

Article

Bond Behavior of Inclined U-Jacket-to-Concrete Joints: Tests and Modeling

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Abstract

Reinforced concrete beams with a fiber-reinforced polymer (FRP) plate bonded to their soffit, known as FRP-plated RC beams, commonly fail due to premature debonding of the FRP plate, limiting the utilization of the FRP strength. Inclined U-jacketing has been demonstrated to be effective as the end anchorage for mitigating debonding failures. The mechanism by which the inclined U-jacketing mitigates debonding failure remains unclear, and no design approach has been developed. Therefore, the present study has been conducted to investigate the mitigating effects of the key parameters of the inclined U-jacket through a series of four-point bending tests and systematic modeling. The test results indicate that both the inclination angle and the chamfer radius significantly affected the bond behavior of inclined U-jacket-to-concrete joints. Compared with the 45° configuration, reducing the inclination angle to 30° increased the peak load and peak displacement by 85.4% and 81.6%, respectively. In contrast, the effect of U-jacket side height became negligible once an effective bonded height had been reached, as increasing the side height from 75 mm to 120 mm changed the peak load by only 2.17%. In addition, a pre-peak parameter identification framework based on a power-function-type cohesive element constitutive relationship was proposed and validated. By analyzing the power-function parameters, namely the coefficient a and exponent b , the influences of U-jacket geometric variables on interfacial mechanical behavior were quantitatively characterized. The proposed approach provides experimentally verifiable parameterization to support the optimized design of inclined U-jacket anchorage systems.

Keywords: FRP plate; reinforced concrete beams; debonding failure; anchorage; U-jackets

1. Introduction

Externally bonding fiber-reinforced polymer (FRP) plates has been widely adopted to enhance the flexural load-carrying capacity of reinforced concrete (RC) beams [1–3], resulting in what are commonly referred to as FRP-plated RC beams. Such beams typically fail through two distinct failure modes: (1) intermediate crack-induced debonding, which initiates at the root of a major flexural crack and propagates toward the FRP plate end; and (2) concrete cover separation, which originates at one of the FRP plate ends and propagates along the level of the tension steel reinforcement toward the beam midspan [4,5]. Both debonding failures are premature in nature, often occurring when the maximum FRP strain reaches only 30% to 60% of its rupture strain, indicating low utilization efficiency of the FRP material [6]. Moreover, the debonding strain has been reported to decrease with increasing FRP thickness, while the strengthening efficiency tends to diminish as the internal steel reinforcement ratio increases [7,8].



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To mitigate debonding failure and improve the utilization efficiency of FRP materials, various anchorage measures have been explored, including metallic anchorages [9–11] and non-metallic solutions [12–14]. Recent studies on structural strengthening and seismic retrofitting have also shown that externally attached strengthening systems can effectively improve structural performance, but their efficiency is highly dependent on the layout, geometric configuration, and connection mechanism of the attached components [15,16]. However, many anchorage measures are susceptible to corrosion or require drilling into concrete, whereas inclined U-jackets have been demonstrated to be a simple and effective alternative. In this configuration, FRP sheets are bonded to the soffit and side faces of the beam, thereby exploiting the concrete cross-section and its compressive resistance to provide anchorage [17,18]. Previous studies have further shown that U-clamps and FRP anchors can delay premature debonding, improve the strain utilization of flexural FRP reinforcement, and, in some cases, shift the governing failure mode from debonding to FRP rupture or concrete crushing [17,19]. The effectiveness of these anchorage systems depends on several design parameters, including anchorage layout, U-wrap width or area ratio, anchorage stiffness, and installation location [20]. Test results from the authors' group have indicated that the inclination angle significantly affects strengthening performance, with 45° U-jackets outperforming 90° configurations in certain cases [21–26]. However, two major research gaps still remain: (1) the effects and interaction trends of key parameters—namely concrete strength, chamfer radius, FRP laminate width and thickness, and U-jacket width, inclination angle, thickness, and side height—have not been systematically quantified, and their influence patterns on anchorage performance are not yet fully understood; (2) existing finite-element (FE) models still lack constitutive representations capable of accurately capturing the anchorage behavior of inclined U-jackets in FRP-strengthened systems.

To address these gaps, a series of lap-joint tests between inclined U-jackets and concrete were conducted using four-point bending configurations [27,28]. The influence of key parameters—including concrete strength, chamfer radius, FRP plate width and thickness (layer number), U-jacket width, inclination angle, and side height—on the bond properties at the U-jacket–concrete interface was systematically investigated. In parallel, a finite-element (FE) model was developed based on established modeling frameworks, theoretical formulations, and a power-function-type cohesive element constitutive model, and its validity was verified by comparison with the experimental load–slip responses. A sensitivity analysis of the power-function parameters a and b was further conducted to clarify how U-jacket geometry governed the load–slip behavior and interfacial mechanical response.

2. Experimental Program

2.1. Specimen Design

An experimental program was designed to evaluate the effects of concrete strength, chamfer radius, FRP width and thickness, and U-jacket width, inclination angle, thickness, and side height on the interfacial bond performance between the U-jacket and the concrete substrate. All specimens were numbered using C30, R25, P50, U25, I45, T2, Z100, and CG120 (Figure 1). The coding definitions are as follows: “C30” denotes a concrete strength grade of C30; “R25” denotes a chamfer radius of 25 mm; “P50” denotes a FRP plate width of 50 mm; “U25” denotes a U-jacket width of 25 mm; “I45” denotes an inclination angle of 45°; “T2” denotes 2 FRP layers; “Z100” denotes that the distance from both the load end and free end of the FRP to the U-jacket within the concrete region is 100 mm; and “CG120” denotes a side height of the U-jacket of 120 mm.

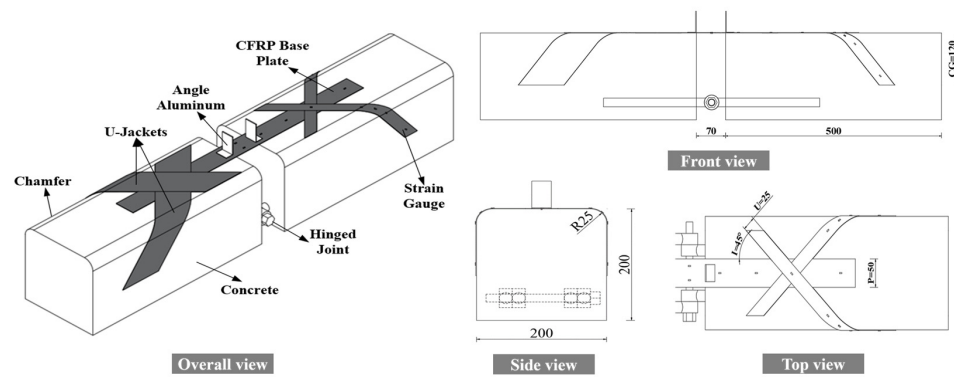


Figure 1. Schematic diagram of research parameters (unit: mm).

To isolate the load–slip response of the U-jacket–concrete interface, no bonding treatment was applied between the FRP base plate and the concrete surface. Potential interference associated with FRP plate–concrete bonding was thereby eliminated. The control variable method was employed to conduct comparative analysis between parameter-adjusted specimens and standard specimens. To minimize random experimental errors and ensure the reliability and accuracy of test data, parallel tests were performed under each variable condition, with each parallel test group comprising two specimens.

Table 1 lists details for all test specimens. For clarity and ease of analysis, all variable names in the table are abbreviated, with each variable corresponding to two specimens. C30R25 serves as the reference specimen, while other designations indicate modified values of the reference variable. For example, R35 denotes a specimen where the chamfer radius was changed from 25 mm to 35 mm for the reference specimen, while all other variables remained unchanged.

Table 1. Detailed information for all test specimens.

Name		Number of Test Specimens
Baseline group	C30R25	2
Base plate width	C30P50	2
	C30P100	2
Chamfer	C30R25	2
	C30R35	2
	C30R45	2
	C50R35	2
	C50R45	2
Inclination angle	C30I30	2
	C30I60	2
	C30I90	2
	C30I135	2
	C50I60	2
	C50I90	2
	C50I135	2
U-Jackets width	C30U25	2
	C30U50	2
	C30U75	2
Side height	C30CG75	2
	C30CG120	2
Thickness	C30T1	2
	C30T2	2
	C30T3	2

The specimen consisted of a concrete block formed with chamfered edges using wooden strips, and its overall dimensions were controlled. The cross-sectional width and height were both 200 mm. The specimen length was 500 mm in most cases, whereas specimens with an inclination angle of 30° were cast with a length of 700 mm. A steel reinforcement cage was embedded in the concrete. The reinforcement grade was HRB400E, and the concrete cover thickness was 25 mm. Four longitudinal bars with a diameter of 14 mm were used, including two top bars and two bottom bars. The ends of the bottom longitudinal tensile bars were bent upward by 70 mm. Stirrups with a diameter of 8 mm were arranged at a spacing of 100 mm, as shown in Figure 2. To promote failure at the right end and facilitate observation, the U-jacket bonding length at the left end was intentionally increased to 75 mm, while all other variables were kept constant. The thickness of a single FRP sheet was 0.167 mm.

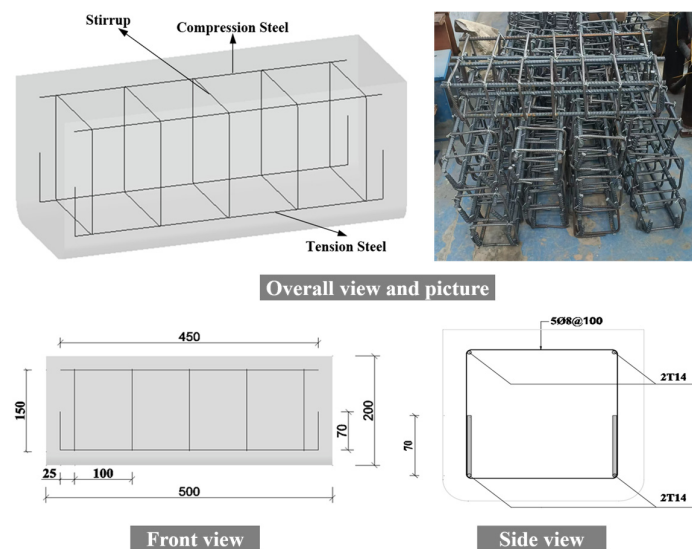


Figure 2. Photograph and schematic diagram of the reinforcement cage (unit: mm).

2.2. Specimen Preparation

Commercially available C30 and C50 concrete were used to cast specimens. During the first 7 days of curing, the specimens were watered and covered with a wet cloth. Cylindrical samples for compressive strength determination were cured under the same conditions and tested at 28 days. Prior to FRP installation, the bonding regions were laid out on the concrete surface using marked guidelines. The designated areas for the U-jackets were then roughened using a pneumatic needle scaler to satisfy the surface-preparation requirements for FRP-to-concrete bonding. The loading assemblies were subsequently installed. At the hinge between the lifting-ring rod and the threaded rod, washers were placed, and industrial lubricating oil was applied to reduce friction. Before FRP bonding, the epoxy primer adhesive Sikadur[®]-330 (Sika (China) Ltd., Suzhou, China) and the carbon-fiber impregnation resin KONY BOND E2500S (Konishi Co., Ltd., Osaka, Japan) were prepared in accordance with the manufacturers' instructions. The carbon-fiber fabric was fully impregnated with the epoxy resin on both sides. The structural adhesive was then uniformly applied to the prepared concrete surface. Afterward, the impregnated fabric was bonded to the predesigned locations. The strengthened specimens were cured for at least 7 days after bonding before testing.

2.3. Material Properties

Composite tensile specimens were fabricated in accordance with Specification D3039/D3039M-17 [29]. Tensile tests were performed to determine the elastic modu-

lus and tensile strength of the composite along the fiber direction. The yield strength, ultimate tensile strength, and elastic modulus of the reinforcing steel bars were determined in accordance with GB/T 28900-2022 [30]. The corresponding results are summarized in Table 2. The elastic modulus and Poisson's ratio of concrete were determined in accordance with ASTM C469/C469M [31]. For the C30 concrete, the elastic modulus and Poisson's ratio were 33,090 MPa and 0.21, respectively. For the C50 concrete, the corresponding values were 42,880 MPa and 0.21, respectively.

Table 2. Material properties.

Name	Type (mm)	Yield Strength (MPa)	Tensile Strength (MPa)	Elastic Modulus (MPa)
Steel	8	460	630	198,000
	14	433	601	178,000
FRP	FRP	/	3908	251,577

2.4. Test Apparatus and Instruments

As no specific standard is available for inclined U-jacket-to-concrete joints, the present test method was developed with reference to established FRP-to-concrete bonded-joint and anchorage test configurations. The adopted four-point bending setup and sectional equilibrium-based load conversion were used to characterize the load–slip behavior of the U-jacket anchorage. The loading arrangement is shown in Figure 3. Stepwise loading was applied using a manual hydraulic jack. The load increment was 2.5 kN per step, and each step was held for 1 min. Loading was continued until a clear anchorage failure was observed, or until interfacial slip continued to develop while the sustained load could no longer increase. During testing, video recording was performed on the underside of the designated failure end to capture the debonding process, and the recordings were used for failure-mode identification and post-test analysis. The maximum load attained in the experimental load–slip response was defined as $P_{\max}$.

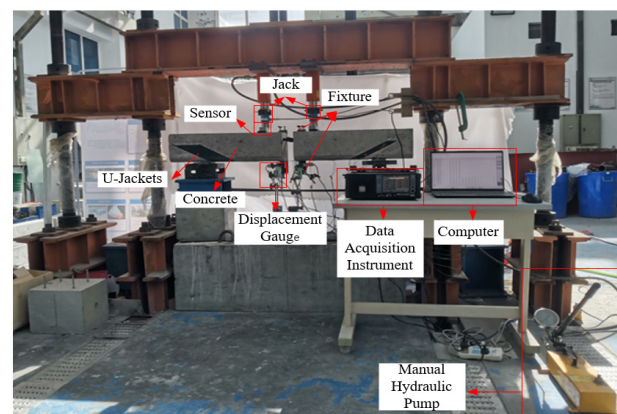


Figure 3. Testing setup.

A four-point bending testing configuration was adopted. Two concrete blocks were connected through lifting-ring rods and the FRP plate. Rotation was permitted at the lifting-ring hinges, and the bending moment at the hinge locations is assumed to be zero. By applying sectional equilibrium at midspan and taking moments at the hinge point M, the tensile force in the FRP at midspan is expressed as:

$$P_{\text{FRP}} = (49.5 - 15)/16 \times F \quad (1)$$

where F denotes the jack load, and P_{FRP} is the midspan tensile force in the FRP (Figure 4). For specimens with an inclination angle of 30° , the relationship becomes:

$$P_{\text{FRP}} = (54 - 15)/16 \times F \quad (2)$$

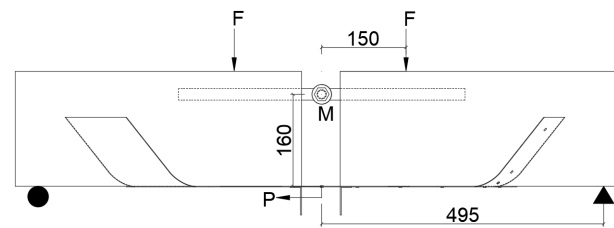


Figure 4. Calculation schematic diagram (unit: mm).

Strain was monitored using 15 strain gauges (Figure 5). Two gauges were attached at the quarter points on the top surface at midspan, and one gauge was attached at the center of the bottom surface at midspan. Five gauges were installed at locations 30 mm away from the free ends and the joint regions. One gauge was placed at the midpoint of the overlap region between the FRP plate and the U-jacket. In addition, six gauges were arranged around the chamfered region, including four at the tangency points and two at the chamfer centers. Relative slip between the FRP plate and concrete was measured using displacement transducers. Angle-aluminum tabs were bonded to the FRP plate, and the transducer supports were fixed to the concrete surface so that the differential displacement across the FRP–concrete interface could be recorded.

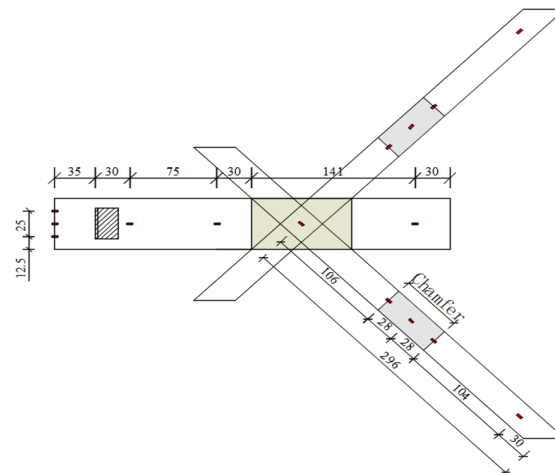


Figure 5. Schematic diagram of strain gauge layout (unit: mm).

3. Results and Discussions

3.1. Effect of Inclination Angle and Concrete Strength

Figure 6 presents the load–displacement responses and key parameters for the C30 concrete with different inclination angles. As shown in Figure 6a, pronounced differences were observed among the load–displacement curves. This indicates that the inclination angle governed the pre-peak response, the peak load level, and the associated slip/displacement capacity. For consistency, the specimen with a 45° inclination (C30I45) was used as the reference ($P_{\text{max}} = 29.97$ kN, $S_{\text{max}} = 2.56$ mm) in the following comparisons (Figure 6b). When the inclination angle was reduced to 30° , P_{max} and S_{max} increased to 55.58 kN and 4.66 mm, corresponding to increases of 85.4% and 81.6%, respectively, relative to I45. This trend suggests that at smaller inclination angles, force transfer at the FRP plate end was

enhanced by the U-jacket, thereby enabling higher load capacity together with greater displacement development [24].

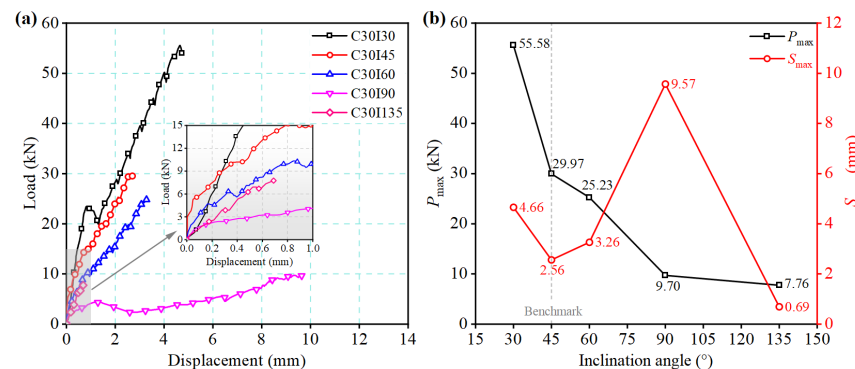


Figure 6. Load–displacement curves and parameter comparisons for the C30 concrete group at different inclination angles: (a) load–displacement curves and (b) peak load and displacement.

When the inclination angle was increased to 60° , P_{max} decreased to 25.23 kN, whereas S_{max} increased to 3.26 mm. This response indicates that the peak load capacity was reduced, while the deformation capacity was maintained or enhanced. A markedly different response was obtained for I90, where P_{max} decreased to 9.70 kN (a 67.6% reduction relative to I45) while S_{max} increased to 9.57 mm (a 273.3% increase). This behavior can be attributed to the limited effective fiber component aligned with the primary force-transfer direction in the 90° configuration, which constrained the peak capacity. Meanwhile, an extended slip development stage could occur at the interface and in localized regions, resulting in substantial displacement growth through progressive degradation and energy dissipation [24]. Meanwhile, both P_{max} and S_{max} of I135 dropped to the lowest levels of 7.76 kN and 0.69 mm, respectively, exhibiting typical weak and brittle behavior. Under the 135° configuration, the interfacial traction component resolved along the FRP fiber direction can become predominantly compressive. Given the pronounced tension–compression asymmetry of anisotropic FRP laminates, compressive loading is prone to inducing fiber micro-buckling (buckling instability) and subsequent kink-band formation, thereby degrading load-transfer efficiency and promoting premature brittle failure [32,33].

Figure 7 shows the load–displacement responses and parameter comparisons for C30 and C50 concrete at different inclination angles. As shown in Figure 7a, similar load–displacement characteristics were observed for the two strength grades. The influence of concrete strength decreased as the inclination angle increased. Specifically, for I60, when the concrete strength increased from C30 to C50, P_{max} increased by 34.2%, whereas S_{max} decreased by 16.9% (Figure 7b). This indicates that, under the I60 configuration, the upper load limit of the anchorage zone was primarily enhanced by the increased strength. Meanwhile, the slip capacity was reduced because interfacial damage propagation was inhibited and the overall stiffness was increased [34]. Conversely, for I90, only minor changes in peak load capacity were observed with increasing concrete strength, whereas displacement development was markedly extended. For C50I90, P_{max} and S_{max} reached 9.92 kN and 13.13 mm, respectively, corresponding to increases of 2.2% and 37.2% relative to C30I90 (9.70 kN and 9.57 mm). This contrast indicates that the peak values under I90 were more likely constrained by non-strength-related factors, such as U-jacket load-transfer efficiency and local fracture, while the higher concrete strength mainly improved energy dissipation and ductility by restraining localized cracking and debonding propagation [24]. For I135, P_{max} and S_{max} were 7.76 kN and 0.69 mm for C30, and 9.27 kN and 0.86 mm for C50, respectively. Although both peak load and displacement were increased by higher concrete strength under the I135 configuration, the absolute response levels remained

within the low-capacity and low-slip range. This indicates that the stress path at an inclination of 135° was inherently unfavorable. Therefore, strength improvement could only partially compensate for concrete-related weaknesses and could not overcome the overall inefficiency associated with brittle response at connections or insufficient effective fiber contribution.

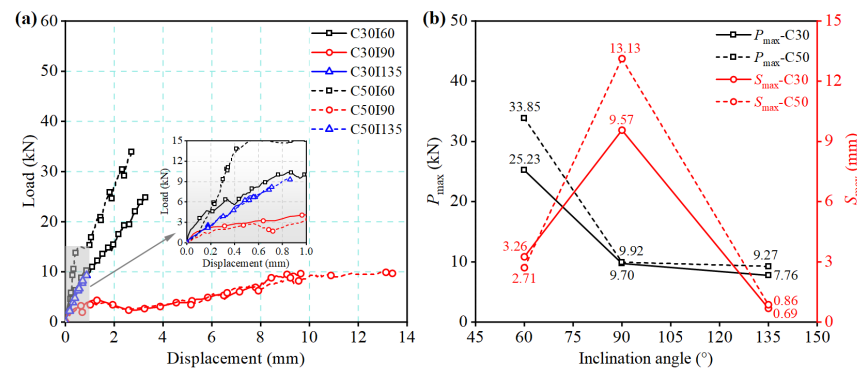


Figure 7. Load–displacement curves and parameter comparisons for the C30 and C50 concrete at different inclination angles: (a) load–displacement curves and (b) peak load and displacement.

Furthermore, Figure 8 presents three representative failure modes observed at different inclination angles. The observed patterns were consistent with the characteristics of the corresponding load–displacement responses. At an inclination angle of 30° , global debonding of the U-jacket was observed (Figure 8a). A relatively high load-carrying capacity was also achieved at this angle. Mechanistically, at 30° the fiber direction was more closely aligned with the primary force-transfer path, and the tensile contribution of the FRP was therefore mobilized more efficiently. As a result, load transfer along the anchorage zone was distributed more uniformly, and severe localization at a single point was less likely. Nevertheless, once a continuous damage path formed at a critical section, rapid unloading was triggered and the response transitioned to global delamination. At an inclination angle of 90° , localized debonding was first initiated at the chamfer and was subsequently followed by fracture of the U-jacket (Figure 8b). The early onset of chamfer debonding suggests that damage evolved progressively prior to final failure, which is consistent with the larger accumulated displacement. In addition, debonding propagation was delayed by higher-strength concrete, allowing the slip stage to develop more fully and thereby enhancing ductility. For the remaining inclination angles, failure was primarily concentrated at the connection between the U-jacket and the FRP plate, and no pronounced sliding was observed prior to rupture (Figure 8c). This indicates that stress concentration at the connection region was the governing weakness, leading to tearing at the joint before sufficient interfacial slip could develop, resulting in premature failure.

3.2. Effect of Chamfer Radius

Figure 9 compares the load–displacement responses and key parameters for C30 and C50 specimens with different chamfer radii. As shown in Figure 9a, changes in chamfer radius produced only minor variations in the overall curve shape. The main difference between the C30 and C50 series was reflected in the peak load level and the displacement corresponding to the peak. For the C30 series, P_{max} increased as the chamfer radius was increased from R25 to R45 (Figure 9b). The P_{max} values for C30R25, C30R35, and C30R45 were 29.97 kN, 30.40 kN, and 34.72 kN, respectively, indicating that a larger chamfer radius enhanced the peak load capacity. In contrast, the corresponding S_{max} values changed only slightly (2.59 mm, 2.49 mm, and 2.69 mm, respectively). The variation in S_{max} was much smaller than that in P_{max} and did not follow a strictly monotonic trend, indicating

a limited influence of chamfer radius on slip/displacement capacity. For the C50 series, C50R45 reached a $P_{\max}$ of 44.63 kN, which was 7.81% higher than that of C50R35 (41.40 kN). Meanwhile, $S_{\max}$ for C50R45 decreased by 9.58% relative to C50R35, indicating that the displacement capacity was not necessarily improved by increasing the chamfer radius at a higher strength level. Further comparisons at identical chamfer radius show that increasing concrete strength significantly increased $P_{\max}$. Specifically, $P_{\max}$ for C50R35 and C50R45 increased by 36.2% and 28.6%, respectively, compared with C30R35 and C30R45. Overall, within the range examined in this study, the primary contribution of chamfer radius to anchorage performance was reflected in the improvement of peak load capacity, with R45 providing the highest load-carrying level for both strength grades. By contrast, the effect of chamfer radius on peak displacement was relatively weak and appeared to depend on the concrete strength.

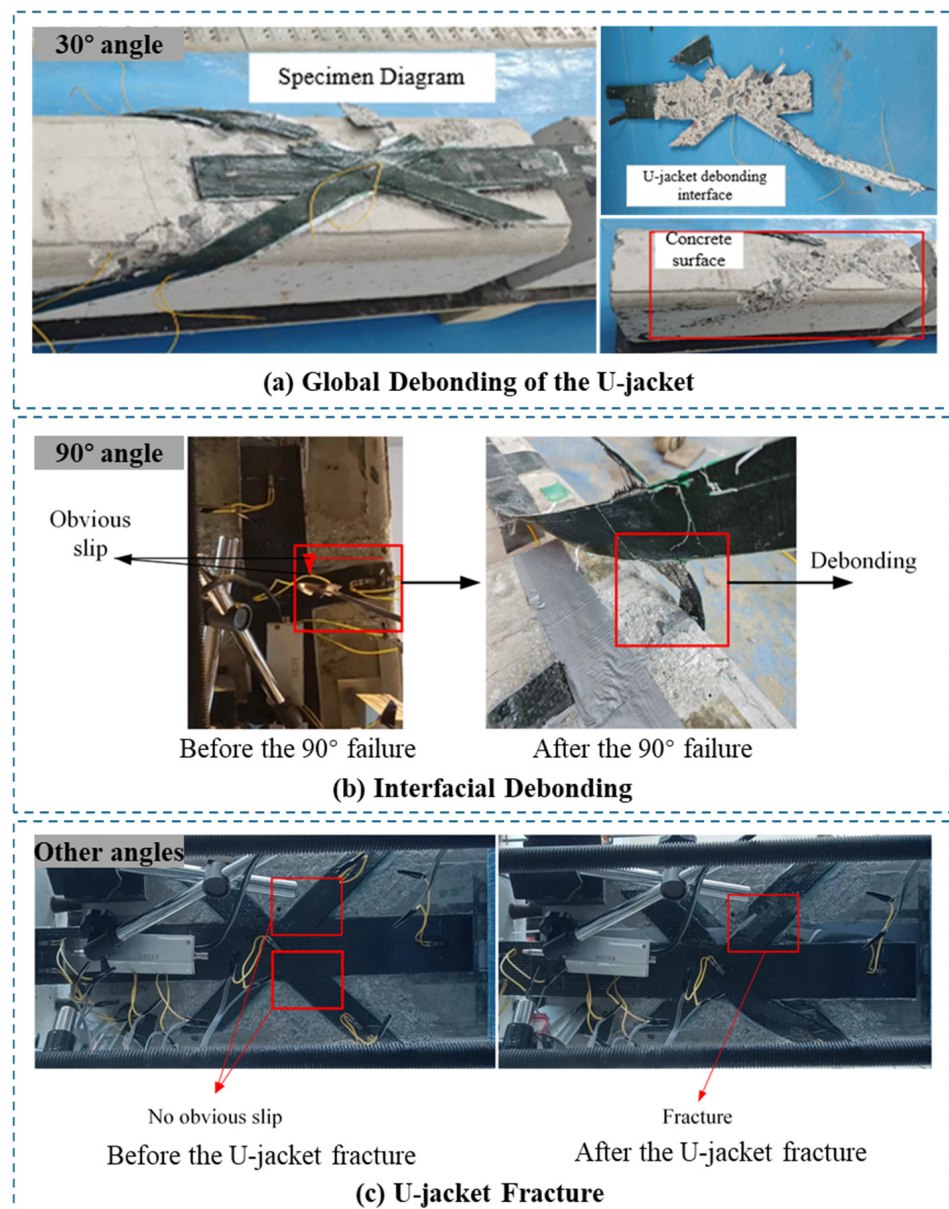


Figure 8. Three representative failure modes observed under different inclination angles.

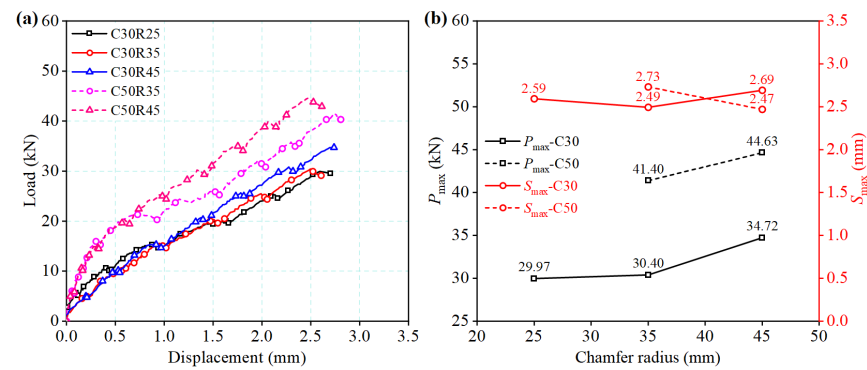


Figure 9. Load–displacement curves and parameter comparisons for the C30 and C50 concrete at different chamfer radii: (a) load–displacement curves and (b) peak load and displacement.

3.3. Effect of FRP Layer Number and Base-Plate Width

Figure 10 illustrates the influence of FRP layer number on the load–displacement responses and key parameters. As shown in Figure 10a, a higher number of FRP layers was associated with a stronger sustained load increase in the pre-peak stage. For C30T1, the load increased rapidly at small displacements and then entered a pronounced plateau-like stage, resulting in a relatively low peak. By contrast, C30T2 and C30T3 maintained a substantial load increase over a wider displacement range, and both the peak load and the peak displacement were markedly enhanced. The P_{max} values for C30T1, C30T2, and C30T3 were 15.53 kN, 29.97 kN, and 48.08 kN, respectively (Figure 10b). Relative to C30T1, P_{max} increased by 93.0% for C30T2 and by 209.6% for C30T3. Meanwhile, S_{max} increased from 2.09 mm (C30T1) to 2.59 mm (C30T2; +23.9%) and 3.19 mm (C30T3; +52.6%). These results indicate that increasing the number of FRP layers not only elevated the peak load capacity but also enhanced the displacement capacity at peak load. Within the tested range, the increase in P_{max} exhibited an approximately linear trend with the number of FRP layers.

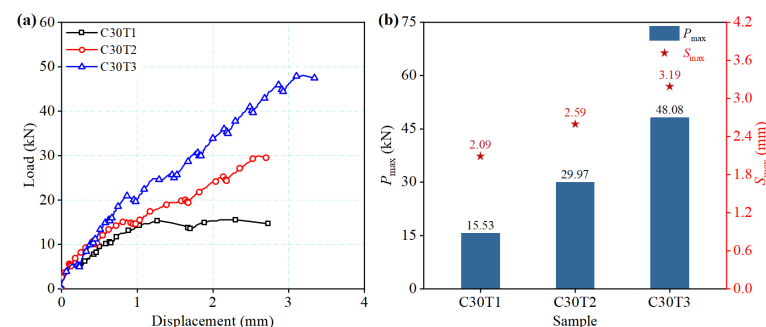


Figure 10. Load–displacement curves and parameter comparisons for different FRP layer numbers: (a) load–displacement curves and (b) peak load and displacement.

Figure 11 shows the influence of FRP base-plate width on the load–displacement responses and key parameters. As shown in Figure 11a, the curves for C30P50 and C30P100 exhibited similar overall shapes, with a continuous load increase up to the peak. However, higher load levels were sustained by C30P100 over most of the displacement range. This indicates that increasing the FRP base-plate width enhanced the load-carrying capacity and slightly increased the displacement attained at peak load. As shown in Figure 11b, P_{max} increased from 29.97 kN (C30P50) to 35.15 kN (C30P100), while S_{max} increased from 2.59 mm to 2.96 mm. Thus, widening the base plate from 50 mm to 100 mm resulted in a 17.3% increase in peak load and a 14.3% increase in peak displacement. Because the U-jacket configuration and the governing failure mode remained unchanged, the improvement was primarily attributed to the base-plate width effect. A wider FRP plate provided a larger effective load-transfer width and higher axial stiffness, thereby reducing the FRP strain

required to transmit a given tensile force at the U-jacket–FRP connection and delaying premature connection fracture [35]. In addition, because the wider plate extended closer to the chamfered corners, force transfer was distributed over a broader region near the chamfer. This interpretation is consistent with the more pronounced strain attenuation in the chamfer-related strain distributions discussed later.

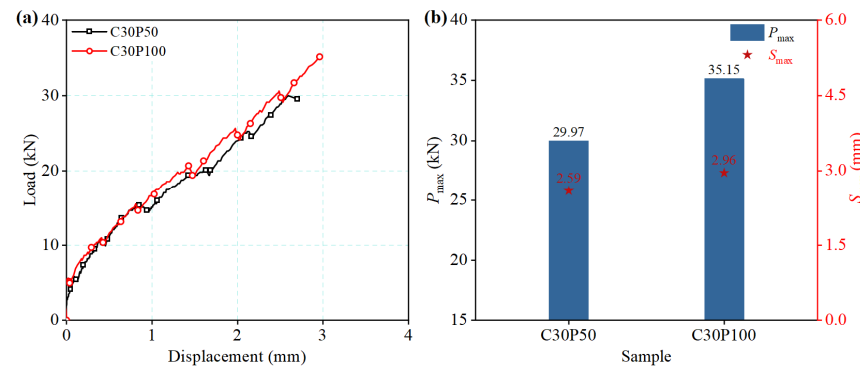


Figure 11. Load–displacement curves and parameter comparisons for different FRP base-plate widths: (a) load–displacement curves and (b) peak load and displacement.

3.4. Effect of U-Jacket Width and Side Height

Figure 12 illustrates the influence of U-jacket width on the load–displacement responses and key parameters. As shown in Figure 12a, all specimens exhibited a generally continuous load increase up to the peak. However, both the peak load and the displacement at peak varied markedly with U-jacket width. For C30U25, P_{max} and S_{max} were 29.63 kN and 2.57 mm, respectively (Figure 12b). When the width was increased to 50 mm, P_{max} and S_{max} for C30U50 increased to 47.65 kN and 3.37 mm, corresponding to increases of 60.8% and 31.1% relative to C30U25. When the width was further increased to 75 mm, P_{max} and S_{max} for C30U75 decreased to 45.50 kN and 2.23 mm, corresponding to reductions of 4.5% and 33.8% relative to C30U50.

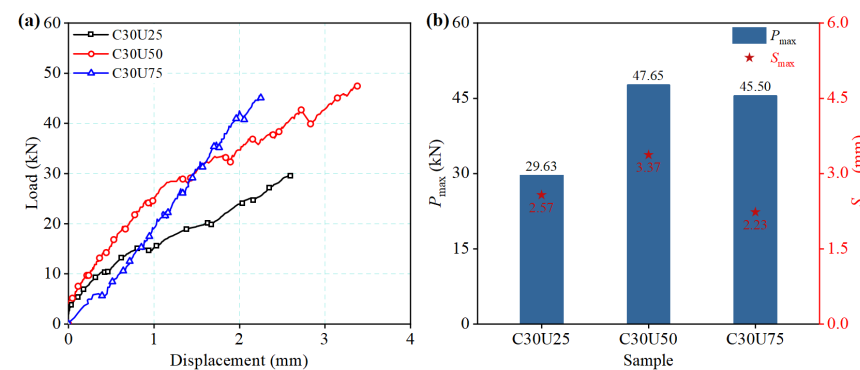


Figure 12. Load–displacement curves and parameter comparisons for different U-jacket widths: (a) load–displacement curves and (b) peak load and displacement.

Failure observations showed that for U-jacket widths of 50 mm and 75 mm, the FRP substrate failed by tensile rupture (Figure 13). This indicates that increasing U-jacket width substantially mobilized the load-carrying capacity of the strengthened system and shifted the governing failure mode from anchorage-controlled debonding to FRP rupture. Consequently, a pronounced increase in both P_{max} and S_{max} was achieved for C30U50 compared with C30U25, whereas C30U75, under FRP-rupture-dominated conditions, did not further increase P_{max} and instead exhibited a reduced S_{max} .

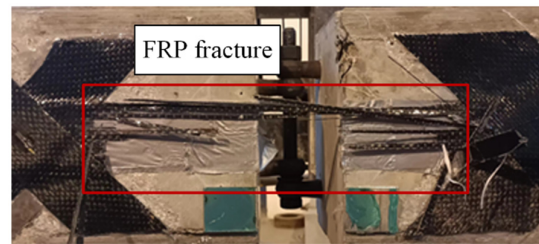


Figure 13. FRP tensile fracture failure diagram.

Figure 14 illustrates the effects of U-jacket side height on the load–displacement responses and key parameters. As shown in Figure 14a, the overall curve shape remained largely unchanged with increasing side height. In addition, the peak load was only marginally affected. The $P_{\max}$ values for CG75 and CG120 were 30.62 kN and 29.97 kN, respectively, corresponding to a difference of only 2.17% (Figure 14b). In contrast, $S_{\max}$ decreased from 3.69 mm (CG75) to 2.59 mm (CG120), i.e., a 29.8% reduction. This result indicates that increasing the side height from 75 mm to 120 mm did not improve the peak load capacity but was associated with a smaller peak displacement. These observations suggest the presence of an effective bonded length of the U-jacket along the fiber direction, beyond which additional side height provides limited contribution to load transfer [34,36]. This inference was further supported by the strain measurements near the free end of the U-jacket, where only minor deformation was recorded (Figure 15).

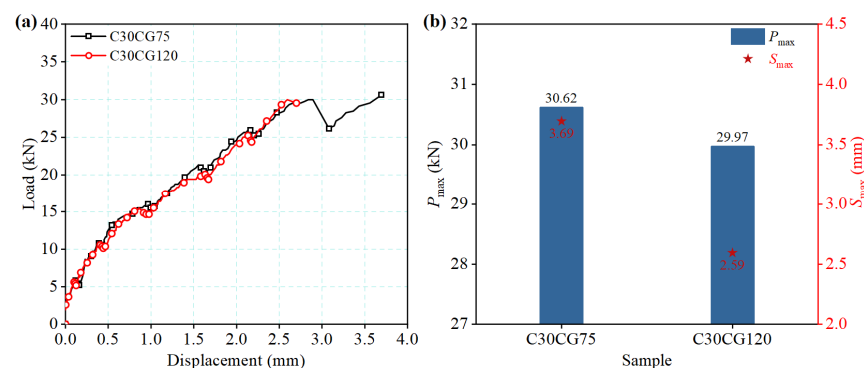


Figure 14. Load–displacement curves and parameter comparisons for different side heights of U-jacket: (a) load–displacement curves and (b) peak load and displacement.

3.5. Strain Characteristics of FRP Plate and U-Jacket

To more accurately represent the stress state in the FRP, the midspan strain profile was referenced to -35 mm from the slip-measurement point. The strain distributions were examined at load levels spaced at 10% intervals of the ultimate load (P_u) to track the evolution of composite strain with increasing load. As shown in Figure 15a, because the FRP base plate was intentionally not bonded to concrete, the strains at the first three measurement points (i.e., prior to the U-jacket region) remained nearly constant. By contrast, the FRP strain near the loaded end increased approximately proportionally with load, then decayed rapidly toward the free end and approached zero at the far end. This indicates that force transfer and strain concentration were mainly confined to a limited region close to the loaded end. At higher load ratios (approximately 0.8–1.0 P_u), the strain profile near the U-jacket connection became increasingly non-monotonic and more scattered. This behavior was attributed to increased deflection at high load levels, which intensified local misalignment between the FRP and the concrete around midspan. The resulting eccentric contact and local stress concentration led to fluctuations in the measured strain response.

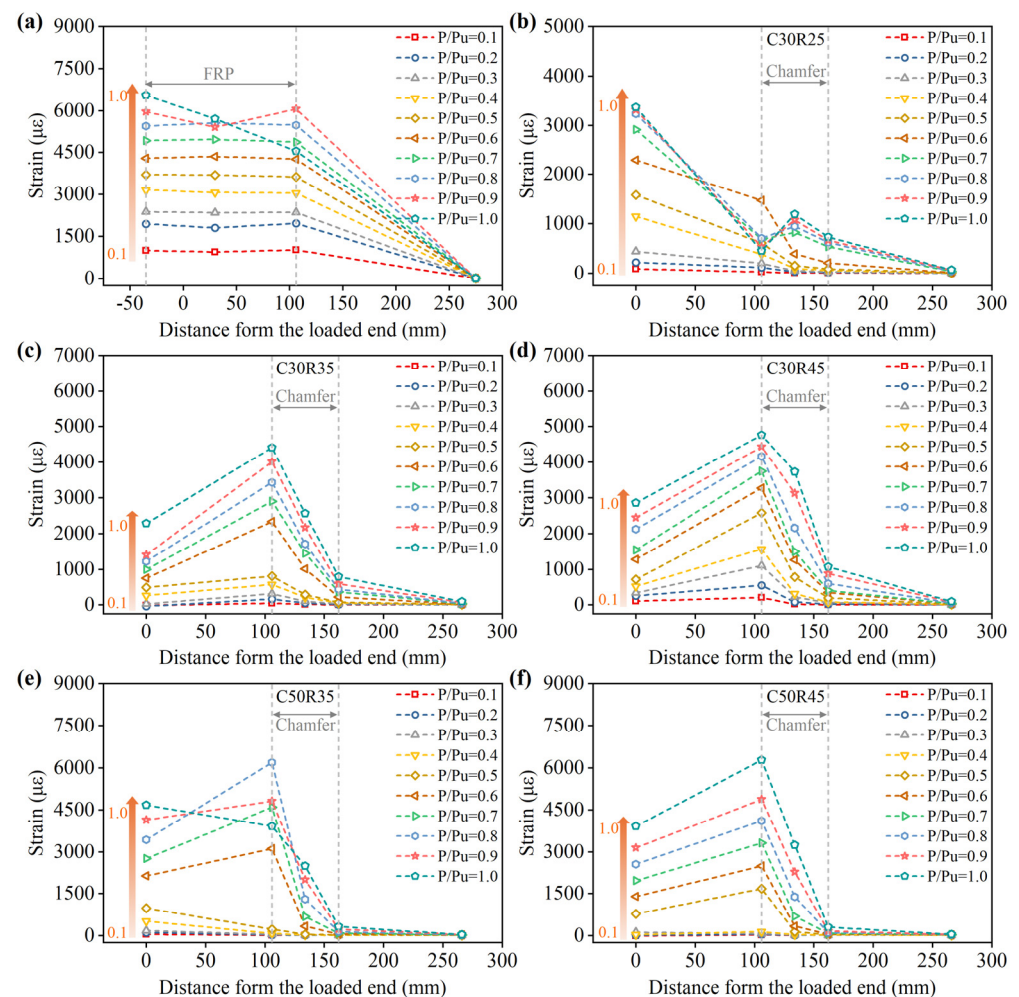


Figure 15. Strain characteristics of FRP plates and U-jackets for different specimens: (a) FRP, (b) C30R25, (c) C30R35, (d) C30R45, (e) C50R35, and (f) C50R45.

The U-jacket strain distributions are shown in Figure 15b–f, where the chamfer region is highlighted to clarify strain evolution in this critical zone. The origin for U-jacket strain was set at the midpoint of the connection between the U-jacket and the FRP plate, so that the relative deformation along the inclined load-transfer path could be captured. Three features were consistently observed in the chamfered region. (1) Strain values near the chamfer-side origin remained relatively low, indicating that compressive confinement provided by the concrete reduced the tensile demand at the loading end. (2) Strain decayed nonlinearly along the chamfered path, with a steeper decay at higher load levels, indicating stronger stress redistribution in the chamfer zone during the high-load stage. (3) With increasing load, the strain decay in the latter half of the chamfered segment became faster than that in the initial half, implying a more fully activated load-transfer mechanism and a stronger contribution of concrete confinement in the latter segment at high load levels. Notably, for specimen C30R25, a distinct change in the local strain trend occurred as the applied load increased from 0.6 P_u to 0.7 P_u . This transition was attributed to localized debonding initiated by stress concentration near the start of the chamfer. Once debonding occurred, the effective stress-transfer location at the U-jacket–concrete interface was shifted slightly toward the free end [27]. Consequently, the strain measured at the chamfer center exceeded that at the origin. A similar response was reported by Yuan et al. [36], who attributed it to stress concentration near the loaded-end interface.

Different chamfer radii also produced discernible changes in the strain-redistribution morphology. For the C30 series, R25 (Figure 15b) more readily developed a secondary

peak or inflection within the chamfered zone at medium-to-high load ratios. By contrast, R35 and R45 (Figure 15c,d) tended to exhibit a single-peak profile concentrated near the chamfer start point, followed by rapid attenuation within the chamfered segment. This indicates that increasing chamfer radius shifted the peak-strain location and altered the strain-gradient distribution within the chamfer zone. A similar pattern was observed for the C50 series (Figure 15e,f), where the strain response near the chamfer became more concentrated and was more strongly amplified as the load ratio increased. This suggests that, under higher stress levels, strain transfer and redistribution within the chamfered region became more pronounced. Overall, the chamfered zone acted as the primary region of strain attenuation and exhibited the highest sensitivity to load level, while the chamfer radius systematically governed both the peak-strain location and the attenuation pattern within this region.

4. Finite-Element Analysis

4.1. Model Design

Based on the experimental load–displacement curves, the power-function cohesive constitutive model given in Equation (6) [37] was adopted to simulate the load–displacement response of the U-jacket–anchored system. This interface-focused modeling strategy follows the established bond–slip and cohesive-zone approaches for FRP-to-concrete bonded joints, in which the interfacial response is represented by local traction–separation or bond–slip laws while the concrete substrate provides the supporting stiffness [34,36,38]. The finite-element (FE) model was developed following established modeling approaches [8,39], as shown in Figure 16.

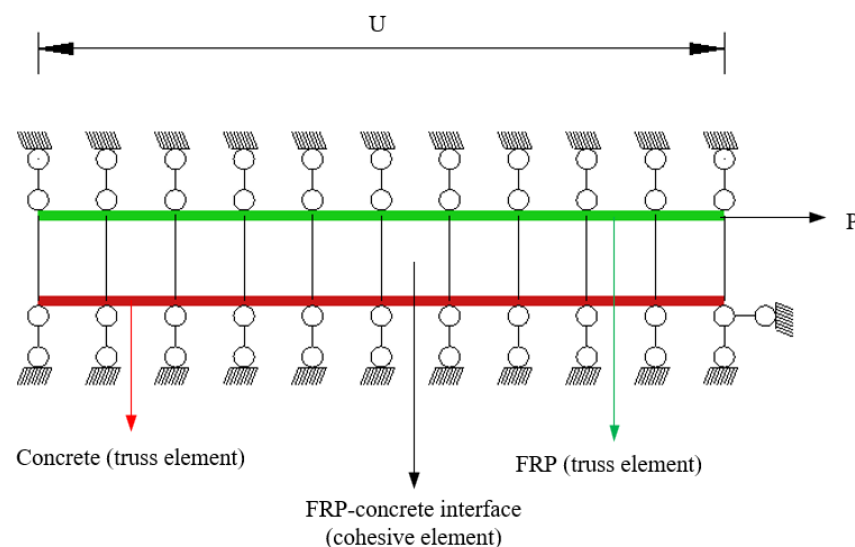


Figure 16. Finite-element modeling.

A mesh sensitivity analysis was conducted using mesh sizes of 20 mm, 15 mm, 10 mm, 5 mm, and 1 mm for the representative specimen [40]. As shown in Figure 17, the predicted peak load approached the experimental value as the mesh size decreased, and little difference was observed between the 5 mm and 1 mm meshes. Therefore, a uniform mesh size of 1 mm was adopted in the interfacial region for the subsequent analyses. Because the dominant interfacial damage was reported to localize within approximately 2–5 mm from the concrete interface [41], cohesive elements were employed to capture interface degradation, whereas the concrete, steel reinforcement, and FRP were idealized as linear-elastic materials. Based on theoretical derivations [36] and FE verification, the model parameters used in this study are summarized in Table 3. Here, $S_{\max}$ denotes the displacement corresponding

to the maximum load $P_{\max}$; $\tau_{\max}$ denotes the peak shear stress of the cohesive elements; and a and b are the coefficients of the adopted power-function cohesive model. The model length and the out-of-plane thickness of the cohesive layer were both set equal to the actual U-jacket width.

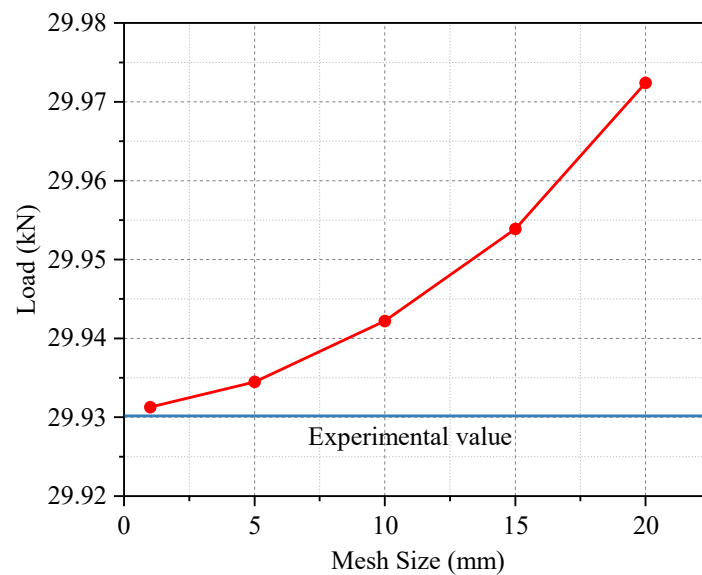


Figure 17. Mesh sensitivity analysis of the predicted peak load with different mesh sizes.

Table 3. Peak load, peak displacement, and constitutive model parameters.

Name		$S_{\max}$ (mm)	$P_{\max}$ (kN)	a	b
Chamfer	C30R25	2.59	29.97	26.947	0.539
	C30R35	2.49	30.4	18.213	0.785
	C30R45	2.74	34.72	26.720	0.755
	C50R35	2.75	41.4	38.950	0.470
	C50R45	2.47	44.63	43.747	0.535
Inclination Angle	C30I30	4.66	55.58	31.813	0.659
	C30I45	2.59	29.97	26.947	0.539
	C30I60	3.26	25.23	17.305	0.675
	C30I90	9.57	9.7	1.631	1.000
	C30I135	0.69	7.76	18.683	0.894
	C50I60	2.71	33.85	28.960	0.617
	C50I90	9.7	9.92	2.418	0.746
	C50I135	0.86	9.27	18.279	0.904
Thickness	C30T1	2.3	15.53	19.469	0.396
	C30T2	2.59	29.97	26.947	0.539
	C30T3	3.19	48.08	33.914	0.738
Side Height	CG75	2.59	29.97	26.958	0.519
	CG120	2.59	29.97	26.947	0.539
Bottom Plate Width	C30P50	2.59	29.97	26.947	0.539
	C30P100	2.59	29.97	28.826	0.566
U-Jacket Thickness	C30U25	2.59	29.97	26.947	0.539
	C30U50	3.37	47.65	35.775	0.669
	C30U75	2.23	45.5	7.773	1.000

4.2. Cohesive Element Constitutive Model

The cohesive constitutive model was calibrated using the following procedure. First, the area under the experimental load–displacement curve (Figure 18) was calculated and

then converted to an equivalent right triangle defined by S_0 and P . Next, S_0 and the peak shear stress $\tau_{\max}$, together with the model energy G , were determined using the elasticity theory expressions proposed by Yuan et al. [36] (Equations (3) and (4)). A power-function cohesive traction–separation relationship was then adopted for numerical simulation. The relationship between the cohesive parameters a and b was obtained by integrating the adopted power function. The exponent b was identified by fitting the experimental response with the power-function form (Figure 19). The coefficient a was subsequently calculated using Equation (9).

$$r_1^2 = \frac{\tau_{\max}}{S_0} \left(\frac{1}{E_{\text{FRP}} t_p} + \frac{b_p}{b_c E_c t_c} \right) \quad (3)$$

$$P = \frac{\tau_{\max} b_p}{r_1} \tanh(r_1 U) \quad (4)$$

$$G = 0.5 S_0 \tau_{\max} \quad (5)$$

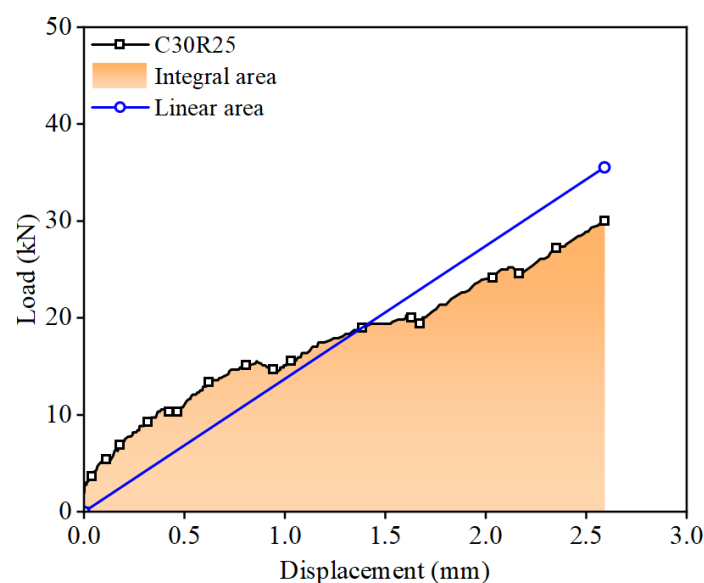


Figure 18. Conversion of test curve area.

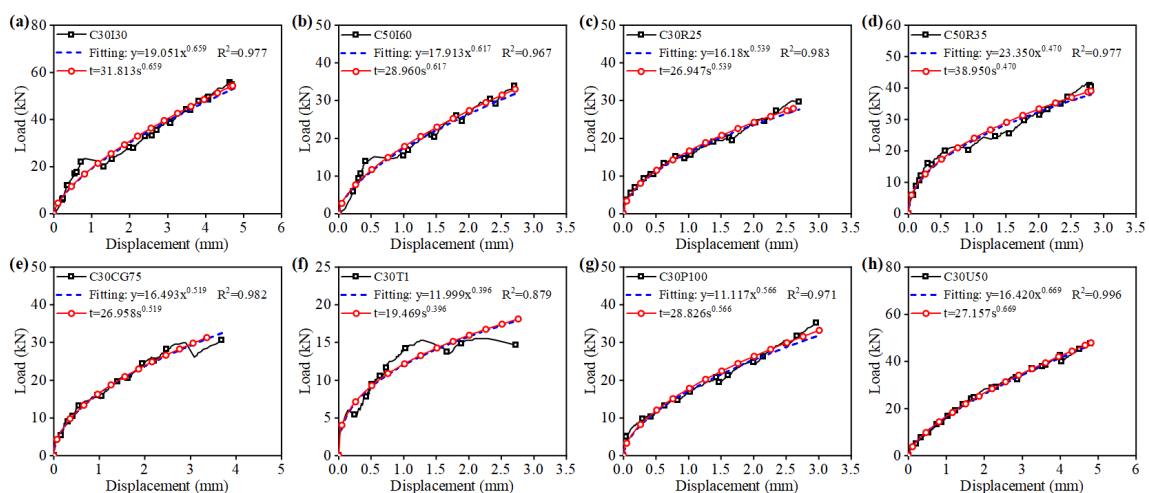


Figure 19. Experimental results and finite-element calculations: (a) C30I30, (b) C50I60, (c) C30R25, (d) C50R35, (e) C30CG75, (f) C30T1, (g) C30P100, and (h) C30U50.

Among them, E_{FRP} and t_p are the elastic modulus and thickness of the FRP, respectively; E_c , t_c , and b_c are the elastic modulus, thickness, and width of the concrete, respectively; b_p

and U both refer to the width of the U-jackets. Here, $\tau_{\max}$ is an unknown quantity, which can be adjusted using MATLAB 2022 to satisfy the aforementioned equations. Then, the energy constitutive G of the element is calculated.

Considering the shape of the experimental load–displacement response and the selected cohesive-function formulation, the bilinear cohesive model was taken as the reference and is expressed in Equation (6) [42]. Because the post-peak (descending) branch of the load–displacement curve could not be reliably captured in the present tests, only the ascending branch was considered in the constitutive description to better represent the experimental observations. The ascending branch was subsequently reformulated using a power-function expression (Equation (7)), and the empirical parameters a and b were re-identified through curve fitting.

$$\begin{cases} \tau = \tau_{\max} \left(\frac{S}{S_0} \right)^a & S \leq S_0 \\ \tau = \tau_{\max} e^{-b \left(\frac{S}{S_0} - 1 \right)} & S > S_0 \end{cases} \quad (6)$$

$$y = ax^b, 0 < b \leq 1 \quad (7)$$

$$\int_0^{S_0} ax^b dx = S \quad (8)$$

$$a = \frac{S(b+1)}{S_0^{b+1}} \quad (9)$$

4.3. Parameter Analysis

The exponent b was obtained by fitting the experimental load–displacement curve (Figure 19a), yielding $b = 0.659$. The coefficient a was then calculated from Equation (9) using the identified b . The resulting constitutive relationship was subsequently implemented in the FE model shown in Figure 16, which was developed and solved using Abaqus. As evidenced by the comparison between the simulated and experimental load–displacement curves in Figure 19, the adopted power-function cohesive model reproduced the experimental response with good accuracy, indicating that the proposed simplified scheme provided a satisfactory fit.

Accordingly, based on the experimental results, theoretical derivations, and FE simulations, the fitted values of the power-function coefficient a and exponent b were compiled and processed, as reported in Tables 3 and 4. Because the cohesive function followed a power function, a controlled the overall scale of the traction–separation response, whereas b governed the rate of nonlinear growth/decay [43]. This constitutive form enabled a quantitative interpretation of how U-jacket parameters influenced anchorage performance. In this context, a primarily reflected the stiffness-related characteristics of the cohesive response, while b characterized the nonlinear evolution of energy dissipation.

When b was held constant, a effectively controlled the initial stiffness (i.e., the effective modulus) of the power-function relationship. Increasing a increased the predicted load P ; however, the marginal increase became less pronounced once a exceeded a threshold level. In addition, the combined trends of a and b were used to infer the variation in P in the load–displacement response. When the trends of a and b were consistent, the trend of P could be assessed directly: (1) if both a and b increased, the corresponding variable was associated with an increase in load; and (2) if both decreased, the load tended to decrease. When a and b varied in opposite directions, the net effect was determined from the P –variable relationship: (a) a positive slope indicated a favorable effect; (b) a negative slope indicated an unfavorable effect; and (c) a near-zero slope indicated a negligible effect, in which case a moderate parameter level could be adopted. Therefore, for the parametric analysis of inclined U-jacket anchorage, the power-function-type cohesive zone model is

suggested as an appropriate interface-level model. It can describe the nonlinear pre-peak load–slip response through two physically interpretable parameters, namely the coefficient (a) and exponent (b). The model is suitable for comparing the effects of U-jacket geometric variables on interfacial load transfer, whereas a more detailed concrete damage model would be required for predicting full post-peak fracture or concrete crushing.

Table 4. Coefficient a , exponent b , and load P .

Name	Strength	Exponent b		Coefficient a		Load P	
		Slope	Intercept	Slope	Intercept	Slope	Intercept
Chamfer	C30	0.0108	0.269	−0.0114	27.232	0.2375	24.033
	C50	0.0065	0.243	0.48	22.163	0.323	30.095
Inclination Angle	C30	0.00283	0.501	−0.484	47.117	−1.012	82.452
Thickness	C30	0.171	0.21567	7.222	12.332	16.275	−1.357
Side Height	C30	0.000444	0.486	−0.00024	26.976	0	29.97
Bottom Plate Width	C30	0.00054	0.512	0.0376	25.069	0.104	24.79
U-Jacket Thickness	C30	0.00922	0.3085	−0.383	36.534	0.311	22.205

5. Conclusions

The present study systematically investigated the effects of key parameters on the bond performance of the U-jacket–concrete interface. A finite-element (FE) model was adopted to develop the constitutive model of the cohesive elements for the U-jacket–concrete interfaces, and its validity was verified by comparison with experimental load–slip responses. A sensitivity analysis of the power-function-type parameters a and b was obtained to clarify how U-jacket geometry governed the load–displacement behavior and interfacial mechanical response. The main conclusions are summarized as follows:

- (1) Among all the parameters examined, the inclination angle had the most significant effect on the bond behavior of inclined U-jacket-to-concrete joints. A decrease in the inclination angle facilitated force transfer from the tension plate to the U-jackets, leading to a change in failure mode and an increase in joint strength. Specifically, the joint strength increased from 7.76 kN at an inclination angle of 135° to 55.58 kN at 30°.
- (2) At the chamfer, the FRP–concrete interface was subjected to a complex stress state involving both shear stress and normal compressive stress. The presence of normal compressive stress enhanced the bond strength of the interface. A larger chamfer radius provided a longer FRP–concrete contact region, thereby increasing the joint strength. Specifically, specimen C30R25 exhibited lower strength than its counterpart with a chamfer radius of 45 mm.
- (3) Based on the numerical results and corresponding parametric analysis, the effects of the power-function coefficient (a), exponent (b), and load response (P) were systematically evaluated for U-jacket anchorage design. The proposed power-function cohesive law enabled a quantitative interpretation of the intrinsic relationship between U-jacket parameters and anchorage behavior: (a) characterized the stiffness-related scaling of the interfacial cohesive response, whereas (b) governed the nonlinear evolution associated with energy dissipation.

Overall, the load-carrying mechanisms and failure patterns of U-jacket–concrete anchorage were clarified through systematic four-point bending tests, with particular emphasis on inclination angle, chamfer geometry, side height, and FRP/U-jacket configurations. A pre-peak parameter identification framework based on power-function-type cohesive mechanics was proposed and validated. This framework translated macroscopic load–slip

responses into transferable interfacial strength and evolution metrics, thereby providing an experimentally verifiable basis for quantitative selection and optimized design of U-jacket anchorage configurations.

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