

Article

Bidirectional Shear Performance of Corroded Stud Connectors in Steel–Concrete Composite Monorail Track Beams

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Abstract

Under the combined action of bidirectional (longitudinal and transverse) shear loads and corrosive environments, the shear performance of stud connectors in steel–concrete composite track beams of straddle-type monorail transit systems is susceptible to degradation, thereby posing a potential risk to the structural safety of the track girders. This study employs push-out tests and numerical simulations to investigate the influence of bidirectional shear loads and stud corrosion on the shear performance of stud connectors. The results showed that both transverse shear loads and stud corrosion lead to a reduction in the shear capacity of stud connectors, with their coupling effect amplifying the degradation. Transverse shear loads induce an accelerated decay trend in the load-bearing capacity of stud connectors, while an increase in corrosion depth results in a linear degradation of the load-bearing capacity. The corrosion depth at the stud root exerts a more pronounced influence on shear performance compared to the corrosion height. Furthermore, the dominant failure mode of stud connectors manifests as root fracture, while transverse shear loads induce alterations in the concrete damage zone. Based on the verified FE model, a shear capacity reduction factor accounting for the coupling effects of bidirectional shear and stud corrosion was established to improve the Oehlers model. This research provides critical theoretical support for the safe design and durability assessment of monorail track girders.

Keywords: stud connector; steel–concrete composite track girder; bidirectional shear force; corrosion effect; shear strength



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1. Introduction

The straddle-type monorail transit system offers numerous advantages, including minimal land occupation, negligible disruption to existing ground transportation networks, exceptional terrain adaptability, rapid construction timelines, and reduced noise pollution [1,2]. This system has been constructed in cities such as Tokyo, Chongqing, and Wuhu [3,4]. As a critical structural component, the track girder not only bears vehicle loads but also serves as a guidance track to ensure operational stability. To enhance the spanning capacity of track girders, a novel steel–concrete composite track girder has been developed. Compared to traditional prestressed concrete track girders, this novel design leverages the complementary mechanical properties of steel and concrete (Figure 1a). The composite system achieves significant weight reduction while enabling longer spans, addressing the limitations of conventional concrete girders. Furthermore, the concrete panel effectively

mitigates vibration and operational noise, establishing steel–concrete composite track girders as a promising advancement in monorail transit systems. Previous study [5] indicates that steel–concrete composite track girders possess high overall stiffness: their vertical displacement under static live loads meets the requirements of relevant codes, and their stress levels under various combined load conditions are far lower than the allowable stress, resulting in high safety performance. In terms of dynamic characteristics [2], although the longitudinal slip at the interface can also change the dynamic performance of the girders, the dynamic response of the track girders tends to stabilize when the interface connection stiffness exceeds a certain critical value. These studies confirm that this type of track girder has broad application prospects.

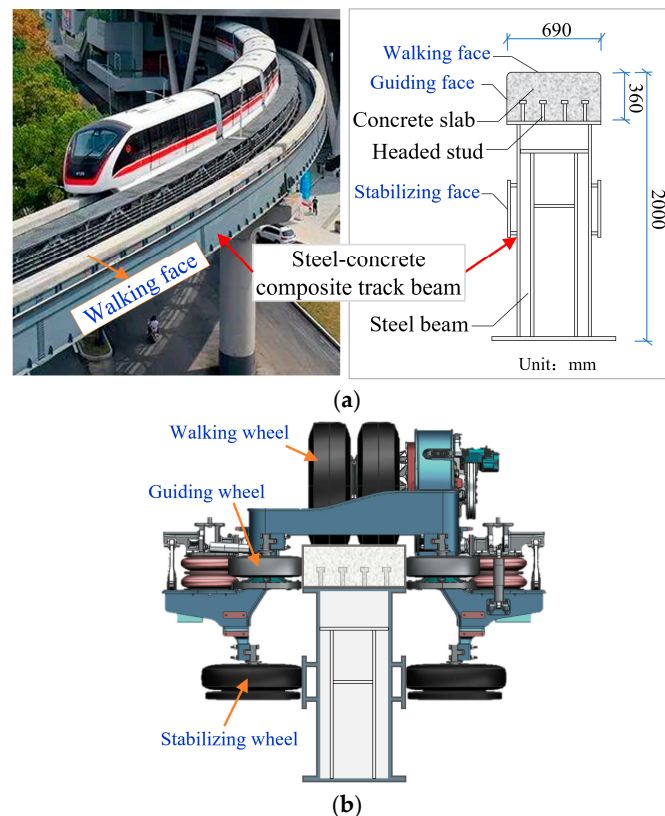


Figure 1. Straddle-type monorail transit in Wuhu [6]: (a) steel–concrete composite track beam and (b) running mechanism.

In steel–concrete composite track girders, stud connectors serve as the critical interface element enabling composite action between steel girders and concrete slabs [7]. These connectors play a pivotal role in transferring longitudinal shear forces and uplift forces, with their mechanical performance directly governing the overall structural safety of the system. Straddle-type monorail systems, however, operate under a distinct mechanical mechanism [8]: trains securely grip the track girder surface using three types of rubber-tired wheels (walking wheels, guiding wheels, and stabilizing wheels), as illustrated in Figure 1b. The unique operational principle subjects the concrete slab to lateral loads, including vehicle sway forces and centrifugal forces. When these lateral loads interact with the longitudinal shear forces induced by the vertical flexural deformation of the slab, bidirectional shear stresses (longitudinal and transverse to the track axis) are generated at the steel–concrete interface, whose mechanical characteristics differ significantly from the shear conditions of conventional steel–concrete bridges. Notably, train–track coupling vibrations can further amplify the longitudinal and transverse dynamic loads borne by stud connectors [9,10].

Extensive research has focused on the static performance [11,12] and fatigue behavior [13,14] of stud connectors in steel–concrete composite systems. However, during long-term service, moisture infiltration through interface gaps usually induces corrosion in studs, which not only degrades their shear capacity but also triggers cracking and spalling of concrete cover, thereby compromising the composite action between steel girders and concrete slabs [15,16]. The corrosion-induced shear performance degradation exhibits significant nonlinear characteristics; as corrosion progresses, shear strength and stiffness initially increase (due to localized compression from rust expansion) but eventually decline sharply, with root corrosion (near the steel–concrete interface) exhibiting far greater detrimental effects than head corrosion (at the stud extremity) [16,17]. But, corrosion impacts on shear capacity exhibit no correlation with stud diameter [18].

In practical engineering, stud connectors in steel–concrete composite girders experience highly complex loading scenarios [19]. Near web openings, stud rods are subjected to additional tensile forces equivalent to approximately 30% of their basic shear capacity due to stress concentration effects [20,21]. In negative bending regions of steel–concrete composite continuous beams, studs further endure combined axial tension and shear stresses, leading to a significant reduction in shear capacity [22,23]. Experimental studies by Shen and Chung [24] revealed that when the tension–shear force ratio reaches 0.267, stud shear capacity decreases by about 10% through push-out tests. The authors' prior study [6] also demonstrated that bidirectional shear loading alters stud failure modes and mechanical responses. However, current research on stud connectors for steel–concrete composite track girders, which considers the coupled effects of environmental influences and bidirectional shear loading, remains relatively scarce, and their shear resistance mechanism has not yet been clarified.

Although design codes such as Eurocode 4 [25] and GB 50917 [26], along with prior studies [27–29], have established shear strength formulas for stud connectors in conventional composite structures, existing studies have not fully considered the coupling effects of bidirectional shear loading and stud corrosion encountered by stud connectors in straddle-type monorail track girders. The shear strength models developed based on unidirectional shear conditions or simplified tension–shear combined scenarios in traditional codes fail to accurately capture the degradation mechanisms induced by the combined action of bidirectional shear and stud corrosion, leading to significant discrepancies between engineering calculations and actual service conditions. Therefore, there is a critical need to further elucidate the degradation mechanisms of stud connectors under the coupling effect of bidirectional shear and corrosion.

Against this background, this paper investigates the degradation laws of stud connectors under the coupled action of bidirectional shear loading and stud corrosion via the design of improved push-out specimens. A finite element model incorporating corrosion height and depth is developed for parametric analysis. Additionally, a shear capacity prediction model accounting for the coupling effect of bidirectional shear and corrosion is proposed, providing theoretical support for the safe design and durability assessment of monorail track girders.

2. Experimental Program

2.1. Specimen Details and Material Property

A total of four specimens were designed, as detailed in Table 1, aiming to investigate the influences of lateral shear loading and stud corrosion on the performance of stud connectors. Among these, Specimen S was designated as the reference specimen. To facilitate the application of bidirectional shear loading, the conventional stud connector configuration was modified, and the push-out specimens were manufactured as illustrated

in Figure 2. Each specimen comprised four key components: steel plates, concrete slabs, stud connectors, and a steel rebar skeleton. Steel plates were fabricated from Q345 steel plate with a thickness of 12 mm. Concrete slabs, with a compressive strength grade of C60, measured 270 mm × 270 mm × 100 mm in dimensions and were internally reinforced with an HRB400-grade steel rebar skeleton (8 mm diameter) to prevent global failure of the concrete during loading. Stud connectors, made of ML15 steel with a diameter of 10 mm and a height of 70 mm, were symmetrically welded to both sides of the steel plates and horizontally spaced at 80 mm intervals.

Table 1. Specimen parameters and test results.

Specimen	RoHV	CD	Peak Shear Capacity (kN)		Shear Stiffness (kN/mm)		Peak Slip (mm)		Ultimate Slip (mm)	
			F_{vp}	Ratio	k_{vs}	Ratio	S_{vp}	Ratio	S_{vu}	Ratio
S	0	0	220.89	-	471.5	-	5.76	-	6.20	-
SH	$0.3P_u$	0	200.53	0.908	473.0	1.003	4.93	0.856	5.12	0.826
SC	0	5%	186.28	0.843	452.5	0.960	4.52	0.785	5.02	0.810
SCH	$0.3P_u$	5%	180.81	0.819	536.0	1.137	3.71	0.644	3.89	0.627

Note: RoHV is the ratio of lateral shear force to the ultimate load of specimen S, P_u is the ultimate load of specimen S, CD is the designed corrosion degree, F_{vp} is the peak shear capacity in vertical direction, k_{vs} is the shear stiffness calculated by vertical shear force–slip curve, S_{vp} is the peak slip corresponding to peak shear load, and S_{vu} and is the ultimate slip corresponding to the ultimate load.

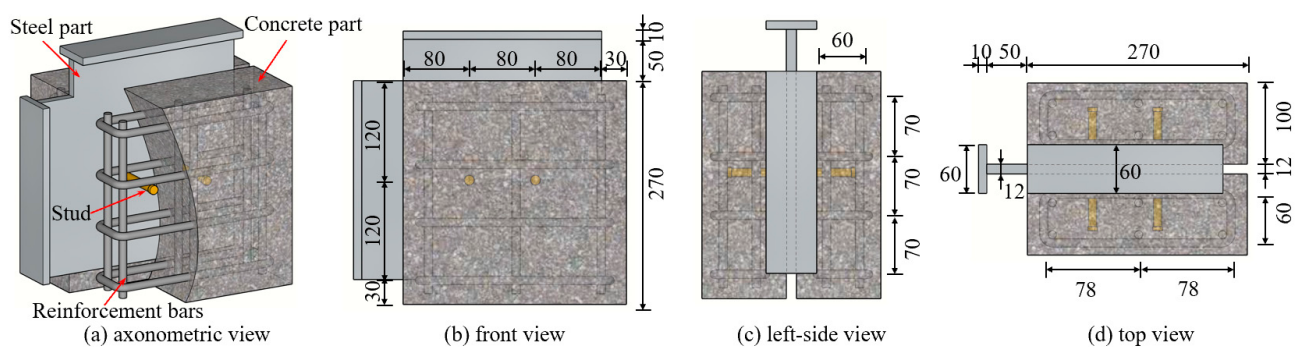


Figure 2. Specimen dimensional diagram.

The mechanical properties of materials were determined through standardized tests. The cube compressive strength of concrete, determined following GB/T 50081-2019 [30], was 80.0 MPa. For steel materials, the yield strength, ultimate strength, and elastic modulus were determined following GB/T 228.1-2021 [31]. Specifically, the yield strengths of the steel plates, studs, and reinforcing bars were 352.56 MPa, 500.11 MPa, and 436.2 MPa, respectively, the ultimate strengths were 521.07 MPa, 520.47 MPa, and 627.07 MPa, respectively, and the elastic moduli were 2.07×10^5 MPa, 2.02×10^5 MPa, and 2.04×10^5 MPa, respectively.

2.2. Specimen Corrosion Treatment

Stud corrosion was induced using a constant-current accelerated corrosion method, as illustrated in Figure 3. First, a 10 mm diameter hole (drilled to a depth reaching the stud head) was created in the concrete slab at the position corresponding to the stud head. Debris within the hole was removed to ensure direct contact between the stud and the corrosion medium. Subsequently, a plastic container was attached to the outer side of the concrete slab, which was then filled with a 5% NaCl solution (with the solution level covering the stud head). A stainless-steel plate was positioned in the solution as the cathode, while the steel plate was connected to the positive terminal of a DC power supply

via a wire, and the stainless steel sheet was linked to the negative terminal. According to Faraday's law [16], electrode dissolution during constant-current corrosion is proportional to current and duration, enabling control of stud corrosion by regulating power supply current and time.

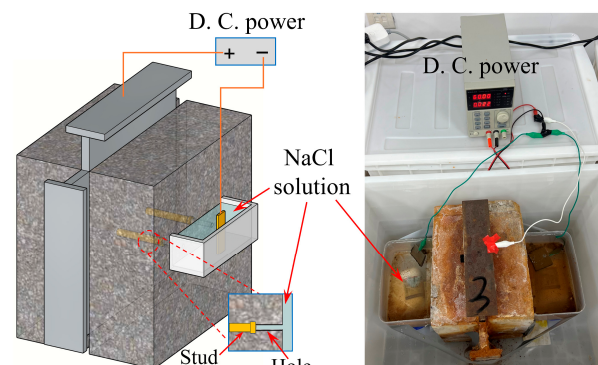


Figure 3. Specimen corrosion treatment.

2.3. Testing Apparatus

The testing setup and loading protocol are presented in Figure 4. An electro-hydraulic servo-controlled universal testing machine was used to conduct push-out tests. Prior to formal loading, a cyclic preloading procedure with 25 cycles within 5–40% of the theoretical failure load was applied at a rate of 1.5 kN/s to eliminate the interfacial bond between the steel plate and concrete slab. For vertically loaded specimens, displacement-controlled loading was performed at a rate of 0.1 mm/min using the configuration shown in Figure 4b, continuing until the peak load decreased by 20% or the specimen failed. For bidirectionally sheared specimens, a lateral loading frame was first employed to fix the jack and force sensor at stud height on the side of the push-out specimen. The lateral shear force was then applied to the design value via the jack, followed by vertical loading (Figure 4c). The loading frame operates on a load-self-balancing principle: one end connects to the jack and force sensor, with the jack linked to the concrete slab, while the other end connects to the steel plate of the specimen. During loading, the lateral force variation was constrained within 5% of the design value.

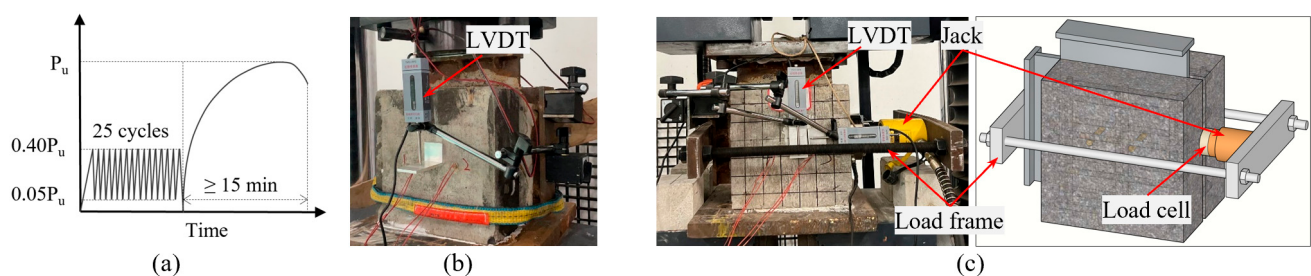


Figure 4. Loading protocol, test setup for specimens. (a) Loading protocol, (b) Test setup for vertical sheared specimen, (c) Test setup for bidirectionally sheared specimen.

To capture the relative slip between the concrete slab and steel plate at the stud locations, an L-shaped aluminum plate was bonded at the midpoint of the line connecting two studs. The LVDT (Linear Variable Differential Transformer, which produced by Liyang Instrument Factory Co., Ltd. in Liyang, China) was then fixed to the steel plate, with its measuring rod in contact with the L-shaped aluminum plate (Figure 4). During loading, the data acquisition system (TDS-540) was employed to synchronously record the load and displacement data.

3. Experimental Results and Discussion

3.1. Vertical Load–Slip Curves

The load–slip relationships are presented in Figure 5, where the vertical axis represents the specimen load and the horizontal axis denotes the vertical interfacial slip between the steel plate and concrete slab. Slip was defined as the average of the displacements measured by the vertical LVDTs. The load–slip curves of all specimens can be categorized into four stages: elastic, yield, stable development, and failure stage. Taking specimen S as an example, before the load reached 60 kN, the specimen remained in the elastic stage, characterized by negligible relative slip between the steel plate and concrete and an approximately linear load–slip relationship. As the load increased beyond 60 kN, the specimen entered the yield stage, where slip accelerated with further load increments. After the load exceeded 176 kN, vertical slip continued to vary linearly with load, marking the onset of the stable development stage. Upon surpassing the peak load of 220 kN, the load rapidly decreased while slip slightly increased, indicating the specimen entered the failure stage.

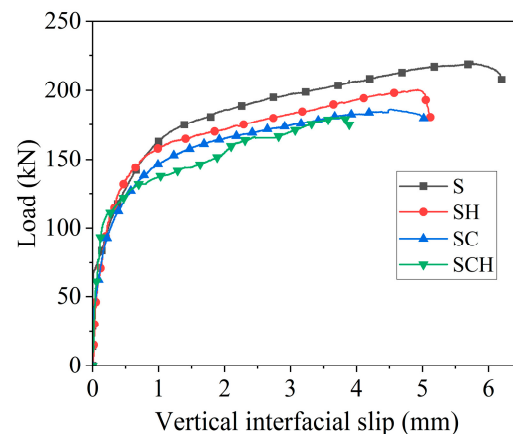


Figure 5. Load–vertical interfacial slip curves.

Furthermore, specimens subjected to transverse shear and stud corrosion entered the yield and failure stages earlier than specimen S, leading to a progressive reduction in their load-bearing capacity. Notably, under the combined effect of transverse shear and stud corrosion, a more pronounced degradation in load-bearing capacity was observed. This phenomenon aligns with the shear behavior of stud connectors under combined tension and shear forces reported in Ref. [24].

In addition, under the action of transverse shear load, the change in the early-stage shear stiffness of Specimen SH was not significant. However, compared with Specimen S, Specimen SH entered the yielding and failure stages earlier, and the maximum interfacial slip also decreased. Similarly, after the corrosion of the studs, both the yield load and failure load of Specimen SC decreased, which is mainly due to the reduction in the effective load-bearing cross-section of the studs caused by corrosion [32]. When transverse load and stud corrosion acted together, the bearing capacity and maximum interfacial slip of Specimen SCH both exhibited more severe degradation. This indicates that in the monorail steel–concrete composite track beam, the impact of transverse load on the long-term mechanical properties of stud connectors cannot be ignored.

3.2. Shear Capacity and Stiffness

Table 1 summarizes the key results of the push-out tests. F_{vp} and S_{vp} denote the peak shear force and its corresponding slip on the load–slip curve, respectively. The shear stiffness of the stud connectors was defined as the secant modulus of the load–slip curve at

a slip of 0.2 mm, and S_{vu} was defined as the slip on the descending branch of the load–slip curve corresponding to $0.9F_{vp}$ [6].

The peak shear capacity of specimen S was 220.89 kN, while those of specimens SH, SC, and SCH were 200.53 kN, 186.28 kN, and 180.81 kN, representing 90.8%, 84.3%, and 81.9% of the peak shear capacity of specimen S, respectively. These results indicate that transverse shear and stud corrosion exert adverse effects on the peak shear capacity of the specimens. Compared to specimen SH, the capacity of specimen SCH further decreased by 9.7%, indicating that the combined action of transverse shear and stud corrosion exacerbated the degradation of the shear performance of the stud connectors.

The shear stiffness of specimen S was 471.5 kN/mm, while that of specimen SH was 473 kN/mm, showing proximity. This similarity likely resulted from incomplete elimination of interfacial adhesion between the steel plate and concrete during the initial loading stage, leading to similar shear stiffness values calculated based on the load corresponding to 0.2 mm slip. The shear stiffness of specimen SC was 452.5 kN/mm, representing a 4% reduction compared to specimen S. In contrast, the shear stiffness of specimen SCH reached 536 kN/mm, showing a 13.7% increase relative to specimen S. This discrepancy is likely due to the persistent substantial interfacial adhesion between the steel plate and concrete, coupled with the corrosion-induced tighter steel–concrete interface, which enhanced the initial-stage interfacial stiffness. Notwithstanding this, specimen SCH exhibited the smallest vertical interface slip at peak load and ultimate load, accounting for 64.4% and 62.7% of those in specimen S, respectively. This indicates that transverse shear and stud corrosion primarily reduce the deformation capacity of the specimen interface.

Additionally, a series of calculation formulas for the shear capacity of individual stud connectors in steel–concrete composite systems have been proposed by researchers. Table 2 summarizes these corresponding calculation models and presents the ultimate shear capacities of the stud connectors in this study under pure vertical loading. Comparative analysis reveals that the ultimate shear capacities derived from other models are relatively conservative. Notably, the calculated value from Oehlers’ formula aligns more closely with the experimental results. However, it is plausible that current formulas may not adequately capture the actual loading conditions of straddle-type monorail transit systems. Therefore, the shear performance of stud connectors in steel–concrete composite track beams (subjected to bidirectional shear) requires systematic assessment. Subsequent analysis will improve upon these formulas to develop an enhanced predictive model for the shear capacity of stud connectors, explicitly accounting for the effects of transverse loading and stud corrosion.

Table 2. Typical design provisions for the shear capacity of the connectors.

Design Code or Literature	Formula	P_u	Deviation
Test result	/	55.2	/
Eurocode 4 [25]	$P_u = \begin{cases} \frac{0.8f_u\pi d^2/4}{\gamma_v} & 3 \leq \frac{H}{d} \leq 4 \\ \frac{0.29\alpha d^2\sqrt{f_{ck}E_c}}{\gamma_v} & \frac{H}{d} > 4 \end{cases}$	35.3	−36.1%
GB 50017-2017 [26]	$P_u = 0.43A_s\sqrt{f_{ck}E_c} \leq 0.7A_s\gamma f_u$	29.7	−46.2%
Oehlers and Foley [27]	$P_u = 5.0(f_{cu}/f_u)^{0.35}(E_c/E_s)^{0.4}A_sf_u$	54.3	−1.6%
Nie et al. [28]	$P_u = 0.43A_s\sqrt{f_{cu}E_c} \leq \alpha A_sf_u$	34.3	−37.9%
Xue et al. [29]	$P_u = 3\lambda A_sf_u \left(\frac{E_c}{E_s}\right)^{0.4} \left(\frac{f_{cu}}{f_u}\right)^{0.2}$	43.1	−21.9%

Note: P_u is ultimate shear capacity of a single stud, f_u is ultimate tensile strength of stud, d is diameter of stud, F_{ck} is cylinder concrete compressive strength, H is height of stud, γ_v is the safety factor – recommended value is 1.25 – γ is the ratio of ultimate tensile strength to yield strength of the stud, α is strength reduction factor, F_{cu} is cubic concrete compressive strength, E_c is elastic modulus of concrete, E_s is elastic modulus of stud, and A_s is cross-sectional area of stud.

3.3. Failure Modes

Generally, three typical failure modes of stud connectors can be observed in standard push-out tests [33]: (i) stud fracture without concrete damage; (ii) concrete crushing without stud shear failure; (iii) a combination of stud shear fracture and concrete crushing near the stud root. The failure modes in this study are presented in Figure 6, with no noticeable deformation observed in the steel plates. All specimens failed due to fracture at the stud root, and the fracture surfaces of the studs were smooth, indicating favorable welding quality. For specimens without transverse shear force (specimens S and SC), only localized concrete crushing was noted near the stud root due to the bending and tensile forces transmitted from the studs. For specimens SH and SCH, the application of transverse shear force induced a change in the deformation direction of the studs, suggesting a notable influence of transverse shear on the failure mode.

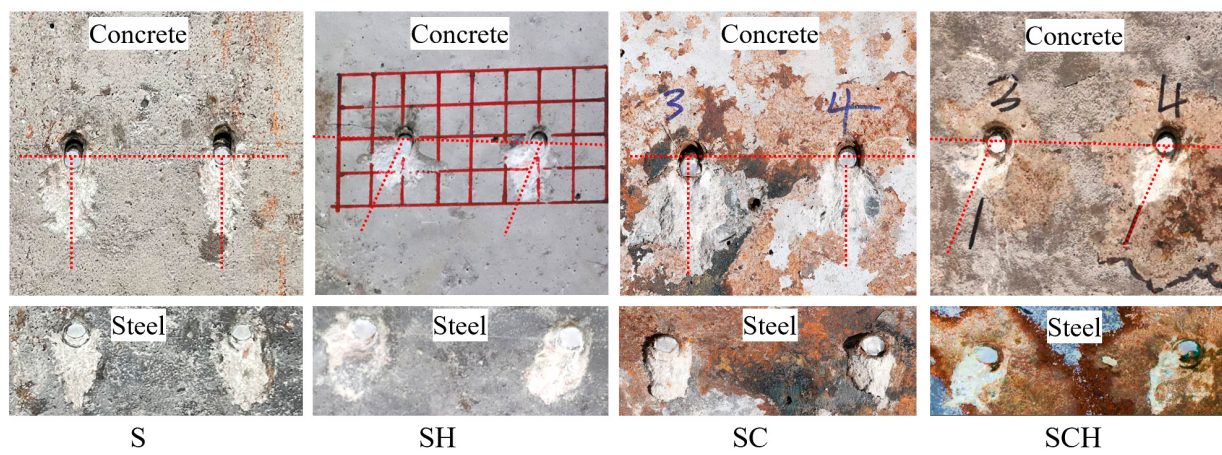


Figure 6. Failure modes.

4. Numerical Simulation

4.1. FEM Model Establishment

A finite element model (FEM) of the push-out specimen was established using ABAQUS (version: 6.14), as shown in Figure 7. To simplify calculations, only a 1/2 symmetric structure was modeled due to symmetry considerations, with ZX-direction constraints applied to the symmetric plane. The concrete slab, steel plate, and stud connectors were discretized using 8-node reduced-integration solid elements (C3D8R). While the steel rebar skeleton was modeled with 2-node linear truss elements (T3D2), via “embedded region” technology, the truss-formed rebar skeleton was embedded in solid concrete elements, constraining its node displacement to concrete deformation [34].

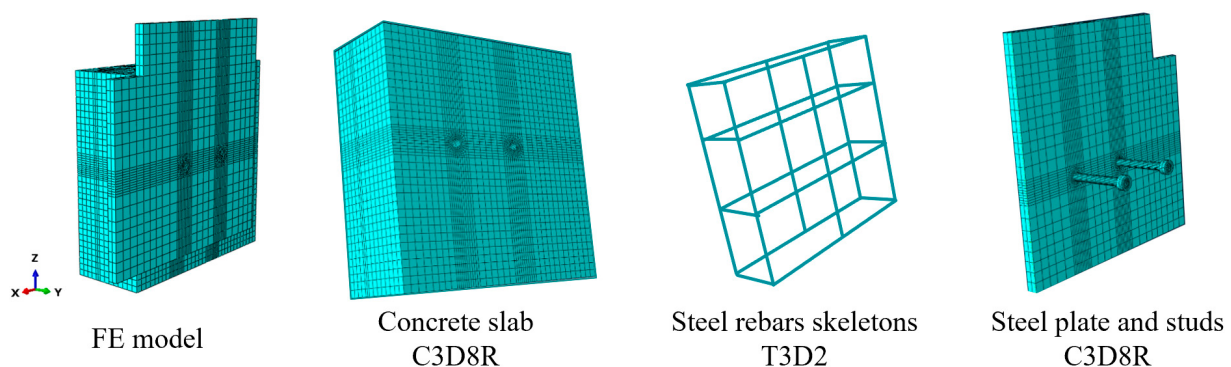


Figure 7. FE model, element type, and mesh.

To establish an appropriate meshing strategy, identical mesh seeds were assigned to the contact interface between the stud and the surrounding concrete, followed by a mesh sensitivity analysis with varying element sizes. Guided by both trial simulations and experimental validation, the final mesh configuration was determined as follows: In the critical regions (studs and adjacent concrete), where the studs experience complex stress states and thus require higher resolution, a biased mesh of 2–7 mm was applied along the stud axis (from root to cap), with 16 uniformly distributed seeds along the circumference. The adjacent concrete adopted the same mesh density to ensure interface compatibility, geometric symmetry, and to minimize distortion. In non-critical regions (concrete remote from the interface and steel plates), where stress gradients were mild, a coarser mesh of 10 mm was employed, with the maximum element size limited to 18 mm, thereby reducing computational demand without compromising accuracy. The reinforcement bars were meshed with an element size of 4 mm.

The surface-to-surface contact was adopted in this study, as it can yield more accurate computational results when compared with experimental data [35]. The contact surfaces between the stud heads and the steel plates were defined as the master surfaces, while the concrete surfaces were set as the slave surfaces. The tangential behavior was simulated using a penalty friction coefficient of 0.3, with normal hard contact applied. For the interaction between the concrete and the steel plate, a penalty friction coefficient of 0 was selected because preloading was applied prior to the experiment to damage the bonding interface between the concrete and the steel plate, thereby eliminating the influence of bonding force on the test results. The steel rebar skeleton was embedded in the concrete slab and selected as the embedded element, and the concrete was assigned as the host element.

4.2. Boundary Conditions and Loading Protocol

Displacements of the concrete slab in the x , y , and z directions were fully constrained at its bottom surface. A YSYM symmetry constraint ($U_2 = UR_1 = UR_3 = 0$) was applied to the steel plate on the XZ symmetry plane, as illustrated in Figure 8. For specimens without transverse loading, vertical displacement-controlled loading was applied at the reference point on the top surface of the steel plate, consistent with the actual loading conditions of the push-out tests. For specimens subjected to transverse loading, the loading process was divided into two steps: transverse loading and formal loading. First, a uniformly distributed load was applied to the steel plate and the side faces of the concrete slab to induce transverse shear forces. The centroid height of this distributed load was aligned with the height of the stud centers, and the resultant forces of the distributed loads on both sides were equal in magnitude and opposite in direction. Subsequently, vertical displacement-controlled loading was applied at the reference point on the top surface of the steel plate.

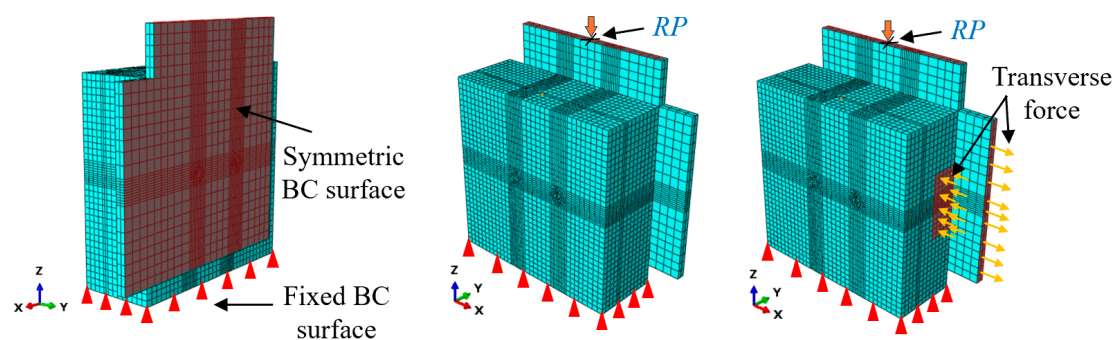


Figure 8. Boundary condition and loading method.

4.3. Material Models

The stress–strain relationships of all materials are presented in Figure 9. Based on the failure modes observed in the specimens, no significant deformation was detected in the steel plates and reinforcing bars. Ideal elastic–plastic models were therefore adopted for the steel plates and rebar skeletons, as illustrated in Figure 9a. The constitutive relationship of the studs was described using a tri-linear elastic–plastic model, as shown in Figure 9b. Specific material property data were taken from the mechanical parameters listed in Section 2.1, with Poisson’s ratio set to 0.3. For the concrete material, the damage plasticity model (Concrete Damaged Plasticity, CDP) was used to simulate the development of damage. The CDP model primarily accounts for concrete behavior under uniaxial compression and uniaxial tension, with the corresponding CDP models illustrated in Figure 9c,d, respectively. The concrete property data, as well as the calculations of strain and damage factors under compression and tension, were adopted based on GB 50010-2010 [36] and our previous studies [6].

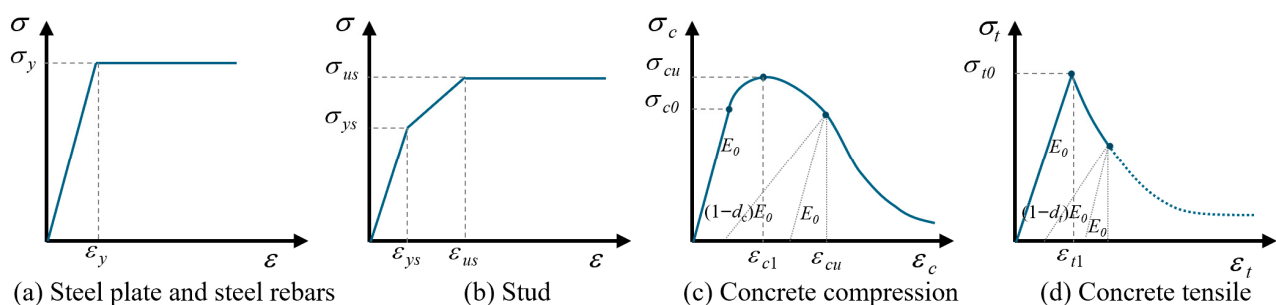


Figure 9. Properties of materials.

4.4. Corrosion Simulation

Due to the embedment of studs in concrete, the corrosion levels at the stud heads and roots differ. This study investigates the effects of two key parameters (corrosion height and depth) on the performance of stud connectors. To better simulate the rust layer within the corroded regions, previous studies [37,38] considered varying corrosion depths and conducted parametric analyses with the elastic modulus of rust layer as a variable, and found that assuming an elastic modulus of 2.1 GPa for rust layer yielded relatively accurate simulation results. Accordingly, this study adopts the same assumption: the elastic modulus of rust layer is set to 2.1 GPa, with both yield strength and ultimate strength reduced to 10% of those of intact studs. All subsequent simulation parameters follow this configuration. Additionally, a uniform circular shell model is employed to represent the corroded regions, as presented in Figure 10. This model uniformly weakens the stud’s cross-section along its radial direction to simulate corrosion depth, with specified depths of 10%, 20%, 30%, and 40% of the stud diameter. Corresponding corrosion heights are defined as 20%, 30%, and 40% of the stud height, respectively.

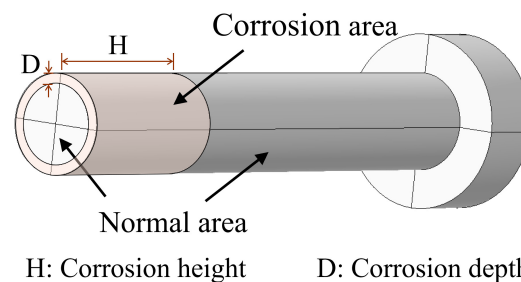


Figure 10. Simulation diagram of corroded studs.

4.5. Model Validation

4.5.1. Load–Slip Curve Comparison

Figure 11 presents the experimental and numerical simulation results for specimens S and SH. For each specimen, the load–slip curves from the tests and simulations exhibit similar trends. During testing, preloading was applied to eliminate the interfacial bond strength between the concrete slab and steel plate; thus, the friction coefficient between the concrete and steel plate was set to 0 in the numerical model. This explains why the initial stiffness predicted by the simulation is relatively low. Nevertheless, the experimental data points are distributed around the simulated curves, indicating acceptable accuracy in the simulation of the shear behavior of stud connectors.

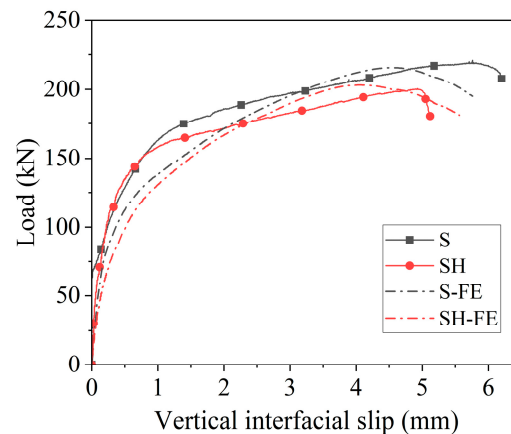


Figure 11. Comparison of load–slip curves between experimental and numerical results.

Table 3 presents the numerical results and corresponding experimental results of the peak shear capacity for specimens S and SH. It is observed that the deviation of the peak load between the numerical and experimental results is within 2.7%, indicating that the established finite element model is relatively accurate in simulating shear capacity and provides a reasonable method for further analysis of the shear behavior of stud connectors under bidirectional shear loading.

It should be noted that comparisons with corroded specimens (SC and SCH) were not performed due to numerical convergence difficulties associated with the very small measured average corrosion depth (0.41 mm for SC and 0.48 mm for SCH). While the numerical model shows good agreement with experimental peak loads and overall load–slip trends for S and SH, slight discrepancies in vertical interface slip are observed, which are attributable to unavoidable residual bond effects between steel plates and concrete in the experiment. These limitations highlight that the present model is primarily applicable for the Ultimate Limit State, and further investigation is required to assess deformation behavior under serviceability conditions.

Table 3. Comparison of peak shear capacity.

Specimen	F_{test}	F_{PE}	F_{PE}/F_{test}
S	220.89	215.79	0.977
SH	200.53	203.16	1.013

4.5.2. Failure Mode Comparison

The failure modes of the concrete in the specimens, derived from both finite element analysis (FEA) and experimental tests, are presented in Figure 12. It was observed that the damage orientation and area of the concrete predicted by FEA were generally con-

sistent with the experimental results. The specimens primarily exhibited failure modes characterized by fracture at the stud root, with both experimental and FEA results showing significant crushing of the concrete adjacent to the stud root. With the introduction of transverse shear, the damage distribution direction of the concrete formed a distinct angle relative to the vertical load direction. The high consistency in failure modes indicates that the failure patterns depicted by the numerical analysis effectively reflect the actual failure process of the experimental specimens.

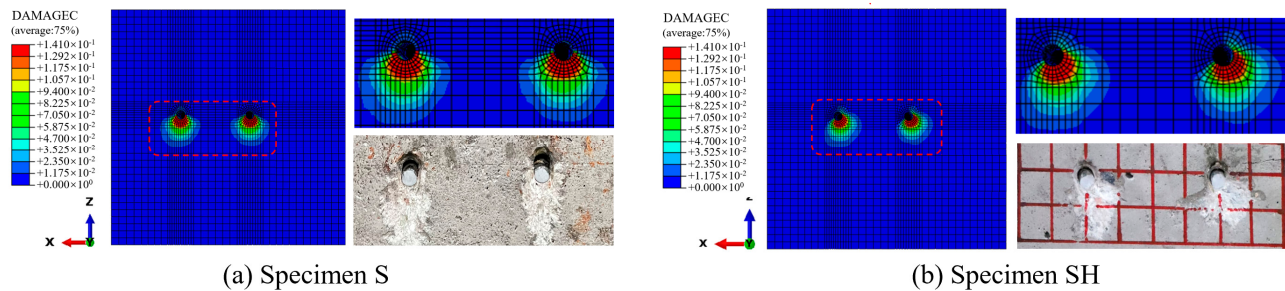


Figure 12. Failure mode comparison between the test and the FE results.

5. Parametric Analysis

Based on the experimentally validated finite element model, this study conducted a systematic parametric investigation into the shear behavior of stud connectors under the combined influences of transverse shear loads and stud corrosion. The parameters examined included corrosion height, corrosion depth, and the magnitude of transverse shear load. By incorporating reduction factors for transverse shear and stud corrosion, a strength prediction model for stud connectors was proposed. Table 4 summarizes the corresponding parameters and numerical simulation results, revealing that both transverse shear loads and stud corrosion exert detrimental effects on the shear capacity of the connectors.

Table 4. Numerical simulation parameters and simulation results.

Scenario	Specimen	RoHV	Height of Corrosion	Depth of Corrosion	Peak Load (kN)
Horizon shear load	S-0-H0-D0	0	/	/	215.7
	S-0.3-H0-D0	0.3	/	/	203.2
	S-0.5-H0-D0	0.5	/	/	181.0
	S-0.7-H0-D0	0.7	/	/	136.1
Corrosion of stud	S-0-H20-D10	/	20%	10%	191.2
	S-0-H20-D20	/	20%	20%	168.4
	S-0-H20-D30	/	20%	30%	146.2
	S-0-H20-D40	/	20%	40%	128.4
	S-0-H30-D30	/	30%	30%	143.5
	S-0-H40-D30	/	40%	30%	140.3
Horizon shear load and corrosion of stud	S-0.3-H20-D10	0.3	20%	10%	180.4
	S-0.3-H20-D20	0.3	20%	20%	157.1
	S-0.3-H20-D30	0.3	20%	30%	135.4
	S-0.3-H20-D40	0.3	20%	40%	113.6
	S-0.5-H20-D10	0.5	20%	10%	153.6
	S-0.5-H20-D20	0.5	20%	20%	123
	S-0.5-H20-D30	0.5	20%	30%	92.2
	S-0.5-H20-D40	0.5	20%	40%	66

5.1. Influence of Transverse Shear Load

The load–slip curves obtained from numerical simulations under different transverse shear ratios are presented in Figure 13a. Four distinct stages, consistent with experimental observations, are identifiable in the simulated curves: the elastic stage, yield stage, stable development stage, and failure stage. With the increase in the transverse shear ratio, the peak vertical shear capacity of the stud connectors decreases consistently, accompanied by a corresponding reduction in the associated slip. Figure 13b illustrates the variation trend of the vertical shear capacity of the stud connectors under different transverse shear ratios. It is evident that the vertical shear capacity exhibits a nearly power-law decreasing trend as the transverse shear ratio increases.

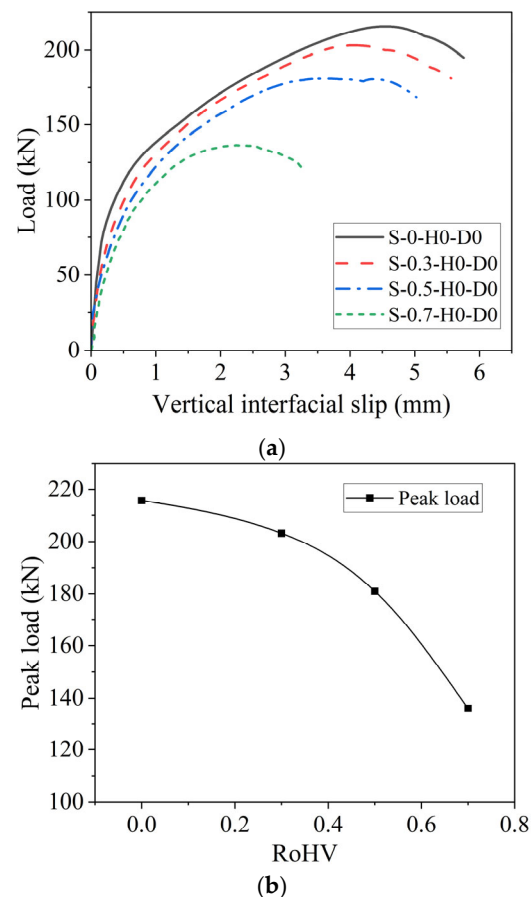


Figure 13. Effect of the transverse shear load. (a) Load-vertical interfacial slip, (b) Peak load vs RoHV.

5.2. Influence of Stud Corrosion

To systematically investigate the effects of varying corrosion heights and depths on the mechanical performance of stud connectors, this study examined four corrosion depth scenarios (10%, 20%, 30%, and 40% of the stud diameter) and three corrosion height scenarios (20%, 30%, and 40% of the stud height). The finite element simulation results, detailed in Table 4, reveal the quantitative impacts of different corrosion zone configurations on the shear behavior of stud connectors, as illustrated in Figure 14a. Specifically, for a corrosion depth of 30% (relative to the stud diameter), the analysis shows that increasing the corrosion height (measured as a percentage of the stud height) leads to progressive degradation in both load-bearing capacity and stiffness. Compared with the uncorroded specimen, the specimen with a 20% corrosion height (of the stud height) exhibited a 32.22% reduction in ultimate shear capacity and a 52.56% decrease in stiffness; for a 30% corrosion height, these reductions increased to 33.47% and 55.13%, respectively; and for a 40%

corrosion height, the capacity and stiffness dropped by 34.96% and 56.41%, respectively. This trend underscores the combined detrimental effect of higher corrosion heights and depths on the structural performance of stud connectors under bidirectional shear loading.

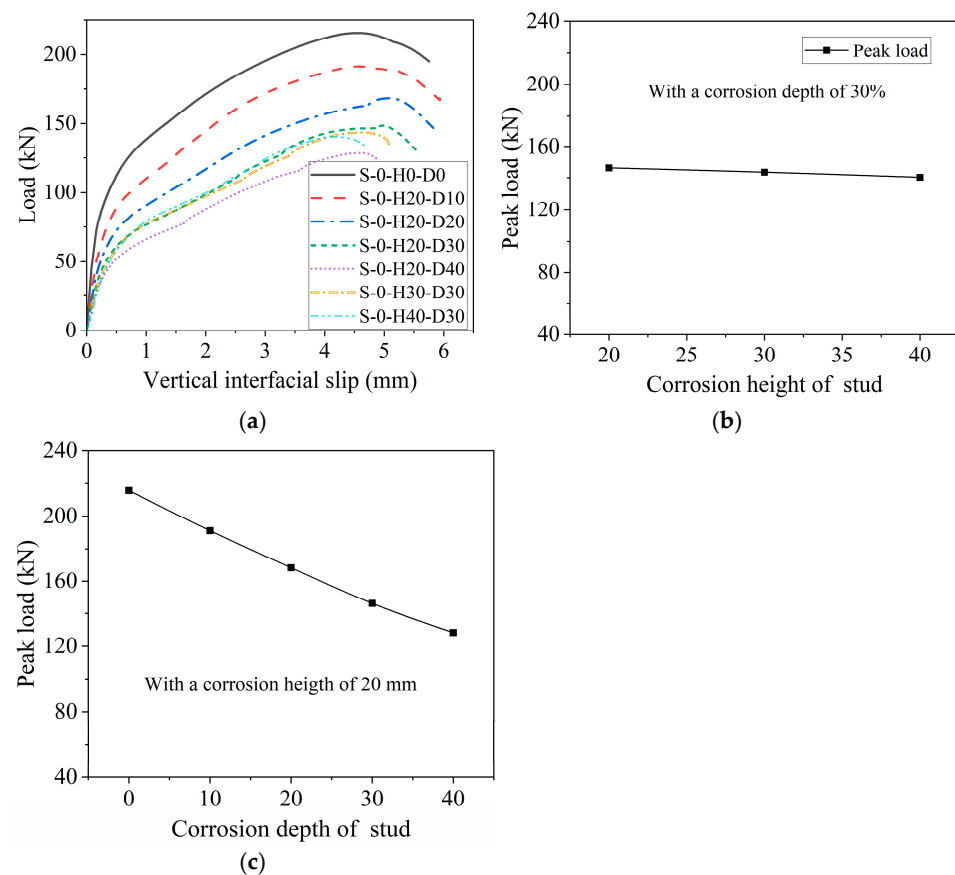


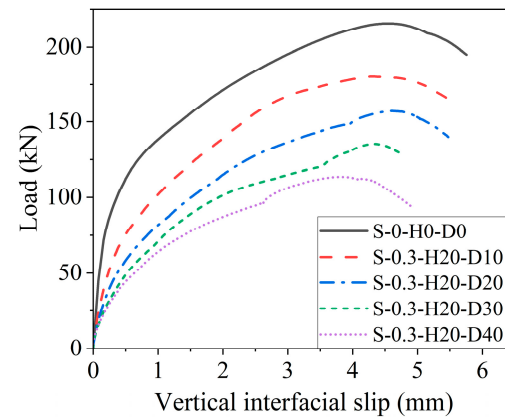
Figure 14. Effect of corrosion of stud. (a) Load-vertical interfacial slip, (b) Peak load vs Corrosion height of stud, (c) Peak load vs Corrosion depth of stud.

Figure 14b illustrates the effect of varying corrosion heights on the shear capacity of studs under a constant corrosion depth, revealing that corrosion height has a negligible impact on the shear capacity. In contrast, Figure 14c demonstrates the variation pattern of stud shear capacity with changing corrosion depths under a fixed corrosion height. For a constant corrosion height (20% of the stud height), changes in corrosion depth exert a more pronounced effect on the load–slip curves of the specimens and significantly alter the shear capacity of the studs. This indicates that stud connectors exhibit greater sensitivity to the corrosion depth parameter compared to corrosion height.

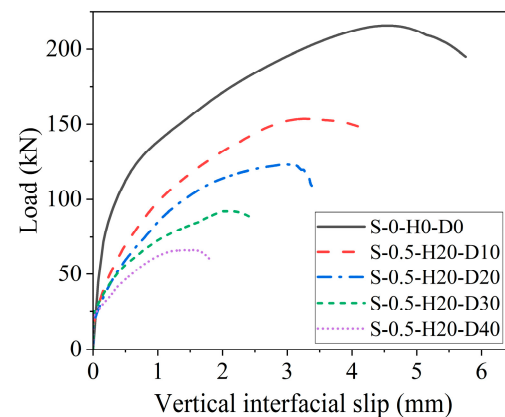
5.3. Coupling Effect of Stud Corrosion and Transverse Shear Load

Figure 15 presents the variation in the mechanical performance of stud connectors under the combined influence of bidirectional shear and stud corrosion. From Figure 15a,b, for a fixed corrosion height of 20% and a transverse shear ratio of 0.3, the ultimate load-bearing capacity of the connectors decreased progressively with increasing corrosion depth. Specifically, compared to the uncorroded specimen, the specimen with a 10% corrosion depth exhibited a 16.38% reduction in load-bearing capacity and a 41.44% decrease in stiffness; for a 20% corrosion depth, these reductions intensified to 27.17% (capacity) and 55.38% (stiffness). When the transverse shear ratio increased to 0.5, the degradation of the ultimate load-bearing capacity became more pronounced, though the overall trend remained consistent. Under this condition (20% corrosion height and 0.5 transverse shear

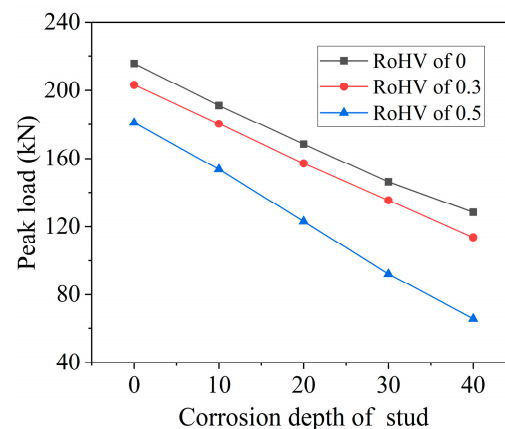
ratio), the 10% corrosion depth specimen showed a 28.82% reduction in capacity and a 43.6% decrease in stiffness; the 20% corrosion depth specimen experienced a 43.33% capacity reduction and 50.87% stiffness loss; the 30% corrosion depth specimen displayed a 57.27% capacity decrease and 55.08% stiffness reduction; and the 40% corrosion depth specimen exhibited a 69.42% capacity drop and 58.05% stiffness reduction.



(a)



(b)



(c)

Figure 15. Effect of transverse shear load and stud corrosion. (a) Load-vertical interfacial slip for specimen with 0.3 RoHV, (b) Load-vertical interfacial slip for specimen with 0.5 RoHV, (c) Peak load vs Corrosion depth of stud.

Notably, Figure 15c reveals that the impact of transverse shear load on the shear capacity of stud connectors became more significant as corrosion depth increased. This observation underscores the necessity of accounting for the coupling effect between bidi-

rectional shear and stud corrosion when developing a precise predictive model for the shear capacity of stud connectors, as neglecting this interaction may lead to inaccurate assessments of their structural performance.

5.4. Shear Strength Model Incorporating Transverse Force and Stud Corrosion

Although previous studies [23,24] have proposed shear strength formulas for stud connectors under combined shear and tensile forces, no established formula currently exists to evaluate the shear strength of stud connectors subjected to bidirectional shear forces induced by monorail trains or the coupled effects of such shear forces and stud corrosion. As revealed by the preceding analysis, the presence of transverse shear force significantly influences the shear behavior of stud connectors, with the degradation of their shear capacity becoming more pronounced when coupled with stud corrosion.

Based on the numerical results in Table 4, the shear strength of stud connectors under different transverse shear loads was normalized against that without transverse shear, and this normalized factor was defined as the strength reduction coefficient β . Figure 16 illustrates the variation trend of the strength reduction coefficient with respect to the transverse shear ratio of the stud and the corrosion depth of the stud. Additionally, as shown in Table 2, Oehlers' formula [27] yields results that most closely align with the experimental data while remaining conservative. Therefore, based on this formula, a proposed equation for the shear capacity of stud connectors under transverse shear force and stud corrosion is given in Equation (6).

$$\begin{cases} \beta = -1.37R_{HV}^{2.66} - 1.14C_d + 1.02 \\ P_u = 5\beta A_{ss}f_u \left(\frac{E_c}{E_s}\right)^{0.4} \left(\frac{f_{cu}}{f_u}\right)^{0.35} \end{cases} \quad (6)$$

where β denotes the reduction factor, R_{HV} represents the transverse shear load ratio, C_d is the corrosion depth, P_u stands for the shear capacity of the stud, A_{ss} is the cross-sectional area of the stud, f_u indicates the tensile strength of the stud, E_s and E_c are the moduli of elasticity of the stud and concrete, respectively, and f_{cu} represents the cubic compressive strength of the concrete.

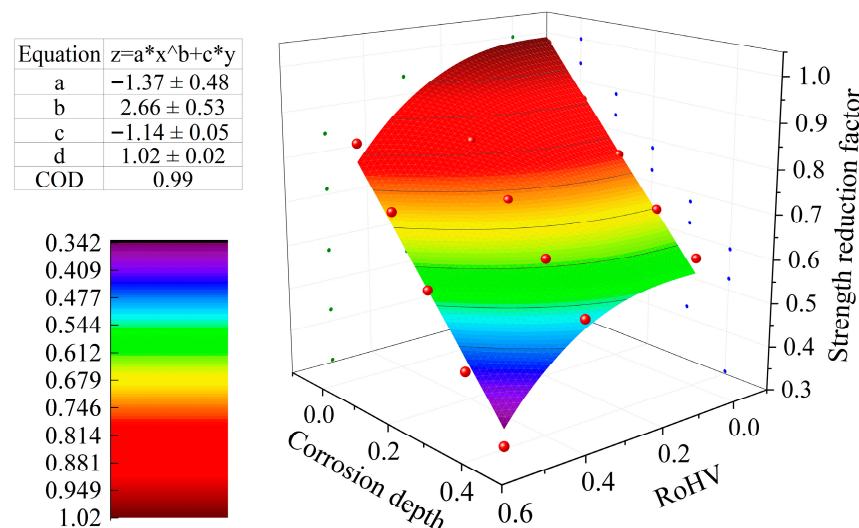


Figure 16. Trend of the strength reduction factor with respect to the transverse shear ratio and corrosion depth of the stud.

The ultimate shear capacity calculated using Equation (6) was compared with the experimental results, as shown in Table 5. The results indicate that the prediction error

of the equation for non-corroded specimens is within 4.4%, whereas the deviation for corroded specimens is approximately 12%. This discrepancy primarily arises from the assumption of a uniform radial corrosion distribution in the equation, while the actual corrosion is not strictly uniform. Moreover, the corrosion morphology of stud connectors and boundary conditions can significantly affect their shear performance [37,39]. Therefore, further investigations under more complex corrosion patterns and bidirectional shear conditions are required to enhance the applicability of the equation.

Table 5. Comparison between experimental results and analytical prediction.

Specimen	RoH	Measured CD	Reduction Factor, β	Shear Capacity Per Stud		F_t/F_p
				Test Results, F_t	Predictive Results, F_p	
S	0	0	1.02	55.20	55.39	1.00
SH	0.3	0	0.96	50.10	52.36	0.96
SC	0	4.1%	0.97	46.60	52.85	0.88
SCH	0.3	4.8%	0.91	45.20	49.39	0.92

6. Conclusions

This study investigates the performance of stud connectors in straddle-type monorail track girders under the coupled effects of bidirectional shear and corrosion through push-out tests and numerical simulations. Moreover, the influence of parameters is analyzed and a predictive model is established. The key conclusions are as follows:

(1) Both transverse shear and corrosion induce degradation of the load-bearing capacity of stud connectors, with the coupling effect being more pronounced. A transverse load equivalent to 0.3 times the ultimate load (P_u) reduces the capacity of uncorroded specimens by 9.2%. A 5% corrosion rate alone causes a 15.7% reduction in stud capacity, while their combined effect leads to an 18.1% decrease.

(2) All specimens exhibit a failure mode characterized by fracture at the stud root with smooth fracture surfaces. For specimens without transverse load, only localized concrete crushing near the stud root is observed. After applying transverse load, the deformation of studs deflects, resulting in angular deviation of the fracture surface.

(3) The transverse shear ratio and corrosion depth are identified as key influencing parameters for the load-bearing capacity of stud connectors, whereas the influence of corrosion height is relatively minor. When the transverse shear ratio increases to 0.7 P_u , the load-bearing capacity decreases to 64.4%. At a corrosion depth of 40%, the capacity linearly decreases by 40.5%.

(4) A predictive model for the shear capacity of stud connectors was developed by incorporating reduction factors for transverse shear and corrosion depth, achieving a coefficient of determination (R^2) of 0.99. This model is capable of predicting the load-bearing capacity of stud connectors under various operating conditions, providing a scientific basis for the design and durability assessment of monorail track girders.

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