

Article

Mechanical Behavior of a Prefabricated Joint for Small-Section Prestressed Concrete Piles Under Tension, Bending and Shear: Full-Scale Tests

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Abstract

Prefabricated pile-cap joints have been applied in concrete pile foundations in waterway revetment structures for decades. However, the mechanical performance of such joints for small-section (≤ 400 mm) prestressed piles has not been systematically validated, which hinders their application in civil and waterway engineering. This study evaluates this joint system through full-scale tests on six identical 400 mm $\times$ 400 mm prestressed concrete pile-cap specimens (C60 pile, C30 joint concrete, HRB400 anchoring bars threaded into embedded steel sleeves) under monotonic tension, bending, and shear, with strain gauges monitoring the response. The results show that the joint sustained a tensile load of 1400 kN, which exceeds the pile's design value by 52%, a bending moment of 277 kN·m, which is 32% higher than the pile's ultimate capacity, and a shear force of 680 kN, which reaches more than 2.5 times the design shear, without any occurrence of joint failure. These findings provide the first comprehensive experimental evidence that this prefabricated joint system can effectively transfer tensile, bending, and shear loads in small-section prestressed piles, thereby supporting its safe application in green, industrialized engineering.

Keywords: prefabricated pile-cap joint; small-section prestressed concrete pile; full-scale test; tensile-bending-shear loading

1. Introduction

Precast concrete construction, characterized by high production efficiency, shortened construction periods, minimized on-site wet operations, enhanced quality control, and significant low-carbon benefits, has become a critical pathway for promoting industrial transformation and sustainable development within the global construction industry [1–3]. Unlike traditional cast-in-place concrete structures, the overall mechanical behavior of precast concrete systems is critically dependent on the joints between prefabricated components [4,5]. As the core load-transfer interface in frame structures, beam–column joints directly govern structural safety under various loading conditions and have therefore been a central research priority in both academia and engineering practice [6–8]. In recent years, with the expanding application of prefabricated construction in logistics warehousing, industrial plants, underground stations, and waterway engineering, higher demands

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have been placed on the mechanical performance of precast connections under complex loading conditions [1,5].

Extensive experimental and theoretical investigations have been conducted on various types of precast beam–column joints. Based on whether cast-in-place concrete is employed, precast connections can be broadly classified into wet connections and dry connections; from the perspectives of load-transfer mechanisms and detailing characteristics, they can be further subdivided into four categories: wet connections with reinforcement splices (WJ-R), wet connections with steel profiles (WJ-S), dry connections with steel profiles (DJ-S), and dry connections with prestressing tendons (DJ-P) [5,9]. Among wet connections, those with reinforcement splices achieve load transfer through lap splicing or mechanical splicing of reinforcing bars within the joint region followed by concrete casting. This approach offers mature construction techniques and good structural integrity, but suffers from reinforcement congestion and difficulties in ensuring casting quality [10]. Wet connections with steel profiles embed structural steel sections in precast members and form the connection through post-cast concrete in the joint region; they provide high load-carrying capacity and stiffness, yet involve complex detailing and higher costs. Among dry connections, those with steel profiles connect precast members through bolted or welded joints between embedded steel components, offering rapid construction and reliable quality control, but with relatively limited ductility and energy dissipation capacity [11–13]. Dry connections with prestressing tendons utilize unbonded post-tensioned tendons to provide self-centering capability with minimal residual deformation; however, they exhibit insufficient energy dissipation and typically require supplemental damping devices [14,15].

Among the various joint types investigated, hybrid connections with embedded steel connectors have attracted considerable attention due to their construction convenience and reliable structural performance [11,12]. Yuan et al. [16] employed pressed sleeve connections to splice longitudinal reinforcement in precast beam–column joints using recycled aggregate concrete, demonstrating that the joint’s hysteretic behavior, load-carrying capacity, and stiffness were comparable to those of cast-in-place counterparts. Fan et al. [17] proposed a dry joint with replaceable multi-slit energy dissipation devices, concentrating plastic deformation and damage within rapidly replaceable components, thereby ensuring that precast beams and columns remain elastic under major earthquakes while enabling post-earthquake rapid repair. To further enhance the durability and strength of the joint region, researchers have introduced Ultra-High Performance Concrete (UHPC) into precast connections: Tang et al. [18] employed prefabricated U-shaped UHPC shells at precast beam ends for local reinforcement, demonstrating that this detailing effectively mitigated concrete damage at the lower portion of beam ends, with specimens incorporating 600 mm-long UHPC shells achieving a ductility of 4.87. Xue et al. [19] utilized UHPC as the post-cast material in beam–column joint cores, significantly reducing the required lap splice length of reinforcing bars. To alleviate reinforcement congestion—a long-standing challenge constraining construction efficiency in precast structures—researchers have also developed alternative solutions including mechanical connectors, notched connections with post-installed connecting bars, and unbonded post-tensioned rocking systems [16]. Chen et al. [20] proposed a new detailing strategy with pre-embedded rebar couplers in precast column joint regions; full-scale quasi-static tests verified that this connection achieved seismic performance comparable to cast-in-place joints. Ghayeb et al. [21] systematically investigated precast beam–column joints with hybrid steel connectors, demonstrating that the precast specimens exhibited superior strength and energy dissipation compared to cast-in-place counterparts. Zhang and Zhang [22] proposed an exterior precast beam–column joint with replaceable dry-connected devices and fully pre-

fabricated slabs; test results indicated excellent load-carrying and energy dissipation capacities, with damage concentrated in the replaceable connection components. Wu et al. [23] conducted systematic tests on semi-rigid joints with bolted angle dissipating connections, revealing the influence of key parameters including angle thickness, bolt diameter, and column bolt gauge on hysteretic performance. In the domain of self-centering joints, Zhu et al. [14] drew inspiration from the mortise–tenon connection concept in traditional Chinese timber structures and proposed a novel precast concrete beam–column joint; numerical analyses demonstrated that its load-carrying capacity exceeded that of comparable cast-in-place joints. These studies collectively indicate that properly detailed precast connections can achieve structural performance comparable to or even exceeding that of cast-in-place construction under cyclic bending and shear loading [22–26].

In UHPC connection technology, Gu et al. [26] systematically investigated the seismic performance of precast reinforced-concrete columns with UHPC connections, demonstrating that UHPC connections enable precast columns to achieve seismic performance comparable to cast-in-place columns while significantly reducing lap splice length. Zhou et al. [27] further proposed two novel UHPC connection configurations; full-scale quasi-static tests verified that both UHPC grouted connections and UHPC cast-in-place layer connections enabled precast bridge piers to achieve seismic performance consistent with cast-in-place counterparts. These advances indicate that the integration of high-performance materials with innovative detailing is emerging as an effective approach to enhance the overall performance of precast connections.

However, the aforementioned studies have predominantly focused on beam–column joints in building structures and pier connections in bridge engineering, whose loading mechanisms and performance requirements differ fundamentally from those of pile-cap joints in waterway engineering. While building and bridge connections primarily resist combined bending and shear actions, pile-cap joints in hydraulic structures must simultaneously accommodate uplift tension, eccentric bending, and horizontal shear, either independently or in combination. In hydraulic structures such as revetments, wharves, and breakwaters, pile foundations are subjected to sustained uplift tension from extreme storm-surge water level differentials, large eccentric bending moments from combined lateral soil-water pressure and mooring forces, and substantial shear forces induced by ship berthing impact and long-term current drag [28]. In engineering practice, these load types are typically designed and verified as independent dominant loading conditions—fundamentally different from the combined bending-shear-dominated seismic design philosophy in building frames—as failure under any single load action may directly trigger progressive structural collapse [29,30].

Despite significant advances in precast connection technology in building and bridge engineering, systematic understanding of precast pile-cap joint performance under these three independent loads in waterway engineering remains critically insufficient. Xiao [31] conducted full-scale tests on precast prestressed concrete pile-to-cast-in-place pile-cap connections, demonstrating that despite relatively shallow embedment depths, the connections could still provide considerable flexural capacity. Yang and Wang [30] experimentally and numerically investigated the behavior of prestressed high-strength concrete pipe pile-to-pile cap connections under combined axial force, bending moment, and shear. However, these studies primarily focused on the performance of pipe piles under cyclic lateral loading combined with axial force, and did not systematically cover the full spectrum of behavior of small-section prestressed square piles under independent tension, bending, and shear actions. Engineering seismic damage and accident investigations have unequivocally demonstrated that the uniaxial load-carrying capacity of pile-cap joints is a critical safety determinant for hydraulic structures [32].

For small-section prestressed square piles, the limited cross-sectional dimensions (side length ≤ 400 mm) impose severe spatial constraints on reinforcement detailing [10], further compounding the difficulty of connection design and making precast connection technology a key factor in overcoming construction efficiency and quality bottlenecks. However, current code systems have not effectively addressed the connection issue for small-section prestressed concrete square piles. Neither the national design drawing set 20G361 [33], the industry standards JTS 151 [34] and JTS 167-4 [35], nor the national codes GB 55008 [36] and GB 50010 [37] provide explicit design provisions for connections between small-section prestressed square piles and pile caps under independent tension, bending, and shear actions. The traditional “chip-off-and-anchor” cast-in-place method is fundamentally infeasible for this pile type: prestress release upon removal of the top concrete instantaneously triggers end splitting and strand retraction [38,39], while on-site welding would damage the prestressing force of high-strength tendons due to hydrogen embrittlement issues [40]. Although preliminary conceptual schemes utilizing UHPC grouting [26,27] or sleeve connections [7,13,28] have been proposed in recent years, their load-carrying performance under independent tension, bending, and shear actions in small-section prestressed square piles has yet to be systematically validated through experimentation.

To address these research gaps, this study adopts a prefabricated pile-cap joint with pre-embedded sleeved anchor bars and conducts full-scale monotonic tension, bending, and shear tests on $400\text{ mm} \times 400\text{ mm}$ small-section prestressed square piles. The objectives are: (1) to determine the ultimate load-carrying capacity and failure modes under the three loading conditions; (2) to elucidate the load-transfer path and pile-body integrity preservation mechanism of the threaded sleeve system; and (3) to establish a comprehensive experimental database under full loading conditions—filling code gaps and providing a foundation for subsequent combined-loading and dynamic performance studies as well as waterway engineering design.

2. Joint Design and Experimental Program

2.1. Description of the Joint Configuration

The pile-cap joint investigated is a prefabricated system for small-section prestressed concrete square piles (side length ≤ 400 mm), intended primarily for civil building and waterway revetment structures. As illustrated in Figure 1, the joint consists of a prefabricated prestressed pile, a cap system (1500×1500 mm plan), and embedded connecting elements. The cap consists of a prefabricated peripheral portion and a central cast-in-place block (450×450 mm) forming the joint region.

The embedded elements include a 20 mm-thick pile-end steel plate, twelve steel sleeves (40 mm, threaded) welded to the plate with open ends flush to its outer surface, twelve pile-side HRB400 anchoring bars (18 mm dia., 400 MPa yield) welded to the sleeves and embedded in the pile concrete, and twelve cap-side HRB400 anchoring bars (25 mm dia., 450 mm effective length) with threaded lower ends placed in the cast-in-place block. The cap-side bars are screwed into the sleeves through the end plate openings before casting the central block with C30 concrete. The pile shaft is made of C60 concrete with twelve prestressing strands (10.7 mm dia., 1420 MPa characteristic strength, 1005 MPa design strength) anchored to the end plate via swaged button-head anchorages.

The load is transferred through the threaded sleeve connections from the cap-side bars to the pile-side bars and the end plate, and finally into the prestressed pile. The connection avoids on-site cutting or welding of prestressing strands, preserving the full integrity of the pile. Key parameters are summarized in Table 1.

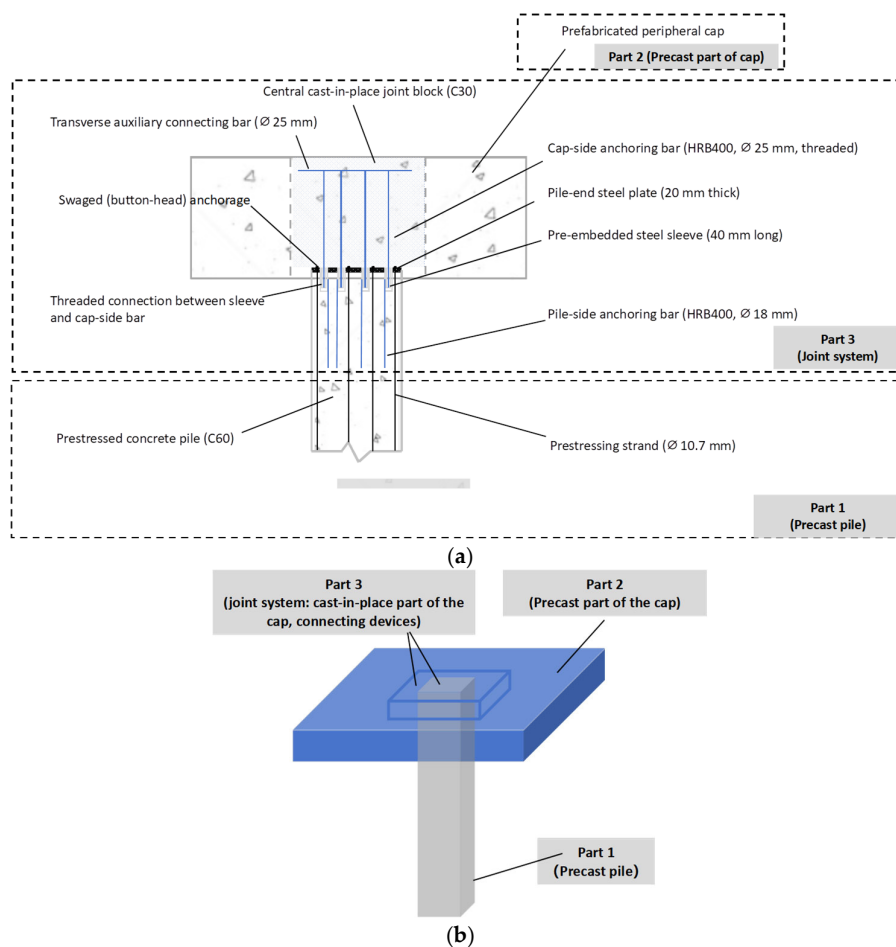


Figure 1. (a) Schematic diagram of the prefabricated pile-cap joint system: detailed connection layout. (b) Schematic diagram of the prefabricated pile-cap joint system: simplified element diagram.

Table 1. Performance parameters of the test prestressed square pile.

Parameter	Value
Pile side length (mm)	400
Single segment length (m)	≤14
Standard value of tensile resistance (crack control level I) (kN)	849.6
Design value of tensile resistance (kN)	917
Cracking moment (kN·m)	105.9
Design value of flexural capacity (kN·m)	159.9
Ultimate value of flexural capacity (kN·m)	191.9
Design value of shear capacity (kN)	260

Note: The values are based on the national design drawing set 20G361 (2020).

2.2. Specimen Preparation

Six full-scale specimens were fabricated with the configuration detailed in Section 2.1 (Figure 1). Prestressed square piles (C60 concrete, 400 × 400 mm) were cast using steel molds. Twelve strands (10.7 mm dia., 1420 MPa) were tensioned to 70% of ultimate strength. A 20 mm steel end plate was fixed at the pile tip, with twelve sleeves (40 mm, threaded) welded flush to its outer surface; twelve HRB400 bars (18 mm dia., 400 MPa yield) were welded to the sleeves. C60 concrete was cast, vibrated, and cured at 20 ± 2 °C, RH ≥ 95% for 28 days. Pile fabrication and finished products are shown in Figure 2.

The cap comprised a precast peripheral portion (C30, 1500 × 1500 mm) and a central void for the cast-in-place joint block (450 × 450 mm). Transverse auxiliary bars (HRB400, 25 mm dia.) were embedded in the peripheral cap. Twelve cap-side HRB400 bars (25 mm dia., 450 mm effective length) with threaded ends were prepared. Cap-side bars were screwed into the pile sleeves through the end plate openings (Figure 3). The central block was cast with C30 concrete and cured at 20 ± 2 °C for 28 days.

Strain gauges were attached to the pile surface (three gauges) and cap-side bars (four gauges) at the locations shown in Figure 4; a 100 × 100 mm crack-observation grid was marked on the pile surface near the joint. All gauges were connected to a DH3820N data acquisition system. The full name of this instrument is DH3820N Distributed Signal Test and Analysis System, which is independently developed and manufactured by Jiangsu Donghua Testing Technology Co., Ltd., a Chinese enterprise. Loading followed GB 50152-2012 [41].

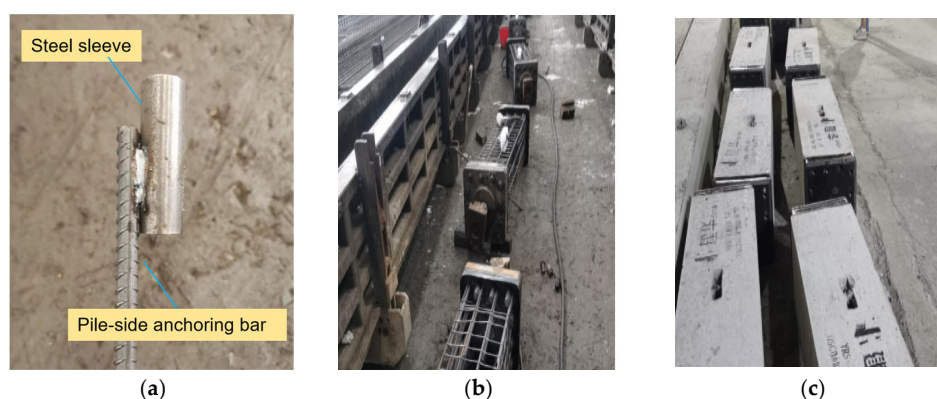


Figure 2. Producing process of the prestressed piles: (a) welded anchorage bar-sleeve assembly; (b) prestressing tendons, anchor bars, and sleeves assembled with pile-end steel plate; (c) finished prestressed pile.

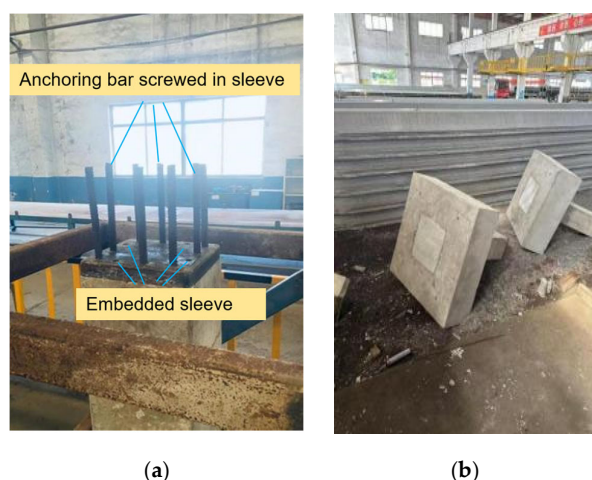


Figure 3. Photographs of the pile, pile cap, and connection components: (a) assembly of anchorage reinforcement to pile-end steel plate (anchoring-bar splicing in progress); (b) completed pile-pile cap connection node.

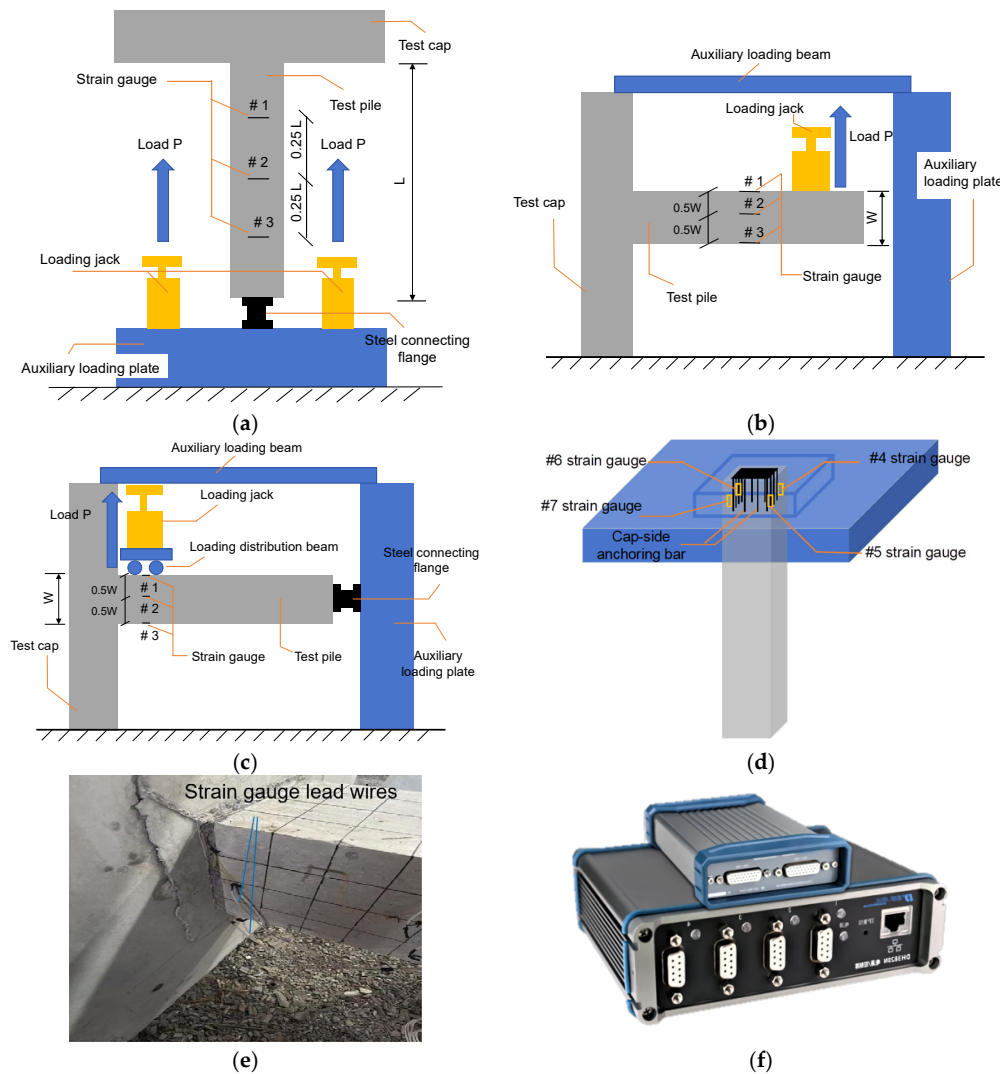


Figure 4. Scheme of tensile test setup. (a) Schematic diagram of the tensile test setup. (b) Schematic diagram of the bending test setup. (c) Schematic diagram of the shear test setup. (d) Schematic of strain gauge layout on anchorage reinforcement in the pile cap. (e) Strain gauge lead wires and the crack observation grid on the pile surface. (f) The DH3820N data acquisition system.

3. Results and Discussion

3.1. Tensile Performance

3.1.1. Theoretical Calculation of Tensile Performance

Per JTS 151-2011, the design and ultimate tensile capacities of the critical sections N_d and N_u were calculated using Equations (1) and (2), respectively. For the cap-side section (longitudinal reinforcement only) and the pile-side section (reinforcement + prestressing strands), the computed ultimate capacities are 2355 kN and 2752 kN, respectively. Both values significantly exceed the design tensile capacity of the pile shaft specified in atlas 20G361 (917 kN), confirming a theoretical strength hierarchy favorable to the joint system.

$$N_d = f_o A_o + f_p A_p \quad (1)$$

where N_d is the design value of axial tensile capacity (N); f_o and f_p are the design tensile strengths of ordinary steel bars and prestressing strands, respectively; and A_o and A_p are the total cross-sectional areas of all longitudinal ordinary reinforcing steel and prestressing steel, respectively.

$$N_u = f_{o, ch} A_s + f_{p, ch} A_p \quad (2)$$

where N_u is the ultimate axial tensile load-carrying capacity (N); $f_{o, ch}$ and $f_{p, ch}$ are the characteristic values of tensile strength (N/mm²) of longitudinal ordinary reinforcement and prestressing tendons, respectively.

3.1.2. Experimental Observations of Tensile Performance

Flexural cracks initiated on the pile surface at 148 kN. As loading progressed to 200 kN, extensive transverse cracks (>3 mm) on the top surface and vertical cracks (>1.5 mm) on the side faces developed, eventually leading to flexural failure of the pile (Figure 5). Throughout this process, no cracking or visible damage was observed in the pile-cap joint region.

At the moment of pile failure, the bending moment sustained at the joint face was calculated as 277 kN·m (based on a lever arm of 1.3 m), which is 32% higher than the pile's theoretical ultimate capacity (210 kN·m). The load–strain responses (Figure 6) show a typical reinforced-concrete behavior, with strains on the tension side increasing sharply post-cracking, while the compression side remained low. Notably, no abrupt strain jumps were recorded, confirming the absence of slip or distress within the threaded sleeve connection.

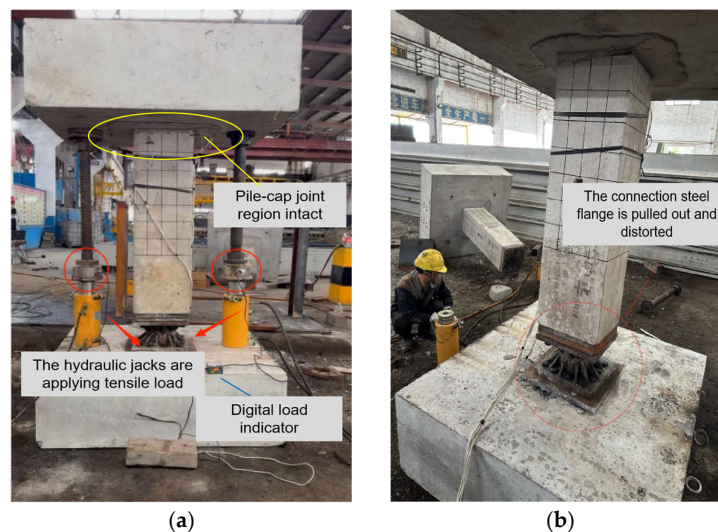


Figure 5. Photograph of pile-cap joint under tensile loading test: (a) pile-cap region intact; (b) the connection steel flange is pulled out and distorted.

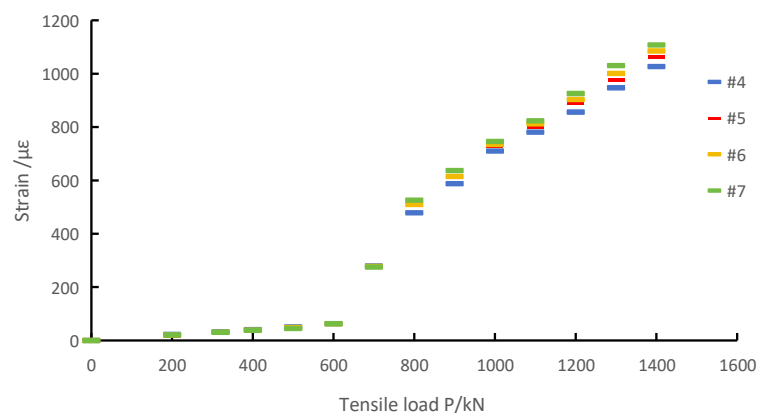


Figure 6. Load–strain curve of anchor reinforcement in pile cap under tensile test.

3.2. Bending Performance

3.2.1. Theoretical Calculation of Bending Performance

Per JTS 151-2011 and GB 50010-2010, the theoretical ultimate flexural capacity of a doubly reinforced section with prestressing and ordinary steel was calculated using Equation (3):

$$M_u = \sum [f_{py}A_{pi} \left(h_{pi} - \frac{x}{2}\right) + f_yA_{si} \left(h_{si} - \frac{x}{2}\right)] \quad (3)$$

where M_u denotes the design value of the flexural capacity (N·mm); A_{pi} is the cross-sectional area of the i -th row of longitudinal prestressing steel in the tensile or compressive zone (mm²); h_{pi} is the distance from the i -th row of longitudinal prestressing steel in the tensile zone to the outer edge of the concrete compressive zone (mm); x is the height of the equivalent rectangular stress block in the concrete compressive zone (mm); A_{si} is the cross-sectional area of the i -th row of non-prestressed reinforcing steel in the tensile zone (mm²); and h_{si} is the distance from the i -th row of non-prestressed reinforcing steel in the tensile zone to the outer edge of the concrete compressive zone (mm).

For the critical sections in this study, the predicted ultimate moments were 210 kN·m for the pile shaft, 267 kN·m for the pile section within the joint region, and 261 kN·m for the cap slab. The predicted capacity of the pile shaft (210 kN·m) is in close agreement with the code-recommended value of 192 kN·m from atlas 20G361. The comparison confirms that both joint-related sections theoretically possess higher flexural strength than the pile shaft itself.

3.2.2. Experimental Observations of Bending Performance

The bending test photograph is shown in Figure 7. Loading was applied in 50 kN increments. Flexural cracks first appeared on the pile surface at 148 kN. As the load increased to 200 kN, extensive transverse cracks (>3 mm wide) developed on the top surface, accompanied by vertical cracks (>1.5 mm) on the side faces, ultimately leading to flexural fracture of the pile (Figure 8). Throughout the entire loading process, the pile-cap joint region remained intact with no visible cracks or damage.

At the moment of pile failure, the bending moment sustained at the joint face was calculated as 277 kN·m (based on a lever arm of 1.3 m from the loading point to the cap face). This value is 32% higher than the pile's theoretical ultimate moment capacity (210 kN·m).



Figure 7. The bending test setup: photograph.

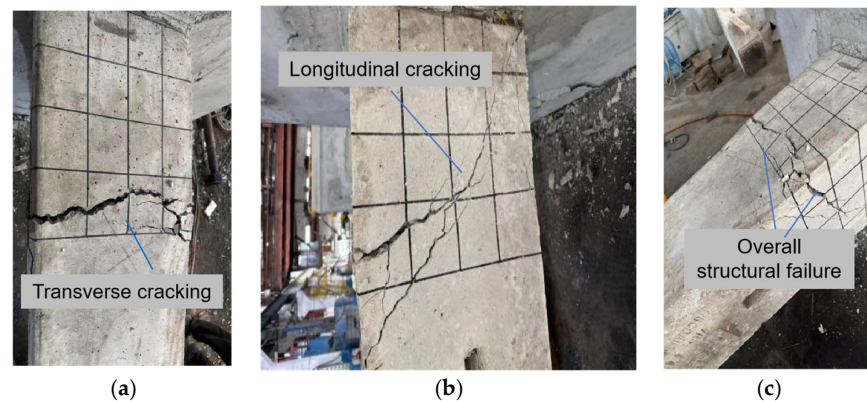


Figure 8. Crack patterns and final structural failure of the pile: (a) transverse cracking; (b) longitudinal cracking; (c) overall structure failure.

Figure 9 presents the load–strain curves of the pile surface (#1–#3) and the cap-side anchoring bars (#4–#7). Prior to cracking, all strains increased linearly with load. After cracking, strains on the tension side (#1 and #2) rose sharply, while the compression side (#3) remained low throughout—a trend characteristic of typical reinforced-concrete flexural behavior. No abrupt strain jumps or anomalies indicative of slip or distress at the threaded sleeve connection were observed.

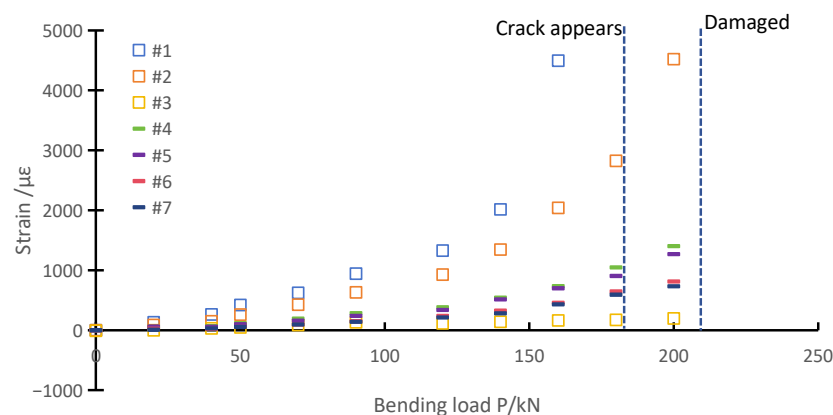


Figure 9. Load–strain curve of the pile surface (#1, #2, and #3) and the anchor reinforcement in cap (#4–#7) under bending test.

3.3. Shear Performance

3.3.1. Theoretical Calculation of Shear Performance

Per JTS 151-2011, the design shear capacity of the pile shaft with stirrups was calculated using Equation (4):

$$V_d = \alpha_{cv} f_t b h_0 + f_{yv} \frac{A_{sv}}{S} h_0 \quad (4)$$

where α_{cv} denotes the shear capacity coefficient of concrete on the diagonal section; f_t denotes the design value of axial tensile strength of concrete; b denotes the width of the cross-section; h_0 denotes the effective depth of the section; f_{yv} denotes the design tensile strength of stirrups; A_{sv} denotes the total cross-sectional area of all stirrup legs within the same section; and S denotes the spacing of stirrups along the longitudinal direction of the member.

For the present specimens, the calculated design shear capacity of the pile shaft was 271 kN, which is slightly higher than the code-recommended value of 260 kN from atlas 20G361, confirming that the pile itself satisfies the serviceability requirement.

3.3.2. Experimental Observations of Shear Performance

The shear test photograph is shown in Figure 10. Loading was applied in 100 kN increments. At 620 kN, a slight vibration was felt and cracks initiated at the connection between the cap top and the auxiliary loading reaction beam. As the load was further increased to approximately 680 kN, the cracks on the cap side widened progressively, and the concrete at the bottom of the cap was crushed (Figure 11a). The test was terminated at this point. Post-test inspection revealed that the pile-cap joint region and the pile itself remained completely free of cracks or damage (Figure 11b). The failure was confined to the precast portion of the concrete cap, rather than the joint system.



Figure 10. The shear test photograph.

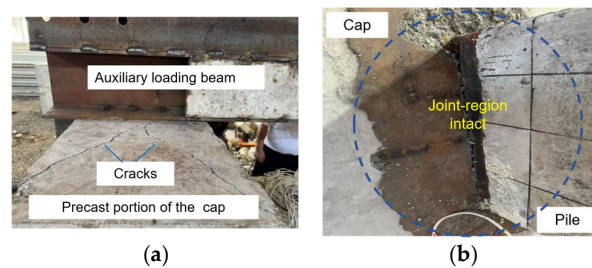


Figure 11. Observation of the shear test: (a) cracks on the precast portion of the cap; (b) intact joint-region.

Figure 12 presents the load–strain curves of the cap-side anchoring bars. On the side away from the loading direction, the rebar exhibited a small negative strain (compression); on the loading-direction side, the strain was positive (tension) but close to zero. Throughout loading, no abrupt changes in strain trends were observed, indicating the absence of abnormal stress concentration or distress at the threaded sleeve connection.

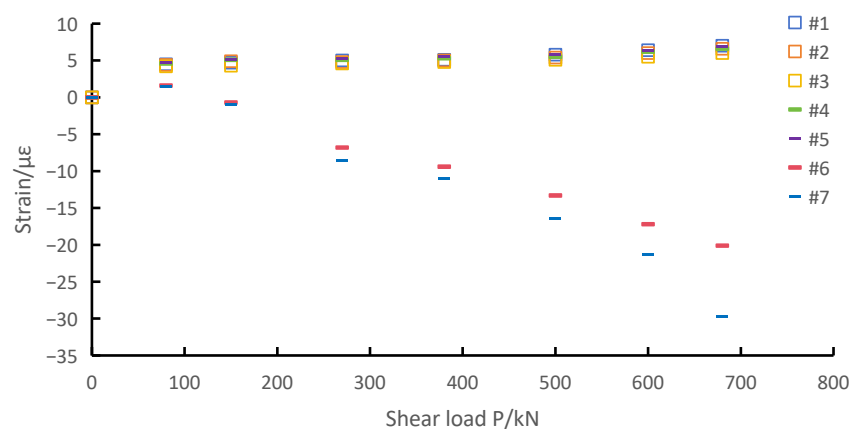


Figure 12. Load–strain curve under shear test.

4. Discussion

4.1. Strength Hierarchy and Failure Mode Analysis

The three sets of full-scale tests consistently demonstrated that the prefabricated pile-cap joint system possessed higher strength than both the prestressed pile shaft and the precast cap under all loading conditions. Table 2 summarizes the key comparative results.

Table 2. Summary of test results and strength hierarchy.

Loading Condition	Load Sustained by Joint	Corresponding Pile Capacity	Failure Location	Joint Status
Tension	1400 kN	917 kN (design, atlas)	Auxiliary steel flange	Intact
Bending	277 kN·m	210 kN·m (ultimate, theoretical)	Pile shaft	Intact
Shear	680 kN	260 kN (design, atlas)	Precast cap	Intact

This hierarchy validates the “strong joint, weak member” design philosophy, ensuring that potential failure would occur in components more accessible for inspection and repair. Notably, in the shear test, failure occurred in the precast portion of the cap rather than the cast-in-place joint block, suggesting that the interface between precast and cast-in-place concrete—or the transverse auxiliary bars within the precast cap—may represent the weakest link in the overall system under shear. This unexpected observation implies that future optimization could focus on enhancing the precast-cap details rather than the joint connection itself, a finding not anticipated by the initial theoretical calculations.

4.2. Load-Transfer Mechanism of the Threaded Sleeve System

The superior performance of the joint is attributed to a carefully designed, multi-component load-transfer system. The identified load path follows a sequential chain: tensile or bending forces applied to the cap are first transmitted to the cap-side anchoring bars, then through the threaded sleeves to the pile-end steel plate, subsequently to the pile-side anchoring bars and prestressing strands, and finally into the prestressed pile shaft. Several features collectively contribute to the observed overstrength:

- (i) Mechanical interlock without slip. The threaded connection provides positive mechanical interlock between the cap-side and pile-side reinforcement. This is evidenced by the linear and continuous load–strain development across all three tests (Figures 6, 9, and 12), where no abrupt strain jumps or plateaus—which would indicate bond-slip or thread stripping—were observed up to the termination of each test. The cast-in-place concrete block further confines the sleeve region, enhancing the bond and load-transfer efficiency.
- (ii) Favorable strength hierarchy at the component level. The cap-side bars (25 mm diameter, 400 MPa yield) have a larger cross-section than the pile-side bars (18 mm diameter, 400 MPa yield). This deliberate design ensures that, under all service and ultimate loading conditions contemplated for the pile, the connection region remains elastic and sustains the applied forces without distress, while the ultimate capacity of the entire system is governed by the strength of the pile shaft rather than by the connection. This hierarchy carries two important implications. First, the location of failure is predesigned: should overloading occur, damage would be confined to the pile shaft—a component with visible cracking, accessible for inspection, and repairable—rather than to the concealed connection region, where distress would be difficult to detect and costly to remedy. Second, the connection is not the system bottleneck: the joint’s capacity has been raised to a level where it no longer controls the overall system strength, which is instead determined by the pile itself. The theoretical

capacities computed in Section 3 consistently support this hierarchy, demonstrating that the joint-related sections possess higher strength than the pile shaft across all three loading modes.

- (iii) Preservation of prestressing integrity. The connection avoids on-site cutting or welding of the prestressing strands, which are instead anchored to the end plate via swaged button-head anchorages. This preserves the full prestressing force and eliminates stress concentration or heat-affected zone weakening that would arise from field welding—a critical advantage over traditional pile-head chipping methods.
- (iv) Composite action of the end plate. The 20 mm thick steel end plate serves not merely as a bearing surface but as a load-distribution diaphragm, transferring the concentrated forces from the twelve sleeves uniformly into the pile cross-section. This explains why no splitting cracks were observed at the pile end, despite the high local stresses introduced by the anchored strands.

4.3. Comparison with Existing Studies and Design Codes

Previous research on precast connections has primarily addressed beam–column joints under seismic loading in building frames [4,8,10,17]. Those studies generally confirmed that properly detailed precast connections can achieve performance comparable to monolithic construction under cyclic bending and shear. The present study extends this knowledge base in three distinct directions:

- (i) Loading regime: Rather than cyclic combined loading, this study verifies the connection under independent tension, bending, and shear—the three loading modes that are individually designed and verified in waterway engineering practice [20,21]. This fills a critical gap, as the combined-load and cyclic studies [14–19] inherently rely on such baseline data as prerequisite conditions.
- (ii) Structural element: While beam–column joints in frames transfer moment and shear between horizontal and vertical members, pile-cap joints must additionally sustain significant uplift tension from storm-surge water level differentials—a load type rarely considered in building frame connections. The tensile capacity demonstrated (1400 kN) directly addresses this waterway-specific demand.
- (iii) Code coverage: Current design codes, including atlas 20G361 [33], JTS 151 [34], JTS 167-4 [35], GB 55008 [36], GB 50010 [37], and GB/T 50152 [41], do not provide experimentally validated design provisions for prefabricated pile-cap connections under independent tension, bending, and shear verification. The data presented in this study (Table 2) provide the first full-scale experimental reference for future code development.

4.4. Engineering Implications and Limitations

4.4.1. Applicable Scenarios

The proposed connection is specifically intended for small-section (≤ 400 mm) pre-stressed concrete square piles in waterway revetment, port wharves, inland waterway regulation structures, and building foundations along waterfront areas—where piles are driven first and subsequently connected to a cast-in-place cap or foundation mat. The system is particularly advantageous in scenarios with: (i) high groundwater table or marine exposure, where on-site welding is difficult or corrosion-prone; (ii) strict environmental requirements, where wet operations and on-site cutting are discouraged; (iii) confined construction sites along riverbanks or coastal zones, where prefabricated components reduce on-site labor and accelerate construction schedules; and (iv) standardized mass production, where the sleeve assembly can be factory-installed with consistent quality control.

In such contexts, the proposed connection offers a practical solution that combines construction efficiency with reliable structural performance, supporting the growing demand for green and industrialized construction in both hydraulic and civil engineering applications.

4.4.2. Limitations and Extrapolation Considerations

While the experimental results are robust for the tested configuration, several factors constrain direct extrapolation:

- (i) Cross-sectional dimensions. The tested pile was 400×400 mm. Extrapolation to larger sections (e.g., 500×500 mm or above) would require verification, as the number and arrangement of sleeves, the end-plate thickness, and the cast-in-place block size would need to be scaled proportionally to maintain the same load-transfer efficiency. Conversely, application to smaller sections (e.g., 300×300 mm) may be feasible but would require careful attention to reinforcement congestion around the sleeves.
- (ii) Prestressing level and concrete grade. The tests employed C60 pile concrete and C30 joint concrete with strands tensioned to 70% of ultimate strength. Higher prestressing levels or higher-strength joint concrete (e.g., UHPC as proposed by Zhou et al. [12,13]) could potentially reduce anchorage length or sleeve dimensions, but these modifications would require separate validation due to changes in bond and confinement characteristics.
- (iii) Environmental durability. The tests were conducted under ambient laboratory conditions. In marine or splash-zone exposure, corrosion of the threaded sleeves, the end plate, and the anchorage bars could degrade long-term performance. Protective measures—such as galvanized or stainless-steel components, epoxy-coated reinforcement, or increased concrete cover—should be considered and validated for aggressive environments.
- (iv) Loading regimes beyond monotonic. This study employed monotonic loading and did not address cyclic, fatigue, or dynamic loading relevant to wave impact, vessel berthing, or seismic events. While the monotonic overstrength observed (32–160% above pile capacities) provides a conservative starting point, the connection's performance under repeated or reversed loading cannot be directly inferred and warrants separate investigation.

4.5. Unexpected Findings and Their Implications

Several observations merit critical reflection beyond the expected “joint is stronger” narrative:

First, in the tensile test, the loading was terminated at 1400 kN due to failure of the auxiliary steel flange rather than any component of the joint system. This means the true ultimate tensile capacity of the joint remains unknown—it is certainly higher than 1400 kN, but by how much remains an open question. This should not be interpreted merely as evidence of over-design; rather, it highlights the need for future tests with a stronger auxiliary system to establish the actual failure limit.

Second, in the shear test, failure occurred in the precast portion of the cap, not the cast-in-place joint block. The theoretical calculations in Section 3.3.1 only considered the pile shaft and did not predict this failure mode. This discrepancy suggests that the precast-to-cast-in-place interface—or the transverse auxiliary bars within the precast cap—may be the actual design bottleneck under shear, rather than the sleeve connection itself. This finding has practical implications: optimization efforts could target the precast cap detailing (e.g., additional shear keys, increased bar diameter, or improved interface roughening) rather than the sleeve joint.

Third, the strain gauges on the cap-side anchoring bars consistently recorded small strains (less than approximately $500 \mu\epsilon$ in tension and near-zero in shear), far below the yield strain of HRB400 steel ($\approx 2000 \mu\epsilon$). This implies that the anchorage bars were not fully mobilized at the point of pile or cap failure. While this confirms the conservative nature of the current design, it also suggests potential for material optimization—reducing the bar diameter or number of sleeves could achieve an economical design while still maintaining the required strength hierarchy.

4.5.1. Design Implications

The experimental results do not imply that the current design is over-engineered. Rather, they reflect the absence of experimentally validated joint details in current codes—atlas 20G361 [37], JTS 151 [38], JTS 147-7 [39], GB 55008 [40], and GB 50010 [41] all lack explicit provisions for prefabricated pile-cap connections under independent tension, bending, and shear verification. In this context, a conservative baseline is both necessary and prudent. The data presented in this study provide an upper-bound reference that enables future rational, rather than empirical, design.

Based on the load-transfer mechanism revealed in Section 4.2 and the strain mobilization patterns observed in Section 3, several optimization opportunities can be considered for future applications. First, the consistently low strains recorded on the cap-side anchoring bars (below $500 \mu\epsilon$ in tension and near-zero in shear, far below the yield strain of $\approx 2000 \mu\epsilon$) suggest that reducing the anchorage bar diameter or the number of sleeves may achieve a more economical design while still maintaining the required strength hierarchy. Second, the cast-in-place block size could potentially be optimized based on the actual confinement requirements, as the current dimensions were conservatively chosen without prior experimental guidance. Third, in shear-critical applications—where failure was observed in the precast cap rather than the joint itself—designers should pay particular attention to the precast-to-cast-in-place interface detailing, such as incorporating additional shear keys, increasing transverse auxiliary bar diameter, or improving interface roughening. These optimizations, however, should be validated through additional testing before implementation in practice.

4.5.2. Future Research Recommendations

The following research directions are recommended to address the limitations identified:

- (i) Combined loading. Tests under tension-bending, bending-shear, and combined tension-bending-shear should be conducted to understand action interaction, as actual service conditions rarely involve pure single loading.
- (ii) Cyclic and dynamic loading. Low-cycle reversed loading tests are needed for seismic regions, while fatigue tests would inform performance under wave-induced or berthing-related repeated loads.
- (iii) Parametric studies. Variations in sleeve length, bar diameter, end-plate thickness, and concrete grade should be systematically investigated to establish design formulae and optimize material usage.
- (iv) Long-term durability. Exposure tests in marine or chloride-rich environments are essential to assess corrosion resistance and service-life performance.
- (v) Full failure limit. For tension, a stronger auxiliary loading system should be developed to determine the true ultimate capacity of the joint, providing a complete picture of the safety margin.

5. Conclusions

This study conducted full-scale monotonic tests on six identical 400 mm × 400 mm prestressed concrete pile-cap specimens to evaluate the mechanical behavior of a prefabricated joint system with threaded sleeve connections under tension, bending, and shear. Based on the experimental results and subsequent analysis, the following conclusions are drawn:

- (1) Tensile performance. The joint system sustained a tensile load of 1400 kN without any damage to the connection, the pile, or the cap—exceeding the design tensile capacity of the pile shaft (917 kN) specified in atlas 20G361 by 52%. The test was terminated due to failure of the auxiliary loading steel flange rather than any component of the joint system, meaning the true ultimate tensile capacity of the joint remains unknown but is certainly higher than 1400 kN.
- (2) Bending performance. The pile failed in flexure at its theoretical ultimate moment (210 kN·m), while the pile-cap joint remained completely intact. The bending moment sustained at the joint face at the time of pile failure was 277 kN·m, which is 32% higher than the pile's theoretical ultimate capacity. The load–strain curves exhibited typical reinforced-concrete flexural behavior with no abrupt jumps indicative of slip or distress at the connection.
- (3) Shear performance. The connection sustained a shear load of 680 kN without any damage to the joint or the pile—exceeding the design shear capacity of the pile (260 kN) by more than 2.5 times. Failure occurred in the precast portion of the cap rather than in the joint system or the cast-in-place block, indicating that the precast-to-cast-in-place interface may be the actual design bottleneck under shear.
- (4) Strength hierarchy. Across all three loading conditions, the joint system exhibited higher strength than both the prestressed pile shaft and the precast cap, confirming that the “strong joint, weak member” design principle is achieved. This hierarchy ensures that potential failure would occur in components that are visible, accessible for inspection, and repairable, rather than in the concealed connection region.
- (5) Load-transfer reliability. The threaded sleeve connection provides a direct and continuous load path from the cap-side anchoring bars to the pile-side bars and the end plate, and finally into the prestressed pile. The linear strain development without abrupt jumps across all tests confirms that the mechanical interlock eliminates slip, while the avoidance of on-site cutting or welding preserves the full integrity of the prestressing strands.
- (6) Design and code implications. Current design codes—including atlas 20G361, JTS 151, JTS 147-7, GB 55008, and GB 50010—lack experimentally validated provisions for prefabricated pile-cap connections under independent tension, bending, and shear verification. The experimental data presented in this study provide the first comprehensive full-scale reference for such connections, supporting future code development and design practice for small-section prestressed piles in waterway and civil engineering applications.

This study demonstrates that the proposed prefabricated pile-cap joint system is a viable and reliable solution for connecting small-section prestressed concrete square piles to superstructures in waterway revetment, port wharves, and waterfront building foundations. The threaded sleeve system offers a practical alternative to traditional cast-in-place methods, eliminating on-site strand cutting and welding while ensuring structural integrity. The findings support the growing adoption of green, industrialized construction in hydraulic and civil engineering. However, the limitations identified—particularly regarding combined loading, cyclic and dynamic actions, environmental durability, and

parametric optimization—warrant further investigation before the connection can be broadly generalized to diverse engineering contexts.

Based on the findings and limitations of this study, the following research directions are recommended: (i) combined loading tests under tension-bending, bending-shear, and full combined actions to understand action interaction under realistic service conditions; (ii) low-cycle reversed loading and fatigue tests to evaluate performance under seismic, wave-induced, and berthing-related repeated loads; (iii) parametric studies varying sleeve length, bar diameter, end-plate thickness, concrete grade, and prestressing level to establish design formulae and optimize material usage; (iv) long-term durability tests in marine or chloride-rich environments to assess corrosion resistance and service-life performance; and (v) tensile tests with a stronger auxiliary loading system to determine the true ultimate capacity of the joint.

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