

Article

Experimental Investigation of the Pull-Out Behavior of Semi-Embedded Anchor Bolts in Concrete Transition Sections

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Abstract

To investigate the pull-out behavior of semi-embedded anchor bolts in the concrete transition sections of steel–concrete hybrid wind turbine towers, five 1:2-scale local specimens were tested under monotonic pull-out loading. The effects of reinforcement, end-plate size, number of anchor bolts, and sleeve installation on the failure mode, load–cumulative surface crack-opening displacement response, reinforcement strain, and ultimate pull-out capacity were analyzed. The results show that reinforcement significantly improved both the pull-out resistance and the apparent deformation capacity characterized by cumulative surface crack opening, changing the failure mode from brittle cracking of plain concrete to a ductile combined mechanism governed by end-plate bearing, concrete splitting, reinforcement restraint, and boundary effects. Increasing the end-plate size, changing the anchor arrangement, and installing a sleeve produced only indicative trends in this limited test series because only one specimen was tested for each configuration. Existing methods were also evaluated as benchmark approaches. The characteristic resistance predicted by EN 1992-4 was conservative, partly because of its fractile basis and partly because the tested configuration was outside the assumptions of an unconstrained concrete cone breakout model. The 45° cone method underestimated the capacity of the reinforced specimens but slightly overestimated that of the plain-concrete specimen, whereas the Nilforoush and LEFM methods overestimated the capacity. These findings provide initial experimental evidence for understanding the load-transfer and failure mechanisms of semi-embedded anchor bolts and clarify the applicability and limitations of the existing calculation methods. The development of a reliable predictive model requires a broader experimental database and systematic numerical investigations.

Keywords: concrete transition section; pull-out capacity; anchor bolt; semi-embedded anchor bolt; reinforcement



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1. Introduction

Against the background of the global transition toward low-carbon energy systems, wind energy has become an important component of renewable energy development. In recent years, the global installed wind power capacity has continued to increase, while wind energy development has gradually expanded from high-wind-speed regions to low-wind-speed areas, complex terrains, and densely populated regions [1–3]. To improve the utilization efficiency of wind energy, wind turbines have continuously evolved toward

higher rated capacities, larger rotor diameters, and greater hub heights. Consequently, tower structures are subjected to increased axial forces, bending moments, and cyclic loads, imposing more stringent requirements on their stiffness, fatigue performance, constructability, and transportability.

Wind turbine towers can generally be classified into steel towers, concrete towers, and steel–concrete hybrid towers [4]. Steel towers offer convenient fabrication and installation; however, transportation constraints, structural stability, and fatigue-related problems become increasingly pronounced as the tower height and diameter increase. Concrete towers exhibit relatively high stiffness and favorable durability, but their considerable self-weight and complex construction procedures limit their wider application. A steel–concrete hybrid wind turbine tower typically consists of a lower concrete segment, a transition section, and an upper steel tower segment [5], as illustrated in Figure 1. This structural system combines the high stiffness of concrete with the relatively low self-weight and high installation efficiency of steel structures and has therefore become an important structural form for tall wind turbine towers [6].



Figure 1. Steel-concrete hybrid wind turbine tower.

The transition section is located between the upper steel tower and the lower concrete tower and serves as a critical component for structural transition and load transfer in steel–concrete hybrid wind turbine towers [7,8]. According to the anchorage configuration, concrete transition sections can be classified into fully embedded and semi-embedded types, whose cross-sectional configurations are illustrated in Figure 2. In the fully embedded configuration, the anchor bolts generally have relatively long cantilevered portions, which may induce pronounced local eccentricity effects under prestressing and external loading. In addition, this configuration requires relatively large quantities of both anchor bolt steel and concrete. By contrast, the semi-embedded configuration shortens the cantilevered length, improves the local force-transfer mechanism, and enhances material utilization, thereby offering greater advantages in engineering applications. Under wind loading, an overturning moment develops in the tower, causing the anchor bolts on the tension side to

sustain substantial axial tensile forces. Consequently, the pull-out capacity of these anchor bolts is a critical issue in the design and safety assessment of concrete transition sections.

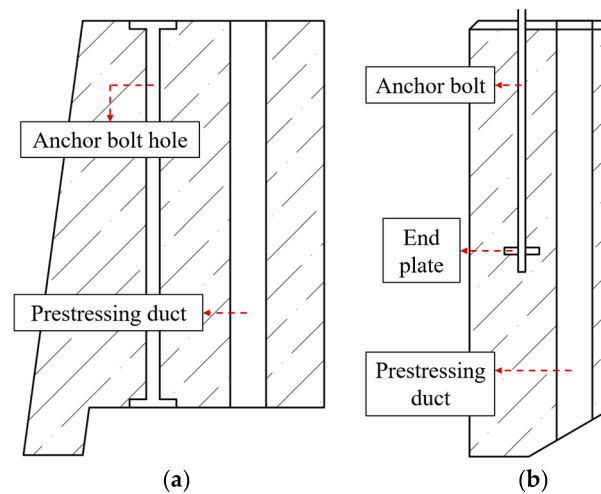


Figure 2. Cross-sectional forms of the concrete transition sections: (a) Fully embedded; (b) Semi-embedded.

In recent years, extensive studies have been conducted on the load-transfer mechanism of anchor bolts, anchor-head configurations, concrete cracking, and the contribution of reinforcement. Miao et al. [9] reported that the anchorage resistance of headed reinforcing bars is jointly provided by bond along the bar and local bearing beneath the anchor head. Xu et al. [10] found that supplementary reinforcement can restrain crack development and participate in resisting tensile forces, thereby improving the pull-out performance of anchor bolts. Jeong et al. [11] investigated the combined tension–shear behavior of cast-in anchors in cracked concrete. Karmokar and Mohyeddin [12] demonstrated that surface cracking, anchor-head geometry, and bearing area can significantly influence the failure mode and ultimate capacity. The tensile resistance and crack-control capacity of the concrete substrate have also received considerable attention. Previous studies have shown that fiber-bridging action can suppress crack propagation in concrete and improve both the pull-out capacity and failure ductility of anchor bolts [13,14]. In terms of numerical analysis, damage-based constitutive models and refined finite element approaches have been employed to simulate the pull-out process of anchor bolts and to evaluate the effects of concrete strength, anchor dimensions, embedment depth, contact interactions, and boundary conditions [15–17].

Most existing studies have focused on anchor bolts with shallow or moderate embedment depths, generally using specimens with regular boundaries and limited reinforcement. However, semi-embedded anchor bolts in the transition sections of hybrid wind turbine towers are typically characterized by large diameters, substantial embedment depths, and densely arranged reinforcement, while also being subjected to complex boundary constraints. Under these conditions, a complete unconstrained concrete breakout cone may be unable to develop. Instead, tensile resistance of the reinforcement, concrete splitting, bearing beneath the end plate, and boundary restraint may jointly contribute to the load-carrying mechanism. Therefore, the applicability of conventional calculation methods for pull-out capacity of anchor bolts remains to be verified. Against this background, five 1:2-scale local specimens representative of actual steel–concrete hybrid wind turbine tower transition sections were tested under pull-out loading of anchor bolts. The effects of structural reinforcement, end-plate size, number of anchor bolts, and sleeve installation on the failure mode, load–cumulative surface crack-opening displacement response, reinforcement strain, and ultimate pull-out capacity were systematically investigated. In addition, the applicability and limitations of several existing capacity calculation meth-

ods were evaluated using the test results as benchmarks. The present study focuses on the initial experimental characterization of the load-transfer and failure mechanisms of semi-embedded anchor bolts. Owing to the limited number of specimens and investigated parameters, the development of a new predictive model is beyond the scope of the present study and requires a broader experimental database and systematic numerical analyses.

The remainder of this paper is organized as follows. Section 2 describes the specimen design, fabrication process, test setup, measurement scheme, and loading protocol. Section 3 presents the observed failure modes, load–cumulative surface crack-opening displacement responses, and reinforcement strain results. Section 4 evaluates the applicability and limitations of existing calculation methods by comparing their predictions with the experimental results. Section 5 summarizes the main conclusions, limitations, and recommendations for future work.

2. Experimental Program

2.1. Specimen Scheme

The test specimens were simplified from a representative local segment of the actual transition section and geometrically scaled at a ratio of 1:2. The specimen represents the tensile-side local anchorage region of the circular concrete transition section, where the pull-out behavior is mainly governed by the interaction among the anchor bolt, end plate, surrounding concrete, and reinforcement. Because the radius of the prototype transition section is much larger than the local anchorage region, the curvature of the transition ring was neglected in the present rectangular local specimen. Nevertheless, this simplification cannot fully reproduce the circumferential continuity and ring action of the prototype, and this limitation is further discussed in Section 5. As shown in Figure 3, the prototype transition section contained circumferentially arranged anchor bolts and prestressing ducts. The horizontal cross-section illustrates their number and arrangement, while the vertical cross-section provides the principal geometric parameters, including the anchor embedment depth, end-plate position, and transition-section dimensions. These geometric characteristics formed the basis for the design of the 1:2-scale local specimens. The anchor bolts in the prototype had a total embedment depth of 1440 mm. The end plate was 200 mm wide and 40 mm thick, and the longitudinal reinforcement ratio was approximately 0.9%. Each scaled specimen had overall dimensions of 405 mm × 300 mm × 1204 mm (length × width × height). The anchor bolt had an edge distance of 150 mm and an embedment depth of 540 mm. A square end plate with a side length of 100 mm and a thickness of 20 mm was installed at a depth of 440 mm. On the opposite side of the specimen, a reserved duct was arranged at a distance of 100 mm from the edge to simulate the prestressing tendon duct in the actual transition section. Specimen LB-1 was a plain-concrete specimen without reinforcement, whereas LB-2 to LB-5 were reinforced concrete members. Except for the absence of reinforcement, the geometry, anchor bolt, end plate, and reserved duct arrangement of LB-1 were the same as those of the reference specimen LB-2. Concrete cube compression tests yielded an average cube compressive strength of 71 MPa. According to the tensile test results, the yield and ultimate strengths of the reinforcing bars (Chongqing Kungang Industrial Co., Ltd., Chongqing, China) were 482 MPa and 620 MPa, respectively. Grade 10.9 M39 high-strength threaded rods (China Shipbuilding Haiwei High Tech Co., Ltd., Zhengzhou, China) were used as the anchor bolts. It is worth noting that although the scaled anchor bolt size was theoretically 21 mm, the anchor bolts were strengthened in the test. This treatment ensured that the failure mode was dominated by concrete breakout rather than anchor bolt fracture.

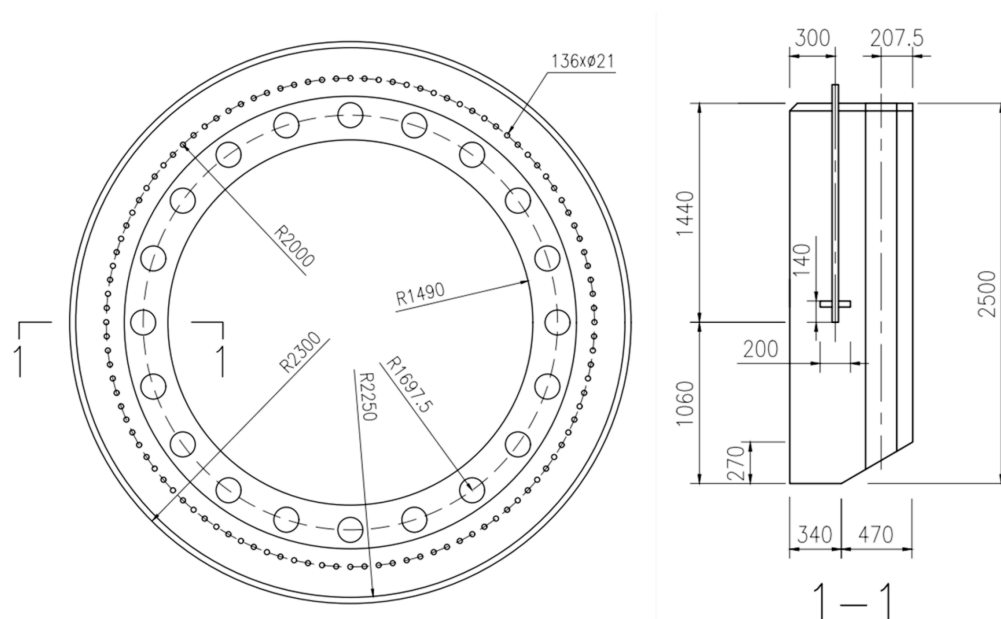


Figure 3. Cross-sectional dimensions of the transition section (mm).

For the two-anchor specimen LB-4, the center-to-center anchor spacing was 144 mm. For the sleeved specimen LB-5, plastic films were wrapped around the anchor bolt shank before concrete casting. The sleeve was intended to reduce the bond interaction between the anchor shaft and the surrounding concrete so that the tensile force was transferred mainly through the end plate. A schematic illustration of the specimens is presented in Figure 4. Five specimens were designed and tested, and their detailed design parameters are summarized in Table 1.

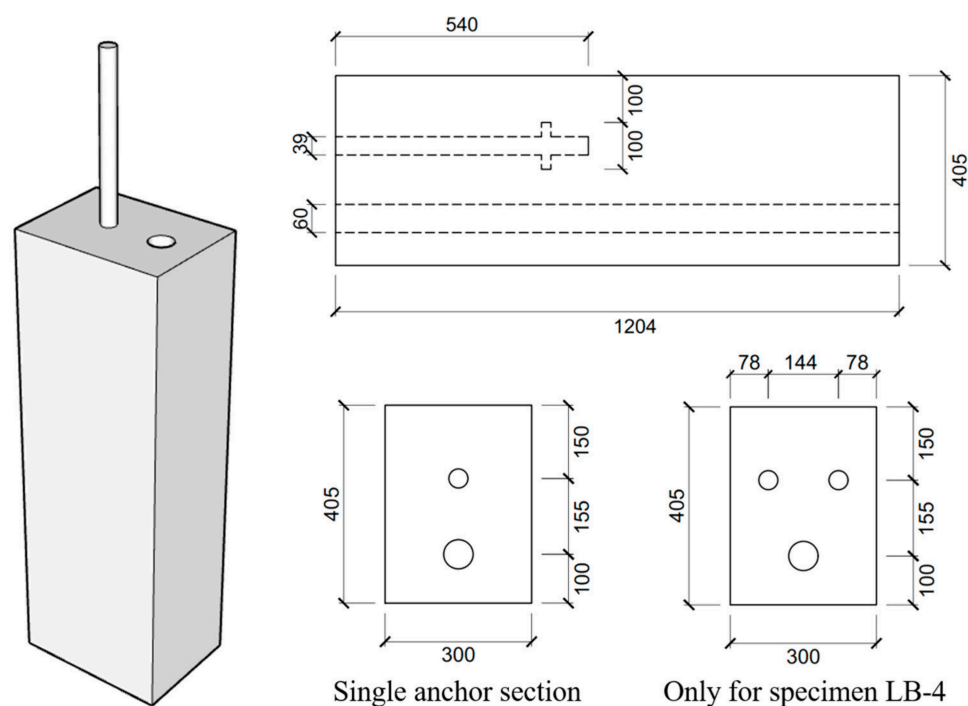


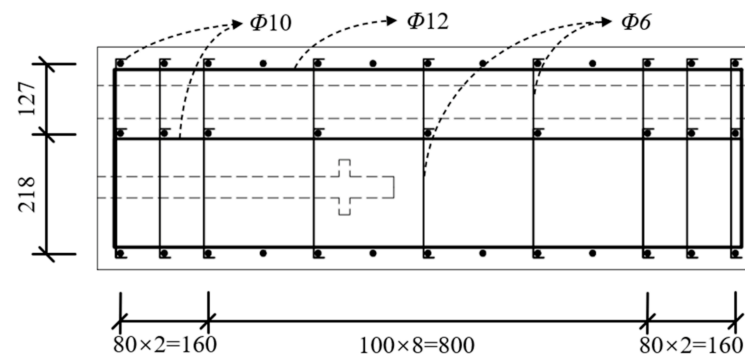
Figure 4. Overall dimensions of the test specimen (mm).

Table 1. Specimen parameters.

Specimen Number	End-Plate Side Length (mm)	Number of Anchor Bolts	Sleeve Installation	Reinforcement Arrangement
LB-1	100	1	×	×
LB-2	100	1	×	✓
LB-3	150	1	×	✓
LB-4	100	2	×	✓
LB-5	100	1	✓	✓

Note: “✓” indicates that the corresponding feature was provided, whereas “×” indicates that it was not provided.

The reinforcement detailing of the specimens was derived by scaling and simplifying that of the prototype transition section. Specifically, the circumferential reinforcement was represented by transverse ties, while the longitudinal reinforcement ratio was maintained at the same level as that of the prototype. The reinforcement layout is shown in Figure 5. In Figure 5, $\Phi 6$, $\Phi 10$, and $\Phi 12$ denote reinforcing bars with nominal diameters of 6 mm, 10 mm, and 12 mm, respectively; the concrete cover thickness was set to 20 mm, and all dimension in Figure 5 are in millimeters.

**Figure 5.** Reinforcement layout (mm).

2.2. Specimen Fabrication

The main fabrication procedures included reinforcement cage assembly, strain-gauge installation, formwork fabrication, and concrete casting. Before fabricating each specimen, all the required reinforcing bars were prepared according to the specified dimensions. The reinforcement cage was initially shaped by positioning and securing four stirrups, after which the remaining reinforcing bars were tied sequentially at their designated locations. Upon completion of the reinforcement cage, the surfaces of the reinforcing bars at the strain-gauge locations were polished. After the strain gauges had been attached, they were covered with silicone sealant (Guangdong Kafuter New Materials Technology Co., Ltd., Huizhou, China) to protect them against moisture and mechanical damage during concrete casting.

Wooden formwork was used for specimen fabrication, with each formwork assembly consisting of five timber panels. Openings were formed at the specified locations, after which the panels were assembled sequentially. Before concrete casting, a uniform layer of release agent was applied to the inner surfaces of the formwork. The reinforcement cage was then placed inside the formwork, and the anchor bolts were positioned and secured. A plastic tube was also installed at the designated location. The concrete was mixed in the laboratory using 372 kg/m³ of cement, 96 kg/m³ of fly ash, 21 kg/m³ of silica fume, 729 kg/m³ of fine sand, 541 kg/m³ of coarse sand, 529 kg/m³ of crushed stone, 5.32 kg/m³ of water-reducing admixture, and 110 kg/m³ of water. During casting, four companion concrete cube specimens were prepared for each test specimen to determine the concrete

compressive strength. The formwork was removed two days after casting. The specimens and concrete cubes were subsequently labeled and cured under identical environmental conditions for 28 days. The principal fabrication procedures are illustrated in Figure 6.

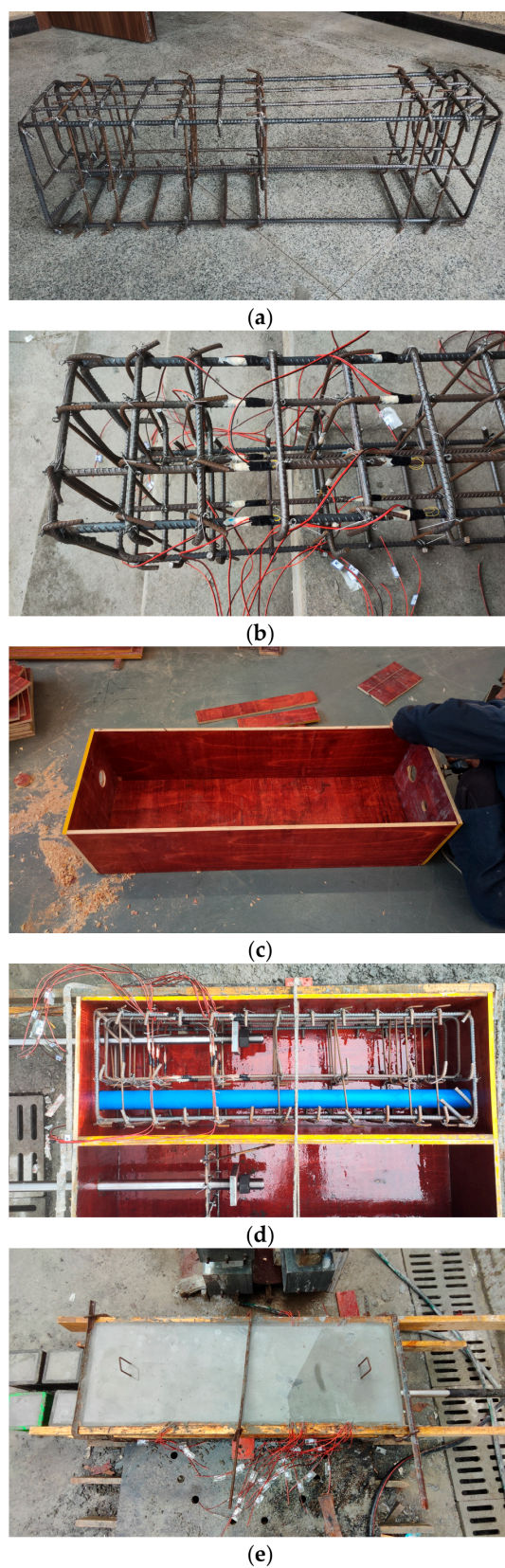


Figure 6. Specimen fabrication: (a) Reinforcement cage assembly; (b) Strain-gauge installation; (c) Formwork fabrication; (d) Formwork assembly; (e) Concrete casting.

2.3. Test Setup

The test setup mainly consisted of a reaction frame, a 2000 kN actuator (Servotest Testing Systems Ltd. Egham, UK), a steel support (Tiegandang Panfa Building Materials Business Department, Jiangjin District, Chongqing, China), and reaction beams (Tiegandang Panfa Building Materials Business Department, Jiangjin District, Chongqing, China). The actuator was installed vertically, with its upper end connected to the reaction frame through a connecting assembly and a loading plate attached to its lower end. The steel support was anchored to the laboratory floor using four threaded rods, and the specimen was positioned within the retaining brackets mounted on the support. The upper end of the embedded anchor bolt was connected to the loading plate to transfer the applied tensile load. Reaction beams were placed over the specimen and connected to the laboratory floor using threaded rods, through which a prescribed pretensioning force was applied to restrain the specimen during loading. The test setup is illustrated in Figure 7. The restraint parameters of the reaction beams are added as follows: the pretensioning force applied to each reaction beam was 700 kN, the clear distance between the reaction beam and the anchor axis was 180 mm, and the contact area was 45,000 mm². The reaction beams influenced the crack trajectory and may have affected the ultimate capacity. The upper end of the anchor bolt was connected to the loading plate through nuts and washers, and the actuator, loading plate, and anchor axis were aligned before testing to reduce sliding, eccentric loading, and unintended transverse clamping.

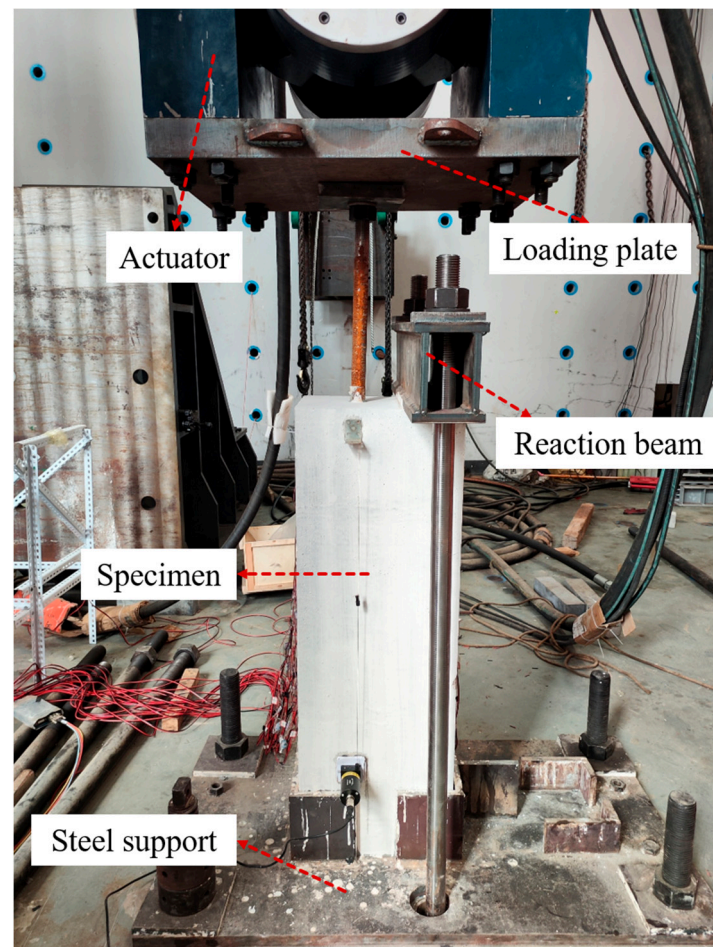


Figure 7. Test setup.

2.4. Measurement Scheme

The primary measurements included the cumulative surface crack-opening displacement and the strains (Huangyan Chengli Engineering Sensor Factory, Taizhou, China) of selected reinforcing bars. The displacement reported in this study does not represent the axial displacement of the anchor bolt or the displacement of the loading plate. It was measured using a draw-wire displacement transducer installed on the front face of the specimen and aligned with the projected position of the anchor bolt. To capture the cumulative opening displacement of the concrete cracks as completely as possible, the upper and lower attachment points of the transducer were positioned 50 mm below the top surface and 250 mm above the bottom surface of the specimen, respectively. This arrangement maximized the coverage of the potential cracking region while reducing the risk of transducer detachment caused by crack propagation. The arrangement of the displacement transducer is shown in Figure 8. Strain gauges were attached to the principal load-carrying reinforcing bars near the end plate. Specifically, the gauges were installed on six longitudinal reinforcing bars and four transverse ties at the elevation corresponding to the end plate, as well as on the reinforcing bars at the corresponding locations in the layer immediately above. In Figure 8, “L” denotes strain gauges attached to transverse ties, whereas “Z” denotes strain gauges attached to longitudinal reinforcing bars. The arrangement of the reinforcement strain gauges is illustrated in Figure 8.

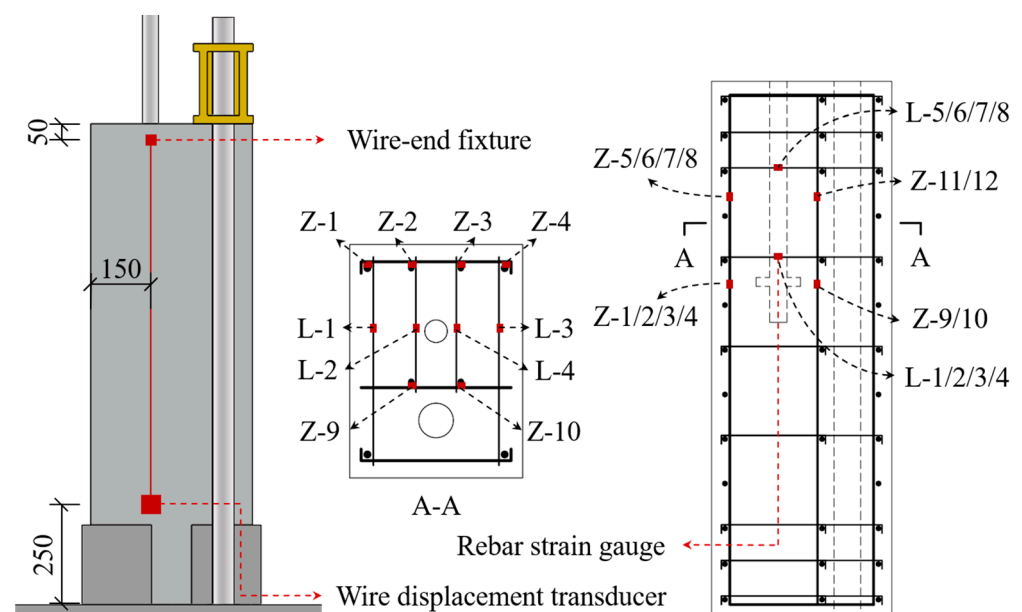


Figure 8. Measurement scheme.

2.5. Loading Protocol

A combined load–displacement controlled loading protocol was adopted. Loading was initially applied under load control in increments of 100 kN. Each 100 kN increment was applied over approximately 1 min and then maintained for 1 min for observation and data recording. Once visible cracks developed in the specimen, the loading mode was switched to displacement control, with an increment of 2 mm at each loading stage. Each specimen was subjected to monotonic static loading only. At each loading stage, the applied load or displacement was maintained for 1–2 min to allow the test phenomena to be observed and recorded. The test was terminated when the specimen exhibited severe failure or when the load-carrying capacity decreased to less than 85% of the peak load. It should be noted that no anchor preload was applied before testing.

3. Test Results and Analysis

3.1. Test Phenomena and Failure Modes

All the specimens exhibited relatively ductile failure characteristics except for the plain-concrete specimen LB-1. During loading, cracks generally initiated near the elevation of the end plate, indicating that the tensile force of the anchor bolt was mainly transferred to the surrounding concrete through local bearing of the end plate. With increasing load, the cracks propagated toward the front and rear surfaces and extended transversely. When the cracks developed to the region constrained by the upper reaction beam, their propagation paths were forced to deflect upward. Consequently, the final failure pattern was mainly composed of transverse cracks near the end-plate level and diagonal cracks between the end plate and the reaction beam, forming an approximately L-shaped crack pattern on the front face.

Specimen LB-1 exhibited a single wide transverse crack, as shown in Figure 9a. Its pull-out resistance was mainly provided by the tensile strength of the concrete. Once the local tensile stress exceeded this strength, the crack propagated rapidly and the load-carrying capacity dropped sharply. This indicates that, without reinforcement, the pull-out resistance was mainly provided by the tensile capacity of concrete, and crack propagation could not be effectively restrained. Compared with LB-1, the reinforced specimens showed more distributed cracking and a more gradual failure process. For the reference specimen LB-2, the primary crack developed from the end-plate region and extended toward the area beneath the reaction beam, while several secondary cracks appeared around the main crack and in the lower tensile region, as shown in Figure 9b. This suggests that reinforcement restrained crack opening after concrete cracking and promoted stress redistribution within the specimen, thereby improving ductility. The crack pattern of LB-3 was generally similar to that of LB-2, but the primary crack was more concentrated near the end-plate region, as shown in Figure 9c. This indicates that changing the local bearing condition mainly affected the crack distribution and stress concentration around the end plate, while the overall failure was still governed by end-plate bearing, concrete splitting and reinforcement restraint. For LB-4, the main crack path was relatively regular and a clear L-shaped crack developed from the end-plate region to the area beneath the reaction beam, as shown in Figure 9d. The concrete in the tensile zone tended to act as a relatively integral block under the combined action of the anchor bolts, but the failure was still controlled by the cracking and splitting of the concrete between the end plate and the reaction beam. LB-5 exhibited a relatively irregular crack distribution, as shown in Figure 9e. The sleeve weakened the bond interaction between the anchor shaft and the surrounding concrete, causing the tensile force to be transferred more directly through the end plate. As a result, the cracking was mainly governed by end-plate bearing and concrete splitting.

Overall, the semi-embedded anchor bolts did not fail through a complete free concrete cone breakout mechanism. Instead, the failure was controlled by the combined effects of end-plate bearing, concrete splitting, reinforcement restraint, and the boundary restraint introduced by the reaction beams. This boundary restraint should be interpreted as a test-boundary-dependent effect; it may partly represent the local confinement of the actual transition section, but its quantitative influence on the ultimate pull-out capacity requires further investigation. Reinforcement was the key factor improving ductility and limiting crack propagation, whereas the end-plate size, anchor-bolt arrangement and sleeve mainly influenced the local load-transfer mechanism and crack distribution.

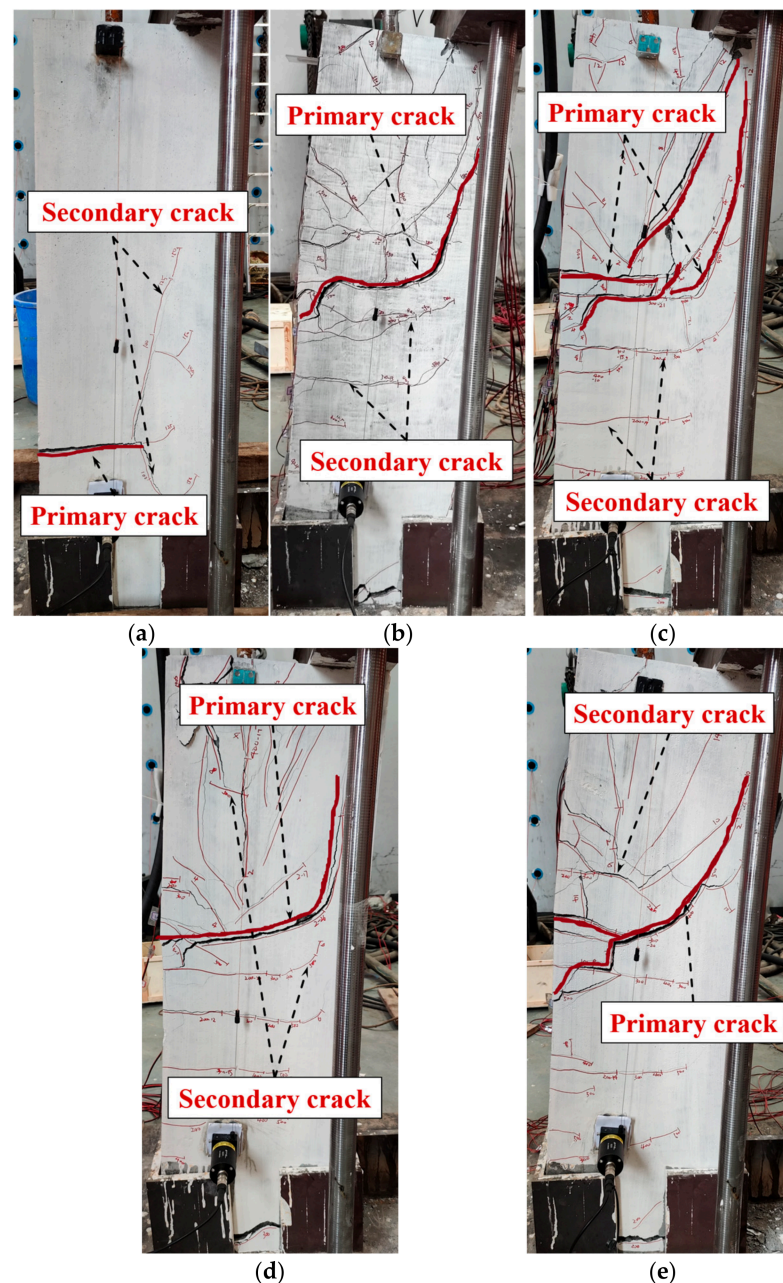


Figure 9. Failure modes of specimens: (a) LB-1; (b) LB-2; (c) LB-3; (d) LB-4; (e) LB-5.

No anchor steel fracture was observed in any specimen. The tensile resistance of the Grade 10.9 M39 threaded rod was higher than the maximum measured load per anchor in this test series; therefore, the failure was governed by concrete cracking and reinforcement restraint rather than anchor steel rupture.

3.2. Result Analysis

The load–cumulative surface crack-opening displacement curves of the specimens are presented in Figure 10, and the corresponding peak cumulative surface crack-opening displacements and ultimate pull-out capacities are summarized in Table 2. The load was recorded by the load cell integrated into the actuator, while the displacement was measured using the draw-wire displacement transducer installed on the specimen surface. It should be noted that the displacement reported in this study represents the cumulative surface crack-opening displacement measured on the front face of the specimen, rather than the axial displacement of the anchor bolt or the displacement of the loading plate.

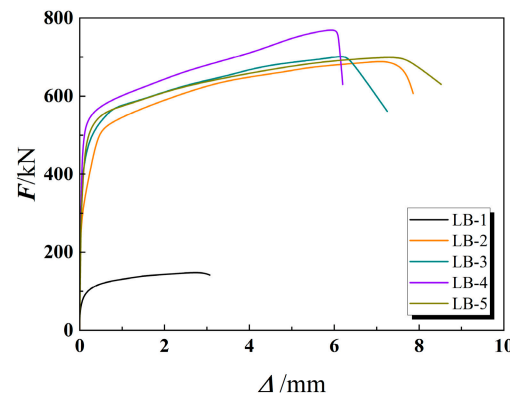


Figure 10. Load–cumulative surface crack-opening displacement curves.

Table 2. Peak cumulative surface crack-opening displacement and ultimate pull-out capacity.

Specimen Number	Peak Cumulative Surface Crack-Opening Displacement/mm	Ultimate Pull-Out Capacity/kN
LB-1	2.853	150
LB-2	7.254	691
LB-3	6.218	712
LB-4	6.051	776
LB-5	7.525	702

The plain-concrete specimen LB-1 showed the lowest bearing capacity and deformation capacity, with an ultimate load of only 150 kN and a peak cumulative surface crack-opening displacement of 2.853 mm. In contrast, all reinforced specimens exhibited much higher pull-out capacity and more stable post-cracking behavior, indicating that reinforcement significantly improved both the pull-out capacity and deformation capacity of the semi-embedded anchor bolts. The reinforced specimens showed similar load–cumulative surface crack-opening displacement responses. At the initial stage, the load increased rapidly with crack-opening displacement, indicating high initial stiffness of the uncracked specimen. After concrete cracking, the stiffness decreased, but the load continued to increase gradually, showing an obvious post-cracking development stage. After reaching the peak load, severe cracking and local damage occurred, followed by a reduction in load-bearing capacity. Compared with the reference specimen LB-2, LB-3 showed an ultimate pull-out capacity increase from 691 kN to 712 kN, corresponding to 3.1%. Because only one specimen was tested for each configuration, this small difference should be regarded as an indicative trend rather than a statistically confirmed improvement. LB-4 reached the highest ultimate pull-out capacity of 776 kN. However, the average capacity per anchor was 388 kN, which was approximately 43.8% lower than the 691 kN capacity of the single-anchor reference specimen LB-2. This confirms a significant group effect caused by overlapping stress fields and a shared concrete tensile zone. LB-5 had an ultimate pull-out capacity of 702 kN, only 1.6% higher than that of LB-2. This difference is within the possible experimental scatter of concrete anchorage tests and should not be interpreted as a clear strengthening effect of the sleeve. Overall, reinforcement was the dominant factor improving pull-out performance, while end-plate size, anchor number and sleeve mainly affected local load transfer, cracking development, and apparent crack-opening behavior.

In addition to the load–cumulative surface crack-opening displacement response, the strain development of the reinforcement was analyzed to clarify the load-transfer mechanism. As shown in Figure 8, strain gauges were arranged on the longitudinal bars and transverse ties near the end-plate region and in the upper part of the reinforcement

cage. Owing to symmetry, one-half of the measured strain results was selected for comparison. The yield strain of the reinforcement was calculated as approximately $2400 \mu\epsilon$ ($\epsilon_y = f_y/E_s = 482 \text{ MPa}/200 \text{ GPa} \approx 2400 \mu\epsilon$, where $E_s = 200 \text{ GPa}$) and is indicated by the dashed line in Figure 11. The load–strain curves of LB-3 and LB-4, representing the enlarged end-plate and two-anchor configurations, respectively, are shown in Figure 11a,b, respectively. For LB-3, the reinforcement near the end plate was mobilized at an early loading stage. After concrete cracking, the strains in the transverse ties and longitudinal bars near the end-plate level increased rapidly, and some gauges exceeded the yield strain at peak load. This indicates that the reinforcement around the end plate effectively restrained crack opening and contributed to the post-cracking bearing capacity. In contrast, the upper longitudinal bars showed relatively small strains, suggesting that the main load-transfer region was concentrated near the end plate. For LB-4, the strain development was generally delayed. Most reinforcement strains remained low before cracking and increased significantly only after the concrete cracked. The upper reinforcement was mobilized mainly near failure, and most measured strains remained below the yield strain at peak load. This may be attributed to the combined action of the two anchor bolts, which enlarged the local load-transfer region and reduced the demand on individual reinforcing bars. Overall, the strain results show that the reinforcement near the end plate played the dominant role in resisting pull-out cracking, while the upper reinforcement mainly contributed during the later damage stage. This is consistent with the observed failure modes governed by end-plate bearing, concrete splitting and reinforcement restraint.

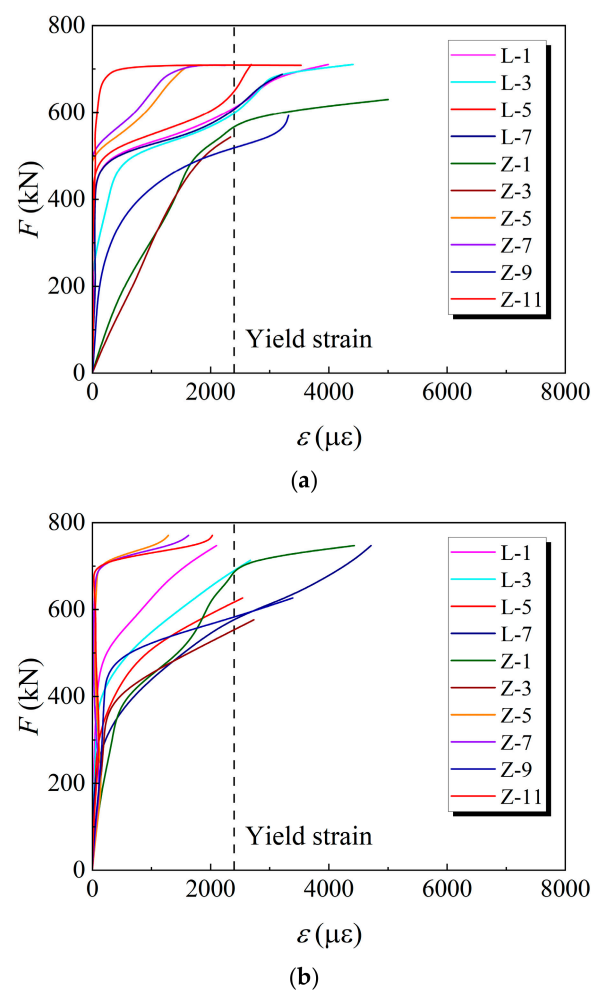


Figure 11. Reinforcement strain curves: (a) LB-3; (b) LB-4.

4. Evaluation of Pull-Out Bearing Performance

4.1. Existing Calculation Methods

The method specified in EN 1992-4 [18] is based on the concrete cone failure theory and calculates the tensile concrete breakout resistance of anchor bolts using the reference resistance, the actual projected area, and the relevant modification factors. This method incorporates several safety reduction factors and is relatively sensitive to variations in edge distance and projected area. Its basic expression is given in Equations (1) and (2). In the present study, the EN 1992-4 calculation is used as a benchmark only. The resistance obtained from the standard formulation is a characteristic resistance and therefore contains statistical and safety margins. In addition, the tested specimens have a large effective embedment depth relative to the limited cross-section and edge distance, so the configuration is outside the normal calibration range of an unconstrained CCD-type concrete breakout model.

$$F_1 = N_{Rk,c} = N_{Rk,c}^0 \frac{A_{c,N}}{A_{c,N}^0} \psi_{s,N} \psi_{re,N} \psi_{ec,N} \quad (1)$$

$$N_{Rk,c}^0 = K_1 \sqrt{f_{ck}} h_{ef}^{1.5} \quad (2)$$

where: $N_{Rk,c}^0$ is the basic resistance of a single anchor without edge or group effects; K_1 is a coefficient related to the condition of the concrete, taken as 8.9 for cracked concrete and 12.7 for uncracked concrete; f_{ck} is the characteristic compressive strength of concrete cylinders, calculated as $f_{ck} \approx 0.85 f_{cc}$ [19]; h_{ef} is the effective embedment depth, taken as the depth to the top surface of the end plate; $A_{c,N}$ is the actual projected area of the concrete cone failure surface, taking the area enclosed between the tension zone edge and the inner edge of the reaction beam; $A_{c,N}^0$ is the projected area of the ideal concrete cone failure surface, calculated using Equation (3); $\psi_{s,N}$ is the modification factor accounting for edge effects, calculated using Equation (4); $\psi_{re,N}$ is the modification factor accounting for concrete surface spalling, calculated using Equation (5); and $\psi_{ec,N}$ is the modification factor accounting for load eccentricity, calculated using Equation (6).

$$A_{c,N}^0 = 9h_{ef}^2 \quad (3)$$

$$\psi_{s,N} = 0.7 + 0.3 \frac{c}{1.5h_{ef}} \leq 1.0 \quad (4)$$

$$\psi_{re,N} = 0.5 + \frac{h_{ef}}{200} \leq 1.0 \quad (5)$$

$$\psi_{ec,N} = \frac{1}{\left(1 + \frac{2e_N}{3h_{ef}}\right)} \leq 1.0 \quad (6)$$

where: c is the distance from the center of the anchor to the nearest free edge of the concrete member, and e_N is the eccentricity of the resultant tensile force acting on the anchor group relative to the centroid of the anchors subjected to tension.

The 45° concrete cone failure method [20] assumes that, under tensile loading, cracks initiate near the embedded end of the anchor and propagate toward the concrete surface, forming an approximately 45° conical failure surface. Owing to its simple formulation, this method provides an intuitive representation of the basic characteristics of concrete cone failure during anchor pull-out. The corresponding pull-out capacity can be calculated using Equation (7). It should be noted that ACI 349-97 is a superseded code; in this study, the 45° cone method is retained only as a historical simplified benchmark for comparison with conventional cone-type assumptions, rather than as a current design recommendation.

$$F_2 = 0.3 f_{cc}^{0.5} A_{c,45} \quad (7)$$

where: F_2 is the pull-out capacity calculated using the 45° cone method; f_{cc} is the concrete cube compressive strength; and $A_{c,45}$ is the effective concrete failure area determined based on the 45° cone failure geometry.

The modified method proposed by Nilforoush [21] was developed based on the concrete capacity model and accounts for the effects of member thickness, anchor-head bearing area, and surface reinforcement on the pull-out capacity, as expressed in Equations (8) and (9).

$$F_3 = N_{Nil} = N_{u,m} \psi_H \psi_{AH} \psi_{Sr} \quad (8)$$

$$N_{u,m} = 6.585 \sqrt{f_{cm}} h_{ef}^{5/3} \quad (9)$$

where: F_3 is the pull-out capacity calculated using the Nilforoush method; $N_{u,m}$ is the reference pull-out capacity; f_{cm} is the concrete cylinder compressive strength, calculated as $f_{cm} = f_{ck} + 8$ [19]; ψ_H is the modification factor accounting for member thickness, calculated using Equation (10); ψ_{AH} is the modification factor accounting for the anchor-head bearing area, calculated using Equations (11) and (12); and ψ_{Sr} is the modification factor accounting for surface reinforcement, calculated using Equation (13).

$$\psi_H = \min \left[\left(\frac{H}{2h_{ef}} \right)^{0.25}, 1.20 \right] \quad (10)$$

$$\psi_{AH} = \left(\frac{A_h}{A_{h,min}} \right)^{0.1} \quad (11)$$

$$A_{h,min} = \frac{N_{u,m}}{15f_c} \quad (12)$$

$$\psi_{Sr} = \begin{cases} \min \left[1.35 \left(\frac{H}{h_{ef}} \right)^{-0.25}, 1.20 \right]; & H \leq 3h_{ef} \\ 1 & ; H > 3h_{ef} \end{cases} \quad (13)$$

where H is the member thickness; A_h is the net bearing area of the anchor head or end plate; and $A_{h,min}$ is the minimum bearing area required to limit the peak bearing stress beneath the end plate to $15f_c$.

The linear elastic fracture mechanics (LEFM) method [22] is based on fracture mechanics and treats the pull-out failure of an anchor bolt as a process involving crack initiation and propagation in concrete. According to this method, the pull-out capacity depends on the elastic modulus and fracture energy of concrete [23], as well as the effective embedment depth, as shown in Equations (14) and (15).

$$F_4 = N_{LEFM} = 2.1 h_{ef}^{1.5} \sqrt{E_c G_f} \quad (14)$$

$$G_f = G_{f0} \left(\frac{f_{cm}}{10} \right)^{0.7} \quad (15)$$

where E_c is the elastic modulus of concrete, calculated as $E_c = 22,000 \times (f_{cm}/10)^{0.3}$ MPa [19]; G_f is the fracture energy of concrete; and G_{f0} is the reference fracture energy determined by the maximum aggregate size, the maximum aggregate size adopted in this test is 25 mm.

4.2. Analysis of Prediction Results

The pull-out capacities of the five specimens were calculated using the EN 1992-4 method [18], the 45° cone method [20], the Nilforoush method [21], and the LEFM method [22], and the predicted values were compared with the test results, as summarized in Table 3. The relationship between the predicted and experimental capacities is presented in Figure 12. The dashed line ($y = x$) represents perfect agreement between

the calculated and measured values. Data points close to this line indicate good predictive accuracy. Points above the line correspond to overestimated capacities and therefore unconservative predictions, whereas points below the line indicate underestimated capacities and conservative predictions. These methods are evaluated here as post-test benchmark approaches. The comparison is intended to clarify the limitations of existing idealized cone or fracture-mechanics assumptions for the tested semi-embedded configuration, rather than to claim that all standard methods are directly applicable to the present out-of-scope geometry.

Table 3. Comparison between test results and calculated results from existing methods.

Specimen Number	F_t	Ref. [18]		Ref. [20]		Ref. [21]		Ref. [22]	
		F_1	F_t/F_1	F_2	F_t/F_2	F_3	F_t/F_3	F_4	F_t/F_4
LB-1	150	30.91	4.85	200.61	0.75	1392.46	0.11	1327.00	0.11
LB-2	691	30.49	22.66	197.89	3.49	1371.68	0.50	1286.38	0.54
LB-3	712	29.63	24.04	192.31	3.70	1452.03	0.49	1274.80	0.56
LB-4	776	25.04	30.97	188.30	4.12	1391.98	0.56	1249.65	0.62
LB-5	702	28.88	24.31	187.46	3.75	1292.38	0.54	1244.38	0.56
Mean ratio for reinforced specimens	-	-	25.49	-	3.77	-	0.52	-	0.57
CoV for reinforced specimens	-	-	14.6%	-	7.0%	-	6.3%	-	6.1%

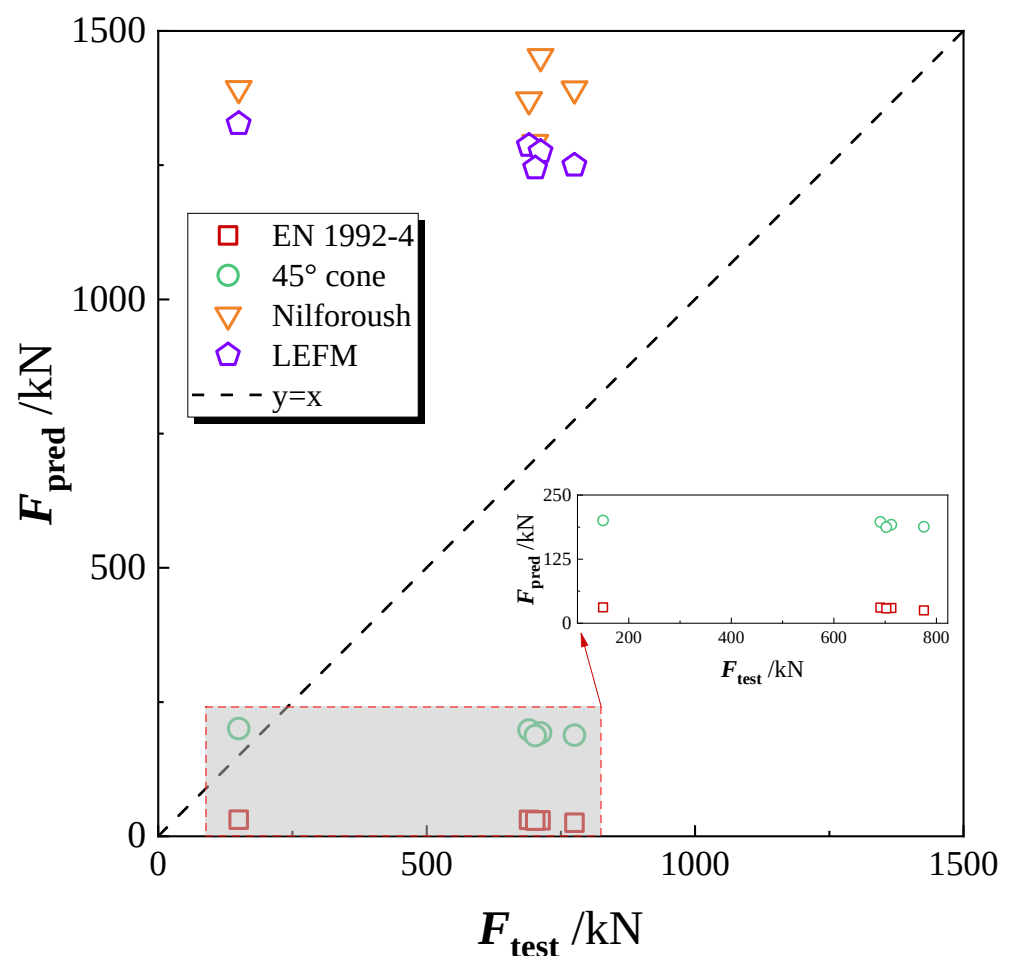


Figure 12. Comparison between predicted and experimental pull-out capacities (Methods: EN 1992-4 [18]; 45° cone [20]; Nilforoush [21]; LEFM [22]).

As shown in Figure 12 and Table 3, the predictions obtained using the four methods differ considerably. The EN 1992-4 method [18] first determines the concrete breakout resistance of a single anchor under ideal conditions and then applies modification factors accounting for edge distance, concrete condition, load eccentricity, and the ratio of the actual to ideal projected failure areas. For the specimens tested in this study, the large effective embedment depth, small cross-sectional dimensions, and limited edge distance led to a very small projected-area ratio. Therefore, the characteristic EN 1992-4 resistance was conservative. This conservatism should be interpreted from two aspects: first, the standard resistance is a characteristic value with embedded statistical margin; second, the tested semi-embedded anchor configuration was outside the assumptions of a complete unconstrained concrete cone breakout. The observed failure mode involved end-plate bearing, concrete splitting, reinforcement restraint, and boundary effects, rather than a classical free cone breakout. The 45° cone method [20] provided closer predictions than EN 1992-4 for the reinforced specimens but still underestimated their capacities; however, it slightly overestimated the plain-concrete specimen LB-1.

The Nilforoush method [21] and the LEFM method [22] both markedly overestimated the pull-out capacity. Although the Nilforoush method incorporates the effects of member thickness, anchor-head bearing area, and surface reinforcement into the basic concrete capacity model, all predicted values were substantially higher than the test results. The mean test-to-predicted capacity ratio, F_t/F_3 , was only 0.52. Similarly, the LEFM method, which accounts for the elastic modulus, fracture energy, and effective embedment depth of concrete, yielded a mean F_t/F_4 ratio of only 0.57, indicating unconservative predictions. The large discrepancies are mainly attributable to the underlying assumptions of these two methods, which were generally developed for conventional cast-in headed anchors or idealized crack-propagation conditions. By contrast, the specimens investigated in this study represent local components of a concrete transition section and are characterized by a semi-embedded configuration, limited cross-sectional dimensions, localized load transfer through the end plate, complex boundary restraints, and significant reinforcement effects. Therefore, their actual load-transfer and failure mechanisms cannot be adequately represented by either a single complete concrete cone breakout mode or an idealized crack-propagation model.

In summary, the four existing methods showed limited applicability to the semi-embedded anchor bolts in the concrete transition section investigated in this study. The characteristic EN 1992-4 resistance [18] was conservative, while the conservatism resulted from both the brittle basis of the standard formulation and the extremely limited projected failure area of the present out-of-scope geometry. The 45° cone method [20] underestimated the capacity of the reinforced specimens but slightly overestimated that of the plain-concrete specimen. In contrast, the Nilforoush method [21] and the LEFM method [22] considerably overestimated the capacities and may therefore lead to unconservative predictions. These discrepancies are mainly attributable to the limited cross-sectional dimensions, localized load transfer through the end plate, complex boundary restraint, reinforcement contribution, and the reduction in effective load-transfer height caused by the deep semi-embedded configuration. Therefore, conventional anchor pull-out models should not be applied directly to this type of component without modification.

It should be emphasized that the purpose of the present comparison is to evaluate the applicability and limitations of the existing methods rather than to establish a new predictive equation. Because only five specimens were tested and each configuration was represented by a single specimen, the available database is insufficient to independently quantify the contributions of concrete cracking, end-plate bearing, reinforcement restraint, boundary restraint, and effective load-transfer height. Therefore, no new calculation model

is proposed in this study. The development of a reliable mechanics-based model requires repeated tests covering broader parameter ranges, together with systematic numerical analyses and independent experimental validation.

5. Conclusions and Recommendations

This study conducted pull-out tests on five 1:2-scale local specimens to investigate the pull-out behavior of semi-embedded anchor bolts in the transition section of steel–concrete hybrid wind turbine towers. The effects of reinforcement, end-plate size, number of anchor bolts, and sleeve installation were examined, and the applicability of existing calculation methods was evaluated. The main conclusions and limitations are as follows: (1) The failure mode of specimens with semi-embedded anchor bolts was different from the conventional unconstrained concrete cone breakout mode. The plain-concrete specimen LB-1 exhibited brittle cracking and a rapid reduction in load-carrying capacity after cracking, whereas the reinforced specimens showed a combined mechanism involving end-plate bearing, concrete splitting, reinforcement restraint, and boundary effects. (2) Reinforcement was the dominant factor improving the pull-out performance. Compared with LB-1, the reinforced reference specimen LB-2 showed a much higher ultimate capacity and cumulative surface crack-opening deformation. Because only one specimen was tested for each configuration, the observed effects of end-plate size and sleeve installation should be regarded as indicative trends rather than statistically confirmed conclusions. (3) The two-anchor specimen reached the highest total ultimate capacity of 776 kN, but its average per-anchor capacity was much lower than that of the single-anchor reference specimen, indicating a significant group effect caused by overlapping stress fields and a shared concrete tensile zone. (4) Existing methods showed limited applicability to the present restrained semi-embedded anchorage configuration. The characteristic EN 1992-4 resistance was conservative, while this conservatism was caused by both the fractile formulation and the extremely limited projected failure area. The 45° cone method underestimated the reinforced specimens but overestimated the plain-concrete specimen. The Nilforoush and LEM methods overestimated the capacities and may be unconservative. (5) Several limitations should be acknowledged. The present tests were monotonic only, whereas wind turbine tower anchorages are subjected to cyclic and fatigue loading in service. The 1:2 specimens were geometrically scaled, but strict fracture-mechanics similitude was not achieved because concrete fracture energy, aggregate size, reinforcement bond behavior, and anchor-preload condition were not scaled. The rectangular local specimens also cannot fully reproduce the circumferential continuity and ring action of the actual circular transition section, and the influence of the reaction-beam boundary restraint on ultimate capacity has not been fully quantified. Future studies should include repeated specimens, cyclic and fatigue loading, varied anchor length and diameter, increased specimen depth or thickness, and refined numerical simulations or prototype-scale tests. These studies are required to establish a broader database, quantify the individual resistance contributions, and support the future development and independent validation of a reliable mechanics-based prediction model.

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