.6</td>
<td>736.5</td>
</tr>
</tbody>
</table>

Table 5 shows the experimental and theoretical results of the flexural stiffnesses of tested piles. The ratio of the secant stiffness at the ultimate load to the initial tangent stiffness is defined as the stiffness degradation ratio. It can be seen that the initial tangent stiffnesses obtained by the simplified calculation method match well with the experimental results. The initial tangent stiffness of other PSRC piles is slightly larger than that of PSC piles with the same diameter, which is consistent with the experimental observations. This is because the main difference in the initial tangent stiffness between these two types of piles comes from the stiffness provided by the deformed steel bars, which constitutes only a small proportion of the whole stiffness. However, the secant stiffness of PSRC piles is markedly larger than that of PSC piles under a similar deflection, and PSC piles show more pronounced stiffness degradation, indicating that the PSRC piles with additional deformed steel bars maintain better flexural stiffness at the ultimate stage.

Table 5 Experimental and theoretical results of flexural stiffnesses.

<table>
<thead>
<tr>
<th>Pile type</th>
<th>$E_l_{c,t}$ (kN·m²)</th>
<th>$E_l_{c}$ (kN·m²)</th>
<th>$E_l_{c}/E_l_{t}$</th>
<th>$E_l_{c,PSRC}/E_l_{c,t}$</th>
<th>$E_l_{sec,c}$ (kN·m²)</th>
<th>$E_l_{sec,c,PSRC}/E_l_{sec,c}$</th>
<th>$E_l_{sec,c}/E_l_{c}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PSC400</td>
<td>54016.1</td>
<td>53730.3</td>
<td>0.99</td>
<td>1.06</td>
<td>4248.5</td>
<td>1.46</td>
<td>0.08</td>
</tr>
<tr>
<td>PSRC400</td>
<td>57399.5</td>
<td>56918.6</td>
<td>0.99</td>
<td>1.04</td>
<td>6220.2</td>
<td>1.06</td>
<td>0.11</td>
</tr>
<tr>
<td>PSC500</td>
<td>116166.6</td>
<td>120802.0</td>
<td>0.94</td>
<td>1.07</td>
<td>11619.8</td>
<td>1.41</td>
<td>0.09</td>
</tr>
<tr>
<td>PSRC500</td>
<td>136887.7</td>
<td>129507.7</td>
<td>0.95</td>
<td>1.07</td>
<td>16410.1</td>
<td>1.41</td>
<td>0.12</td>
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<tr>
<td>PSC600</td>
<td>234472.5</td>
<td>241490.0</td>
<td>1.03</td>
<td>1.07</td>
<td>24194.1</td>
<td>1.43</td>
<td>0.10</td>
</tr>
<tr>
<td>PSRC600</td>
<td>246461.7</td>
<td>258700.1</td>
<td>1.05</td>
<td>1.07</td>
<td>34620.1</td>
<td>1.43</td>
<td>0.13</td>
</tr>
</tbody>
</table>
5. Finite element analysis

In this section, a detailed finite element (FE) analysis of the tested piles is presented. A suitable and validated FE model can be used to further identify the damage process and perform more extended parametric investigations.

5.1. Finite element model

The finite element analysis is conducted to simulate the flexural performance of tested piles using the software ABAQUS. As shown in Fig. 14, the geometric dimensions and loading conditions in the finite element model are the same as the actual tests. Two elastic loading blocks and two elastic supporting blocks are set up to prevent abnormal stress concentration at the loading and supporting points. An eight-node three-dimensional solid element with reduced integration (C3D8R) is used to model the concrete and a two-node three-dimensional truss element (T3D2) is chosen to model the reinforcing bars, which are embedded into the concrete. The initial compressive prestress of the concrete is realized by equivalently decreasing the temperature of the prestressing strands. For the lateral load, a displacement loading scheme is adopted to ensure the convergence of the solution process.

After a sensitivity analysis and convergence trial of mesh size of the pile models, the wall of the pile cross-section is discretized into 4 layers in the radial direction and 36 equal parts along the circumferential direction. The mesh density in the length direction gradually becomes coarser from midspan segment to two sides of the overhanging segments, and the mesh length is 25 mm, 50 mm and 80 mm in turn.
5.2. Material parameters

The concrete damaged plasticity (CDP) model available in ABAQUS is used to model the high-strength concrete in the FE model of the piles. CDP characterizes the inelastic behavior of the concrete based on the concept of isotropic elastic damage combined with isotropic tensile and compressive plasticity. The main material parameters are defined by specifying the uniaxial compressive and tensile stress-strain curves, and in the present model these curves follow the material test results described in Section 4.2. Other parameters of the CDP model are determined in accordance with relevant studies in the literature [31,32] and tailored trial calculations, as follows: Poisson’s ratio \( \nu = 0.2 \), dilation angle \( \psi = 45^\circ \), eccentricity \( \varepsilon = 0.1 \), stress ratio \( \sigma_{bo} / \sigma_{co} = 1.16 \), shape factor \( K = 0.6667 \), and the viscosity parameter \( \mu = 0.001 \).

The standard plastic model defined in ABAQUS is applied to simulate the mechanical properties of reinforcing bars. A thermal expansion coefficient is defined so as to realize the initial compressive pre-stress of concrete, as mentioned earlier. The stress-strain curves of the reinforcing bars described in Section 4.2 is used to define the basic constitutive relationships. The detailed parameter values of these stress-strain curves for reinforcing bars are presented in Table 6.

<table>
<thead>
<tr>
<th>Reinforcing bar</th>
<th>( E_s ) (GPa)</th>
<th>( f_y ) (MPa)</th>
<th>( \varepsilon_y )</th>
<th>( k_1 )</th>
<th>( k_2 )</th>
<th>( k_3 )</th>
<th>( k_4 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Phi^s 11.1 )</td>
<td>195</td>
<td>1725</td>
<td>0.0088</td>
<td>1.0</td>
<td>7.56</td>
<td>7.83</td>
<td>1.11</td>
</tr>
<tr>
<td>( \Phi 16 )</td>
<td>191</td>
<td>503</td>
<td>0.0026</td>
<td>9.2</td>
<td>44.2</td>
<td>52.5</td>
<td>1.22</td>
</tr>
<tr>
<td>( \Phi^b 4/5 )</td>
<td>199</td>
<td>477</td>
<td>0.0024</td>
<td>1.0</td>
<td>10.8</td>
<td>12.1</td>
<td>1.15</td>
</tr>
</tbody>
</table>

5.3. Numerical results

With the above finite element model and the material parameter settings, the bending responses of the PSC and PSRC piles have been analysed. Fig. 15 illustrates the comparison of load-deflection curves at the midspan between numerical and experimental results. It can be seen that the development trend of the simulated curves is consistent with the experimental curves. The simulated curves match well the experimental curves before cracking, and still...
show good consistency after cracking until failure. As in the experiment, the bending stiffness of the simulated curves decreases rapidly; however, the peak loads are generally lower than those of experimental curves, and this could be due to a smaller concrete compressive stress in the FE model than actually developed in the test specimen.

![Comparison of load-deflection curves between numerical and experimental results.](image)

**Fig. 15.** Comparison of load-deflection curves between numerical and experimental results.

Table 7 presents a comparison of some detailed flexural performance parameters for the six tested piles between numerical and experimental results. It can be observed that the cracking moment $M_{cr,n}$ and the ultimate bending moment $M_{u,n}$ obtained from the numerical simulation show good consistency with the experimental results; although most of the simulated values are lower than the experimental results, the differences are within a margin of 10%.

The maximum midspan deflections $f_{u,n}$ obtained by the numerical simulation also match well with the experimental results, with a difference margin about 10%. These comparisons demonstrate that the finite element analysis adopted in this study can reasonably represent the flexural performance of PSC and PSRC piles.

**Table 7 Comparison of flexural performance between numerical and experimental results.**

<table>
<thead>
<tr>
<th>Pile type</th>
<th>$M_{cr,n}$ (kN·m)</th>
<th>$M_{cr,e}$ (kN·m)</th>
<th>$M_{cr,n}/M_{cr,e}$</th>
<th>$M_{u,n}$ (kN·m)</th>
<th>$M_{u,e}$ (kN·m)</th>
<th>$M_{u,n}/M_{u,e}$</th>
<th>$f_{u,n}$ (mm)</th>
<th>$f_{u,e}$ (mm)</th>
<th>$f_{u,n}/f_{u,e}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PSC400</td>
<td>89.6</td>
<td>73.8</td>
<td>1.21</td>
<td>158.4</td>
<td>166.5</td>
<td>0.95</td>
<td>59.71</td>
<td>63.22</td>
<td>0.94</td>
</tr>
<tr>
<td>PSRC400</td>
<td>94.5</td>
<td>86.3</td>
<td>1.10</td>
<td>243.8</td>
<td>267.8</td>
<td>0.91</td>
<td>54.11</td>
<td>61.28</td>
<td>0.88</td>
</tr>
<tr>
<td>PSC500</td>
<td>174.6</td>
<td>170.1</td>
<td>1.03</td>
<td>306.6</td>
<td>349.8</td>
<td>0.88</td>
<td>48.95</td>
<td>52.02</td>
<td>0.94</td>
</tr>
<tr>
<td>PSRC500</td>
<td>181.4</td>
<td>172.3</td>
<td>1.05</td>
<td>485.6</td>
<td>543.8</td>
<td>0.89</td>
<td>45.14</td>
<td>47.33</td>
<td>0.95</td>
</tr>
<tr>
<td>PSC600</td>
<td>274.3</td>
<td>294.5</td>
<td>0.93</td>
<td>482.3</td>
<td>540.0</td>
<td>0.89</td>
<td>41.20</td>
<td>38.63</td>
<td>1.07</td>
</tr>
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<td>PSRC600</td>
<td>284.9</td>
<td>297.6</td>
<td>0.96</td>
<td>754.5</td>
<td>812.5</td>
<td>0.93</td>
<td>37.26</td>
<td>42.51</td>
<td>0.88</td>
</tr>
</tbody>
</table>

The strain development of longitudinal reinforcements in the whole loading process, which is difficult to be
measured by the experiment due to the centrifugal production process, has been further studied by the finite element analysis. Fig. 16 shows the strain distribution of the longitudinal reinforcements over the depth of the midspan cross-section at some key loading stages, including concrete cracking, yield of longitudinal reinforcements and the final failure. It can be found some useful results from the images.
Fig. 16. Strain distribution of longitudinal reinforcements over the depth of cross-section in the midspan: (a) PSC400; (b) PSRC400-PT; (c) PSC500; (d) PSRC500-PT; (e) PSC600; (f) PSRC600-PT; (g) PSRC400-NPT; (h) PSRC500-NPT; and (i) PSRC600-NPT.

For PSC pile test specimens, before the yield of the bottom prestressing strand, the strain of prestressing strands over the depth of the cross-section is linearly distributed and this conforms to the assumption of plane section. All the prestressing strands below the central axis of the pile cross-section reach the yield strength, and the strain distribution in the prestressing strands is approximately linear. For PSRC pile test specimens, on the other hand, the strain of prestressing strands is relatively low, and the number of the prestressing strands entering the yield stage is smaller. The strain distribution of prestressing strands approximately satisfies the assumption of plane section until the failure of the pile test specimens. The strain distribution of deformed steel bars is also similar to that of prestressing strands. In addition, it can be found that the non-prestressing tendons tend to yield before the
Fig. 17. Stress distribution of longitudinal reinforcements in the midspan section at failure.

Fig. 17 shows the stress distributions of the longitudinal reinforcements at the failure stage. It can be observed that the stress of the prestressing strands in PSC pile specimens is approximately a two-segment linear distribution over the depth of the cross-section. The turning point is above the central axis of the cross-section, and the stress is close to the yield strength. The prestressing strands at and below the central axis all exceed the yield strength, and the bottom prestressing strand approach the tensile strength. While in PSRC pile specimens, because of the arrangement of the non-prestressing bars, the stress in the prestressing strands is relatively lower than that of PSC pile specimens. The stress distribution of the prestressing strands is also approximately a two-segment linear distribution, but the turning point is below the central axis of the cross-section. Moreover, the stress distribution of
the non-prestressing tendons in PSRC pile specimens is similar to that of the prestressing strands, and the non-prestressing tendons below the central axis all reach the yield strength.

In the CDP concrete material model, it is assumed that the cracks occur when the maximum principal plastic strain of concrete is positive and the direction of the cracks can be visualized through the maximum principal plastic strain [32]. Thus, the maximum principal plastic strain of concrete can be used to describe the development of cracks in the piles. Fig. 18 shows a comparison of crack distribution between numerical and experimental results when the ultimate flexural capacity is reached. It can be seen that the development of cracks from the numerical model, including the density and spread of the crack region, are in good agreement with the experimental results.

![Fig. 18. Crack distribution of numerical and experimental results for test specimens: (a) PSC400; (b) PSRC400; (c) PSC500; (d) PSRC500; (e) PSC600; and (f) PSRC600.](image)

### 6. Parametric analysis

Based on the verified FE model, parametric analyses are performed to investigate the flexural performance of PSRC piles in this section. The influences of three parameters are studied, including the prestressing level of the prestressing strands, prestressing strand ratio and deformed steel bar ratio.

#### 6.1. Prestressing level of prestressing strands

The prestressing level of the prestressing strands determines the compressive pre-stress of concrete and it is considered as an important design parameter. To evaluate the influence of the prestressing level of the prestressing
strands on the flexural strength of PSRC piles, the tensioning control stresses of the prestressing strands in the
models are selected as 35%, 50% and 70% of the nominal tensile strength of 1860 MPa, respectively.

The load-deflection curves of three size-groups of PSRC piles with different prestressing levels are plotted in Fig.
19. It can be seen that the prestressing level has little impact on the flexural strength of PSRC piles. However, the
deflection at the maximum load increases by 18.1%, 17.0% and 23.6%, as the prestressing level decreases from 70%
to 35% for PSRC400, PSRC500 and PSRC600 piles, respectively. However, it is worth noting that there is a decrease
in the anti-cracking capacity with a decrease of the prestressing level in the prestressing strands.

![Load-deflection curves of PSRC piles with different prestressing levels.](image)

6.2. Prestressing strand ratio

As mentioned in the review in Section 1, the prestressing strand ratio has a certain effect on the characteristic
behavior of concrete as the compressive pre-stress of concrete is determined by the amount of the prestressing
strands with the design control prestressing level. Three groups of PSRC pile models with similar layout of
prestressing strands but in diameters of 11.1 mm, 12.7 mm and 15.2 mm are investigated. The corresponding
prestressing strand ratios are 0.57%, 0.76%, 1.08% (PSRC400 group); 0.65%, 0.86%, 1.23% (PSRC500 group);
and 0.61%, 0.82%, 1.16% (PSRC600 group), respectively. Fig. 20 shows the load-deflection curves of the three
groups of PSRC piles with different prestressing strand ratios. As the prestressing strands ratio increases, the flexural
strengths of PSRC400, PSRC500 and PSRC600 piles increase by 31.1%, 31.3% and 29.4%, respectively, while the
ultimate deflections decrease by 9.1%, 16.6% and 19.7%. The decrease in the deformation capacity is mainly due
to a higher compressive pre-stress of concrete, which causes earlier crushing of concrete in the compression zone.

![Graph showing load-deflection curves of PSRC piles with different prestressing strand ratios.]

**Fig. 20.** Load-deflection curves of PSRC piles with different prestressing strand ratios.

### 6.3. Deformed steel bar ratio

To study the influence of deformed steel bar ratio on the flexural performance of PSRC piles, the deformed steel
bars of the same arrangement but with varying diameters of 12 mm, 14 mm and 16 mm, are numerically investigated
in this section. The corresponding reinforcement ratios of deformed steel bars are 0.87%, 1.18%, 1.55% (PSRC400
group); 0.99%, 1.35%, 1.76% (PSRC500 group), and 0.94%, 1.27%, 1.66% (PSRC group), respectively.

Fig. 21 shows the load-deflection curves of PSRC piles with the above different deformed steel bar ratios. As the
deformed steel bars ratio increases, the ultimate deflections of PSRC400, PSRC500 and PSRC600 piles decrease
by 3.0%, 3.8% and 1.2%, while the flexural strengths increase almost equally by about 18%. As with increasing the
prestressing strand ratio, increasing deformed steel bar ratio also improves the flexural strength of PSRC piles.

However, it has a minimum effect on the deformation capacity, and this is because increasing the amount of
deformed steel bars has an insignificant influence on the initial compressive pre-stress of concrete. From this point
of view, increasing the amount of deformed steel reinforcement may be regarded as a more favorable option when
an increase in the load capacity is required.

![Load-deflection curves of PSRC piles with different deformed steel bar ratios.](image)

**Fig. 21.** Load-deflection curves of PSRC piles with different deformed steel bar ratios.

7. Conclusions

Pretensioned centrifugal spun concrete piles with steel strands (PSC piles) and with a combination of steel strands and deformed rebars (PSRC piles) are developed to enable application of pretensioned spun high strength concrete piles in seismic areas. To evaluate the comparative flexural performance of the PSRC piles with respect to the PSC piles, three pairs of full-scale pile test specimens with different pile diameters have been investigated experimentally. In association with the experiments, theoretical analysis and numerical simulation of tested piles have also been carried out. The similarities and differences in terms of the crack resistance, flexural strength and stiffness, deformation capacity and crack distributions between the PSC and PSRC piles are compared and analysed. And then parametric analyses are also conducted to study the flexural performance of PSRC piles. The main conclusions can be summarized as follows:

1. Due to the addition of the non-prestressing deformed rebars, the ratio of longitudinal reinforcement of PSRC piles was greatly enhanced, and as a result the ultimate bending moments of PSRC piles were 61%, 55% and 50% higher than those of PSC piles, respectively. The initial concrete compressive pre-stress of PSRC piles was slightly smaller than that of the PSC piles with the same pile diameter.
2. The cracking moments in both piles were differed only slightly, and so were the initial bending stiffnesses. However, the crack distributions in the PSRC piles became more favourable with an extended crack region, denser cracks and generally reduced crack widths. This demonstrated that the arrangement of non-prestressing rebars can effectively improve the cracking behavior and control the crack width.

3. While the addition of non-prestressing rebars significantly improves the hardening stiffness of the piles after cracking and the ultimate bending strength, the PSRC piles maintained good deformation capacity, and the maximum midspan deflection at failure was close to the corresponding PSC piles.

4. The failure modes of PSC and PSRC piles were both governed by gradual concrete crushing in the compression zone, indicating that the compressive performance of high strength concrete was fully mobilised.

5. The two theoretical methods implemented for the calculation of the bending strength of the piles produced comparable results, indicating that the simplified method is effective for the bending strength calculation. In general, both methods tend to yield conservative predictions on the flexural strength of tested piles, and the deviation is within 15%.

6. Using a general-purpose finite element method (with ABAQUS herein), the flexural performance of the tested piles can be simulated satisfactorily. The simulated load-deflection curves are consistent with the experimental counterparts, and the simulated values of cracking moment, ultimate bending moment and maximum midspan deflection for test specimens also agree well with the experimental results. Additionally, the maximum principal plastic strain of concrete can be used to describe the development of cracks in the piles with reasonable accuracy. Overall, the finite element model can reliably reflect the flexural performance of piles.

7. Parametric analyses show that the prestressing level of prestressing strands has limited influence on the flexural strength of PSRC piles. Although the deformation capacity of piles tends to improve by reducing the prestressing level of prestressing strands, this may not be a useful approach as reducing the prestressing level
generally reduces the crack resistance of the piles. The flexural strength of PSRC piles can be improved with the increase of the prestressing strand ratio and the deformed steel bar ratio. However, increasing the prestressing strand ratio is shown to reduce the deformation capacity of piles, while this negative effect appears to be negligible when increasing the deformed steel bar ratio. From this point of view, increasing the amount of deformed steel bars may be regarded as a more favorable approach when an increase of the load capacity in PSRC piles is required.

In future research, for general application of PSRC piles in high intensity seismic regions, the cyclic behavior of PSRC piles under combined lateral and axial loading will need to be evaluated. Studies in this direction are underway and the results will be reported subsequently.

**Declaration of Competing Interest**

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

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