Rail Structure Interaction Analysis of Steel Composite Metro Bridge

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Abstract - Continuously welded rail (CWR) with the track directly fixed on concrete deck is used for most elevated metro bridge structures. The interaction between the CWR and elevated metro structure due to temperature variation, train loadings like braking/traction and vertical loading takes place through directly fixed rail fasteners, which have a non-linear force-displacement relationship. According to UIC 774-3R parameters that influence this interaction includes: Type of superstructure, Span length, Expansion length, Bending Stiffness of the Deck and Support Stiffness. This paper presents rail structure interaction analysis of steel composite metro bridge and case study of an under-construction metro bridge project D.N. Nagar – Mandale, Mumbai. The total length considered for the present study is 743.43 m out of which 260.43 m is steel composite bridge portion and remaining portion are of U-Girder and PSC girder superstructure span as a boundary condition. A three dimensional (3D) finite element analysis was carried out using the software SOFiSTiK. For present case study results are represented in the form of axial rail stresses along the length of the bridge and checked with UIC 774-3R permissible limits.

Key words: Rail structure interaction; Continuous welded rail; Metro bridge; UIC 774-3R

1. INTRODUCTION

A continuously welded rail (CWR) track has been extensively used many advantages. Compared with a jointed rail track, CWR reduces the maintenance cost of the track, increase the riding quality and increases the service of track components. The main components of CWR track bridge is elastic CWR, elastic rail fasteners which attach the rails to the concrete plinth on the deck of the bridge, superstructure, elastic bearings which supporting the superstructure on the substructure and substructure including foundations. Relative displacement of the rails and structure of the bridge caused by the temperature variation, braking and vertical loading of train. Due to this track bridge interaction phenomenon results in additional stresses to be generated in structure and therefore in rails. Mainly interaction between rails and structure take place through the rail fasteners, which have a taking nonlinear stiffness law under consideration.

CWR directly connected to deck of the bridge by fasteners, since rails is not able to expand or contract when temperature variation occurred in bridge. Temperature increases above the rail installation temperature causes compressive forces in rails that would buckle the rail and temperature decrease below the rail installation temperature causes tensile forces in the rails that would be break the rail. Also, due to rail and structure interaction phenomenon, transverse and longitudinal shear force has been developed at bearing level that should be considered for design of foundation and substructure. This paper focus has mainly on the rail structure interaction analysis of steel composite bridge, for which analysis have been carried by SOFiSTiK software considering the nonlinear spring for rail and deck connection and shear force on pier which need to be consider for substructure and foundation design.

2. LITERATURE REVIEW

The investigations on CWR forces and their influence on the design of structures have been in discussion for the last 30 years at national and international levels.

Longitudinal forces in continuously welded rails on bridge decks due to nonlinear track bridge interaction (2006) illustrates the longitudinal stresses generated in continuously welded rails on the railway bridge. Longitudinal loads are caused because of braking action of railway, the uniform temperature change of bridge as well as a sudden change of ballast stiffness at a moment when train reaches the bridge. Based on study results it was found that longitudinal stresses obtained by conventional separate treatment of loading case are higher compared to stresses calculated using proposed correct combination of loading cases. Based on results it was concluded that in certain situations expansion devices are not required. Maximum compressive rail stresses were considerably reduced because of proposed truly nonlinear simulation

Longitudinal track bridge interaction for load sequence 2008) illustrates that results obtained from statical approach are sufficiently accurate and change of coupling stiffness results in increase of largest compression force by 10% is realistic. Author also found that multiple unloading-loading-unloading track after seasonal temperature there is increase linear elastic part along track bridge coupling interface.

Nonlinear rail structure interaction analysis of an elevated skewed steel guideway (2011). This paper focused on determination of rail break gap value and quantifying rail axial stresses and bearing forces and their distribution along length of bridge. Based on result it was found that bearing transferred negligible amount of lateral forces to substructure. Based on study it was found that 3-D modelling give broad insight into RSI forces. Author concludes broken rail gap must be less than stipulated in design criteria.

3. OBJECTIVES OF PRESENT STUDY

This study focuses on following objectives,

i) To carry out the RSI analysis of under construction steel composite bridge for Mumbai metro line 2B.

ii) To find out the additional stresses in rail and longitudinal force at bearing level.

iii) To check the additional rail stresses is under permissible limit given by UIC 774-3R.

4. BRIDGE STRUCTURE OF PRESENT STUDY

The Mumbai Metro Line-2B Bridge from D.N. Nagar to Mandale has been used as a basis for this study. The configuration of the bridge is a two-track, multi-span simply supported with steel composite girder superstructure. Adjacent spans of main steel composite bridge are U- Girder, Pretension Prestressed Girder and Post Tensioned Prestressed Girder superstructure. Each track structure consists essentially of two parallel rails that are directly fixed to the deck. The rails are attached to the deck by fasteners, which are placed at fixed equal intervals along the length of the track. The reinforced concrete (RC) substructure members are circular type piers at U girder superstructure and rectangular type piers at steel composite and PSC superstructure are present. Foundation consists of a circular reinforced concrete group of piles. The total length considered for the present study is 743.43 m out of which 260.43 m is steel composite bridge portion and remaining portion are of U-Girder and PSC girder superstructure span.

The superstructure considered for the present study consists of a total 23 numbers of span with varying span length and substructure is considered with varying height as provided in general arrangement drawing. The curvature of the bridge is considered to measure special effect of curvature on considered steel composite bridge. The U girder and Pretensioned girder superstructure is supported by elastomeric bearing and steel composite and post tensioned girder superstructure supported by POT-PTFE bearing. Between two successive girder expansion gap of 50 mm is provided. There are four number of piles of each 1m diameter and circular pier of 2m diameter is used under the standard U girder spans. And six number of piles of each 1 m diameter and 2.2 x 2.4m rectangular pier is used under special span.

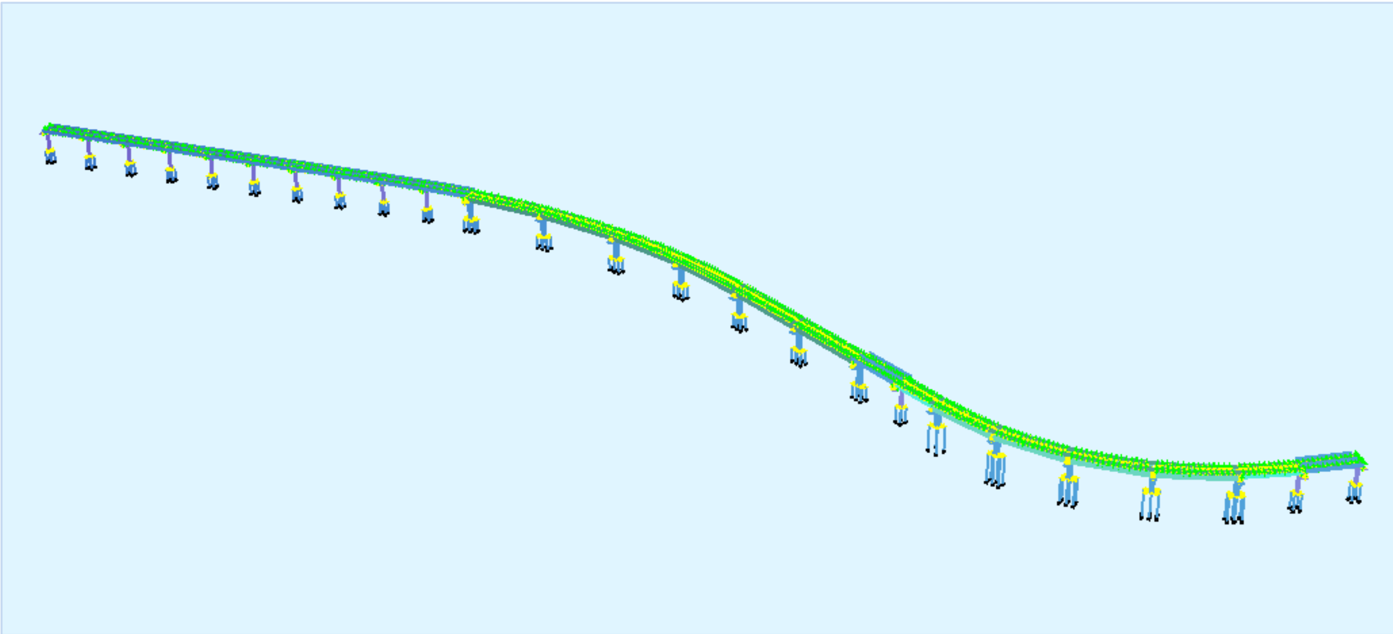


Figure 1: Perspective view of Bridge

5. NUMERICAL MODELING OF TRACK BRIDGE INTERACTION

To carry out numerical studies RSI of steel composite metro bridge, a numerical model has been created and the analysis are carried out using this model. The model created for the rail structure interaction has been shown above in Fig. 1.

The numerical model is developed to simulate the rail structure interaction using the software SOFiSTiK which is based on stiffness approach. The bridge and rails are modeled using the beam elements and the connection between the two is modeled by nonlinear springs. The experimental data enabled to idealize the behavior by means of the adoption of a bilinear elasto-plastic law, characterized by the maximum value of the frictional force and the value of the displacement of yield u0. As per UIC 774 -3R,

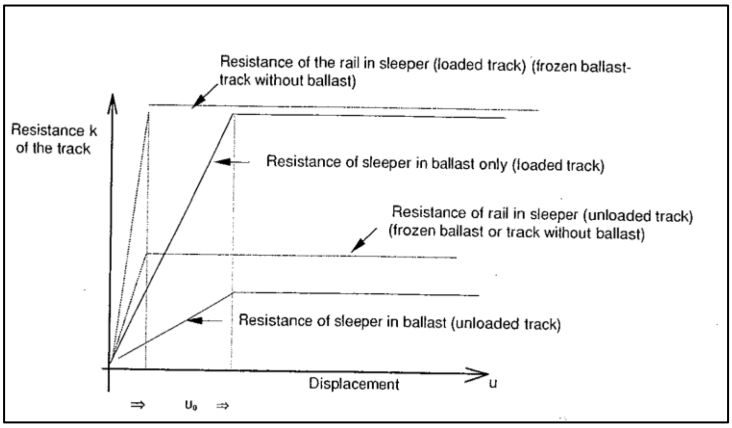


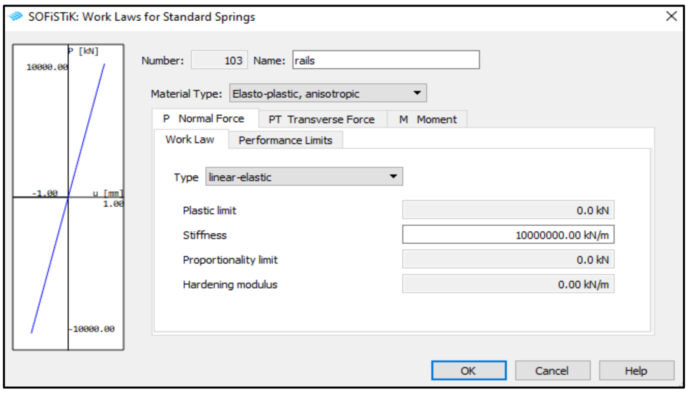
Figure 2: Force-displacement interaction law between track and deck

Displacement between elastic and plastic zone
u0 = 0.5 mm
Resistances k per unit of length for one track in the plastic zone:

- ktr = 40kN/m for unloaded track (80000 kN/m/m)

- ktr = 60kN/m for loaded track (120000 kN/m/m)

Using the above force displacement diagram, connection between superstructure and rail is modelled as a spring with bilinear elasto-plastic behaviour in transverse direction with stiffness as shown in below fig. 3 and fig. 4



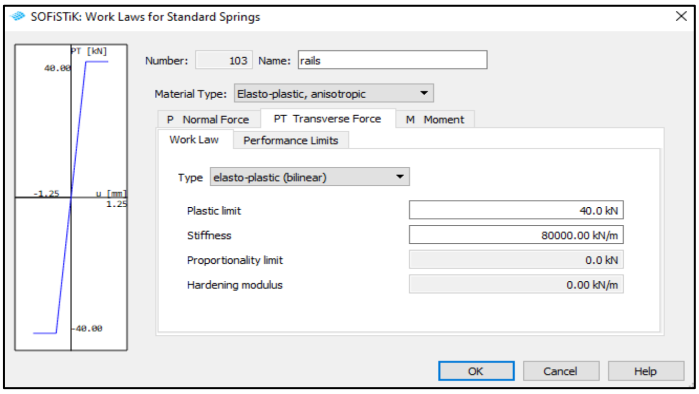


Figure 3: Definition of normal force of rail spring in SOFiSTiK Figure 4: Definition of trans. force of rail spring in SOFiSTiK

The rail is connected to deck using rigid link and spring as defined above. Typical connection of the rail and U girder superstructure are shown below in fig. 5

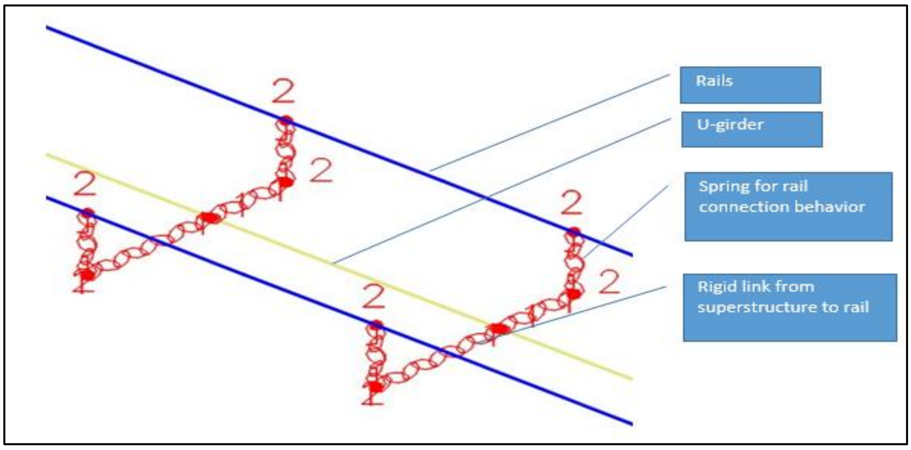


Figure 5: Typical connection of rail and U girder superstructure

The bearings are modelled using spring. In this study two types of bearing have been used, first type of bearing is POT PTFE fixed and guided bearing used below steel composite and special span superstructure and second type of bearing is elastomeric bearing which is used below standard U girder and I girder superstructure. Bearing is defined in SOFiSTiK using work laws.

The pier cap has been modelled as beam element. The modelling has been done at the top of the pier cap and the pier cap is connected to the pier using rigid links up to depth of pier cap. And also, superstructure connected to pier cap using rigid links and springs to simulate the offset and elastomeric / Pot-bearings.

The piles are modelled as beam element, and fixity level is considered at 6m below pile cut off level. The piles are connected to pier center using rigid links to simulate the rigid pile cap. 4 piles per pier have been considered for standard spans and 6 piles have been considered for steel composite and special span in the analysis. The spacing of the piles has been considered as 3m in transverse direction and longitudinal direction. For simplicity, the same foundation has been considered for all piers.

6. LOADS AND ACTION

6.1 Load case 1 - Thermal effects in the combined structure and track system

a. Rail

As per UIC 774-3, in case of CWR a variation in the temperature of the track does not cause a displacement of the track. Thus, there is no interaction effect due to the variation of the temperature in the track.

b. Superstructure

The temperature load on superstructure has been applied as per clause 1.4.2 of UIC 774-3R, which is ± 35°C.

6.2 Load case 2 - Live load effects on rails

As per clause 1.4.3 of UIC 774-3R, loads from only two tracks are to be considered for the analysis. The same clause also specifies that braking on one track and traction on second track shall be considered.

a. Longitudinal effects

Braking and traction effects are applied along with the moving load definition in SOFiSTiK. To be on conservative side both braking and traction are considered as 20% of the vertical axle load.

b. Vertical effects

Traffic loads cause a bending in the deck. This introduces a rotation of the end sections and displacement of upper edge of deck. The effect of this rotation on the rails is to be evaluated. A single 6 coach train with 4 axles per coach as defined in DBR has been considered for the analysis. The fig. 6 below shows a vertical load of a single car of metro rail. Mumbai metro has six number of cars; each car has a length of 22.1 m and a load of 680 kN

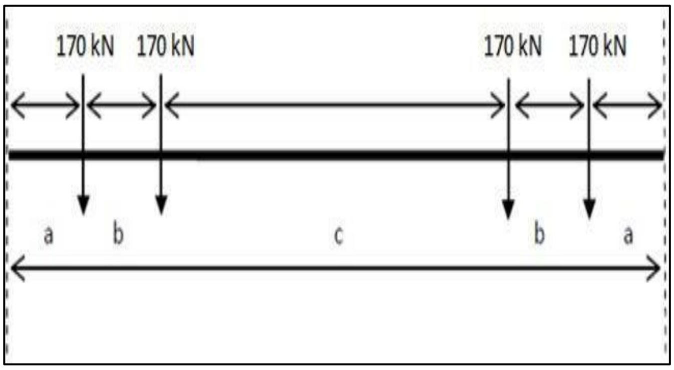


Figure 6: Vertical load of train

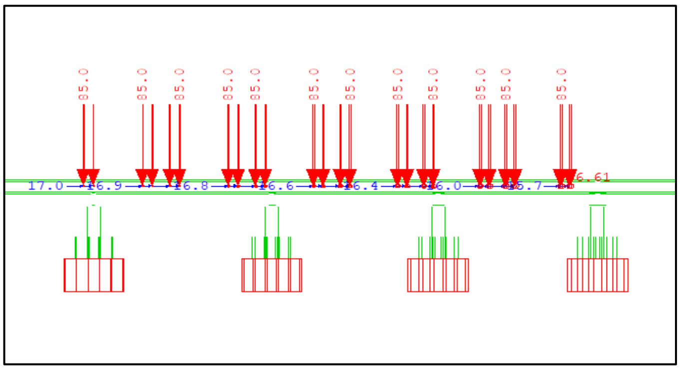


Figure 7: Vertical, braking and acceleration load applied in SOFiSTiK

6.3 Combination of actions and conditions to be checked

As per clause 1.5.1. of UIC 774-3R, the load factor for all loads shall be 1.

a. Check for additional stresses in rail

As per clause 1.5.2 of UIC 774-3R, for ballast less track, the additional compressive and tensile stresses in rails due to temperature variation of the deck, barking/acceleration and deck end rotation shall be less than 92 MPa. For this check, the results of load case 1 and 2 will be combined without any load factor.

7. RESULTS AND DISCUSSION

In this case study the Rail Structure Interaction analysis for Steel Composite Mumbai Metro line 2B rail bridge is performed, and results are represented for individual load. Results for effect of rail continuity on forces transferred to substructure is also represented.

7.1 Additional stresses in rail due to temperature variation

Axial force in rail due to temperature variation are shown in below figures.

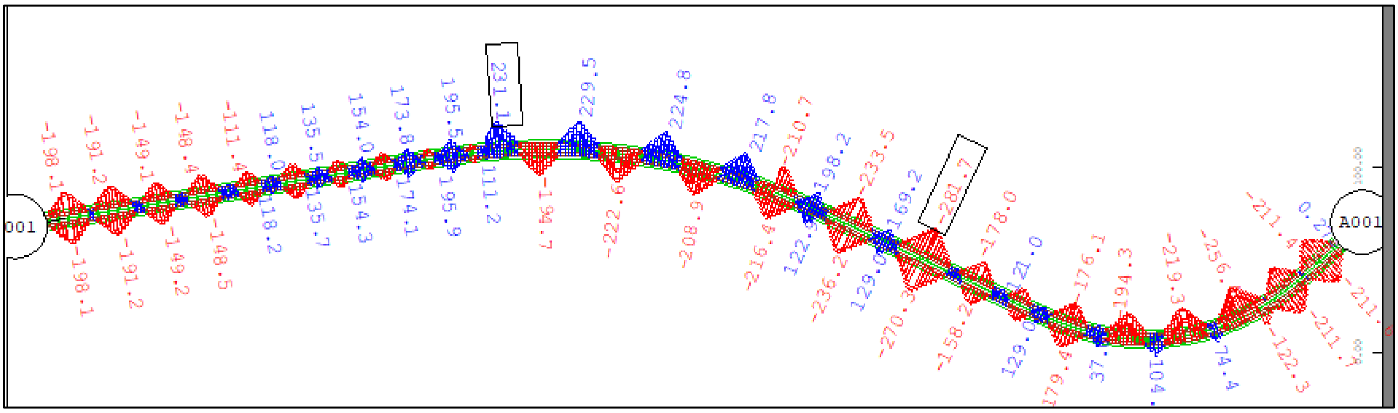


Figure 8: Axial tension force in rail due to temperature variation

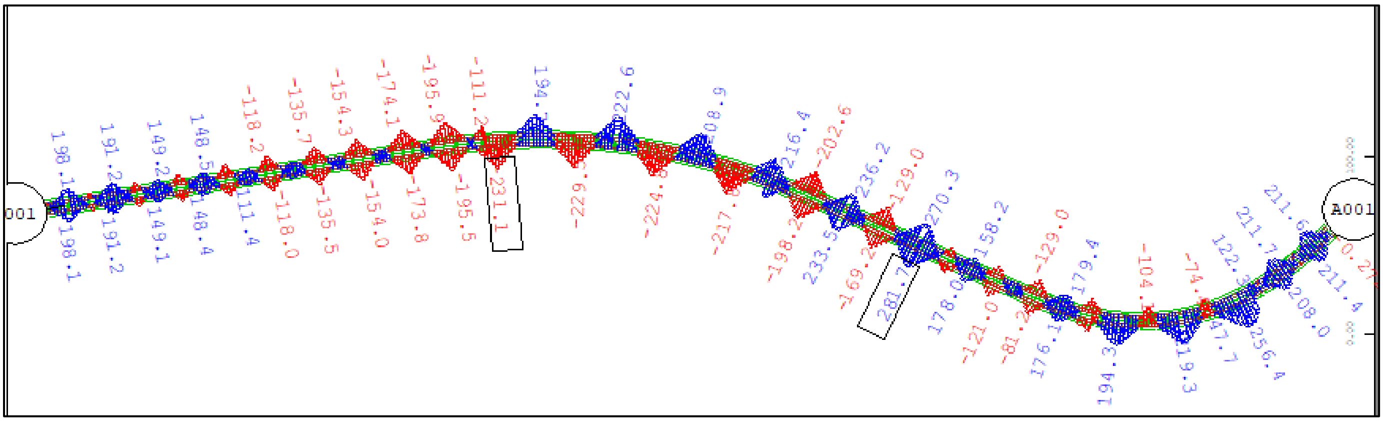


Figure 9: Axial compression force in rail due to temperature variation

The maximum axial tension force in rail due to temperature = 231.1 kN

Therefore, the tensile stress in rail due to temperature

$$= \frac{231.1 \times 1000}{7670}$$

$$= 30.13 \text{ MPa} \quad \text{- Tension}$$

The maximum axial compression force in rail due to temperature = -231.1 kN

Therefore, the compressive stress in rail due to temperature

$$= \frac{-231.1 \times 1000}{7670}$$

$$= -30.13 \text{ MPa} \quad \text{- Compression}$$

7.2 Additional stresses in rail due to live load

The critical position of live load for the governing forces in rail for steel composite span is shown below fig 10. The critical case is when both tracks of the main line are loaded. This load takes into effect the longitudinal force due to braking / traction and rotation of span due to vertical loads.

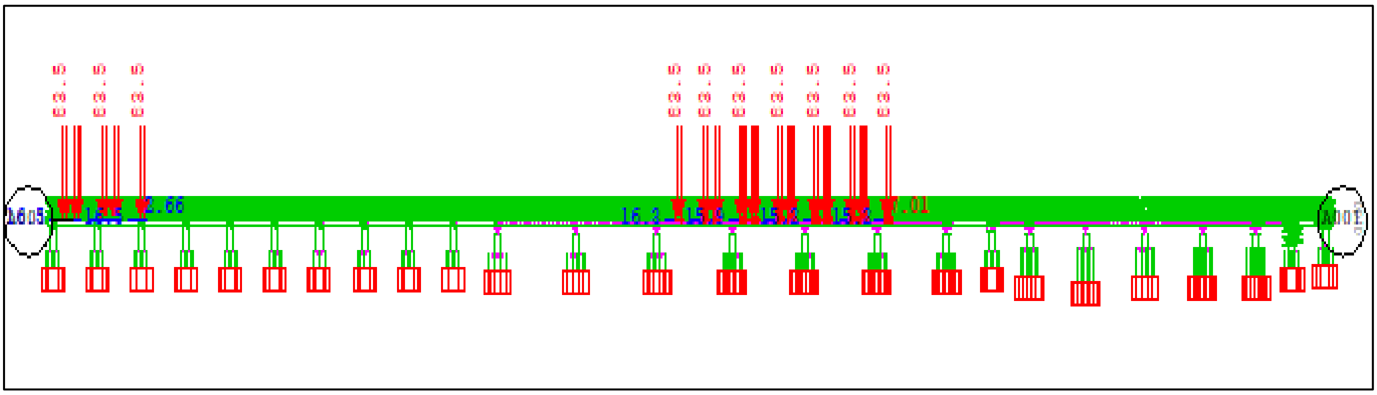


Figure 10: Critical live load position for steel composite span

The axial force in rail due to braking/traction and vertical live loading are shown in the below figures.

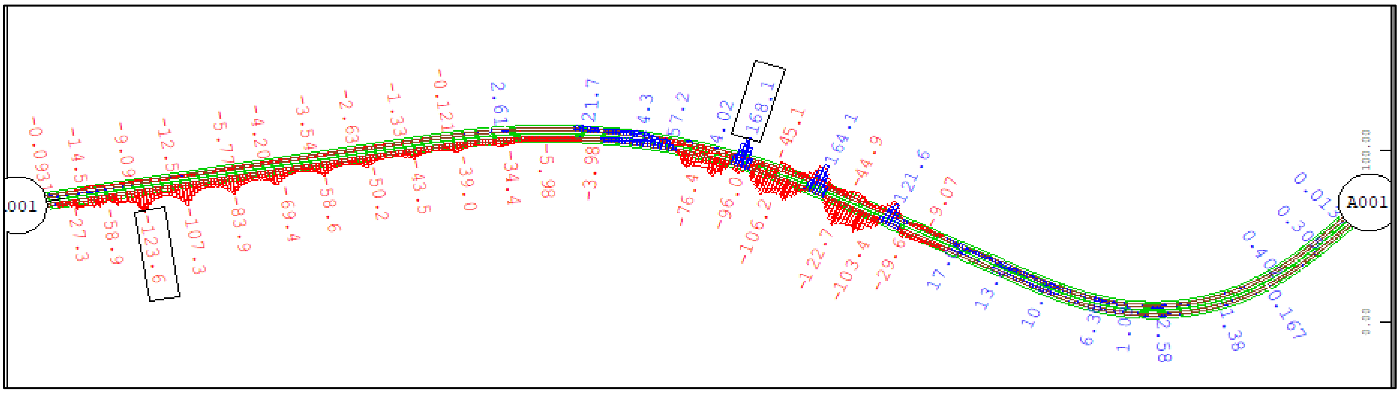


Figure 11: Axial tension force in rail due to live load

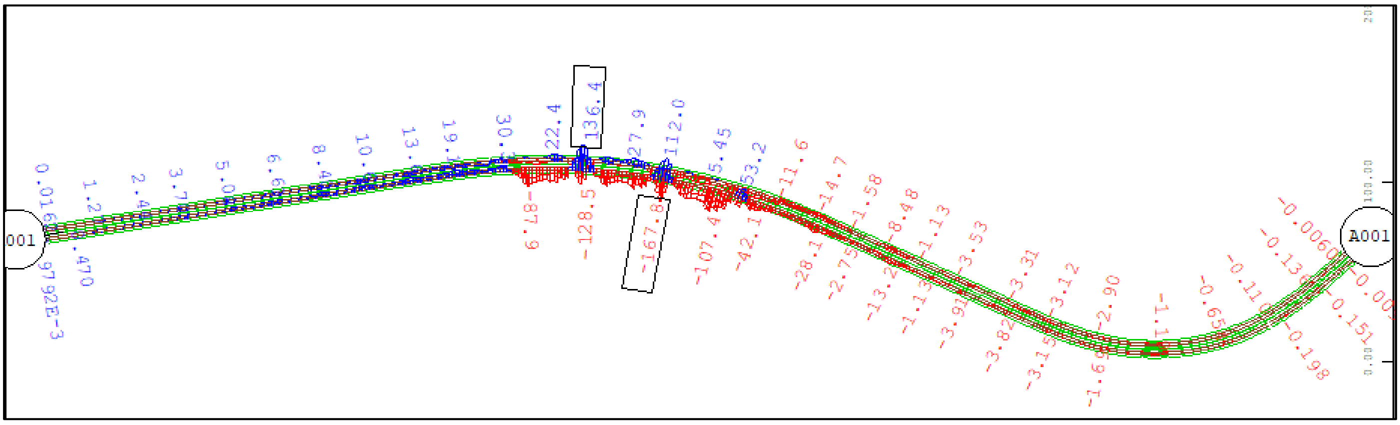


Figure 12: Axial compression force in rail due to live load

The maximum axial tension force in rail due to live load = 168.1 kN

Therefore, the tensile stress in rail due to live load $= \dfrac{168.1 \times 1000}{7670}$

= 21.91 MPa - Tension

The combined tensile stresses in rail due to Temperature & Live load = 30.13 + 21.91

= **52.04 MPa < 92 MPa**

The maximum axial compression force in rail due to live load = - 167.8 kN

Therefore, the compressive stress in rail due to temperature $= \dfrac{-167.8 \times 1000}{7670}$

= - 21.87 MPa - Compression

The combined compressive stresses in rail due to Temperature & Live load = - 30.13 - 21.87

= **- 52.00 MPa < 92 MPa**

Table 1: Axial rail stresses for present study

Load	Temperature	Braking / Traction and Vertical live load	Total
Tensile Stress MPa	30.13	21.91	52.04
Compressive Stress MPa	- 30.13	- 21.87	- 52.00

RSI analysis for present study shows that combined axial tensile stress in the rail is 52.04 MPa and combined compressive stress in the rail is 52.04 MPa. Which are within the permissible limit given by UIC 774-3R

7.3 LWR FORCE TRANSFERRED TO SUBSTRUCTURE

For the applied temperature loads as defined in load action point no. 6, the shear force in each pier for considered span for RSI have been summarized below. The LWR force to be considered for analysis shall be the shear force obtained divided by the contributary length of superstructure for that particular pier

Table 2: LWR forces transferred to substructure

Pier No	Span Length		Contributary length	Shear Force (kN)	Force per meter (kN/m)
	Preceding	succeeding			
P465	25m	25m	25m	113.4	4.54
P466	25m	25m	25m	95.3	3.81
P467	25m	25m	25m	82.4	3.30
P468	25m	25m	25m	74.5	2.98
P469	25m	25m	25m	70.4	2.82
P470	25m	25m	25m	70.7	2.83

P471	25m	25m	25m	74.7	2.99
P472	25m	25m	25m	84.5	3.38
P473	25m	25m	25m	101.2	4.05
P474	25m	43.5m	34.25m	29.4	0.86
P475	43.5m	43.43m	43.46m	12.4	0.29
P476	43.43m	43.5m	43.46m	17.1	0.39
P477	43.5m	43.5m	43.5m	12.9	0.30
P477A	43.5m	43.0m	43.25m	63.3	1.46
P478	43.0m	43.5m	43.25m	129.3	2.99
P479	43.5m	28m	35.75m	244	6.83
P480	28m	23m	25.5m	17.3	0.68
P486	23m	33m	28m	38.9	1.39
P487	33m	33m	33m	101.2	3.07
P488	33m	33m	33m	118.6	3.39
P490	33m	33m	33m	99.3	3.01
P491	33m	25m	19.33m	33.6	1.74
P492	25m	25m	25m	8.43	0.34

The maximum LWR force generated in the pier per meter running = 6.83 kN/m
The maximum force per track is = 3.41 kN/m

In the design of substructure and foundation, the LWR force considered is 4 kN/m per track

8. CONCLUSION

1. Rail Structure Interaction analysis for this case study shows that combined axial compression and tensile rail stresses due to temperature variation and braking/traction and vertical live loads are 52.00 MPa and 52.04 MPa respectively. This rail stresses are within the permissible limit given by UIC 774-3R. Hence no need to provide expansion devices in rail. If stresses exceed permissible limits given by UIC 774-3R, these stresses are reduced by providing either expansion devices or changing the other parameter like, bearing arrangement, deck expansion length.

2. Rail and structure interaction for different action of loading, transferred horizontal forces (LWR) to substructure which is 3.41 kN/m per track. This LWR force should be considered for the design of substructure and foundation.

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