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Numerical Assessment of Track-Bridge Interaction in Continuous Welded Rail on Concrete Bridges Subjected to Extreme Thermal Loading

[1]Okorie Austine Uche, [2]Danjuma Dauda Abubakar, [3]Efiok Etim Nyah

[1]Professor of Civil Engineering, [2]Master of Engineering Student, [3]Lecturer I
[1,2]Department of Civil Engineering, Nigerian Defence Academy, Kaduna, Nigeria, [3]Department of Civil Engineering, University of Cross River State, Calabar, Nigeria

Abstract : Continuous welded rail (CWR) on long concrete viaducts in hot climates experiences pronounced track-bridge interaction (TBI) when thermal expansion is restrained. This study examines a 791 m, 33-span precast prestressed concrete bridge on Nigeria's Kano-Maradi-Dutse standard-gauge corridor, where rail surface temperatures routinely reach 45 °C under intense solar exposure. A SAP2000 finite-element model, fully compliant with BS EN 1991-1-5, BS EN 1991-2 and UIC 774-3, incorporated non-linear bi-linear ballast and fastener resistance, sliding pot-bearing friction, extended approach-track sections, and the actual bridge geometry and bearing layout. Analyses combined uniform temperature changes (±20 °C per the design report and +45 °C for local extremes), braking (20 kN/m), traction (33 kN/m) and RU vertical loading (80 kN/m + 4 × 250 kN). Thermal loading proved dominant. At the rail temperature (20 °C), axial rail stresses remained below ±3 MPa. At 45 °C they rose to peak values of +20 MPa (tension) and -15 MPa (compression), still comfortably within the UIC 60 permissible additional stresses of +92 MPa (tension) and -72 MPa (compression). Superposition of braking increased peak compression by only 4-6 MPa. Relative displacements between deck segments reached a maximum of ±6.0 mm under braking at 45 °C (well below the 8 mm serviceability limit for 160 km/h lines), while vertical RU loading produced only 1.7-2.5 mm longitudinal deck movement. The results confirm that the existing CWR system satisfies all relevant serviceability criteria without additional expansion devices, provided the rail temperature is selected from local meteorological records. The work offers practical guidance for the design and assessment of CWR on concrete bridges in thermally demanding regions of sub-Saharan Africa.

***IndexTerms* - Continuous Welded Rail, Track-Bridge Interaction, Extreme Thermal Loading, Concrete Viaduct, Finite Element Model, Serviceability Assessment, hot-climate railway**

Abbreviations:
BS: British Standard
CWR: Continuous Welded Rail
EN: European Norm (Eurocode)
RU: Railway loading (equivalent to UIC 71)
SAP2000: Structural analysis software
TBI: Track-Bridge Interaction
UIC: International Union of Railways
UTM: Universal Transverse Mercator

1. Introduction

1.1 Background

Continuous welded rail (CWR) has emerged as the dominant track form in contemporary railway networks, prized for its smooth ride, lower maintenance needs, and improved efficiency over traditional jointed rail. By removing joints, CWR ensures seamless structural integrity, which enhances vehicle-track dynamics, accommodates faster speeds, and minimizes wear on components. These merits have fuelled its broad implementation in standard-gauge systems globally, including key developments in sub-Saharan Africa such as Nigeria's expanding corridors [1,2].

Yet, this continuity reshapes the rail's longitudinal response, especially on bridges. Unlike embankment-laid track, where ballast and sleepers can absorb some expansion, bridge-mounted rails encounter extra constraints from the bridge's rigidity, bearings, and deck layout. This builds up axial forces when thermal movements are impeded, positioning temperature as a key design driver in bridge-CWR setups [3]. In hot regions like northern Nigeria, where rail temperatures can climb well above ambient due to solar exposure, these stresses intensify [4].

The surge in standard-gauge infrastructure, exemplified by the Kano-Maradi-Dutse line, underscores the need to grasp track-bridge interplay amid harsh climates. Extended bridges with continuous spans and precast concrete elements heighten sensitivity to thermal loads, often overshadowing traffic effects in serviceability checks [5].

1.2 Track-Bridge Interaction as a Design-Critical Phenomenon

Track-bridge interaction (TBI) describes the linked mechanical behaviour between the track and bridge, driven by longitudinal ties. In CWR, thermal shifts induce rail strains resisted by ballast shear, sleeper-ballast friction, fastener rigidity, bearing friction, and bridge stiffness. This interplay dictates axial force and displacement patterns along the assembly [6].

When a bridge expands, shrinks, or bends under loads, the fixed rail has to tag along. In turn, rail expansion from heat pushes forces back into the bridge via the track. This two-way exchange leads to nonlinear behaviour, as resistances like ballast friction activate gradually with movement. Studies show braking and traction amplify stresses by layering onto thermal baselines, often peaking in combined cases [7].

Poorly managed TBI risks buckling from compression in heat, fractures from tension in cold, or degraded fasteners, bearings, and joints - threats amplified on long concrete bridges [8]. Thus, robust TBI evaluation is vital for meeting safety and service limits over the structure's life, spurring advanced numerical tools to model thermo-mechanical responses [1].

1.3 Motivation and Scope of the Study

This work is driven by the imperative to better comprehend CWR behaviour on concrete bridges under severe thermal loads, especially in hot climates where standards like Eurocodes and UIC guidelines are rooted in milder conditions and may fall short. These frameworks assume typical temperature swings, but Nigerian zones often see rail peaks of 40-45°C, heightening distress risks [4]. Recent data signal more frequent heat extremes across Nigeria's regions, straining rail assets [5].

The study delivers a numerical evaluation of TBI using a concrete bridge from the Kano-Maradi-Dutse corridor at coordinates N 1435143.6964, E 412851.8994 as a prototype. A SAP2000 finite element model integrates rail stresses, displacements, and loads from thermal gradients (±20°C uniform, per design report; up to 45°C extremes), braking (20 kN/m), traction (33 kN/m), and vertical traffic (RU loading: 80 kN/m uniform + 4×250 kN concentrated). Focus is on serviceability metrics like axial stresses and displacements against Eurocode limits.

Unlike optimization-centric research, the goal is to clarify key drivers, measure effects in severe heat, and check against service limits. Drawing on local climate information, real configurations, and standards, this study offers actionable insights for designing, evaluating, and sustaining CWR-bridge systems in challenging thermal settings.

2. Review of Design Standards and Engineering Practice

2.1 International Provisions for Track-Bridge Interaction

Design and assessment of continuous welded rail on bridges rest on a well-established body of international standards that combine general structural loading rules with railway-specific guidance on longitudinal interaction. In the European framework, which underpins much of the project design for the Kano-Maradi-Dutse corridor, EN 1991-2 defines the characteristic vertical, braking and traction actions from rail traffic, together with rules for their combination with thermal effects. Complementary provisions in EN 1991-1-5 set out the uniform and differential temperature components to be applied to the bridge deck [9,10].

These Eurocode clauses, however, do not address TBI as a self-contained problem. That role falls to the dedicated UIC recommendations, above all UIC 774-3 [6], which supplies simplified mechanical models, permissible additional rail stresses and relative displacement limits for serviceability verification. The document explicitly recognises that restrained thermal expansion constitutes the dominant source of longitudinal rail force on concrete bridges, where deck stiffness, bearing restraint and continuity amplify the interaction. Permissible rail stresses are therefore checked at the serviceability limit state, with emphasis on preventing buckling in compression and fracture in tension under the most adverse combinations of temperature change, braking/traction and bridge deformation.

Supporting UIC leaflets complete the framework: UIC 772-R [11] governs the selection and modelling of bearings, particularly the longitudinal friction characteristics of sliding pot bearings used on the case-study structure, while UIC 860-R [12] provides the mechanical properties of rail steel that enter the stiffness and strength calculations. When applied together, these documents encourage engineers to treat the track and bridge as an integrated system rather than isolated elements- an approach that has become standard practice for long concrete viaducts carrying CWR [1].

2.2 Limitations of Existing Standards under Extreme Thermal Conditions

The Eurocode-UIC suite remains robust for temperate climates, yet its calibration reveals clear boundaries when applied to corridors experiencing sustained high temperatures. Temperature ranges embedded in the models derive largely from European meteorological records, where rail temperatures rarely remain above 35-40 °C for prolonged periods. In contrast, site-specific data collected for the Kano region show rail surface temperatures frequently reaching 45 °C during the peak dry season, driven by intense solar radiation, low cloud cover and reduced ballast depth on bridge decks [13].

This discrepancy matters because thermal strain grows linearly with temperature while restraint forces like ballast shear, fastener stiffness and bearing friction, mobilise non-linearly. Standard simplified methods therefore risk under-estimating peak compressive stresses when default parameters are used without regional adjustment. The same limitation appears in the treatment of combined actions: although Eurocodes permit the superposition of thermal, braking (20 kN/m, capped at 6000 kN) and traction (33 kN/m, capped at 1000 kN) effects through partial factors and simultaneity coefficients, numerical studies consistently demonstrate that braking applied onto an already compressed rail can produce stress peaks noticeably higher than linear addition would suggest [1,5].

A further practical gap concerns selection of the rail neutral (or "stress-free") temperature. Current guidance offers limited direction for climates with large seasonal and diurnal swings, yet an inappropriate choice can shift the entire stress range toward either excessive compression in hot weather or tension in cooler periods, eroding the safety margin against buckling or fracture. For rapidly expanding networks in semi-arid zones, this absence of explicit calibration provisions represents a material shortcoming that must be bridged by detailed numerical assessment grounded in local climatic records.

These observations underline the continuing value of standards-compliant analysis while highlighting the necessity, in thermally demanding environments, of supplementing codified rules with site-specific data and advanced modelling. The present study therefore adopts the full Eurocode–UIC loading framework but calibrates temperature actions and restraint parameters directly from the Kano-Maradi-Dutse design documentation and meteorological archives, ensuring the results remain both code-conformant and realistic for the operating conditions encountered in northern Nigeria.

3. Case Study Description

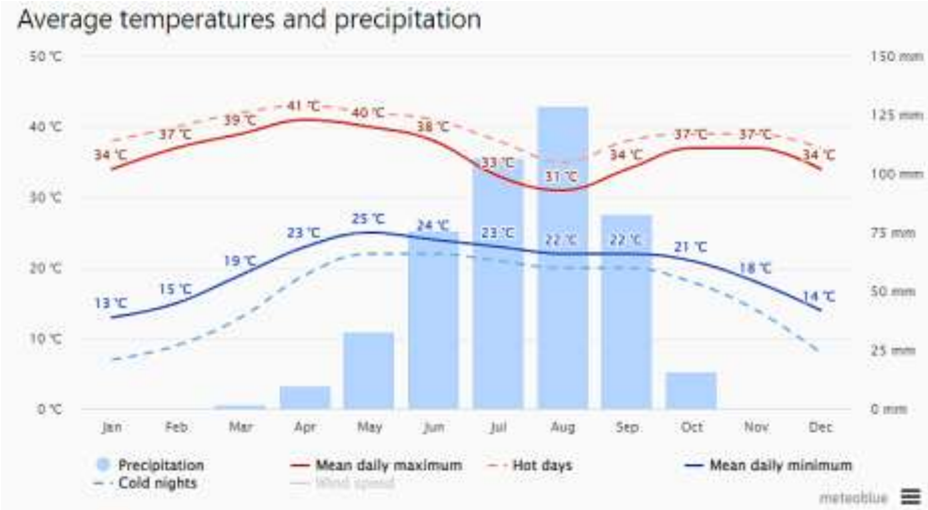
3.1 Railway Corridor and Climatic Conditions

The case study focuses on a bridge along the Kano-Maradi-Dutse Standard Gauge Railway Corridor in northern Nigeria. This cross-border line supports mixed passenger and freight traffic through a semi-arid zone marked by intense solar radiation, minimal cloud cover, and sharp seasonal shifts. These conditions place heavy thermal demands on rail infrastructure. Long-term records from the Kano area show that dry-season ambient temperatures often exceed 40 °C, with large diurnal swings layered on already high daily means (see Figure 1).

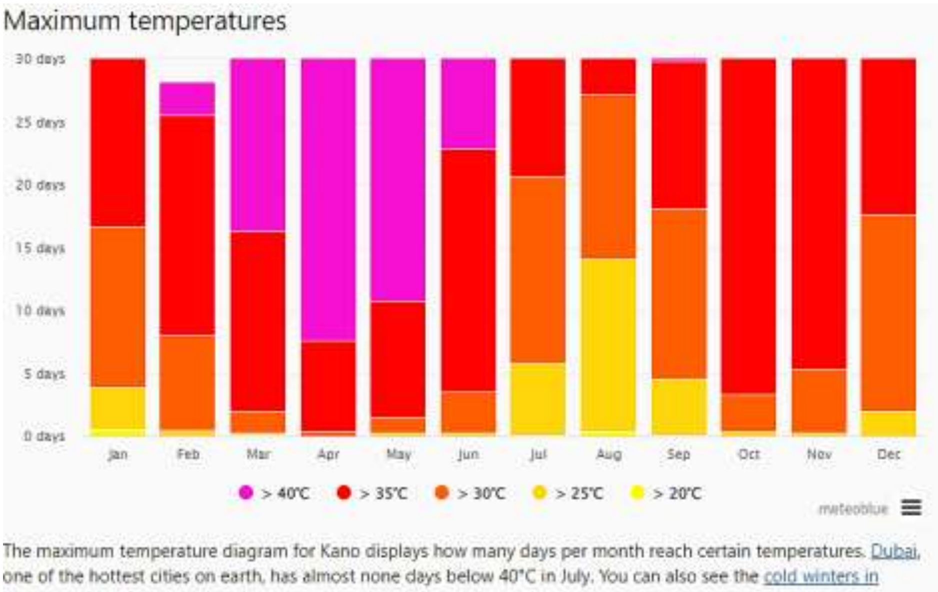
Rail surface temperatures matter more than air temperatures for continuous welded rail performance. Direct sun exposure and thinner ballast on bridge decks limit cooling, so rail temperatures commonly reach or exceed

45 °C, well above the ranges assumed in most design standards. This gap between ambient and rail temperatures drives the need for careful assessment of track-bridge interaction under extreme heat [1].

The corridor therefore offers a realistic and demanding setting for studying thermal effects on CWR systems. Sustained high temperatures, daily gradients, and long bridge structures combine to amplify restrained expansion, making it possible to examine serviceability limits and pinpoint the dominant interaction mechanisms with confidence.



(a) Kano Average Temperature



(b) Kano Maximum Temperatures

Figure 1: Long-term average and maximum temperature profiles for the Kano region, illustrating the climatic context of the case study [13]

3.2 Bridge Location and Structural Context

The selected bridge lies on the Kano-Maradi-Dutse alignment at UTM coordinates N 1 435 143.6964, E 412 851.8994 (Zone 32N). The section features straight horizontal geometry and gentle longitudinal gradients, which reduce secondary effects on longitudinal force distribution and allow the direct influence of thermal and mechanical actions to be isolated more clearly.

This bridge forms part of a series of concrete structures designed to carry continuous welded rail without intermediate expansion joints. Longitudinal restraint therefore builds up across multiple spans, heightening sensitivity to temperature-induced forces. Adjacent approach tracks rest on conventional ballasted formation, creating a stiffness transition at the abutments and piers that is known to concentrate interaction effects in these zones.

3.3 Bridge Geometry and Structural Configuration

The bridge is a multi-span simply supported reinforced-concrete viaduct built with precast prestressed girders. Typical span lengths strike a practical balance between hydraulic requirements, construction methods, and structural efficiency, producing a repetitive layout that lends itself well to numerical modelling. Overall length measures 791.30 meters, with 33 spans: two end spans of 23.65 meters and 31 middle spans of 24.0 meters - a scale at which cumulative restraint under thermal loading becomes significant.

Each span sits on pot bearings arranged to combine fixed and longitudinally free supports. Fixed bearings at one abutment anchor the structure, while the opposite abutment and selected piers allow controlled movement, in line with established guidance for railway bridges. The deck carries a single-track ballasted section with continuous welded UIC 60 rails on prestressed concrete sleepers. Ballast depth is deliberately reduced on the deck compared with embankment sections, increasing longitudinal stiffness and limiting the track's ability to accommodate thermal strain- a common feature in modern concrete-bridge practice that tends to intensify TBI under elevated temperatures (Figure 2).

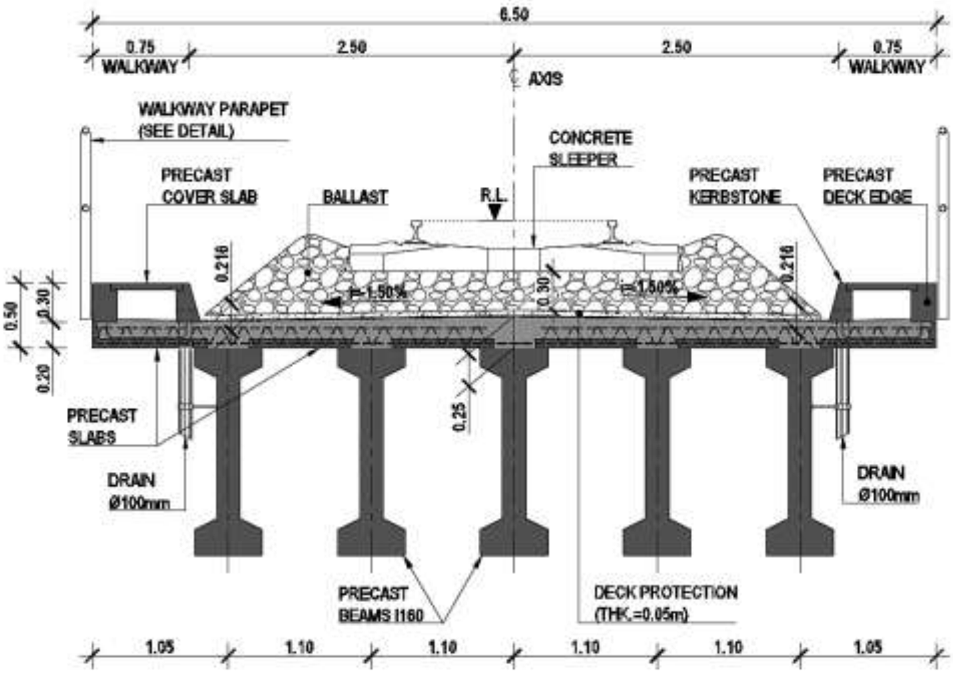
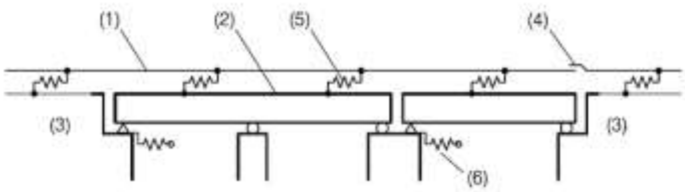


Figure 2: Typical cross-section of the railway bridge deck showing track components and structural arrangement.

3.4 Track System and Restraint Characteristics

UIC 60 rails run continuously across the bridge and into the approach tracks. Fasteners supply vertical, lateral, and longitudinal restraint between rails and sleepers, while the sleeper-ballast interface transfers loads into the deck. Longitudinal resistance is inherently non-linear: initial movement occurs against low stiffness, after which ballast shear resistance engages progressively with increasing displacement.

This non-linear behaviour lies at the heart of TBI (Figure3). On long concrete bridges it produces noticeable stress gradients along the rail, especially near fixed bearings and stiffness transitions. Regions closest to these points experience the highest compressive stresses under hot conditions and therefore require particular attention in numerical work.



Key

(1) Track
(2) Superstructure (a single deck comprising two spans and a single deck with one span shown)
(3) Embankment
(4) Rail expansion device (if present)
(5) Longitudinal non-linear springs reproducing the longitudinal load/ displacement behaviour of the track
(6) Longitudinal springs reproducing the longitudinal stiffness K of a fixed support to the deck, taking into account the stiffness of the foundation, piers, and bearings, etc.

Figure 3: Conceptual representation of longitudinal restraint mechanisms in a ballasted CWR system on a concrete bridge [9]

3.5 Relevance of the Case Study to Design Practice

The chosen bridge and track system reflect the typical challenges faced by modern railway infrastructure in hot-climate regions: long continuous spans, extensive precast concrete elements, jointless CWR, and exposure to extreme thermal cycles. As a real project example, it supplies a solid, site-specific basis for testing whether current design methods remain adequate when thermal demands exceed those assumed in temperate-zone standards.

Grounding the analysis in actual geometry, local climate data, and construction details ensures the findings translate directly into engineering decisions. The case study therefore does more than illustrate theoretical mechanisms; it informs practical design checks, risk management, and long-term maintenance strategies for CWR on concrete bridges operating under severe thermal conditions.

4. Numerical Modelling Methodology

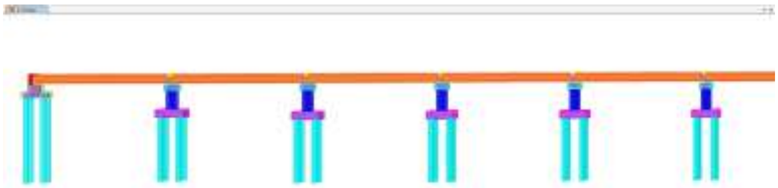
4.1 Modelling Approach and Software Selection

The track-bridge interaction was modelled using the finite element method implemented in SAP2000, a widely applied structural analysis package suited to capturing the coupled thermo-mechanical behaviour of continuous welded rail on concrete bridges. This choice aligns with established practice in railway engineering, where the software supports non-linear link elements to represent the progressive mobilisation of longitudinal track resistance, beam elements for the rail and deck, and appropriate boundary conditions to simulate bearing restraint and abutment fixity.

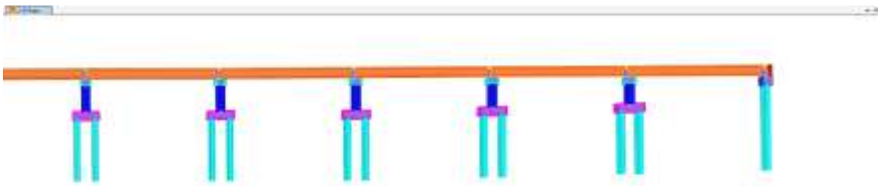
The model treats the rail as a continuous axial member interacting with the bridge deck through distributed non-linear springs that replicate the elasto-plastic character of ballast and fastener resistance. This approach follows the principles outlined in UIC 774-3 [6], which recommends simplified yet realistic representation of longitudinal restraint for serviceability verification under combined thermal and mechanical actions. The finite element framework allows direct incorporation of temperature-induced strains, braking and traction forces, and vertical traffic effects, enabling a unified assessment of axial rail stresses and relative displacements.

4.2 Geometric and Structural Idealisation

The bridge geometry was idealised as a series of simply supported spans with precast prestressed concrete girders carrying a single ballasted track. Deck elements were modelled using frame objects with properties derived from the design drawings, including cross-sectional area, second moments of area, and material stiffness corresponding to high-strength concrete. The continuous welded UIC 60 rails were represented by frame elements running the full length of the structure and extending into the approach embankments to capture the decay of interaction forces away from the bridge ends (Figure 4). Longitudinal track restraint was discretised using non-linear link/support elements connecting the rail nodes to the corresponding deck nodes. These elements were assigned a bi-linear force-displacement relationship consistent with UIC 774-3 recommendations for ballasted track: an initial elastic stiffness up to a characteristic displacement (2 mm), followed by a plastic plateau representing the ultimate shear resistance of the ballast-sleeper interface under unloaded and loaded conditions. Separate curves were applied for unloaded (thermal-dominant) and loaded (train-present) scenarios to reflect the increase in longitudinal resistance when vertical axle loads compress the ballast.



(a) Model M1 – 3D Extruded General View near Abutment A1



(b) Model M1 – 3D Extruded General View near Abutment A2

Figure 4: 3D view of the SAP2000 finite-element model showing the continuous rail, bridge deck, pier supports, and boundary conditions at fixed and free bearings

4.3 Material Properties and Restraint Parameters

Rail properties were adopted from standard UIC 60 section data, giving a cross-sectional area of 7686 mm², Young's modulus of 210 GPa, and coefficient of thermal expansion of 1.2×10^{-5} /°C, in accordance with UIC 860-R (2005). For the concrete deck, Young's modulus was taken as 35 GPa and the coefficient of thermal expansion as 1.0×10^{-5} /°C, which are typical for prestressed elements in such applications.

Longitudinal resistance parameters were drawn from the UIC 774-3 guidelines for ballasted track on concrete decks. For unloaded track under moderate maintenance conditions, the initial stiffness was approximately 20-25 kN/mm per metre of track length (equivalent to around 10–12.5 kN/m/mm when discretised), with an ultimate resistance of about 20 kN/m per rail. Under loaded track conditions (with vertical traffic present), the stiffness and resistance increased, typically to 1.5-2 times the unloaded values, reflecting ballast compaction - often reaching an ultimate resistance of around 60 kN/m per rail in many benchmark cases and practical implementations aligned with UIC recommendations.

These conventional values were cross-checked against the site-specific ballast conditions and fastener performance detailed in the bridge design documentation to ensure appropriate regional calibration. Friction at sliding pot bearings was modelled in accordance with BS EN 1337-2, adopting a nominal coefficient of 0.03-0.05 under service conditions and applying this as equivalent longitudinal link stiffness at the movable supports.

4.4 Load Cases and Combinations

Thermal loading was applied as uniform temperature changes to both the rail and the deck. The characteristic range of ±20 °C was used for standard serviceability checks, as specified in the design report and aligned with BS EN 1991-1-5. To capture the extreme local conditions prevalent in northern Nigeria, an additional case considered a rail temperature rise up to +45 °C, corresponding to a differential of +25 °C relative to the neutral temperature and representing peak solar-heated rail surfaces documented in the region.

Mechanical actions included vertical traffic in the form of RU loading (equivalent to UIC 71), consisting of a uniform distributed load of 80 kN/m together with four concentrated loads of 250 kN each, applied with the appropriate dynamic factor φ in accordance with BS EN 1991-2. Longitudinal traction was modelled as a distributed force of 33 kN/m, capped at a total of 1000 kN, while longitudinal braking was represented by a distributed force of 20 kN/m, capped at 6000 kN.

Individual load cases were analysed separately, followed by critical combinations that adhered to Eurocode principles for simultaneous occurrence, typically thermal effects combined with one horizontal action and, where relevant, vertical traffic effects. Non-linear static analysis procedures were employed throughout to account for the progressive activation of track resistance (Figure 5) and the consequent redistribution of internal forces.

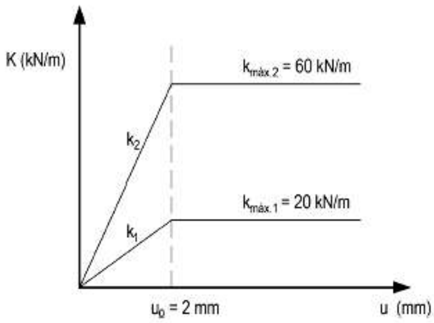


Figure 5: Typical non-linear longitudinal resistance curve for ballasted track (unloaded and loaded conditions), as implemented in the model [6]

4.5 Boundary Conditions and Analysis Type

Fixed bearings at one abutment restrained longitudinal deck movement, while sliding bearings at the opposite abutment and intermediate piers permitted movement subject to friction. Approach track sections were extended sufficiently (60 m) with gradually increasing longitudinal restraint to simulate free-field conditions far from the structure.

Analyses employed non-linear static solution procedures to handle the bi-linear link behaviour and geometric non-linearity where large displacements were anticipated. Output focused on serviceability metrics: maximum compressive and tensile rail stresses, relative rail-deck displacements, and bearing movements, all checked against permissible values in UIC 774-3 and Eurocode Annexes.

4.6 Model Validation and Benchmarking

The modelling approach was benchmarked against simplified analytical solutions for a representative single-span case under pure thermal loading, using closed-form expressions and reference test cases from UIC 774-3 Appendix D. The single-span idealisation extracted from the full model showed good agreement with these benchmarks, with peak rail stresses and displacements differing by 5-8 % in critical zones which is well within the 10 % validation tolerance in clause 1.7 (or up to 20 % when conservative). Discrepancies arose mainly from the full model's inclusion of multi-span continuity, variable pier stiffness, and non-uniform approach-track restraint - effects deliberately retained to better reflect real bridge behaviour but idealised or absent in the hand-calculation benchmarks.

This methodology provides a robust, standards-compliant framework for quantifying TBI under the severe thermal regime experienced in the study corridor, without reliance on optimisation of fastener parameters.

5. Results

The finite-element analyses conducted in SAP2000 yielded clear and consistent patterns in the response of the continuous welded rail-bridge system under the combined actions of thermal loading, braking, traction, and vertical traffic. All results are presented for rail temperatures of 20 °C and 45 °C, corresponding to the neutral and extreme service conditions relevant to the Kano-Maradi-Dutse corridor. The focus here is on serviceability metrics such as the axial rail stresses and longitudinal displacements, benchmarked directly against the limits in BS EN 1991-2, BS EN 1991-1-5, and UIC 774-3.

5.1 Axial Rail Stresses

Thermal loading proved to be the dominant driver of track-bridge interaction, as expected for long concrete bridges carrying jointless rail in a hot climate. At a rail temperature of 20 °C the axial stresses remained modest, generally within ±3 MPa along the rail length, with only minor local peaks near the fixed bearings. When the rail temperature rose to 45 °C the stress envelope widened markedly: peak tensile stresses reached approximately +20 MPa and compressive stresses approached -15 MPa (Figures 6 and 7). These values are well below the permissible limits of +92 MPa in tension and -72 MPa in compression for UIC 60 rail [14,12], confirming a comfortable margin against fracture or buckling under the design temperature range.

The stress distribution exhibited the characteristic oscillatory pattern produced by discrete support restraint, with higher magnitudes concentrated near the fixed abutment and pier lines where longitudinal movement is most constrained. Superposition of braking or traction forces onto the thermal baseline increased the peak compressive stress by only 4-6 MPa, underscoring that temperature change, rather than train actions, governs the overall stress level in this configuration.

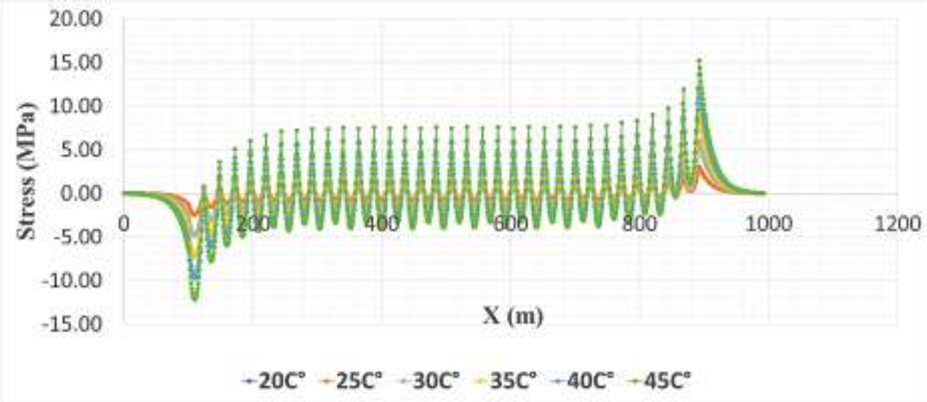


Figure 6: Axial rail stress distribution along the bridge under pure thermal loading at rail temperatures of 20 °C and 45 °C.

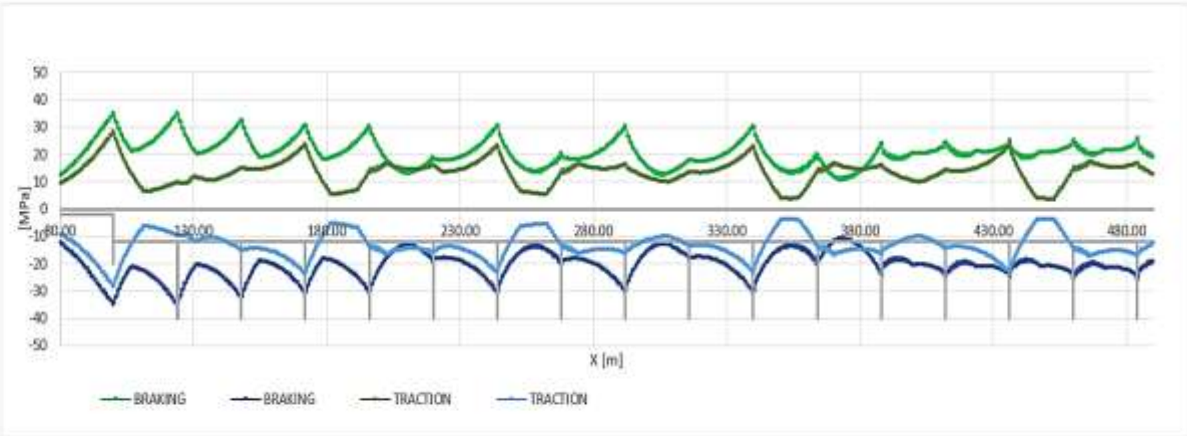


Figure 7a: Combined axial rail stress envelope at 45 °C rail temperature for braking and traction effects (1/2).

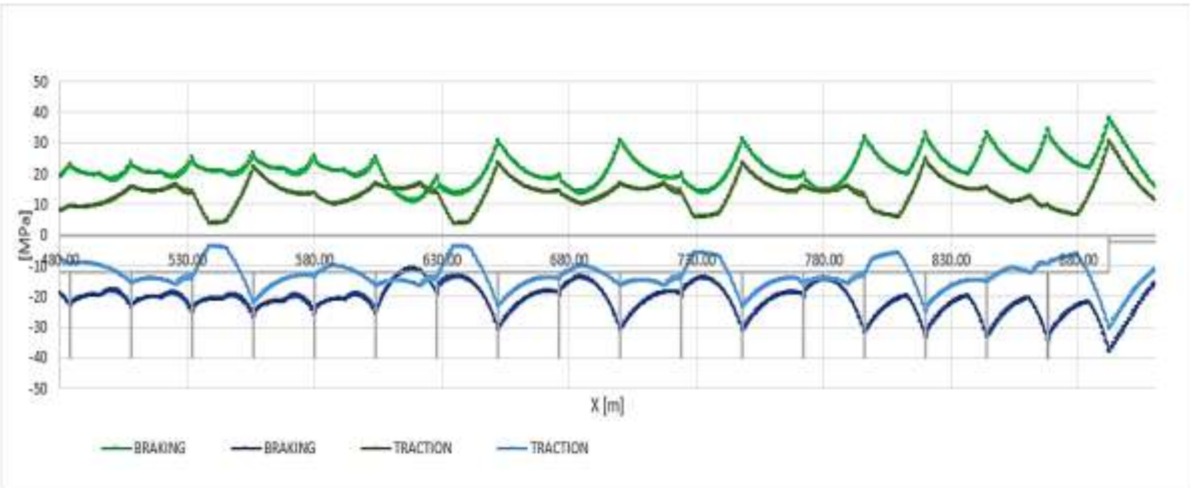
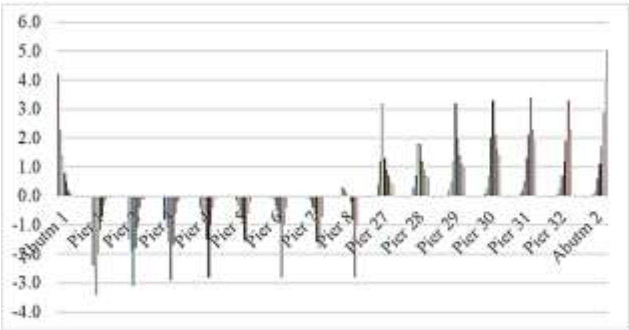


Figure 7b: Combined axial rail stress envelope at 45 °C rail temperature for braking and traction effects (2/2).

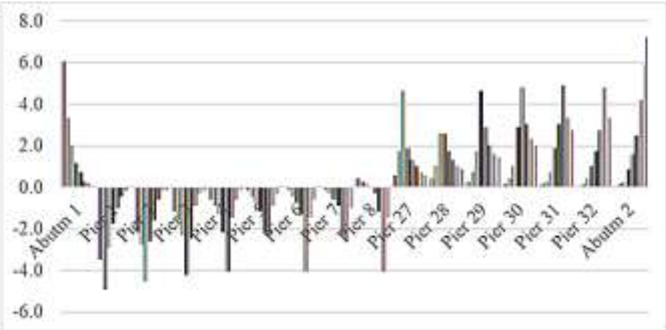
5.2 Longitudinal Displacements under Braking and Traction

Braking actions produced the largest relative displacements between adjacent deck segments. At 20 °C rail temperature the maximum relative displacement remained within ±4.0 mm; at 45 °C this increased to ±6.0 mm (Figure 8). Traction induced smaller movements, reaching ±3.0 mm at 20 °C and ±4.5 mm at 45 °C. These values fall comfortably inside the 8 mm serviceability threshold stipulated in BS EN 1991-2 for lines operating up to 160 km/h and are consistent with the non-linear mobilisation of ballast and fastener resistance.

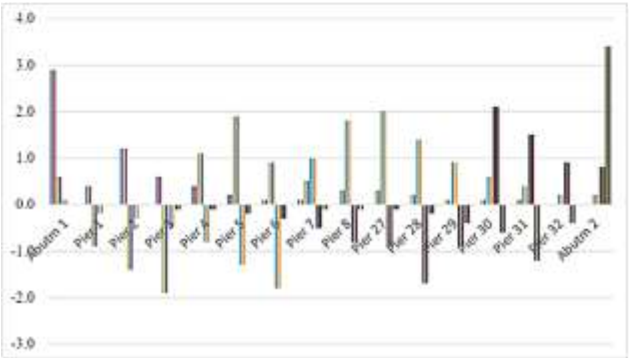
The displacement profiles revealed that the largest relative movements occurred at the transitions between spans and near the fixed bearings, precisely the locations where track–bridge interaction is most intense. The modest increase with temperature reflects the additional expansion of the deck itself, which slightly reduces the effective stiffness of the restraint system.



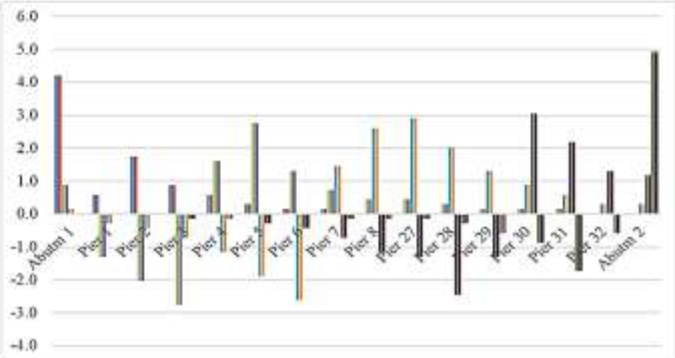
Relative Longitudinal Displacements Due to Braking at 20 °C



Relative Longitudinal Displacements Due to Braking at 45 °C



Relative Longitudinal Displacements Due to Traction at 20 °C

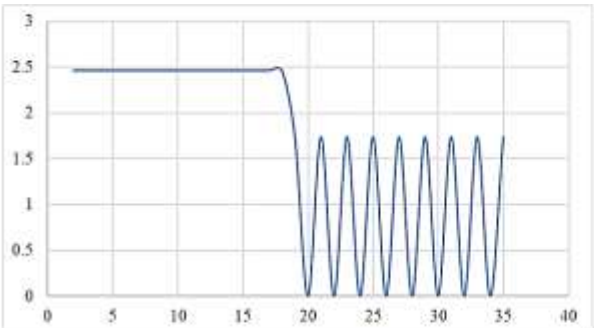


Relative Longitudinal Displacements Due to Traction at 20 °C

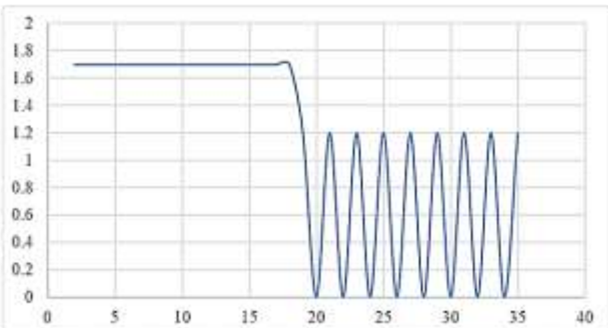
Figure 8: Relative Longitudinal Displacements Between Decks Under Braking and Traction Actions.

5.3 Deck Displacements under Vertical Traffic

Vertical traffic loads (80 kN/m uniform plus 4 × 250 kN concentrated) produced comparatively small longitudinal movements of the deck upper surface: 1.7 mm at 20 °C rising to 2.5 mm at 45 °C (Figure 9). Vertical deflections of the deck remained negligible, between 0.2 mm and 0.3 mm. These responses lie far below the 3 mm vertical and 8 mm longitudinal limits prescribed for high-speed compatibility, confirming that vertical actions have only a secondary influence on longitudinal behaviour in this ballasted concrete-bridge system.



Absolute Longitudinal Displacement of the Upper Surface of the Deck Due to Vertical Traffic at 20 °C [δH (deck edge) dv = 1.9 m]



Absolute Longitudinal Displacement of the Upper Surface of the Deck Due to Vertical Traffic at 45 °C [δH (deck edge) dv = 1.9 m]

Figure 9: Absolute longitudinal displacement of the deck upper surface under RU vertical loading at 20 °C and 45 °C rail temperatures.

5.4 Serviceability Assessment

The results demonstrate that the bridge and its continuous welded rail satisfy all relevant serviceability criteria under the most adverse combinations of thermal and mechanical loading encountered in northern Nigeria. Thermal effects remain the governing action, yet the induced stresses and displacements stay within safe margins when the system is analysed with realistic non-linear restraint and site-specific temperature ranges. Braking emerges as the critical mechanical contributor to relative deck movement, while vertical traffic plays a minor role.

The modelling approach was fully compliant with BS EN 1991-1-5, BS EN 1991-2, and UIC 774-3, therefore provides a reliable basis for design verification and highlights the importance of adopting rail temperatures up to 45 °C in regions with intense solar exposure. No further mitigation measures (such as additional expansion joints) appear necessary for this structure, provided the neutral temperature is selected with due regard to local climatic records.

6. Discussion

6.1 Governing Mechanisms of TBI under Extreme Thermal Loading

Building directly on the stress distributions presented in Section 5, the analyses confirm that restrained thermal expansion remains the overriding driver of TBI on this long concrete viaduct. The marked widening of the axial stress envelope at 45 °C rail temperature, from the modest ±3 MPa seen at 20 °C to peaks of approximately +20 MPa in tension and -15 MPa in compression, arises because the rail's natural tendency to lengthen is resisted by the relatively stiff deck, bearings and ballast restraint. The characteristic oscillatory pattern, with stress concentrations at fixed bearings and span transitions, is exactly what would be expected from the discrete nature of the support system and the progressive mobilisation of non-linear track resistance.

This behaviour aligns closely with both theoretical expectations and field evidence from similar hot-climate installations. It also explains why thermal effects frequently overshadow traffic loading in serviceability assessments for structures of this length and construction type.

6.2 Interaction between Thermal and Mechanical Actions

When the thermal baseline is combined with train-induced forces, the interaction becomes more nuanced yet still predictable. As the displacement profiles in Figures 8 and 9 show, braking generates the largest relative movements between deck segments (±6.0 mm at 45 °C), producing additional compressive stress increments of 4-6 MPa that cannot be captured by linear superposition. Traction forces, by comparison, remain modest in their effect on peak values and serve mainly to redistribute stress along the rail length. Vertical RU loading exerts the smallest longitudinal influence, increasing ballast compaction locally but contributing little to the overall interaction response.

These findings reinforce a key practical point: for bridges in regions where rail temperatures routinely approach 45 °C, the governing serviceability combination is almost invariably maximum thermal expansion paired with full braking, a scenario that must be checked explicitly rather than through separate load cases.

6.3 Adequacy of Existing Design Provisions

The computed responses provide reassuring evidence of the robustness of current standards when applied thoughtfully. All axial stresses remain well below the permissible limits of +92 MPa (tension) and -72 MPa (compression) for UIC 60 rail, while the largest relative displacements (6 mm under braking) sit comfortably within the 8 mm threshold given in BS EN 1991-2 for lines up to 160 km/h. Even under the most severe thermal-mechanical pairing, the system satisfies the serviceability criteria of UIC 774-3 and the supporting Eurocode clauses.

The progressive narrowing of safety margins at 45 °C rail temperature, together with the clear sensitivity to neutral-temperature selection, highlights an important qualification. Default parameters drawn from temperate-zone calibration can still be used, but only when supplemented by site-specific climatic data and realistic restraint modelling, precisely the approach adopted in this study.

6.4 Implications for Design and Assessment Practice

Several information for permanent-way and bridge engineers emerge clearly from these results. First, thermal loading should be elevated to a primary design factor for any concrete bridge carrying continuous welded rail in hot climates, especially where total length exceeds a few hundred metres. Second, the combination of peak rail temperature and braking force deserves explicit attention in every serviceability check. Third, the neutral temperature must be chosen with care, informed by local meteorological records and the anticipated construction season, rather than by generic tables.

The modelling methodology itself; non-linear springs, extended approach-track lengths, and accurate bearing friction, offers a practical and defensible route for verifying performance when standard assumptions are pushed toward their limits. Finally, the concentration of peak effects at fixed bearings and bridge-embankment transitions points to the value of targeted monitoring during the early years of operation. Simple instrumentation in these zones can provide early warning of any unexpected behaviour and support proactive maintenance before minor issues escalate.

The study demonstrates that, with proper attention to local conditions and realistic numerical representation, continuous welded rail on concrete bridges can operate safely and reliably even under the demanding thermal regime of northern Nigeria. It also offers a template for similar assessments on the growing network of standard-gauge lines across sub-Saharan Africa.

7. Conclusion

This numerical study has demonstrated that TBI on concrete viaducts carrying continuous welded rail remains safely within established serviceability limits, even when subjected to the extreme rail temperatures characteristic of northern Nigeria's semi-arid climate.

The finite-element model, built in SAP2000 and fully compliant with UIC 774-3, BS EN 1991-1-5, and BS EN 1991-2, captured the non-linear mobilisation of longitudinal track resistance and the coupled thermo-mechanical response of the system. Key findings include:

i. Thermal loading dominates the interaction, with axial rail stresses rising from modest levels (±3 MPa at 20 °C rail temperature) to peaks of approximately +20 MPa (tension) and -15 MPa (compression) at 45 °C. These values lie well below the permissible additional stresses of +92 MPa in tension and -72 MPa in compression for UIC 60 rail, as set out in UIC 774-3.
ii. Braking emerges as the most significant mechanical contributor, producing the largest relative displacements between deck segments (up to ±6 mm at peak thermal conditions) and modest additional compressive stress increments (4-6 MPa). Traction and vertical RU loading exert comparatively minor effects on longitudinal behaviour.
iii. All computed displacements and stresses satisfy the relevant Eurocode and UIC serviceability thresholds, confirming the structural adequacy of the bridge and track system under the combined actions examined.

The results highlight the critical importance of adopting realistic, site-specific rail temperature ranges, rather than relying solely on the milder uniform gradients (±20 °C) often assumed in temperate-zone calibrations, when designing or assessing CWR on concrete bridges in hot regions. The pronounced influence of hot rail surfaces, which can routinely exceed 40-45 °C in the Kano area, narrows safety margins noticeably and demands careful selection of the neutral (stress-free) temperature based on local meteorological records and construction sequencing.

From a practical standpoint, the work reinforces several guiding principles for railway engineers involved in similar projects across sub-Saharan Africa's expanding standard-gauge corridors:

i. Thermal expansion should be treated as the primary serviceability factor for multi-span concrete structures carrying jointless rail.

ii. The combination of maximum rail temperature and braking force constitutes the governing load case and must be verified explicitly.
iii. Non-linear finite-element modelling, incorporating accurate boundary conditions, extended approach-track lengths, and progressive ballast resistance, provides a robust and defensible method for performance demonstration when standard simplifications are extended to extreme conditions.

No additional mitigation measures, such as supplementary expansion devices, appear necessary for the prototype bridge studied, provided the neutral temperature is appropriately chosen and long-term monitoring of critical zones (fixed bearings and abutment transitions) is implemented during early service life.

The investigation offers confidence that well-executed, standards-compliant analysis, grounded in realistic climatic data and detailed structural representation enables continuous welded rail to perform reliably on concrete bridges under severe thermal demand. The approach presented here can serve as a practical template for future evaluations on Nigeria's growing railway network and analogous hot-climate infrastructures elsewhere.

8. Recommendations for Practice and Further Study

The numerical evaluation of TBI on the prototype concrete bridge from the Kano-Maradi-Dutse corridor has confirmed that continuous welded rail systems can operate reliably under extreme thermal conditions when assessed with realistic, site-specific inputs and standards-compliant methods. The findings point to several actionable steps for designers, constructors, and maintainers working on similar standard-gauge projects in hot-climate regions of sub-Saharan Africa.

8. Recommendations for Practice and Further Study

The findings confirm that the prototype bridge and its continuous welded rail system remain within serviceability limits under extreme thermal conditions, provided assessments incorporate realistic local data and standards-compliant methods. The following recommendations offer practical guidance for railway engineers on similar standard-gauge projects in hot-climate regions.

8.1 Design and Construction Practice

Thermal loading must be treated as the primary serviceability driver for multi-span concrete bridges carrying jointless rail, particularly where rail surface temperatures routinely reach or exceed 45 °C due to solar exposure, as in northern Nigeria. Design reports should supplement the standard uniform temperature range (±20 °C) with an extreme case based on local peak rail temperatures to capture the governing combination of thermal expansion and braking forces.

The neutral (stress-free) temperature for the continuous welded rail should be selected using long-term meteorological records for the corridor (e.g., Kano-area data showing frequent dry-season ambient maxima above 40 °C), with preference given to the upper end of the annual range and alignment with the anticipated rail-laying season. This minimises peak compressive stresses in hot periods while controlling tensile risks in cooler months. Rail installation should target this neutral temperature where sequencing permits, with allowance for minor post-installation adjustment informed by early monitoring.

Non-linear finite-element modelling (e.g., in SAP2000) provides a reliable means of verifying compliance when simplified UIC 774-3 methods are extended to severe conditions. Key features should include:

i. bi-linear longitudinal resistance curves for unloaded and loaded ballast conditions;
ii. realistic sliding-bearing friction per BS EN 1337-2;
iii. extended approach-track lengths to model free-field force decay
iv. site-specific thermal gradients where differential deck heating is relevant.

Special attention is required at fixed-bearing zones and abutment transitions, where peak stresses and displacements concentrate. These locations must satisfy UIC 774-3 permissible additional rail stresses (+92

MPa tension, -72 MPa compression for UIC 60 rail) and relative displacement limits (typically ≤8 mm for speeds up to 160 km/h).

During construction, rigorous quality control of ballast compaction, fastener torque, and initial rail prestressing is essential to achieve the design restraint characteristics. Post-construction baseline surveys of rail stresses (using strain gauges or vibration methods where feasible) at critical points should establish the as-built neutral temperature.

8.2 Operation and Maintenance

Critical zones such as fixed bearings, pier transitions, and bridge-embankment interfaces, warrant routine monitoring as part of the asset management plan, with heightened focus during the first 3-5 years of service and in advance of peak dry-season temperatures. Low-cost instrumentation (reference marks for displacement or portable rail-temperature sensors) can detect anomalies early, enabling proactive measures.

Hot-weather patrols should be scheduled when ambient temperatures exceed 35-40 °C or rail temperatures approach 45 °C, inspecting for indicators of tight track (ballast disturbance, rail canting, lateral misalignment). Where necessary, apply temporary speed restrictions or intensified inspections in accordance with established heat-risk protocols.

8.3 Further Study

To support refinement of design practice across Nigeria's expanding network, future work should include:

i. field instrumentation of operational bridges to validate numerical models against measured rail stresses, displacements, and temperatures under seasonal extremes, accounting for real ballast variability and fastener ageing;
ii. parametric sensitivity analyses of interaction forces to changes in ballast condition (fouling, compaction loss) and fastener degradation;
iii. comparative evaluation of mitigation options, such as centralised fixed-bearing arrangements or selective expansion devices on very long viaducts, including cost–benefit assessment in hot climates;
iv. extension to ballastless (slab) track systems, where higher inherent restraint may amplify thermal effects in high-speed or heavy-haul applications.

Studies grounded in local climate data and operational feedback would provide a stronger basis for updating national railway guidelines to address extreme thermal demands more effectively.

Finally, proactive adoption of site-specific thermal realism, careful neutral-temperature selection, and targeted monitoring will enhance the long-term resilience of continuous welded rail on concrete bridges throughout Nigeria's standard-gauge corridors and similar thermally demanding environments.

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Author contributions

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EEN, (Formal analysis: Supporting; Investigation: Supporting; Methodology: Supporting; Project administration: Equal; Software: Equal; Supervision: Supporting; Writing-review & editing: Supporting).

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