

Advanced Finite Element Analysis, Multi-Scale Mechanics, and Digital Twins for Steel–Concrete Composite Interfaces: A State-of-the-Art Review

Arjun Chhaiya¹, Dr. Chirag Odedra², Dr. Prashant Bhuva³, Dr. Kalpesh Kapadiya⁴, Shekhar Parmar⁵, Ashish Kachhadiya⁶

¹P.G. Student, Department of Civil Engineering, Dr. Subhash University, Gujarat, India

^{2,3,4} Associate Professor, Department of Civil Engineering, Dr. Subhash University, Gujarat, India

^{5,6} Assistant Professor, Department of Civil Engineering, Dr. Subhash University, Gujarat, India

Abstract - Modern civil engineering relies heavily on steel–concrete composite systems. They pair the tensile capacity and ductility of structural steel with the bulk stiffness, compressive strength, and thermal protection of concrete. That entire structural mechanism stands or falls on the interface between the two materials. Over the past few decades, our design methodology has moved past basic empirical push-out setups like Eurocode 4 Annex B toward non-linear 3D finite element simulations, and more recently, toward physics-informed neural networks (PINNs) and field-deployable digital twins. This paper breaks down the core computational mechanics across six main themes: (1) macro-kinematics, partial interaction equations, interfacial bonding stages, and standard push-out rigs; (2) interface element formulation (cohesive zone models, discrete spring networks, fiber beam-columns, three-field mixed elements) and concrete damaged plasticity (CDP) calibration; (3) complex field conditions: 3D concrete pry-out cones, longitudinal splitting, connector cluster effects, long-term creep/shrinkage, multi-million cycle traffic fatigue, and fire exposure; (4) newer connection hardware: Perfobond Ribs (PBL), dismountable high-strength bolts for circular construction, and ultra-high performance concrete (UHPC) interfaces; (5) numerical objectivity, crack band energy regularization, stress singularity relief at stud welds, and model updating routines; and (6) machine learning surrogates, PINNs, and real-time digital twins built for structural health monitoring. We conclude by outlining remaining experimental and computational challenges that need attention in future field and laboratory work.

Key Words: Steel-concrete composite interfaces, Partial interaction, Concrete Damaged Plasticity, Perfobond ribs (PBL), Dismountable bolted connectors, Ultra-High Performance Concrete (UHPC), Crack Band model, Physics-Informed Neural Networks, Digital Twins.

1. INTRODUCTION

Steel–concrete composite girders are a staple in highway overpasses, multi-story buildings, and heavy industrial plants. Balancing concrete under compression against a structural steel flange under tension lets designers stretch spans further while cutting overall tonnage [1, 2]. But

composite behavior is not guaranteed. Everything depends on whether the shear connection along the joint can transfer horizontal forces and prevent vertical uplift [3, 4]. Without working connectors, the slab and the steel girder simply bend on their own, resulting in severe flexural sag and early serviceability issues.

For a long time, shear connector design leaned on empirical fits derived from standard push-out tests, most notably Eurocode 4 Annex B [5]. Yet standard push-out tests bring their own baggage: boundary friction can artificially brace the concrete, testing frames introduce parasitic moments, and the actual multi-axial stress field around the stud remains locked away inside a testing "black box" [6, 7].

Over the last twenty years, interface mechanics has shifted through three main phases:

1. High-fidelity non-linear FE simulations handling 3D contact friction and Concrete Damaged Plasticity [8, 9].
2. Advanced hardware alternatives like fatigue-resistant Perfobond Ribs (PBL), demountable bolts designed for building disassembly and reuse, and compact steel–UHPC decks [10, 11, 12].
3. Hybrid numerical and machine learning workflows, including Physics-Informed Neural Networks (PINNs), fast regression surrogates, and operational digital twins for predictive bridge maintenance [13, 14, 15].

Here, we trace these technical threads to give a clear, unified view of interface mechanics as it stands today.

2. Physical Foundations & Standard Testing

2.1 Macro-Kinematics & Partial Interaction Theory

Under classic Euler–Bernoulli beam assumptions with an infinitely stiff shear connection (full interaction), plane sections stay plane throughout the total depth ($s = 0$) [16]. Real shear studs, however, deform under horizontal shear flow. This movement creates a slip gradient across the interface [2]:

$$\Delta \epsilon = ds / dx = \epsilon_s(y_{int}) - \epsilon_c(y_{int}) \quad (1)$$

Because of this relative slip, the cross-section splits into two distinct layers. Each layer bends about its own neutral axis while sharing the overall curvature ($\phi = -v''$). The internal bending moment decomposes into three distinct parts [2, 17]:

$$M = M_c + M_s + F \cdot z \quad (2)$$

where $M_c = E_c I_c \phi$ reflects slab bending, $M_s = E_s I_s \phi$ covers steel girder bending, F represents the equal-and-opposite axial force produced by shear transfer, and z is the lever arm between cross-sectional centroids. If connector stiffness drops toward zero ($k_s \rightarrow 0$), then $F \rightarrow 0$, and both layers bend independently. As connection stiffness picks up, the axial couple $F \cdot z$ carries the lion's share of the applied moment [3].

Newmark, Siess, and Viest formulated the differential equation governing linear-elastic partial interaction [16]:

$$d^2F/dx^2 - \alpha^2 F = -\beta M(x) \quad (3)$$

with governing coefficients:

$$\alpha^2 = (k/p) [1/(E_s A_s) + 1/(E_c A_c) + z^2/(\Sigma EI)],$$

$$\beta = (k/p) z / (\Sigma EI) \quad (4)$$

Newmark introduced the dimensionless factor $1/C$ to evaluate connection efficiency. Girders with $1/C \geq 20$ generally reach more than 95% of their full-interaction bending capacity. His analytical solutions also show where interfacial slip bites hardest: it peaks near simple end-supports and abrupt point loads, tapering off in regions of constant bending moment.

2.2 Degree of Connection & Plastic Resistance

Under ultimate limit state (ULS) plastic design, sections are calculated with rigid-plastic stress blocks [5, 18]. The degree of connection (η) is defined by how many connectors are installed (N) versus the count required for full composite action (N_f):

$$\eta = N / N_f = (N \cdot P_{Rd}) / \min(A_a f_{yd}, A_c \cdot 0.85 f_{cd}) \quad (5)$$

When $\eta \geq 1.0$, the concrete slab develops its total compressive capacity ($N_{c,f} = A_c \cdot 0.85 f_{cd}$), meaning extra studs will not raise the plastic moment resistance ($M_{pl,Rd}$). When $\eta < 1.0$, the compressive force in the slab is limited to $N_c = \eta N_{c,f}$. The plastic neutral axis then shifts downward into the steel web, lowering the moment capacity (M_{Rd}) below $M_{pl,Rd}$ [3].

Eurocode 4 permits calculating M_{Rd} either through rigorous plastic stress blocks or a straightforward linear approximation:

$$M_{Rd} = M_{a,Rd} + \eta (M_{pl,Rd} - M_{a,Rd}) \quad (6)$$

To stop brittle connector failure from happening before the beam can form a ductile plastic hinge, Eurocode 4 sets a minimum degree of shear connection ($\eta_{min} \geq 0.40$ for spans under 25 m) and insists on a characteristic slip capacity $\delta_{uk} \geq 6.0$ mm for any connector categorized as ductile [3, 5].

2.3 The Tripartite Bonding Mechanism

As Oehlers and Bradford detailed, shear transfer across the interface actually happens through three overlapping physical steps [1]:

$$\tau_{total}(s) = \tau_{adhesion}(s) + \tau_{friction}(s) + \tau_{dowel}(s) \quad (7)$$

- **Chemical Adhesion** ($s < 0.1$ mm): Microscopic cement hydration crystals bond to the steel flange, giving very stiff initial behavior. But this bond fractures abruptly at tiny slip displacements and offers zero post-break strength. Because it cannot be counted on in practice, testing standards require greasing steel flanges during push-out tests [5].
- **Coulomb Friction** ($0.1 \text{ mm} \leq s \leq 0.5 \text{ mm}$): Driven by $\tau_f = \mu \sigma_n$ (where $\mu \approx 0.40\text{--}0.50$). Normal compressive stress (σ_n) develops from gravity loads, slab prying, and the downward clamping provided by stud heads [1].
- **Mechanical Dowel Action** ($s > 0.5 \text{ mm}$): The stud shank bends, yields, and crushes the local concrete pocket directly in front of its base. Ollgaard, Slutter, and Fisher laid out the classic expressions [4]:

$$Q_u = 0.5 A_s \sqrt{f'_c E_c} \leq A_s f_u \quad (8)$$

$$Q(s) = Q_u (1 - e^{(-0.71 s)})^{0.55} \quad (s \text{ in mm}) \quad (9)$$

Notice that the concrete elastic modulus (E_c) carries just as much weight as compressive strength (f'_c). That is why lightweight concrete drops stud shear resistance by 15% to 25% compared to regular mixes of identical compressive strength [4].

Standard push-out testing under Eurocode 4 Annex B uses an HEB 260 profile with two $600 \times 600 \times 150$ mm reinforced slabs attached to the flanges [5]. The sample undergoes 25 cycles between 5% and 40% of estimated ultimate capacity to seat the assembly, followed by displacement-controlled loading that takes at least 15 minutes to reach failure. The characteristic shear resistance is calculated as $P_{Rk} = 0.9 P_{test,min}$, with design resistance taken as $P_{Rd} = P_{Rk} / 1.25$.

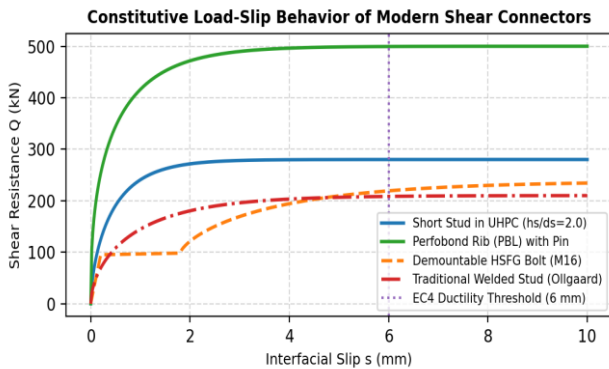


Fig -1: Constitutive load-slip response across connector typologies comparing traditional welded studs, PBL ribs with pins, demountable HSFG bolts, and short studs in UHPC against the EC4 ductility limit (6 mm).

3. FINITE ELEMENT FORMULATIONS FOR INTERFACES

3.1 Interface Kinematics & Element Formulations

Standard displacement-based continuum elements tend to suffer from numerical shear locking along thin composite joints. When subjected to steep shear gradients, lower-order formulations over-constrain the interface, making the connection artificially rigid and masking true slip [19]. Engineers generally turn to four numerical strategies to circumvent this:

- **Cohesive Zone Models (CZM):** CZM treats the contact plane as a traction-separation layer (τ - s). Lu et al. developed exponential, simplified, and bilinear softening laws for these interfaces [20]. They showed that ultimate bond capacity (P_u) scales with interfacial fracture energy (G_f): $P_u = b_f \sqrt{(2 E_f t_f G_f)}$. Yuan et al. worked out the governing equations across five progressive debonding phases, defining the effective bond length L_e where load capacity hits an asymptotic ceiling [21].
- **Discrete Non-Linear Springs (e.g., COMBIN39):** Rather than modeling detailed 3D contact geometries, researchers often use multi-linear zero-length spring elements. Devaraj et al. combined these with artificial neural networks (ANNs) that predict load-slip backbones straight from connector geometry and material strengths [14]. Assigning these curves to spring elements maintains roughly 95% deflection accuracy while cutting CPU run times by over 99% compared to full continuum models.
- **Distributed-Slip Fiber Beam Elements:** Salari et al. formulated a 13-DOF element that bakes longitudinal slip directly into beam kinematics [22]. The slab and steel beam have independent axial displacement fields ($u_c(x)$ and $u_s(x)$) but share vertical deflections ($w_c = w_s$). Fiber-section integration handles material yielding

cleanly without generating the artificial stress spikes seen with discrete spring nodes.

- **Three-Field Mixed Variational Formulations:** Dall'Asta and Zona used a Hu-Washizu variational approach, treating displacements (u), strain fields (e), and internal stress fields (s) as separate entities (the HW112 element) [19]. Decoupling stresses from displacement derivatives eliminates shear locking and erratic stress ripples, yielding smooth profiles even on modest mesh sizes.

3.2 Concrete Damaged Plasticity (CDP) Calibration

Simulating concrete around connectors requires capturing non-linear crushing alongside tensile cracking. The Concrete Damaged Plasticity model [8] pairs multi-axial yield criteria with isotropic continuum damage:

$$\sigma = (1 - d) D_0 \epsilon^p : (\epsilon - \epsilon^p) \quad (10)$$

Here, d tracks stiffness degradation (compressive damage d_c or tensile damage d_t), softening the initial elastic matrix: $E = (1 - d) E_0$. The yield surface applies a modified Drucker-Prager criterion using a non-associated hyperbolic potential dictated by the dilation angle (ψ).

Without regularization, post-peak strain-softening creates heavy mesh dependency: as mesh size shrinks, the energy dissipated by cracked elements trends toward zero, causing unrealistic brittle failure [23]. To enforce mesh objectivity, Alfarah et al. incorporated Bažant's Crack Band Model into CDP [9, 23]. The cracking strain scales by the characteristic element length ($l_{ch} = \sqrt[3]{V_{elem}}$ for 3D elements):

$$\tilde{\epsilon}_{ck} = \tilde{\epsilon}_{cr} + w / l_{ch} \quad (11)$$

This keeps the area beneath the tensile softening curve ($G_f = \int \sigma_t dw$) consistent across different element sizes.

Tao et al. ran extensive parametric calibrations for the primary CDP inputs [24]:

1. **Dilation Angle (ψ):** Dictates plastic volume expansion. Unconfined concrete works well with $\psi = 30^\circ$ – 31° , while heavily confined steel-concrete pockets require 37° – 40° .
2. **Flow Potential Eccentricity (ϵ):** A default value of $\epsilon = 0.10$ maintains smooth numerical derivatives.
3. **Biaxial Ratio (f_{b0} / f_{c0}):** Typically 1.16 for conventional concrete, shifting to 1.20–1.25 under multi-axial confinement.
4. **Deviatoric Parameter (K_c):** Kept at $K_c = 2/3 \approx 0.667$ to preserve yield surface convexity.
5. **Viscoplastic Regularization (μ):** Relaxes the yield criterion in implicit solvers to clear tangent stiffness singularities. Setting $\mu = 10^{-5}$ to 10^{-4} helps Newton-Raphson convergence without distorting structural

capacity. However, pushing $\mu > 10^{-3}$ can generate unphysical overstrength of 15% to 20% [24].

Ultra-High Performance Concrete (UHPC) behaves differently. Fakeh and Jawdhari found that standard CDP values fail because of the dense particle packing and fiber pullout resistance [25]. UHPC demands a dilation angle around $\psi = 55^\circ$ and a biaxial ratio approaching $\sigma_{b0}/\sigma_{c0} = 3.00$. Birtel and Mark also proposed 1D strain-based damage relations using damage ratios $b_c = 0.70$ and $b_t = 0.10$ to prevent negative plastic strain rates and solver crashes during sharp load drops [26].

Table -1: Recommended CDP parameter settings across concrete types.

Parameter	Normal Concrete (NSC)	UHPC Matrix	Physical Mechanism
Dilation Angle (ψ)	30°–35°	55°	Controls plastic volumetric expansion & confinement
Biaxial Ratio (f_{b0}/f_{c0})	1.16	3.00	Expands yield surface under biaxial compression
Viscoplastic Parameter (μ)	10^{-5} – 10^{-4}	10^{-5} – 10^{-4}	Stabilizes implicit Newton–Raphson iterations
Compressive Plastic Ratio (b_c)	0.70	0.85	Controls accumulated plastic strain in compression
Crack Band Length (l_{ch})	$l_{ch} = \sqrt[3]{V_{elem}}$	$l_{ch} = \sqrt[3]{V_{elem}}$	Enforces mesh-independent fracture energy (G_f)

4. MULTI-AXIAL HAZARDS AND LONG-TERM EFFECTS

4.1 3D Uplift and Contact Separation

Basic 2D models assume the concrete slab remains firmly glued to the steel flange ($v = 0$). In reality, flexural curvature and stud bending generate vertical separation ($v(x) = w_c - w_s$) [27]. Gara et al. showed that uplift cuts the effective contact area by 6% to 14%. That loss concentrates horizontal shear into a smaller footprint directly ahead of the gap, moving the critical damage zone toward internal supports. Realistic 3D models require surface-to-surface Master-Slave penalty contact pairs (hard normal contact with Coulomb friction $\mu = 0.40$ – 0.50) so the gap can open naturally under tension while resisting interpenetration under compression [28].

4.2 3D Concrete Failure: Pry-Out, Splitting, and Group Effects

When large headed studs ($d_s \geq 22$ mm) sit inside thin slabs, failure often switches from stud shearing to 3D concrete pry-out [7, 29]. The shear load (V_s) introduces an overturning moment ($M = V_s \cdot e$). Very high triaxial compressive stresses ($\sigma_b \geq 3$ – $6 f_{ck}$) form at the front of the stud base, while the backward prying motion yanks the stud head upward. Conical failure surfaces break outward at angles between 35° and 45° .

Experimental data points to an embedment-to-diameter ratio threshold around $h_{ef}/d_s \approx 4.2$: studs below this ratio tend to pull out concrete cones abruptly, whereas deeper studs develop ductile steel shear [7].

In narrow concrete flanges, radial bursting forces can crack the slab longitudinally right down the line of connectors [30]. Providing adequate transverse rebar (at least 0.8% rebar ratio) clamps the concrete and restores ductile bearing [5]. When studs are clustered tightly with longitudinal spacing $p_L < 4.5 d_s$, their compressive stress bulbs overlap, creating an inter-stud damage band [31]. In UHPC matrices, Cao et al. demonstrated that load sharing among cluster rows is uneven: the leading row takes roughly 48% of the shear, the middle row handles 32%, and the trailing row carries just 20% [32]. This imbalance can spark progressive fatigue failure. They established an empirical group reduction factor:

$$\eta_{group} = 0.70 + 0.067 (p_L / d_s) \quad \text{for } 2.0 d_s \leq p_L < 4.5 d_s \quad (12)$$

4.3 Creep, High-Cycle Fatigue, and Fire

- **Concrete Creep and Shrinkage:** Steel does not creep or shrink under normal conditions, so differential movement gets restrained by the interface [33]. Over months and years, compressive stresses in the concrete relax, transferring axial load into the steel girder. This relaxation drives long-term slip and can double or triple initial deflections. Bradford and Gilbert accounted for this using the Age-Adjusted Effective Modulus Method (AAEMM):

$$E_e = E_c / (1 + \chi \phi), \quad \text{where } \chi \approx 0.80\text{--}0.85 \quad (13)$$

Liu et al. noted that seasonal environmental swings degrade effective connector stiffness by another 10% to 20% across normal operational life [34].

- **High-Cycle Traffic Fatigue:** Bridge decks handle millions of heavy axle passes. Fatigue damage generally runs through three stages: Stage I (rapid shakedown slip over the first 10^4 cycles), Stage II (steady-state crack propagation across the stud base, $ds/dN \approx \text{const}$), and Stage III (unstable acceleration leading to shank

rupture) [35, 36]. Eurocode 4 uses a Category 90 S-N curve:

$$(\Delta\tau_R)^8 \cdot N_R = (90)^8 \cdot (2 \times 10^6) \quad (14)$$

FE solvers track this using cycle-jump routines tied to the Palmgren–Miner cumulative damage sum ($D = \sum n_i / N_{f,i}$) to degrade spring stiffness incrementally [37].

- **Elevated Temperatures and Fire:** During a fire, steel and concrete heat up at very different rates. Mirza and Uy, along with Shahabi et al., observed that shear stud capacity stays largely intact up to 300°C (>90%), but drops off sharply between 400°C and 600°C (holding just 20%–25% capacity at 600°C, and under 10% at 800°C) [38, 39]. Above 500°C, the failure mode changes from concrete breakout to stud shearing because steel softens faster than concrete loses integrity. Furthermore, thermal expansion mismatch ($\alpha_s > \alpha_c$) generates secondary prying moments directly at the weld collar [40].

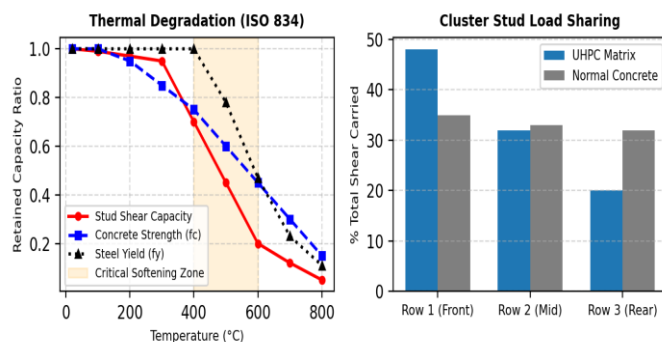


Fig -2: Multi-axial hazard mechanics: (a) Thermal degradation response under ISO 834 fire exposure showing critical softening between 400°C and 600°C; (b) Cluster stud load sharing across rows in UHPC vs. normal concrete matrices.

5. MODERN SHEAR CONNECTION SYSTEMS

5.1 Perfobond Rib (PBL) Connectors

To eliminate weld-toe fatigue and concrete breakout in shallow slabs, Andr  introduced the Perfobond Rib (PBL): a continuous steel plate with punched holes welded along the beam flange [10]. Force transfers via plate end bearing, surface friction, and the concrete pins that form inside the holes [41]:

$$Q_{PBL} = 4.54 h t b_c \sqrt{f_{ck}} + 1.41 n d d_h^2 \sqrt{f_{ck}} + 1.22 A_{tr} f_{y, tr} \quad (15)$$

Because PBL plates remove the sharp weld notch of headed studs and spread shear evenly, their fatigue endurance limit is 2.5 to 3.0 times higher [42]. Plain perforations tend to fail

abruptly once the concrete pin shears off. Threading a transverse steel rebar through the hole prevents this sudden drop, producing a ductile yield plateau with slips exceeding $\delta_{uk} > 10.0$ mm [43, 44].

5.2 Demountable Bolted Connectors for Circular Construction

Cast-in welded studs make it virtually impossible to take a structure apart without destroying the concrete slab. High-strength structural bolts (Grades 8.8 and 10.9) solve this, enabling circular building design and fast precast slab placement [11, 45]. Their load-slip response works through four distinct phases:

- **Phase I (Friction Resistance, $s < 0.1$ mm):** Shear is resisted strictly by friction between plates: $V_{slip} = k_s n_f \mu F_{p,C}$, giving very high initial stiffness.
- **Phase II (Hole Clearance Slip, $0.1 \text{ mm} \leq s \leq \Delta_{gap}$):** The bolt slips through hole clearances ($\Delta_{gap} = 1.0$ – 2.0 mm), creating a brief flat plateau with nearly zero stiffness.
- **Phase III (Shank Bearing):** The bolt shank hits the edge of the hole, restoring high stiffness ($K_{bearing} \approx 80$ – 120 kN/mm).
- **Phase IV (Plastic Deformation and Failure):** Bearing stresses ovalize the flange holes until threads shear or plates tear at large slips ($s_u > 8.0$ mm) [11, 46].

When running FE simulations of bolted joints, Dai, Lam, and Saveri pointed out that a two-step loading scheme is critical: Step 1 tightens bolt pre-tension ($F_{p,C} = 0.70 A_s f_{ub}$) across the shank section, and Step 2 locks the bolt length before applying shear [47]. Skipping the bolt pre-tension underestimates initial joint stiffness by over 300%. When a building reaches end-of-life, these bolts can simply be unbolted, allowing precast panels to be lifted away without cutting torch damage. That leaves the steel frames ready for reuse, saving upwards of 85% in embodied carbon [48, 49].

5.3 Steel–UHPC Interfaces

Ultra-High Performance Concrete ($f'_c = 120$ – 200 MPa, $f_{ct} = 8$ – 15 MPa) provides dense particle packing and strong internal fiber bridging that prevents concrete cone pry-out entirely [12, 37]. This means designers no longer need deep stud embedment ($h_s / d_s \geq 4.0$). Instead, stubby studs ($h_s / d_s = 1.5$ – 2.5) or headless dowels perform cleanly.

Short studs in UHPC match or exceed the capacity of full-length studs in conventional concrete while offering exceptionally high initial stiffness ($K_s > 400$ kN/mm) [37]. Kim et al. calibrated a modified load-slip relationship for these interfaces [12]:

$$Q(s) = Q_u (1 - e^{-(1.5 s)})^{0.60} \quad (16)$$

Ding et al. built dedicated UMAT/VUMAT subroutines to model fiber-bridging stresses across tensile micro-cracks without having to mesh every individual steel fiber explicitly [50].

Table -2: Overview of structural shear connection systems.

System	Primary Shear Transfer	Fatigue Life	Ductility & Demountability
Traditional Welded Stud	Shank bending & concrete bearing	Baseline (EC4 Category 90)	Ductile ($\delta_{uk} \geq 6$ mm); Non-demountable
Perfobond Rib (PBL)	Concrete dowel bearing & rebar shear	2.5x–3.0x higher than studs	High ductility ($\delta_{uk} > 10$ mm); Non-demountable
Demountable HSFG Bolt	Four phases: Friction → Slip → Bearing → Plastic	High (no weld notches)	Ductile ($s_u > 8$ mm); Fully reusable
Short Stud in UHPC	Pure shank shear (no concrete breakout)	High, but cluster row imbalance	Extremely stiff ($K_s > 400$ kN/mm); Non-demountable

6. MODEL VERIFICATION, CONVERGENCE, AND MODEL UPDATING

6.1 Scaling from Push-Out Tests to Full-Scale Beams

Applying push-out test results directly to full-size flexural girders often leads to errors [29, 51]. In push-out setups, minor loading eccentricities ($M_{ecc} = P \cdot e$) pry slabs outward and pull on the studs, while friction against machine baseplates artificially confines the slab base, inflating measured capacities by 10% to 15% [6]. In actual profiled deck slabs, adjacent spans create rotational clamping:

$$N_{tie-down} = (2 E_c L_{slab} \theta_{rot}) / (L_{span} \cdot b_{rib}) \quad (17)$$

This clamping holds the concrete rib into the steel trough, boosting shear capacity by 20% to 35% compared to standalone test specimens [52].

Nguyen and Kim proposed a two-tier calibration workflow: Tier 1 matches a local 3D push-out model against experimental load-slip curves, local concrete strains, and stud yielding. Tier 2 applies an empirical adjustment factor to reflect the beneficial longitudinal compression found in sagging beam regions [29]:

$$P_{beam} = P_{push} [1 + 0.12 (\sigma_{c,long} / f_{ck})] \quad (18)$$

This correction accounts for an 8% to 15% gain in real shear transfer.

Over continuous bridge supports, negative bending puts the deck into tension. Duan et al. split the hogging response into a Truss Mode (longitudinal shear pulls the slab into tension) and a Bending Mode (the two components flex separately) [53]. Crack opening can be checked using Uplift-Restricted Slip-Permitted (URSP) connectors. Concrete between cracks continues carrying tensile load through tension stiffening; omitting tension stiffening in FE runs overpredicts mid-span deflections by 15% to 25% [9, 53].

6.2 Mesh Objectivity and Singularities

- Crack Band Regularization: Scaling the ultimate tensile cracking strain inversely against element length ($\epsilon_{tu} = 2 G_f / (f_{ct} l_{ch})$) keeps fracture energy dissipation constant as mesh sizing changes between 2 mm and 100 mm [9, 23].
- Singularity Smoothing at Stud Roots: The sharp 90° corner between a stud shank and the steel flange acts as a mathematical stress singularity ($\sigma_{ij} \propto r^{-(\lambda-1)}$) [54]. Refining the mesh around sharp square corners will simply cause artificial localized crushing in the concrete. To avoid this, modelers must include an explicit weld collar radius ($r_w = 2.0$ – 4.0 mm), use structured circular O-grid meshing (1.5–2.5 mm elements), and apply volume-weighted non-local stress averaging [46, 54].
- Hourglass and Energy Checks: With first-order reduced integration bricks (C3D8R), users should check that artificial hourglass energy (ALLAE) stays well below total internal strain energy (ALLSE): $ALLAE / ALLSE \leq 5\%$ [24, 55]. Viscoplastic regularization ($\mu = 10^{-5}$ – 10^{-4}) stabilizes Newton–Raphson solver iterations across post-peak softening branches without adding unphysical stiffness.

6.3 Finite Element Model Updating (FEMU)

Composite FE models carry several uncertain parameters (concrete modulus E_c , spring stiffness K_s , friction coefficient μ , dilation angle ψ , bolt pre-stress). Closed-loop model updating frames this as an inverse optimization problem balancing frequency shifts, Modal Assurance Criterion (MAC) values, and static deflections [56, 57, 58]:

$$\Pi(\theta) = w_f \Sigma (\Delta \omega_i / \omega_i)^2 + w_\phi \Sigma [1 - MAC(\phi_i, \phi_{FE})] + w_u |\Delta u|^2 \quad (19)$$

Parametric physical updating (tweaking actual engineering constants like K_s and E_c) works much better than direct stiffness matrix manipulation because it preserves mesh connectivity and positive definiteness [58]. Tikhonov regularization:

$$\Delta \theta = (S^A T S + \lambda^2 L^A T L)^{-1} S^A T \Delta z \quad (20)$$

combined with L-curve criteria keeps sensitivity matrices stable in the presence of sensor noise [57]. Particle Swarm Optimization (PSO) typically converges 40% to 60% faster than classic Genetic Algorithms on these continuous multi-variable problems [59, 60]. Fast neural surrogates trained on 50 to 200 FE runs evaluate performance functions in milliseconds, shaving optimization runtimes by 95% [14, 59]. Bridge structures also see ambient temperature swings that shift natural frequencies by 5% to 15%—often eclipsing the subtle 1% to 3% frequency drops caused by local shear connector failure [61]. Simoen et al. combined Bayesian updating with Principal Component Analysis (PCA) to separate thermal variations from real mechanical damage, preventing false alarms [61, 62]. Spatial material variations across wide deck slabs can be handled through Karhunen–Loève random expansions and Maximum Likelihood Estimation [63].

7. MACHINE LEARNING, PINNS, AND DIGITAL TWIN PARADIGMS

7.1 Physics-Informed Neural Networks (PINNs)

Standard deep learning models evaluate inputs rapidly, but as pure black-box regressors, they can easily violate equilibrium or energy conservation when extrapolating beyond their training boundaries [13, 64]. Physics-Informed Neural Networks (PINNs) address this by embedding the governing differential equations directly into the network's loss function via automatic differentiation:

$$L_{PINN}(\theta) = w_d L_{data}(\theta) + w_p L_{PDE}(\theta) + w_b L_{BC}(\theta) + w_i L_{int}(\theta) \quad (21)$$

Here, L_{PDE} penalizes violations of partial interaction equilibrium ($E_c A_c (d^2 u_c / dx^2) - q_s(s) = 0$), L_{BC} enforces physical boundary conditions, and L_{int} governs interfacial traction-separation laws [65]. By forcing outputs to respect mechanical laws, PINNs generate reliable predictions even from sparse physical sensor measurements. Huang et al. demonstrated that PINNs can tackle inverse identification problems directly, extracting bond-slip backbones ($\tau(s)$) and fracture release rates (G_f) from sparse strain gauges without running full FE meshes [20, 65].

7.2 Machine Learning Surrogates for Structural Reliability

Ensemble models like XGBoost, Random Forests, and deep MLPs trained on validated FE databases predict ultimate shear capacities (Q_u) and failure modes with strong accuracy ($R^2 > 0.99$, $RMSE < 4.0$ kN) [14, 66].

These surrogates help resolve the massive computational bottleneck in structural reliability calculations [66]. Evaluating failure probabilities ($P_f = P(g(X) \leq 0)$) through classical Monte Carlo sampling requires 10^5 to 10^7 runs of the

limit state function $g(X)$. Running that many non-linear 3D FE iterations would demand months on a high-performance cluster. A neural surrogate evaluates each configuration in microseconds, running 10^6 stochastic samples in seconds while preserving non-linear tail distributions.

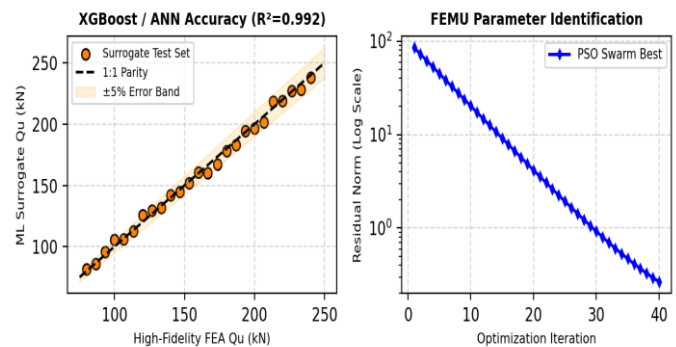


Fig -3: Advanced computational paradigms: (a) XGBoost / ANN surrogate accuracy against high-fidelity FEA predictions ($R^2 = 0.992$); (b) Finite Element Model Updating (FEMU) residual convergence using Particle Swarm Optimization (PSO).

7.3 Operational Real-Time Digital Twins

An operational Digital Twin links a physical bridge to an updating numerical model, tracking structural health over its design life [15, 67, 68]:

- Tier 1 (Sensing Array): Fiber Bragg Grating (FBG) strain sensors, wireless triaxial accelerometers, LVDTs, and thermocouple strings collect continuous field data.
- Tier 2 (Edge Processing Gateway): On-site microprocessors clean incoming signals, filter spikes, compress telemetry, and push clean features over MQTT or REST protocols.
- Tier 3 (Simulation Core): High-dimensional FE matrices cannot run at edge speeds. System matrices are projected onto reduced subspace bases using Proper Orthogonal Decomposition (POD) or modal truncation. These Reduced-Order Models (ROMs) evaluate structural responses in sub-milliseconds while capturing over 99% of dynamic system energy [15]. Paired with Unscented Kalman Filtering (UKF), the core tracks connector stiffness degradation ($K_{s,eff}$) on the fly.
- Tier 4 (Asset Dashboard): Updated damage indicators feed into Paris–Erdogan crack growth models ($da/dN = C(\Delta K)^m$) to estimate remaining service life (RSL) and push maintenance alerts before connectors fail under traffic [67, 68].

Table -3: Comparison of 3D FEA, black-box machine learning, and hybrid digital twins.

Metric	Traditional 3D FEA	Pure Machine Learning	Hybrid Digital Twins
Governing Basis	Continuum mechanics & CDP damage	Empirical pattern matching	PINNs + ROMs + Kalman filtering
Execution Time	Hours to days per simulation	Sub-millisecond inference	Sub-millisecond edge or cloud runs
Data Requirements	None (purely physics-based)	Large training sets ($> 10^3$)	Works with sparse, physics-guided data
Extrapolation	High; constrained by physical laws	Poor; yields unphysical trends	High; preserves balance of forces
SHM Integration	Unusable in real time (solver delay)	Fast, but lacks physical causality	Continuous streaming & live RSL updates

8. CURRENT RESEARCH GAPS

Even with recent strides across modeling and sensor technology, several key issues remain open:

- **Bridging Micro-Cracking to Macro-Scale Twins:** Most current digital twins track macroscopic bridge dynamics (like modal frequencies) and attempt to infer localized damage (like a fatigue crack at a single stud weld). Developing practical homogenization techniques that bridge micro-scale fatigue fracture with macro-scale reduced-order models remains an active challenge.
- **Coupled Multi-Hazard Damage:** Most experimental and numerical campaigns examine degradation mechanisms in isolation looking at creep, fatigue, or elevated temperature independently. In the field, these hazards hit simultaneously: traffic cycles trigger concrete micro-cracking, which accelerates moisture entry and shrinkage, while daily temperature cycles widen fatigue stress ranges. We need coupled constitutive models that handle these interactions together.
- **Updating Design Codes for Modern Hardware:** Mainstream design codes (Eurocode 4, AASHTO, AISC) are still oriented around standard welded studs in normal-weight concrete. Explicit design equations, reduction factors (k_t, k_l), and group reduction formulas (η_{group}) tailored for demountable bolts and ultra-dense UHPC matrices need to be formalized in upcoming code revisions.
- **Handling Kinematic Discontinuities in PINNs:** Current PINN formulations struggle when confronted with sharp kinematic jumps, such as clearance slip plateaus, friction transitions, and discrete interfacial debonding.

Formulating Physics-Informed DeepONets and Fourier Neural Operators that cleanly process non-smooth interface jumps without numerical instability is an important next step.

- **Sensor Reliability and Long-Term Drift:** Operating digital twins on outdoor highway infrastructure means dealing with weather extremes, dropped wireless packets, and sensor drift over time. Developing robust Bayesian routines that can identify and correct sensor drift without misinterpreting it as structural damage is essential for long-term deployments.

9. CONCLUSIONS

This review brings together the current state of knowledge on steel-concrete composite interfaces, covering physical experimentation, advanced FE modeling, alternative connector hardware, and emerging digital twin workflows. Key takeaways:

- Standard push-out tests (Eurocode 4 Annex B) establish essential baselines, but they introduce frame prying and friction artifacts that require two-tier numerical adjustments when scaling up to full beams.
- Robust non-linear FE simulations depend on specialized interface formulations (CZM, discrete neural springs, distributed fiber beams, or mixed elements) to prevent shear locking, alongside Bazant's Crack Band regularization to avoid mesh-dependent concrete softening.
- Realistic 3D modeling highlights multi-axial failure modes like pry-out cones, deck splitting, and front-row stud overload in dense clusters that simple 2D assumptions overlook.
- Newer hardware choices address historic weak points: Perfobond Ribs bypass weld-toe fatigue, demountable bolted connections make structural reuse practical with minimal demolition waste, and UHPC matrices enable short, high-capacity connector layouts.
- Combining physics-informed AI with reduced-order digital twins helps bridge the gap between heavy FE solvers and high-speed sensor streams, opening the door to live condition tracking and proactive lifecycle maintenance.

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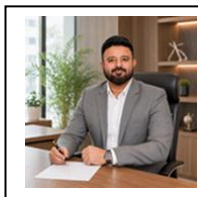
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BIOGRAPHIES



Arjun M. Chhaiya holds a Bachelor of Engineering in Civil Engineering and is currently pursuing his Master of Technology degree specializing in Structural Engineering. His research expertise encompasses non-linear 3D finite element analysis, multi-scale interface mechanics, steel-concrete composite infrastructure, and surrogate-assisted structural health monitoring.



Dr. Chirag Odedra is an Associate Professor in the Department of Civil Engineering at Dr. Subhash University, Junagadh, Gujarat, India. He has more than 11 years of teaching and academic experience in Structural Engineering. He holds a Ph.D. in Civil Engineering and his research interests include nano-concrete, nano-silica, nano-alumina, mechanical and durability performance of concrete, and Artificial Neural Network (ANN)-based strength prediction and modelling.