

Article

Track–Bridge Interaction and Low-Resistance Fastener Layout for a 4×40 m Continuous Rigid-Frame Bridge on the Nan-zhu-Zhong Intercity Railway

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Abstract

Understanding the non-linear dynamic interaction between tracks and bridge structures is essential for maintaining the safety of continuous rigid-frame bridges. To accurately capture these beam–rail interactions, this study develops a detailed 3D finite element model based on the principle of stationary total potential energy. This framework fully integrates the track, main girders, and piers into a single system. Based on a 4×40 m continuous rigid-frame viaduct in an urban transit network, the numerical model accounts for the bilinear mechanical behavior of the rail fasteners. The study compares the transmission of longitudinal forces along the continuously welded rail (CWR) under two fastening layouts. The baseline case uses uniform constant-resistance fasteners across the entire bridge, while the optimized scheme places small-resistance fasteners at the final 20% of each structural segment. Analysis shows that placing low-resistance fasteners in the high-displacement areas near the girder ends creates an effective longitudinal “release zone.” This design effectively interrupts the buildup of longitudinal forces, resulting in a much smoother force distribution along the rails. Quantitative results indicate that this optimized fastener layout has only a minor effect on structural deflection and braking-induced rail stresses, keeping deviations below 11%. At the same time, it significantly reduces the peak expansion stress and broken-rail stress by 43.4% and 22.2%, respectively. By shifting the stress regulation philosophy from “rigid resistance” to dynamic “force channeling,” these findings demonstrate that local low-resistance fastener deployment improves the overall mechanical compatibility of the track–bridge infrastructure. Ultimately, this work offers a solid theoretical basis for the design and maintenance of CWR systems on long-span rigid-frame bridges.



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Keywords: urban rapid rail transit; continuous rigid-frame bridge; beam–rail interaction; small-resistance fasteners; non-linear mechanism

1. Introduction

Deploying continuous welded rails (CWR) across elevated rail transit systems eliminates joint-related impacts, significantly improving dynamic stability and lowering maintenance demands [1,2]. However, the structural integration of CWR introduces severe track–bridge interaction (TBI) phenomena. The mutual constraint between the rails and the supporting deck, driven by ambient thermal fluctuations, train braking, and girder bending, induces excessive longitudinal additional forces [3,4]. These localized stress accumulations

pose a direct threat to track integrity, increasing the risk of track buckling, rail fracture, or system fatigue [5,6].

To address these multi-factor coupled mechanical challenges, the engineering community has explored various mitigation strategies. Most numerical and experimental investigations prioritize evaluating expansion joints, bearing friction, and customized fastener systems [7,8]. Specifically, adjusting the longitudinal sliding resistance of the fasteners provides a cost-effective method to redistribute stress. Previous studies indicate that deploying small-resistance fasteners can effectively release the locked-in braking and expansion forces, provided that the resistance profile is carefully calibrated to avoid excessive rail gap expansions and track instability [9–11]. Recent models have also incorporated advanced boundary conditions, such as unsupported track defects and seismic excitation, to assess the dynamic reliability of these systems [12–15].

Despite these advancements, a review of the recent literature reveals a disproportionate focus on simply-supported girders [4,16] and ultra-long-span suspension structures [17]. Continuous rigid-frame bridges—which are extensively utilized in modern urban transit for their structural efficiency—remain insufficiently investigated. In these systems, the monolithic connection between the piers and the girder severely amplifies the coupling between pier-top deformation and deck displacement. This inherent geometric stiffness makes the spatial optimization of fastener resistance much more complex than in traditional configurations [18,19].

Aiming to fill this specific gap, this research evaluates the longitudinal force transmission on a four-unit 4×40 m continuous rigid-frame viaduct. A comprehensive finite element approach is employed to compare two fastener distribution topologies: a conventional uniform resistance layout versus a targeted configuration where small-resistance fasteners are applied only to the outer 20% of each unit's span. By tracing the force evolution of individual fasteners and mapping the overall CWR stress profile, this study clarifies the stress-release mechanisms specific to continuous rigid-frame architectures. The quantitative results will support the optimized design and lifecycle maintenance of CWR systems in complex railway infrastructures [20].

2. Engineering Overview

Serving as the primary structural model, this research evaluates a 4×40 m continuous rigid-frame bridge comprising four interconnected units. As delineated in Figure 1, the primary girder adopts a single-box, single-cell concrete topology, maintaining a characteristic structural height of 2.5 m, coupled with top and bottom plate widths of 11.20 m and 3.00 m, respectively. Supporting this superstructure, the substructure integrates transversely waisted plate-type piers, visually mapped in Figure 1. Measured along the longitudinal bridge axis, the central and peripheral piers are assigned discrete thicknesses of 1.6 m and 0.8 m, respectively. Geomechanically, this structural framework is securely anchored by cast-in-place cylindrical concrete bored piles, each featuring an embedment length of 60 m and a diameter of 1.5 m. To accurately define the subsurface boundary conditions, the subgrade reaction coefficient is designated at 1000 kPa/m^2 , while the overarching structural design load is conservatively parameterized at 150 kN/m .

Within the numerical framework, the track domain is mathematically idealized as a linear dual-track configuration, strictly subjected to the ZK standard live loading protocols. For the kinematic simulation, the vehicular subsystem deploys a CRH6 Electric Multiple Unit (EMU) assembled in a standard eight-car formation. With individual carriage modules measuring 24.5 m, the cumulative length of the traversing trainset reaches 201.4 m. Operationally, this specified EMU operates under a localized axle load of 17 t and is dynamically assessed at a baseline design velocity of 200 km/h.

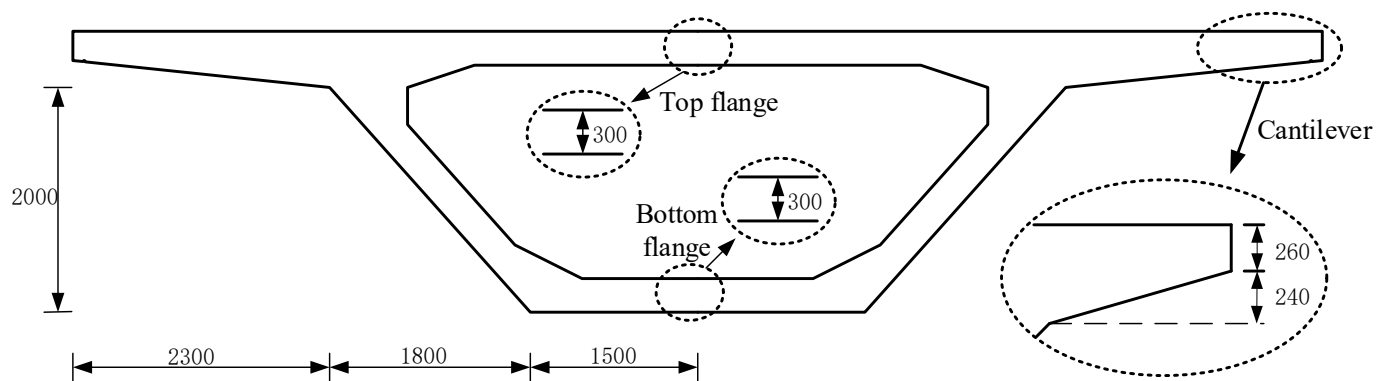


Figure 1. Cross-section of the main girder (Unit: mm).

3. Track–Bridge–Pier Calculation Model and Parameter Selection

3.1. Train Dynamics Model

The individual train carriage is conceptualized as a multi-body dynamic assembly, fundamentally constructed from a primary car body, a pair of bogies, and four independent wheelsets. In an unconstrained spatial domain, each of these structural masses intrinsically exhibits six degrees of freedom (DOFs): three orthogonal translations (surge, sway, and heave) alongside three corresponding rotations (roll, pitch, and yaw). To optimize computational expenditure while preserving the fidelity of the dynamic simulation, the spatial vibration framework integrates several core simplifications. Primarily, all principal vehicular components are treated as strictly non-deformable masses. Longitudinal dynamic interactions are deliberately excluded under the premise that the train traverses the infrastructure at a uniform velocity. Furthermore, the kinematic behavior of the entire multi-body architecture is assumed to conform strictly to small-perturbation theory. Regarding the suspension mechanics, the stiffness and damping attributes are mathematically idealized using linear elastic and viscous models, respectively. Concurrently, the wheel–rail creep dynamics are approximated via a linear formulation, intentionally bypassing higher-order non-linearities. Finally, governed by the rigid-contact hypothesis, the vertical displacements of the wheelsets are presumed to be perfectly synchronous with the underlying rail geometry.

Dictated by these defined boundary conditions, the kinematic complexity of a solitary four-axle carriage is mathematically condensed to 23 DOFs. By eliminating the longitudinal translation, the primary car body and both bogies are allocated five DOFs each. Meanwhile, the kinematic constraints restrict the four wheelsets to merely two DOFs apiece: lateral translation (sway) and rotation about the vertical axis (yaw).

3.2. Spatial Finite Element Model of the Track–Bridge–Pier System

Within the numerical architecture, the rail profiles are discretized via continuous beam elements, whereas the longitudinal track resistances are mathematically represented by non-linear spring components, designated as K_1 and K_2 . The initial spring element, K_1 , quantifies the fastening constraint acting between the rail and the underlying track slab. To accurately map the structural stiffness bifurcation that occurs prior to and following fastener slippage, the mechanical signature of this interface is governed by a bilinear constitutive relationship. Conversely, the K_2 parameter defines the longitudinal friction boundary separating the track slab from the bridge deck. To replicate its continuous yielding mechanics under sustained loads, this interface is mathematically idealized through a perfectly elastoplastic material formulation.

Expanding to the primary load-bearing infrastructure, three-dimensional spatial elements are deployed to render the main girder, the supporting piers, and the deep pile foundations. To rigorously capture the complex geomechanical boundary conditions, the established m-method is integrated to resolve the dynamic soil–pile coupling interactions. Synthesizing these localized constituent models yields a high-fidelity, three-dimensional finite element (FE) framework dedicated entirely to evaluating the multi-body track–bridge–pier interplay. The complete topological assembly of this continuous rigid-frame viaduct system is visually documented in Figure 2.

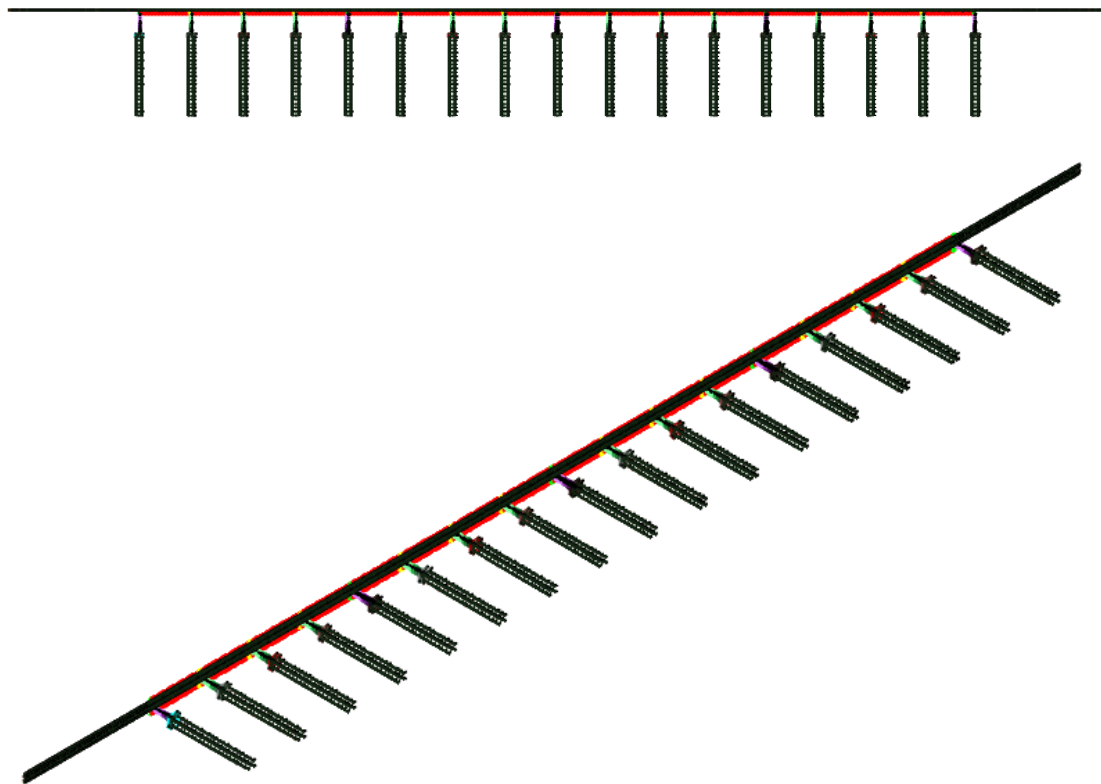


Figure 2. Spatial finite element model of longitudinal track–bridge interaction for the continuous rigid-frame bridge.

To rigorously capture the non-linear contact mechanics inherent to the track–bridge interaction (TBI), the governing equations of the coupled framework are derived anchoring on the stationary total potential energy postulate of elastodynamics. Designating the global potential energy functional as Π , the fundamental equilibrium state is mathematically articulated via the variational principle:

$$\delta \Pi = \delta (U + N - M) = 0 \quad (1)$$

Herein, the variables U and V denote the internal strain energy and the potential energy attributed to inertial forces, respectively, whereas W quantifies the external work exerted upon the multi-body system. Within this integrated track–bridge matrix, the overarching energy functional aggregates the elastic deformation energies stored across the rail profiles, principal girders, and supporting piers, alongside the localized potential energy generated by the interlayer structural constraints.

To precisely accommodate the intrinsic non-linearities defining the longitudinal restraint within the continuously welded rail (CWR) assembly, the longitudinal fastener resistance, designated as $r(x)$, is mathematically formulated applying a bilinear constitutive law:

$$r(x) = \begin{cases} k_0 u(x), & |u(x)| \leq u_a \\ r_{\max} \cdot \text{sgn}(u(x)), & |u(x)| > u_a \end{cases} \quad (2)$$

where:

$u(x)$ is the relative longitudinal displacement between the rail and the main girder;

k_0 is the initial elastic stiffness of the fastener;

u_{cr} is the critical slip displacement;

$r_{\max}$ is the maximum longitudinal resistance of the fastener (the limit for constant or low resistance).

By employing the direct stiffness technique to synthesize the global mass, damping, and stiffness matrices of the structural assembly, the comprehensive spatial coupled governing matrix equation for the unified track–bridge–pier system is formulated as follows:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K(u)]\{u\} = \{F(t)\} \quad (3)$$

Herein, $[K(u)]$ represents the global system stiffness matrix embedding the non-linear interface contact stiffness, a parameter that mathematically characterizes the displacement-dependent non-linear behavior inherent to the localized fastener resistance.

The numerical framework captures track–bridge interaction by configuring thermal and braking conditions based on established railway codes. Rather than simulating complex thermal fields, the model subjects the main girder elements to a uniform temperature shift (ΔT). As the bridge superstructure undergoes thermal expansion or shrinkage, it shifts longitudinally relative to the continuous welded rail (CWR). This relative movement directly engages the non-linear fastener springs, thereby generating thermal additional stresses within the rail. Regarding train deceleration, the model applies a uniform longitudinal line load directly to the rail nodes. The loaded segment length exactly matches the actual trainset. Once applied, these braking forces propagate downwards—from the rails through the sliding fasteners, reaching the deck, bearings, and supporting piers. This specific load path setup ensures a realistic assessment of braking-induced structural responses.

3.3. Parameter Selection and Research Schemes

The physical track topology evaluated within this numerical framework utilizes standard 60 kg/m continuous welded rail (CWR) segments fabricated from high-performance U75VG grade steel. To establish a rigorous analytical foundation, both the critical computational boundary parameters and the strategically designed comparative research matrices governing the simulation model are comprehensively cataloged in Table 1.

Table 1. Detailed parameters and research schemes.

Scheme	Material		Temperature				
	Yield Strength	Safety Factor	Allowable Rail Stress	Maximum Rail Temperature	Minimum Rail Temperature	Intermediate Rail Temperature	Design Stress-free Rail Temperature
	472 MPa	1.3	363 Mpa	59.3 °C	−1.3 °C	29.0 °C	30 ± 5 °C
Scheme 1	Uniform installation of constant-resistance fasteners across the entire bridge.						
Scheme 2	Installation of low-resistance fasteners within the 20% length at both ends of each unit, with constant-resistance fasteners elsewhere.						

Note to Table 1: For the ballastless track, WJ-8 constant-resistance fasteners and WJ-8 low-resistance fasteners are adopted.

4. Coupled Dynamics Model and Characteristic Analysis of the Track–Bridge–Pier System

4.1. Regulation Mechanism of Expansion Force Under Thermal Loads

Subjected to a uniform thermal differential scenario of $\pm 30\text{ }^{\circ}\text{C}$, the primary superstructure naturally undergoes unconstrained longitudinal thermomechanical expansion and contraction. Within the structural paradigm of Scheme 2, strategically embedding a “low-resistance release zone” across the high relative displacement sectors near the girder extremities profoundly attenuates the kinematic coupling between the supporting bridge deck and the overlying rail infrastructure. As a result of this localized friction mitigation, the numerical results are explicitly summarized in Table 2.

Table 2. Calculated results and comparison of rail expansion forces.

Scheme	Maximum Rail Expansion Force (kN)	Maximum Rail Expansion Stress (MPa)	Maximum Longitudinal Displacement of Rail (mm)	Maximum Rail–Girder Relative Displacement (mm)
Scheme 1	865.2	111.7	14.7	26.5
Scheme 2	497.3	63.2	10.5	27.7

Mechanistically, by intentionally downgrading the tangential boundary stiffness, denoted as $k(x)$, near the structural extremities of the girder, the optimized configuration (Scheme 2) dramatically homogenizes the longitudinal expansion force distribution profile, as graphically evidenced in Figure 3. This strategic topological adjustment inherently suppresses the development of acute stress concentrations typically localized within the negative bending moment regions overlying the pier supports. To rigorously unpack the physical mechanics governing this phenomenon, a differential segment of the continuous rail is isolated for a static equilibrium evaluation. The supplementary longitudinal axial force propagating through the rail, designated as $N_r(x)$, is fundamentally generated by the spatial accumulation of the interfacial fastener resistance, $r(x)$. This distributed friction is mechanically activated by the relative kinematic slip occurring between the rail and the subjacent girder. Consequently, the fundamental differential equilibrium governing this rail segment is formulated as:

$$\frac{dN_r(x)}{dx} = -r[u(x)] \quad (4)$$

Executing a spatial integration over this governing differential equation yields the absolute longitudinal force at an arbitrary cross-section, mathematically defined as the definite integral of the localized fastener restraints distributed along the track alignment:

$$N_r(x) = \int_0^x -r[u(\xi)]d\xi \quad (5)$$

Within the kinematic framework of a continuous rigid-frame viaduct, the thermally induced longitudinal translation of the primary girder, denoted as $u_b(x)$, inherently adopts a linear spatial profile, culminating in peak displacements strictly at the structural termini. Under the baseline paradigm deploying uniform constant-resistance constraints (Scheme 1), these terminal zones of exaggerated displacement prematurely trigger the plastic yield state of the fasteners, locking in the ultimate resistance threshold, r_{max} . Mechanically, this saturated friction forces the resultant axial rail load, N_r , to escalate precipitously along a severe gradient, precipitating aggressive and localized stress accumulations. Conversely, the optimized topological framework explicitly allocates low-resistance fasteners across these highly sensitive, elevated-displacement sectors—specifically targeting the terminal

20% of the girder span. This deliberate structural modification conceptualizes a functional “stress-release zone.” Within this defined spatial domain, the ultimate yield resistance of the sliding interfaces is deliberately suppressed from the baseline r_{max} to a diminished threshold, r'_{max} ($r'_{max} < r_{max}$). Grounded in the fundamental theorems of integral calculus, this localized friction attenuation mathematically degrades the growth velocity of the axial force accumulation function, yielding a quantifiable force mitigation gradient:

$$\Delta N_r = \int_{L_{end}} (r_{max} - r'_{max}) dx \quad (6)$$

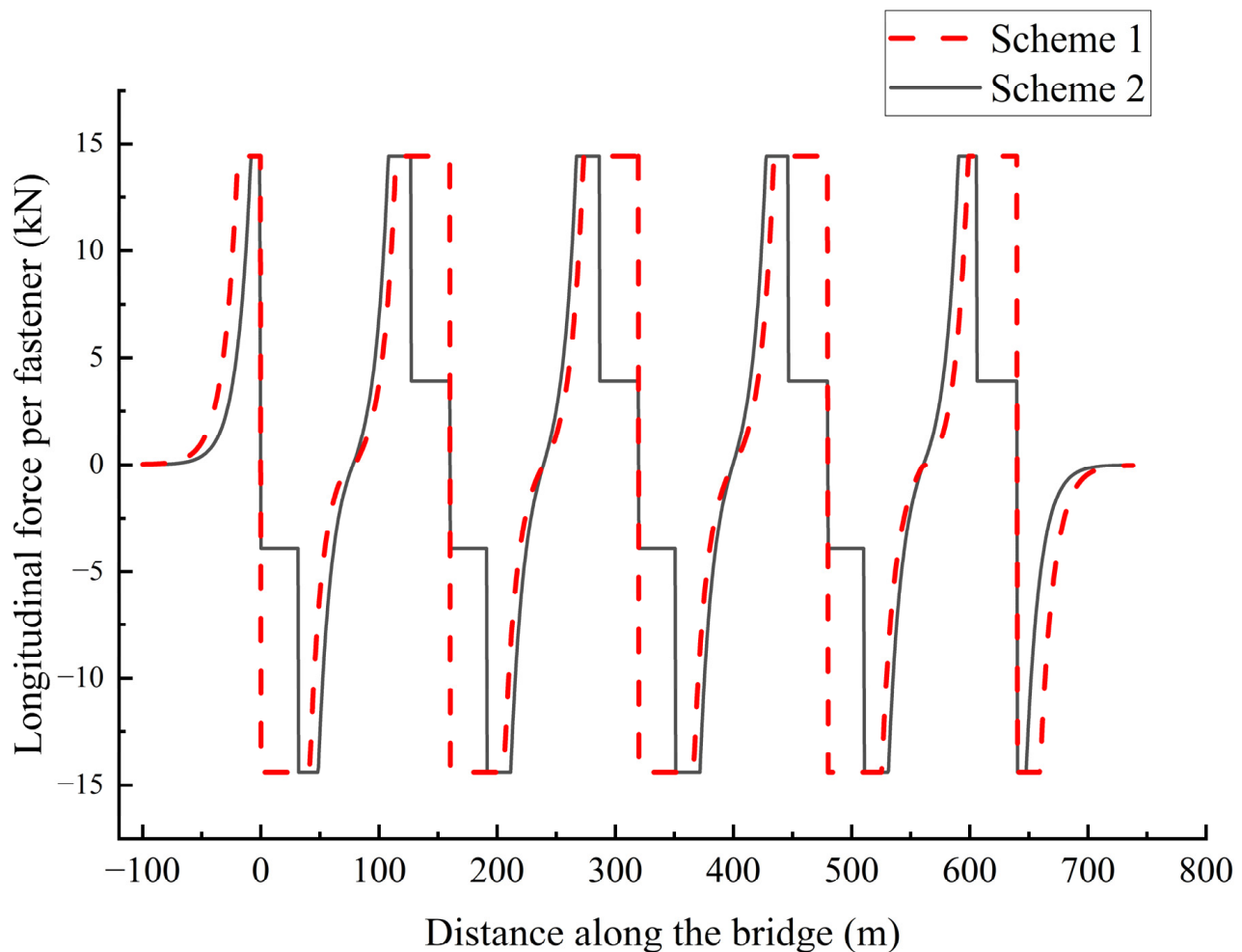


Figure 3. Longitudinal force distribution of individual fasteners on a single rail under the expansion force load case.

This integral explicitly formulates the substantive reduction in longitudinal rail forces achieved through the optimized configuration. By systematically severing the resistance accumulation trajectory within the heavily deformed girder-end regions, the coupled matrix safely dissipates the excessive strain energy previously trapped within the rail infrastructure. On a macroscopic scale, this energy dissipation translates into an exceptional 43.4% attenuation in the peak expansion stress, concurrently yielding a profoundly smoothed longitudinal force gradient. Analyzed through a thermodynamic and structural energy lens, this nuanced regulation philosophy—prioritizing dynamic “alleviation” over brute “rigid resistance”—unequivocally validates the superior mechanical efficacy and resilience of the refined layout scheme.

4.2. System Dynamic Responses Under Train Loads: Bending and Braking

Transitioning to the operational load configurations, the mechanical behaviors associated with train-induced vertical flexure and longitudinal braking mechanics are fundamentally governed by transient vehicular live loads. A rigorous numerical evaluation of these dynamic boundary conditions yields the following quantitative insights into the structural response framework, as summarized in Tables 3 and 4, Figures 4 and 5:

Table 3. Calculated results and comparison of rail bending forces.

Scheme	Maximum Rail Bending Force (kN)	Maximum Rail Axial Stress (MPa)	Maximum Combined Rail Stress (MPa)	Maximum Lateral Force per Fastener on a Single Rail (kN)
Scheme 1	30.3	3.9	107.9	29.7
Scheme 2	50.3	4.1	115.8	29.7

Table 4. Calculated results and comparison of rail braking forces.

Scheme	Maximum Rail Braking Force (kN)	Maximum Rail Braking Stress (MPa)	Maximum Longitudinal Rail–Girder Relative Displacement (mm)
Scheme 1	205.4	26.5	1.4
Scheme 2	184.8	23.8	1.4

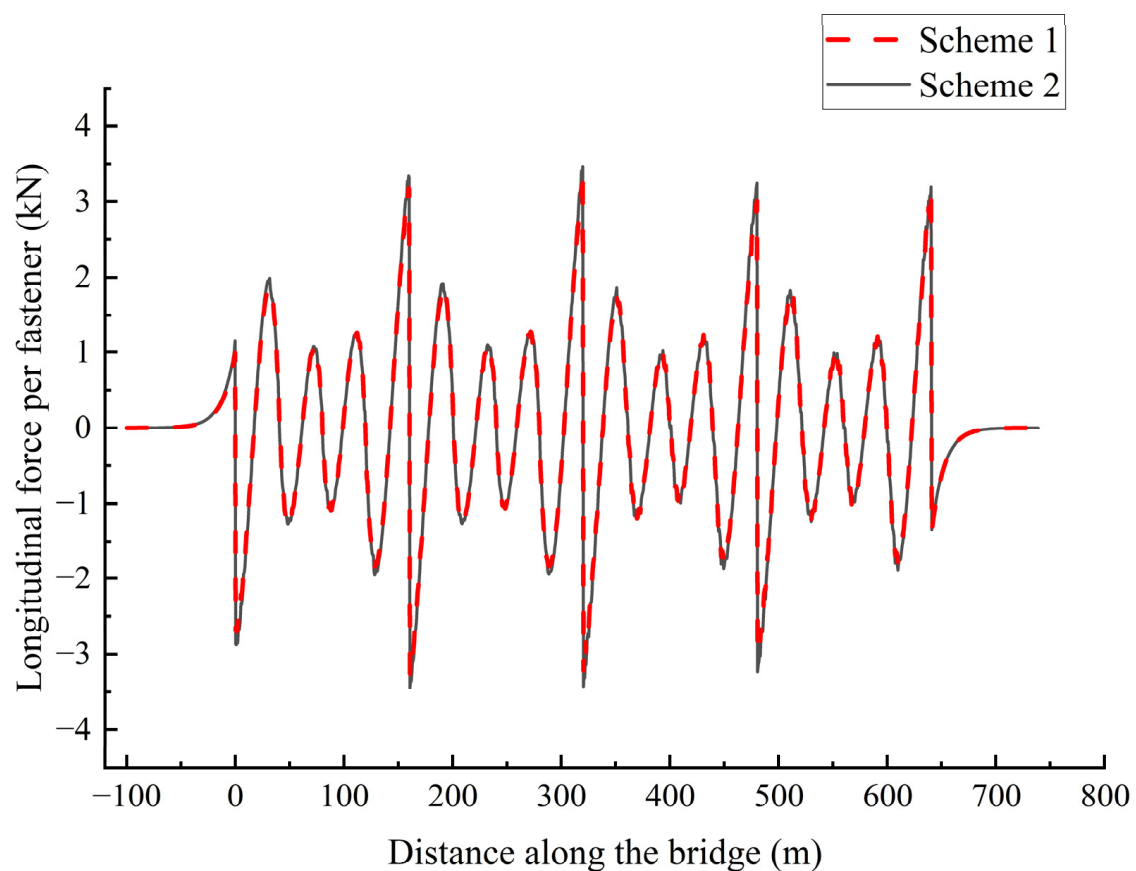


Figure 4. Longitudinal force distribution of individual fasteners on a single rail under the bending force load case.

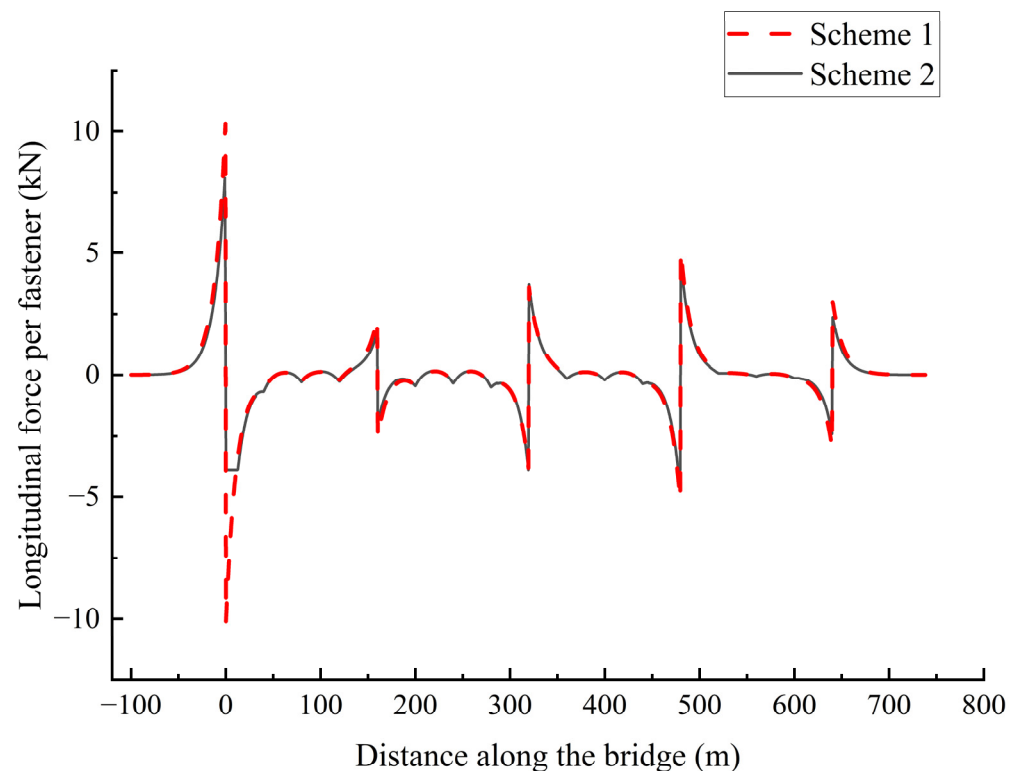


Figure 5. Longitudinal force distribution of individual fasteners on a single rail under the braking force load case.

Evaluation of the operational load scenarios indicates that the stress differences between the two fastening setups remain highly manageable. Specifically, the braking and combined rail stresses exhibit relative deviations of roughly 10.2% and 7.3%, respectively. This acceptable level of variance confirms that the optimized arrangement reduces severe thermal stress peaks without sacrificing overall structural stiffness or the stability of moving trains.

Furthermore, the data suggests that placing low-resistance fasteners in targeted areas does not seriously compromise the track's capacity to withstand sudden, intense dynamic forces. Most importantly, the modified track–bridge system maintains excellent longitudinal restraint. In both tested layouts, the maximum braking-induced slip between the rail and girder is kept strictly under 1.4 mm.

4.3. Extreme Scenarios: Broken-Rail Safety Analysis

Operating under the extreme limit state hypothesis wherein a structural rail rupture initiates exactly at the critical cross-section subjected to peak superimposed tensile stresses, the ensuing sudden dissipation and spatial propagation of the post-fracture internal forces inherently precipitate a profoundly non-linear mechanical cascade, as summarized in Table 5, Figure 6.

Table 5. Calculated results and comparison of broken-rail forces.

Scheme	Maximum Rail Tensile Force (kN)	Maximum Broken-Rail Stress (MPa)	Maximum Broken-Rail Gap (mm)
Scheme 1	1523.6	195.1	38.4
Scheme 2	1178.6	151.7	38.4

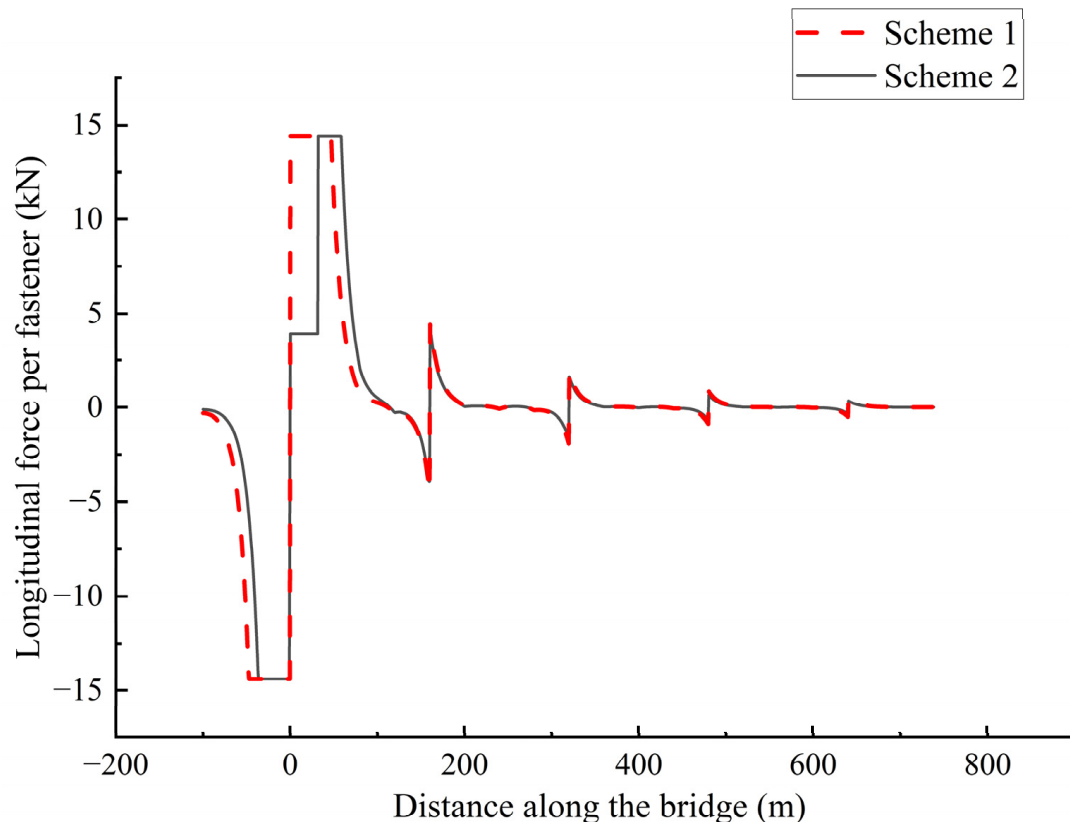


Figure 6. Longitudinal force distribution of individual fasteners on a single rail under the broken-rail force load case.

Quantitative extraction of the post-fracture mechanics data indicates that the terminal broken-rail gap under the optimized configuration stabilizes at 38.4 mm, demonstrating strict kinematic equivalence with the baseline layout. Despite this macroscopic geometric consistency, the peak post-fracture rail stress within the refined matrix undergoes a precipitous attenuation of 22.2% (decreasing from 195.1 MPa to 151.7 MPa). This stark parametric contrast unequivocally confirms that upon catastrophic rail rupture, the modified topological strategy expertly mobilizes the plastic yield and sequential slip of the adjacent low-resistance fasteners. By efficiently dissipating the sudden surge of transient fracture energy, this localized sliding mechanism endows the global track–bridge coupled system with profoundly enhanced “fail-safe” structural resilience.

4.4. Track Strength Verification and Safety Evaluation

In accordance with the Code for Design of Railway Continuous Welded Rail (TB 10015-2012) [21], a strength envelope verification was conducted for both proposed schemes, as summarized in Figures 7 and 8.

A comprehensive assessment of the verification metrics confirms that the aggregate stress margin inherent to the optimized configuration decisively eclipses the baseline capacity. Crucially, when the infrastructure is subjected to severe operational perturbations—specifically unpredictable deviations such as an indeterminate zero-stress thermal baseline or localized fastener degradation—this refined topological strategy guarantees a substantially expanded safety envelope. Consequently, the localized friction mitigation system not only satisfies static design criteria but also endows the global track network with formidable structural redundancy against unpredictable in-service deterioration.

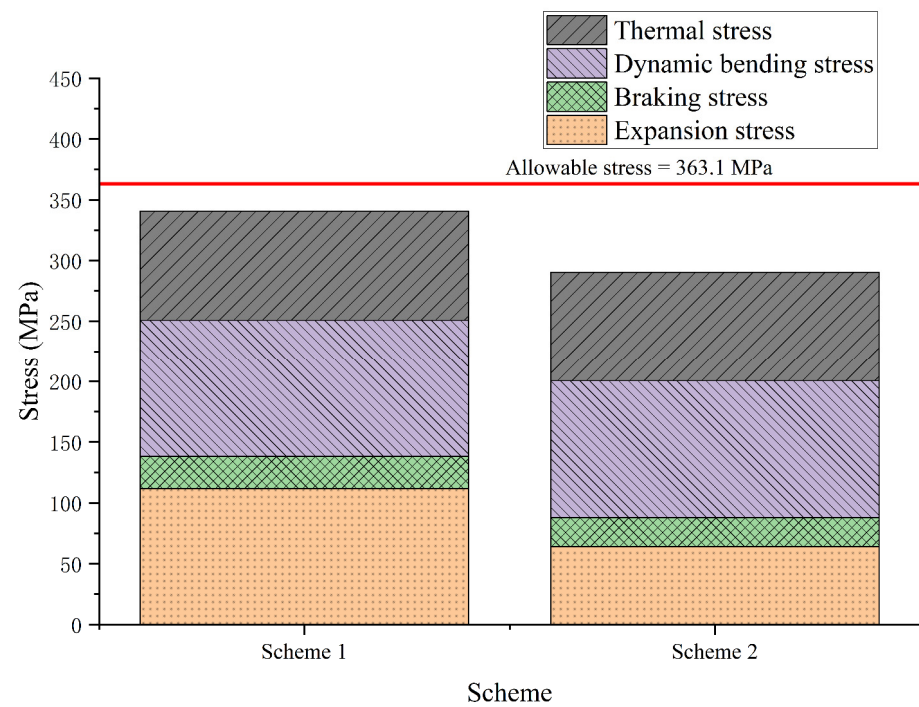


Figure 7. Comparison of rail-strength verification for continuous welded rail (CWR) on bridges.

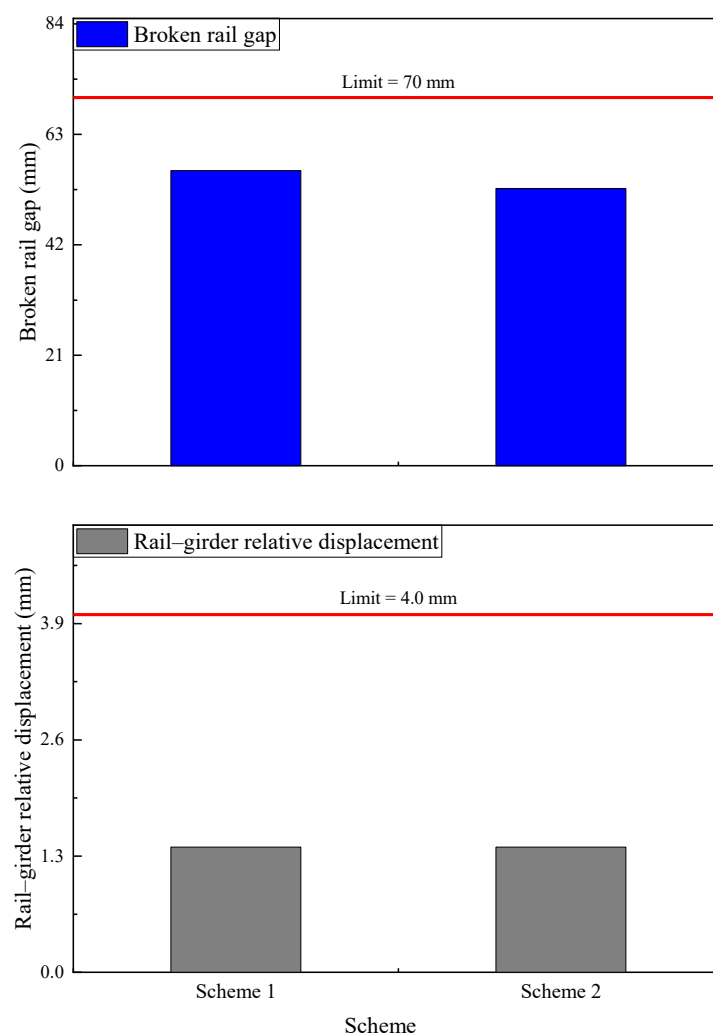


Figure 8. Comparison of broken-rail gap and relative displacement results.

5. Conclusions

Synthesizing the principal findings regarding the global rail stress response, the implementation of the optimized configuration—specifically deploying low-resistance fasteners exclusively across the terminal 20% of each structural unit—precipitates a profound attenuation in extreme load effects. Compared to the uniform constant-resistance baseline, this targeted topological adjustment slashes the peak rail expansion stress and broken-rail stress by 43.4% (decreasing from 111.7 MPa to 63.2 MPa) and 22.2% (decreasing from 195.1 MPa to 151.7 MPa), respectively. This quantifiable reduction unequivocally demonstrates a superior mechanical capacity for regulating severe longitudinal forces triggered by aggressive thermomechanical variations and catastrophic fracture events. Conversely, the structural discrepancies observed in vertical flexural and braking-induced stresses remain strictly marginal, validating that this localized friction mitigation exerts virtually no adverse kinematic influence on stress paradigms governed by dynamic train payloads or emergency decelerations.

Beyond macroscopic stress mitigation, evaluating the system from a localized micro-mechanical perspective reveals that this terminal low-resistance deployment expertly optimizes the spatial force distribution along individual rail strands. By fundamentally flattening the longitudinal force transmission gradient and suppressing aggressive peak load accumulations, the configuration systematically enhances the global mechanical coordination of the sliding interface. Ultimately, this homogenized load-sharing mechanism proves highly conducive to extending the lifecycle serviceability of the track infrastructure, profoundly delaying the onset of residual plastic deformations and localized fatigue degradation.

Culminating in the ultimate strength and stability verification of the continuous welded rail (CWR) assembly, analytical comparisons reveal a critical vulnerability within the uniform constant-resistance paradigm. While satisfying prevailing statutory codes, the baseline scheme operates precariously close to permissible structural limits, leaving a highly constrained safety margin that is exceptionally susceptible to failure under adverse spatial alignments, such as high-curvature track segments. In stark contrast, the optimized low-resistance framework reserves an expansive structural safety envelope. This profound surplus capacity fundamentally accommodates the inevitable uncertainties arising from parametric perturbations, numerical model simplifications, and stochastic fluctuations in real-world operating environments, thereby substantially amplifying the environmental adaptability and global reliability of the coupled network. Beyond these theoretical structural merits, this targeted optimization scheme exhibits exceptional practical feasibility and cost-effectiveness. From an installation perspective, it necessitates only the selective substitution of sliding fasteners within designated terminal zones during the track-laying phase, thereby circumventing any capital-intensive modifications to the primary track-slab or bridge superstructure. Consequently, for the advanced engineering of long-span continuous viaducts within regional rapid transit architectures, it is strongly advocated that the strategic 20% terminal deployment of low-resistance sliding fasteners be prioritized as a foundational structural directive, warranting formal integration into future technical design specifications and engineering manuals.

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