



## ANALYSING RAIL STRUCTURE INTERACTION IN OPEN WEB STEEL GIRDERS WITH VARIED SPANS: A COMPREHENSIVE CASE STUDY

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### Abstract

Long welded rails are extensively used these days due to their many advantages over traditional short welded rails, such as better load distribution, longer rail lifespans, increased safety, lower maintenance costs, faster speeds, and more efficiency etc. The provision of continuous long welded rails over bridges and adjacent embankment can be adopted and strictly adhered based on the specific guidelines outlined in UIC 774-3R. Rail structure interaction effect is one of the fundamental aspects to be considered in the design of railway bridges. However, the usage of long welded rails over a steel open web through girder bridge carries its own unique challenges since the stress developed in long welded rails primarily depends on the stiffness of bridge elements. This paper presents a comprehensive case study on three ongoing bridge projects, featuring two single span and one multi span simply supported steel open web through girder bridge with varying span length (55.51 m, 79.90 m and 145.5 m) for Dedicated Freight Corridor Corporation. Using industry-standard finite element method software package, rigorous Rail-Structure Interaction assessments were carried out in accordance with the guidelines outlined in the UIC 774-3R code of practice. Open web through girder bridges increases the flexibility of structure and poses difficulty to satisfy the stress and displacement criteria mentioned in UIC and Indian Railway Standards codal provisions. Comparisons are made between additional rail stresses and displacements obtained from the previously mentioned rail-structure interaction analyses and conclusions were reached. In this paper, alternate proposals required for identifying the acceptable economically feasible solution that meets the codal requirements are suggested. The possibilities of continuous long welded rails over steel open web through girder bridges are collectively scrutinized, and conclusions are drawn.

*Keywords: track, long welded rails, rail interaction, switch expansion joints, rail free fasteners, open web girders, steel bridges*

### 1 Introduction

To meet current and future expectations, the difficult tasks associated with designing railway bridges for higher speeds and larger loads are always growing. Additional research is necessary to determine the safety of the track system under increasing static and dynamic loads and this can be achieved with Rail structure interaction (RSI) analysis. RSI gets affected by numerous factors, such as the bridge's structural characteristics, the rail cross-section, the superstructure stiffness, substructure stiffness, loading type, the support conditions, etc. Since the time that bridges have been used for commercial transportation, extensive research has been conducted on the phenomena of RSI caused by live loads coming from trains and thermal loads generated on the bridges. Although this research has been done in

several nations, a few of the studies are highlighted [1-5]. The present paper focuses mainly on the study of RSI analysis on both unballasted single span and multi span steel open web through girder (OWG) bridges carrying very heavy axle load of 32.5 t. For this purpose, initially the bridge is being analysed for RSI considering dedicated freight corridor (DFC) loading. Secondly, the axial stresses in the rail are examined to provide a provision of Long welded rails (LWR) on bridge. Finally, the relative displacement of track and bridge are examined for train live loads acting horizontally are evaluated at each abutment and pier location accordingly. Provision of LWR on bridge reduces the excessive vibrations which in-turn increases the passenger ride comfort, reduces derailment risks and rail fractures at joints and essentially reduces very high maintenance cost accordingly. Therefore, in line with the results obtained from RSI analysis the provision of providing LWR or switch expansion joints (SEJ) on bridge shall be concluded accordingly.

## 2 Scope and objective

The present study focuses on the following objectives:

- To illustrate the RSI analysis results of Bridge-1 (SA 39), Bridge 2 (MUP 10) and Bridge 3 (MJB 7)
- To compare the additional rail stresses between two single span bridges i.e, Bridge 1 and Bridge 2 and one single span and multi-span bridge i.e, Bridge 2 and Bridge 3.
- To conclude the provision of providing LWR/SEJ on the bridge based on the permissible limit given by UIC 774-3R [7] and RDSO Guidelines [8].

## 3 Methodology

### 3.1 Design criteria

The UIC 774-3R code of practice explains the interaction between the bridge and the track by bilinear relationship as mentioned in the Cl. 1.2.1.2. The resistance of the rail to longitudinal displacement is a function of displacement of rail relative to supporting structure and the loading on the track. The resistance increases for lower displacements and remains constant after reaching a certain magnitude of displacement. The criteria to be met in the calculations done with a computer program are as per Cl. 1.7.2 of UIC 774-3R.

### 3.2 Input data for Track Stiffness

Track stiffness for unballasted bridge region and for embankment region have been considered as per Cl. 1.2.2 and Cl.1.2.1.2 of UIC 774-3R.

### 3.3 Bridge geometry

Table 1 Material Properties

Rail		Superstructure		Portal piers and gravity abutment		Open and pile foundation	
Rail steel modulus	210000 [MPa]	Height	10.8 [m]	Concrete grade ( $f_{ck}$ )	35 [MPa]	Concrete grade ( $f_{ck}$ )	35 [MPa]
Cross sectional area	7781 [mm <sup>2</sup> ]	Steel grade	E250 & E350	Youngs modulus ( $E_c$ )	29500 [MPa]	Youngs modulus ( $E_c$ )	29500 [MPa]
Thermal expansion coefficient	$1 \times 10^{-5}$ per °C	Thermal expansion coefficient	$2.16 \times 10^{-5}$ per °C				

### 3.4 Boundary conditions

Table 2 Open Web Steel Girder Bridge Details of Bridge 1, Bridge 2 and Bridge 3

Bridge name	Bridge-1		Bridge-2		Bridge-3	
Span Configuration	55.51 m		79.90 m		3 x 48.5 m	
Type of foundation	Open		Pile		Pile	
Abutment	1	2	1	2	1	2
Length of the abutment wall	7.22 m	7.22 m	7.26 m	7.30 m	15.9 m	15.9 m
Thickness of abutment at top	0.75 m	0.75 m	0.7 m	0.7 m	0.7 m	0.7 m
Thickness of abutment at bottom	3.2 m	3.2 m	3 m	3 m	2.6 m	1.5 m
Open foundation thickness	3 m	3 m				
No of Piles	-		16 Nos	16 Nos	12 Nos	12 Nos
Diameter of Pile	-		1.5 m		1.5 m	
Pier with Pile Foundation						
Pier dimension (Circular radius)	-		-		1 m	
Depth of Pile Cap	-		-		1 m	
Piles in Pier Location	-		-		8 Nos	

In case of open foundation, elastic behaviour of soil is considered by means of linear springs to extract the accurate results. The stiffness value is calculated based on the soil properties and plan area of the supporting structure. Whereas, in case of pile foundation, to extract the accurate results pile is modelled up to the depth of fixity and given fixed supports at the fixity point.

**Table 3** Substructure stiffness of Bridge 1, Bridge 2 and Bridge 3

Bridge name	Bridge-1		Bridge-2		Bridge-3			
	A1	A2	A1	A2	A1	P1	P2	A2
Live Load stiffness [kN/m]	154000	159000	232000	231000	491000	105000	116000	385000
Temperature stiffness [kN/m]	77000	79000	116000	115000	245000	52000	58000	192000

Both temperature and live load cause displacements and stresses in the bridge structure, but the stiffness of the structure under live loads is typically higher than that due to temperature effects because live loads result in more significant and immediate deformations. Therefore, the stiffness contribution from temperature effects is assumed to be proportionally smaller than that from live loads.

### 3.5 Loading conditions

The various possible combinations of live load is considered as per IRS Bridge rules [6], Appendix XXV. Based on the 5 numbers of loco-wagon combination given in Appendix XXV and Appendix XXVI of IRS bridge rules, the Equivalent Uniformly Distributed Loads (EUDL) producing same bending moment due to standard DFC loading for the given span length is considered. After that, the vertical loads are multiplied with coefficient of dynamic augmentation (CDA). For analysis purpose, uniform vertical load has been considered in conservative side.

**Table 4** Load cases of Bridge 1, Bridge 2 and Bridge 3

Load case	Bridge-1	Bridge-2	Bridge-3
Train vertical load [kN/m]	154.23	148.72	157.04
Temperature load, $\Delta T_{\text{Deck}}$ [°C]	±35	±35	±35
Tractive force [kN/m]	37.35	37.35	37.35
Braking force [kN/m]	19.08	18.26	19.78

From Appendix XXVII of IRS bridge Rules, the braking force is considered for (EJ-EJ) length of the bridge. Maximum tractive force of loco = 1234.8 kN. This force has been applied on length of 33.06 m, which is the distance between the first axle and last axle as per the combination 4 (as given in Appendix XXVIII of IRS bridge Rules). Also, the temperature load on the bridge is applied as per Cl.1.4.2, UIC 774-3R with a maximum uniform temperature variation of the rails i.e,  $\Delta T_{\text{Rail}} = \pm 50$  °C .

### 3.6 Analysis of model

The assumptions for model preparation are considered as per the guidelines provided in UIC 774-3R. As per Cl.1.7.3 of UIC 774-3R, the substructure and superstructure modelling for all the bridges were done. Full substructure model has been assumed to be supported by appropriate soil springs calculated based on subgrade modulus defined as per soil strata in IS 2950 Part 1: 1981.

For proper bearing arrangement, fixed spherical bearing is provided at the fixed location and free spherical bearing is provided at the free end for a single-track configuration.

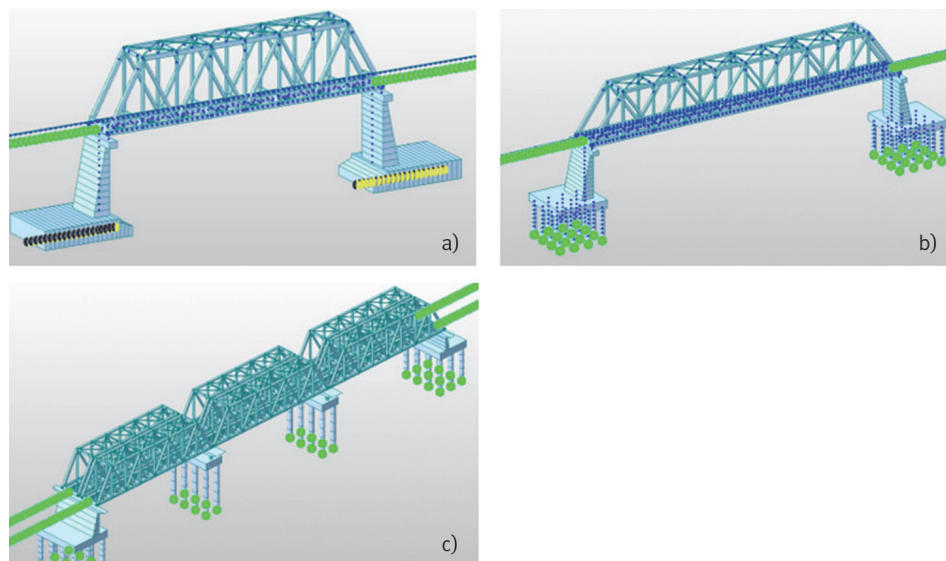


Figure 1 Rendered Superstructure Model for: a) Bridge 1, b) Bridge 2 and (c) Bridge 3

## 4. Result and discussion

Bridge parameters considered for RSI Analysis in the present paperwork are – (i) Stiffness of substructure, (ii) Stiffness of superstructure, (iii) Track stiffness on bridge, (iv) Track stiffness on the embankment, (v) End rotation of the deck, (vi) Vertical deflection of the girder, (vii) Thermal loads, (viii) Braking/Traction loads and (ix) Train vertical live load.

Based on the above-considered parameters, the axial stress in rail (both tensile and compressive), the longitudinal relative displacement between the rail and bridge deck, Horizontal displacement due to deck rotation and maximum absolute displacement of deck are calculated as per the guidelines given in UIC774-3R to propose a provision for providing LWR on the bridge or to adopt SEJ.

To meet the objective of the current study, two unballasted single-span open web steel girder bridge having two different bridge span configurations of 55.51 m (Bridge 1) and 79.90 m (Bridge 2) respectively carrying DFC loading (32.5t Axle load) are analyzed using the guidelines given in UIC774-3R. Also, another analysis has been carried out between bridge 2 (single span) and Bridge 3 (multi span) both resting upon Pile foundation following the guidelines provided in UIC 774-3R. As per the guidelines, the loads considered for the analysis are braking and traction loads, thermal loads and train vertical live loads (as shown in Figures 2-5).

### 4.1 Stresses in rails

From Figure 2, it is evident that the total tensile stress and compressive stress generated at the fixed support of bridge 1 is 18.89% and 21.10% higher than that of Bridge 2.

The total tensile and compressive stress values obtained at the free support of Bridge 1 is 31.84% and 19.11% lesser than that of Bridge 2 which is completely opposite to that obtained from fixed support condition (as shown in Figure 3).

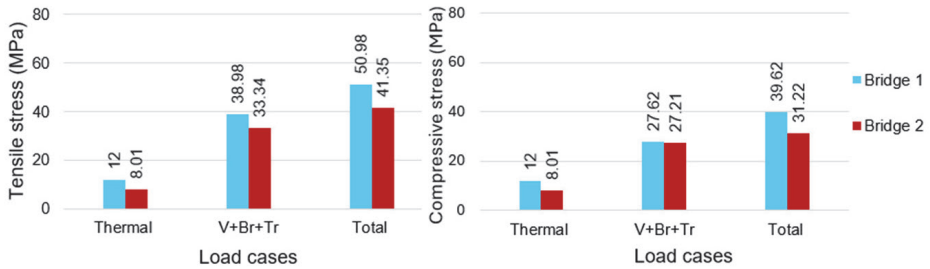


Figure 2 Comparison between Bridge 1 and Bridge 2 for fixed support condition under (a) Tensile stress (b) Compressive stress

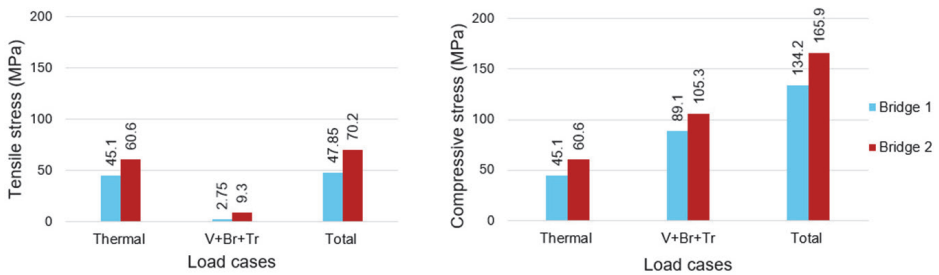


Figure 3 Comparison between Bridge 1 and Bridge 2 for free support condition under (a) Tensile stress and (b) Compressive stress

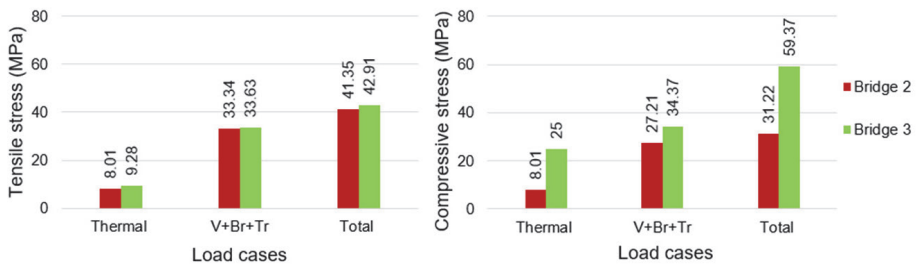


Figure 4 Comparison between Bridge 2 and Bridge 3 for fixed support condition under (a) Tensile stress and (b) Compressive stress

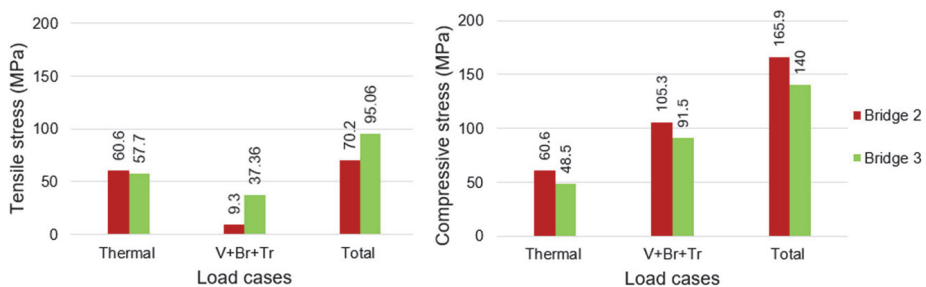


Figure 5 Comparison between Bridge 2 and Bridge 3 for free support condition under (a) Tensile stress and (b) Compressive stress

From Figure 4, it is observed that the total tensile stress and compressive stress generated at the fixed support of bridge 3 is 3.63% and 47.41% higher than that of Bridge 2. Also from Figure 5, the total tensile stress values obtained at the free support of Bridge 2 is 26.15% lesser than that of Bridge-3 and on the contrary, the compressive stress value of Bridge 2 is 15.61% higher than that of Bridge 3.

From figures 2-5, total rail stresses at the free support for all the bridges were found to be exceeding the recommended limit as per Cl.2.8.2.4.3 of IRS Bridge Rules. To be more precise, for Bridge 1 and Bridge 2, even though the total tensile stress for both fixed and free support condition, it is under the permissible limit but the total compressive stress values exceed the recommended limit.

For Bridge-3, total tensile and compressive stresses exceed allowable limits under free support but are within limits under fixed support. Rail stress due to thermal loads differs between fixed and free support conditions due to the bridge's truss and span configuration. The distribution of thermal load is influenced by pier stiffness, potentially causing differential movement. From (Figure 2 and 4), stress reduction due to temperature variation is observed for fixed support, while it increases significantly for free support(from Figure 3 and 5). As shown in Figure (2-5), total tensile stress at free support is minimally impacted by vertical, braking, and traction loads, but compressive stress is significantly influenced by these loads. Tensile stresses from vertical, traction, and braking forces differ greatly between fixed and free support, with larger tensile stress at fixed support due to load transfer to the embankment, while larger compressive stress accumulates at free support.

## 4.2 Displacements in rails

**Table 5** Displacement values for Bridge 1, Bridge 2 and Bridge 3

Bridge name	Bridge 1	Bridge 2	Bridge 3	Max. allowable limit
Longitudinal relative displacements between deck and rail due to braking and traction force				
Max. $\delta_{rel}$ [mm]	1.44	2.44	1.36	4
Horizontal displacement due to deck rotation (vertical bending)				
Max. $\Delta\theta$ [mm]	5.893	11.252	9.26	8
Horizontal displacement of deck due to braking and traction force				
Max. $\delta$ [mm]	1.80	2.92	4.09	5

The maximum displacement values of the rails for various load cases are illustrated in Table 5. From Table 5, it can be clearly observed that only the displacement values obtained for Bridge-1 does not exceed the maximum allowable limit provided in Cl.1.72 of UIC 774-3R. On the other hand, Bridge-2 and Bridge-3 do not meet the criteria to pass the recommended values stated in the code. From the above discussion, it has become certain that the results of RSI analysis performed for the unballasted steel bridges under consideration do not fulfil UIC recommendations and Addendum / Corrigendum Slip No. 45 of IRS Bridge Rules concerning stresses and displacements due to RSI. To optimize bridge durability and minimize structural damage risks from high stress loads, several preventive measures can be taken:

- **Single-Side Switch Expansion Joint:** Install on one side of the bridge to accommodate thermal expansion and contraction, ensuring proper load distribution.
- **Dual-Side Switch Expansion Joints:** If stresses persist after installing a single joint, place joints on both ends to dissipate stress across the span, enhancing overall structure durability.

- **Rail-Free Fasteners with Switch Expansion Joint:** Use rail-free fasteners alongside SEJs to further alleviate stress distribution issues. This approach reduces stress transfer from track to bridge during construction, improving overall performance and longevity.

## 5 Conclusion

Based on the RSI analysis conducted for the present study, the following conclusions can be drawn:

- By making some changes in the bearing arrangement rather than changing the dimensional values of the structure or applying SEJ which are very much costly in practicality, the stresses as well as the displacement values of the rails can be brought below the permissible limit for both single span and multi span bridges.
- It has been observed that provision of LWR accumulates the axial compressive stresses generated due to live loads in the free/ movable end of the rail significantly. Hence, it becomes necessary to perform RSI analysis to check whether the axial stresses in the rail are within the permissible stress limits or not. Also, by installing additional expansion devices such as SEJ, the strains in rail can be reduced.
- The temperature variation of the deck doesnot contribute much additional stress at the integral part of the structure as the RSI effect is counteracting the rail stresses caused due to temperature change in the rail.

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