RDSO GUIDELINES FOR CARRYING OUT RAIL-STRUCTURE INTERACTION STUDIES ON METRO SYSTEMS

Ver 2

Provisions with Commentary

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RESEARCH DESIGNS AND STANDARDS ORGANISATION, LUCKNOW

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Provisions with Commentary

Developed for
Indian Railways

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PREFACE

Where track alignment is on a viaduct, which is especially true in major portions of metro-rail transit system, the movements and rotations of the supporting structure can impact significantly on the track form design, and the limitations posed by the safe performance of track can impact significantly the design of components of viaduct.

There is pressure on the viaduct designer from the owners to offer the most cost effective structure, which usually means slender sub structure elements and long, continuous spans. But there is also conflicting pressure from the permanent way designers to allow continuously welded rails over the viaduct and eliminate the need for joints/rail expansion switches to the maximum extent possible. This latter often means that the viaduct elements must have certain minimum stiffness. Thus design of the viaduct/track form for metro systems requires extensive interaction between the viaduct designer and the track designer to give a safe and optimum solution. The viaduct designer is called upon to provide ingenious solutions, which requires deep understanding of the Rail-Structure interaction phenomenon and the various options available at his disposal to address the concerns of track design engineers.

As there are no Indian standards which cover the problem of track-structure interaction, difficulties were being experienced in carrying out proper RSI studies for metro systems in India. To address this issue, these guidelines have been written which attempt to explain the various dimensions of the effect of continuous welded rails (LWR/CWR) on the viaduct structure and vice versa under variety of circumstances actually encountered during design of metro viaducts. This document is based on UIC leaflet 774-3R 2001. Attempt has been made to guide designers in use of UIC leaflet and to explain the importance of various checks. For cases not covered by the UIC leaflet, other codes or technical literature available on the subject have been referred to provide complete guidance to the designers. Hope these guidelines will be able to fulfill the needs of design engineers regarding carrying out Rail-Structure Interaction studies for metro systems.

Feedback/ suggestions/ questions on issues regarding these guidelines may kindly be directed to directorsteel2@gmail.com.

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RDSO GUIDELINES FOR CARRYING OUT RAIL-STRUCTURE INTERACTION STUDIES ON METRO SYSTEMS

Provisions with Commentary and Explanatory Examples

Part 1 – PROVISIONS & COMMENTARY

PROVISIONS

1.0 INTRODUCTION

1.1 GENERAL

The purpose of these guidelines is to define the methodology for carrying out the Rail Structure Interaction (RSI) to be considered on metro viaducts. The guidelines provide a basis for carrying out RSI studies and thus to work out the forces induced in rails and bridge components due to the interaction effects and to assess if the arrangement will be safe under the interaction effects. The interaction takes place due to expansion/ contraction of deck/rails under change of temperature, longitudinal deformation of sub structure under braking / tractive forces from rolling stock and vertical bending of deck under vertical live loads.

The present guidelines are formulated as an extension of UIC 774-3R, for facilitating the adoption of the leaflet to the metro specific scenario in India. Certain aspects of RSI on which UIC 774-3R is silent have been explained and provisions available in other codes like European codes, Spanish National codes or Korean codes etc have been recommended for use.

1.2 ADAPTATION / MODIFICATION TO EXISTING RULES

At present, there is no Indian code which covers the RSI phenomenon. These guidelines present the methodology to be followed for RSI analysis on metro systems in India.

These guidelines are meant to supplement IRS Bridge Rules and other codes/specifications already in vogue for design/ detailing of the viaducts on metro systems.

1.3 RELEVANT CODES & STANDARDS

- Rules specifying the loads for design of super-structure and sub-structure of bridges and for assessment of the strength of existing bridges (**Bridge Rules**)
- **UIC 774-3R** October 2001: Track/Bridge Interaction – Recommendations for calculations.

COMMENTARY

C1.1

UIC Leaflet 774-3R 2001 is the basic code on which subsequent codes have evolved. The UIC leaflet is based on earlier research on the related phenomena. The leaflet describes methodology to be adopted for carrying out interaction studies, based upon numerical methods that idealize the behavior of all the elements and actions involved for the computation of stresses and displacements. Specific clauses of other codes, wherever used, have been mentioned in the commentary.

C1.2

These specifications cover only one aspect, namely RSI, and do not specify the loads to be used for design, and other checks required for stability/ safety of the structure. There are other closely linked phenomena like dynamic analysis and the vehicle – track – bridge interaction etc which are not covered in these guidelines.

C1.3

PROVISIONS

- IRS Code Of Practice For Plain, Reinforced & Prestressed Concrete For General Bridge Construction (**IRS Concrete Bridge Code, 19**)
- **UIC 776 2R**: Design requirements for rail bridges based on interaction phenomena between train, track and bridge.
- **Korean Design Standard:** Railway Design Manual (*Volume* Track)
- Eurocode 1: Actions on structures — Part 2: Traffic loads on bridges (**EN 1991-2 – 2003**)
- **TCRP report 155, 2012:** Track Design Handbook for Light Rail Transit, second volume,

COMMENTARY

2.0 SCOPE

These guidelines explain the interaction phenomenon, parameters affecting RSI, provide guidance on choosing representative stretches for conducting RSI, methodology to be adopted for carrying out RSI, special cases in RSI, use of computer programs for carrying out RSI and options available for modification in track if the RSI results indicate excessive stresses/ deformations.

These guidelines cover steel/concrete bridges with simply supported or continuous spans, whether on straight or curved alignment and whether level or on gradient having any type of bearings on metro systems in India. However, these guidelines do not cover the bridges with long spans/special geometry such as cable stayed bridges, Bow-string girders etc where the typical structural behaviour of spans affects RSI phenomenon requiring specialized studies to be carried out or where span arrangement necessitates excessive movement in track which is beyond the capacity of a typical track Expansion Joint to accommodate.

C2.0

Different bearings/ bridge forms need to be modelled such as to reflect their actual behaviour under the RSI phenomenon. In structures like cable stayed bridges, the flexibility of deck and non-linear response due to presence of cables supporting the deck require more complex models which capture all phenomenon accurately. These are beyond the scope of these guidelines. Similarly, for large movements in track, specialized solutions are required to be worked out. These are site-specific solutions, and track experts are required to study the site conditions and design these arrangements.

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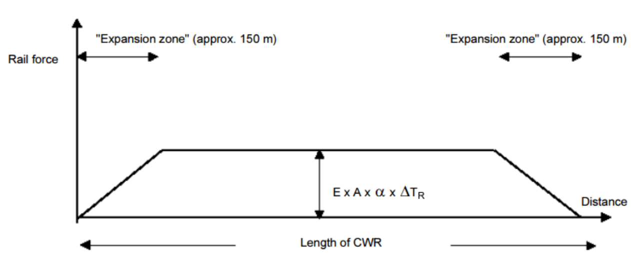
3.0 GENERAL CONCEPTS

This section of the code deals with the general concepts describing the RSI phenomenon covering the effect of train loads (vertical as well as longitudinal) and the effect of thermal changes.

3.1 INTERACTION PHENOMENON

In jointed track, the analysis of effect of various loads on rails and on bridge is carried out separately. However, this type of analysis is not appropriate when the continuously welded rails (which restrict the free movements) are laid on the structure because then the track-structure interaction shows non-negligible effects.

The presence of deck under the tracks induces extra stresses in the rails due to interaction phenomenon and this affects stresses in bridge components also. The extra stresses in the rail are induced by thermal expansion/ contraction of bridge deck(s)/tracks, deflection of sub-structure under tractive/ braking forces from the trains and the end rotations caused by vertical bending under vertical train loads. The magnitude of these extra stresses in the CWR mainly depends on the stiffness of various elements of bridge, resistance offered by the track structure to deformation and the boundary condition of rails (i.e. whether these are continuous or have expansion joint(s) in between). The RSI describes the effects, under various loads, of structural collaboration of rails and bridge by means of their connection elements.



Distribution of rail stresses along length of an LWR

COMMENTARY

C 3.0

The long term phenomena like effect of dead loads, deformation of deck under creep/ shrinkage etc are considered to be dissipated during various track maintenance operations and hence are not considered while carrying out the RSI studies.

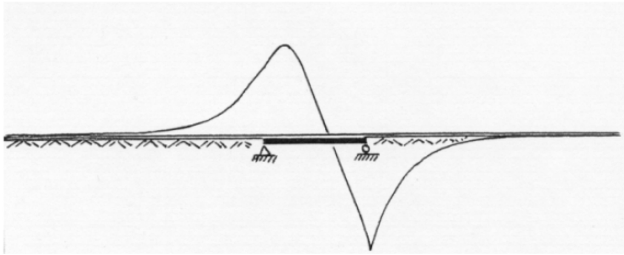
C 3.1

For rail-free fastenings also, the track and bridge move independently, so there is no interaction between these. Hence, there is no requirement for carrying out RSI studies.

The purpose of RSI analysis is to examine these extra stresses in rails due to the actions of temperature change, braking / traction of rolling stock combined with the vertical bending caused due to live loads. These stresses are required to be kept within allowable limits so that the track is safe under tension as well as compression, and the bridge elements are to be proportioned to take all the loads.

If the RSI study indicates extra stresses in rails beyond permissible limits, these can be brought within limits by altering either the stiffness of the structure and/or the fixing arrangement of the rails to the bridge structure and/ or introducing expansion joints.

PROVISIONS



Distribution of rail stresses affected by presence of bridge in non – breathing length due to change in length of bridge deck when temperature changes

The difference between LWR/CWR on ground vis-à-vis LWR/CWR on bridge is that a bridge has lesser stiffness which results in its deformation under various loads/ thermal effects. **The track is supported on the bridge and has to respond to these movements. But the rails, being continuous, are not free to move and resist these movements, which induces loads in them. These loads cause the track as a whole to move, which relieves part of the loads, which are transferred back to the structure. The final deformations/ stresses in track and viaduct depend on this interaction, which is basically governed by the stiffness of track and that of the bridge.** This interaction between track and the bridge structure is studied as RSI effect.

The RSI phenomenon, as explained above, is non-linear which can only be solved by an iterative procedure to get a solution that satisfies all boundary conditions. There can be no formula to be directly used to determine the stresses or deformations etc. The results can be obtained by two ways: we can use the charts given in the UIC 774-3R or we can model the bridge, track and approaches and find out the results using Finite Element Method based computer programs which try to solve the non-linear problem through convergence of results through iterations. The relative stiffness's of different elements like track, deck, sub structure and bearings play important role in determining the results. The designer has to change the stiffness's/ arrangement to optimize the performance and costs.

COMMENTARY

The interaction phenomenon can be summed up as the interplay of stiffnesses. The component which is stiffer will attract more stresses. If the sub-structure is flexible, it will move under loads and the rails will be subjected to higher stresses, which can be unsafe for the train operations. Quite often the purpose of RSI is to ensure that the bridge is stiff enough such that the track is safe.

3.2 PARAMETERS AFFECTING RSI

3.2.1 Expansion Length

Expansion length is defined as the distance between the thermal center point and the opposite end of deck.

C 3.2.1

For a series of simply supported decks with one end having fixed bearing and other end having free

PROVISIONS

In simple terms, this means it is the length over which structure is allowed to expand/ contract by the supports. Free/ moveable bearings allow expansion/ contraction to take place whereas fixed bearings do not allow the same. Expansion length depends on the type of support configuration adopted in a structure. Expansion length is defined and indicated on different type of structures in para 1.1.3.1 of UIC 774-3R 2001.

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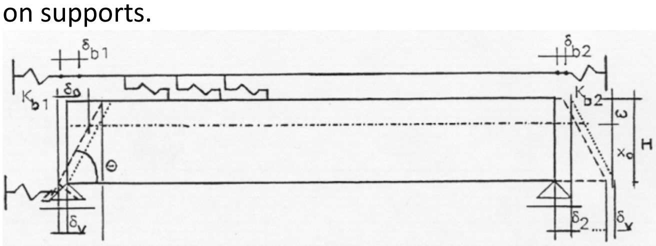
bearing, expansion length is the distance between the fixed bearings. For continuous spans having a fixed bearing somewhere in the middle, there are two expansion lengths, one on each side. For succession of decks, the expansion length at a joint is equal to the sum of expansion lengths of nearest two spans. However, if the structure does not have fixed bearings and arrangement has neoprene or sliding bearings, the expansion length has to be worked out between thermal center-points (i.e. points which will not move under thermal effects) on the deck.

Para 1.3.1 (Figure 6) in UIC 774-3R 2001 indicates expansion length for different types of support configurations commonly adopted in bridges.

3.2.2 Span Length

The vertical train loads cause rotation at ends of decks. Since the rails are not fitted at the neutral axis of the deck, the length of fibers at the track level changes under these loads. This leads to longitudinal displacement, and thus, stresses in tracks which depends on the magnitude of load as well as span length. The span length is measured as center to center of bearings on supports.



Schematic indication of longitudinal displacement of deck fibres at rail level under vertical train loads (The springs indicate restraints due to track and supports)

C 3.2.2

The center to center distance (or effective span) between supports is the span length for this purpose. As against this, normally the expansion length for simply supported spans is overall length of girder. To simplify the computations, sometimes, analysis is done considering overall length for both.

3.2.3 Bending Stiffness of Deck

The bending stiffness of each deck is required to calculate the longitudinal deformation effects of the structure under the vertical loading by rolling stock.

C 3.2.3

3.2.4 Deck Height & Rotation Distance

The change in length of the deck fibers supporting rails is affected by the distance between deck level supporting track and the neutral axis of the deck which is

C 3.2.4

The longitudinal displacement of deck is described in clause 1.3.3 of UIC 774-3R 2001. It is evident that if the track is supported nearer to neutral axis of

PROVISIONS

termed as rotational distance and the total distance from bearing to the top of deck, called deck height.

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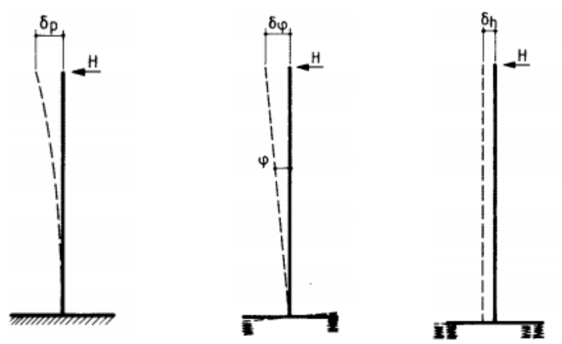
girder, this effect will be lesser, and vice versa.

3.2.5 Support Stiffness

Major source of interaction phenomenon is the stiffness (or, more correctly, flexibility) of bridge supports under longitudinal actions (braking/tractive loads and temperature variations). The longitudinal stiffness of the sub structure depends on stiffness of individual components viz, foundation, sub structure and bearing. Stiffness of different parts should be combined to get total stiffness Ktotal as follows:

$$\frac{1}{\mathrm{K_{total}}} = \frac{1}{\mathrm{K_{pier}}} + \frac{1}{\mathrm{K_{bearing}}}$$

where Kpier is stiffness of each sub-structure (pier/abutment) and Kbearing is stiffness of bearing. Kpier has further components as described in figure below:



a) b) c)

Longitudinal displacement of deck due to a) Elastic deformation of sub structure, b) bending of foundation and c) longitudinal movement of foundation.

C 3.2.5

The effects of longitudinal loads on the substructure is described in clause 1.3.2.2 of UIC 774-3R 2001.
The stiffness of the sub structure, $K_{pier} = H/\Sigma\delta_i$ where δ_i is the deflection of sub structure due to:

1. Displacement due to elastic deformation of sub structure.
2. Displacement due to rotation of support.
3. Displacement due to longitudinal movement of foundation.

All the above displacements have to be worked out at the top of bearing level.

While computing stiffness, for sustained temperature loading analysis, long-term Young's modulus shall be used, whereas for the short-term effects of braking and tractive loading, instantaneous modulus shall be used.

The Young's modulus should be determined as per IRS-CBC-1999 and the Young's modulus for long term effect is normally taken as half the Young's modulus for short term.

3.2.6 Track Stiffness

C 3.2.6

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The track stiffness is a measure of longitudinal resistance offered by the track to longitudinal movement. The track stiffness is dependent on multiple factors like:

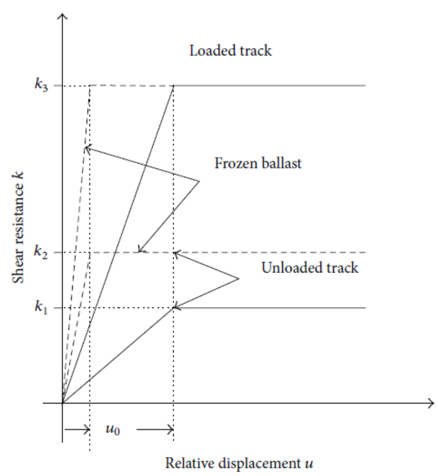
- Type and condition of track structure
- Load on Track
- Maintenance condition of Track

COMMENTARY

In RSI, the movement of bridge under different loads is considered. The stresses induced in the track due to these movements and actual movement of track depends on the interaction effect which is dependent on the track stiffness curve. This is a very important parameter in RSI and can be manipulated by changing the track characteristics to allow movement at critical locations thus ensuring that extra stresses on account of RSI are within limits.

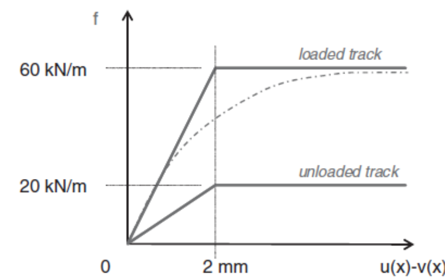
The track stiffness is calculated by the measure of the track deformation. The deformation is in a bilinear curve as suggested in clause 1.2.1 of UIC 774-3R.

Typical track stiffness curve is as follows:

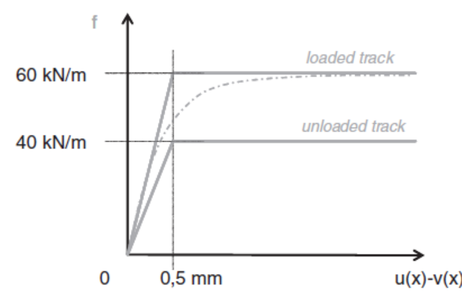


The longitudinal resistance of the track is provided by the fasteners in ballast less track and by ballast in ballasted track is proportional to the displacement of the rail relative to the supporting deck until a relative displacement of u_0 is reached, which corresponds to elastic limit. At this point, the fasteners/ballast cannot resist any further load; the resistance force is constant while the movement continues (plastic shear resistance). The elastic limit is different for frozen and unfrozen ballasts. Analogously to frictional behavior, plastic shear resistance of the ballast is higher when an additional vertical load i.e. train load is applied to the track.

The curve is idealized form of actual behaviour of track.



ballasted track



unballasted track

The elastic limit is 0.5 mm for ballastless track and 2 mm for ballasted track as per clauses 1.2.1 and 1.2.2 of UIC 774-3R 2001. However, the code allows different values to be adopted as per actual track behavior. The track behavior to be considered under longitudinal forces shall be finalized in consultation with track design engineers.

The limiting plastic track resistance given in the clause 1.2.1.2 of UIC 774-3R 2001 for ballasted deck are

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- 12 kN/mm for ballasted track (unloaded) with moderate maintenance.
- 20 kN/mm for ballasted track (unloaded) with good maintenance.
- 60 kN/mm for frozen ballasted track (loaded).

The frozen ballast also acts quite like ballastless track. However, most of places in India do not experience sub-zero temperatures and hence this effect is not important for Indian metros at the moment.

The limiting plastic track resistance given in the clause 1.2.2 of UIC 774-3R 2001 for unballasted deck are

- 40 kN/mm for unballasted track (unloaded).
- 60 kN/mm for unballasted track (loaded).

Other values of plastic track resistance as per actual track behaviour are also allowed by the code.

A note for capturing this behaviour by computer programs: The behavior of track under longitudinal forces is quite complex. When the direction of displacement changes, the ballast behavior becomes elastic again, but the relative displacement from sliding is not recovered. Any computer program to be used for carrying out RSI studies ought be capable of capturing this behaviour realistically as per actual behaviour in field.

For capturing this behaviour properly, a computer program must take into account, for example, that the connector elements that are in the sliding state before applying the live load will return to elastic behavior, while their relative displacement and their connector force will remain unchanged. The implementation of connector elements representing ballast/ fasteners in the interaction phenomenon causes many other complications, including that the activation and deactivation of elements is a function of the presence of load. These cannot be realized in many engineering FEM programs commercially available. This aspect needs to be examined before an FEM program is chosen for carrying out RSI analysis.

3.2.7 Sectional Properties of Rails

The cross sectional area of the rails in track, Young's modulus of the rail-steel and other parameters of rails are required to work out stresses in rails.

C 3.2.7

3.2.8 Temperature variations

The temperature changes induce change in length of deck and/or rails. The decks are almost always (except in

C 3.2.8

Reference temperature for deck is temperature at which the rails are fastened to the deck. The

PROVISIONS

case such as integral bridges) permitted to expand/ contract with change in temperature. If the deck length changes, the interaction phenomenon described above kicks in.

If there are no expansion joints on the bridge or within 100 m either side of the bridge, the rails are in non-breathing length of LWR/CWR and cannot expand/ contract with changing temperature. However, if the expansion joint is provided, the rails can also move and interaction effect of the same has to be studied. In this case, the variation of temperatures of deck and rail from the respective reference temperatures and the difference between temperature of deck and rail has to be considered.

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fastening of rails to deck can be at the time of initial laying or during subsequent distressing/ other maintenance operations. It shall be attempted that the rail be fixed to deck at about the same temperature for which the RSI computations were carried out for the viaduct. Reference temperature for rails is the stress-free temperature of the rail.

3.3 VERIFICATION OF TRACK AND BRIDGE CONFIGURATION THROUGH RSI COMPUTATIONS

Parameters to be verified during RSI studies are the following:

C 3.3

The combined response of track and structure, as studied by RSI, can have unfavorable effects on either the bridge structure or the rails. Design/ layout/ dimensions of the bridge or the track configuration may have to be changed to keep these unfavorable effects within limits.

3.3.1 Additional Stresses in Rails

The additional rail stresses due to the various actions should be limited to ensure that no rail fracture takes place due to overstressing and the track structure does not buckle. The additional stresses permitted for the RSI phenomenon shall be laid down by track design engineers looking at the rail stress computations done for the rolling stock, LWR and other effects. The additional stresses have to be specified for different curvatures of track.

C 3.3.1

The additional stresses allowed for the interaction effect are different from normal track tresses computed, and this check is additional for verifying proper functioning of track under RSI phenomenon.

From theoretical stability calculations, UIC 774-3R 2001 Clause 1.5.2 specifies that on UIC 60 Kg (rails) CWR, having steel of minimum Ultimate Tensile Strength 900 N/mm^2, minimum curve radius 1500 m, laid on ballasted track with concrete sleepers and >30 cm, well consolidated ballast, the additional stresses in rails shall be less than 72 N/mm^2 in compression and 92 N/mm^2 in tension. The additional allowable stresses are lower for compression as compared to tension to keep additional factor of safety towards possibility of buckling. For ballastless track, the possibility of buckling is not there, hence these values for 60 kg rails in above conditions can be taken as 92 N/mm^2 in compression as well as in tension.

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3.3.2 Displacements of Bridge Elements

Too much displacement in the bridge structure can cause instability in track structure. Therefore, the UIC 774-3R 2001 imposes certain restrictions on the displacements of bridge elements given below:

3.3.2.1 Longitudinal displacement of Deck due to movement of substructure

Due to tractive/braking loads, the relative longitudinal displacement between end of a deck and abutment, or that between two consecutive spans needs to be checked. This value should not go above ±5 mm in case CWR runs through one or both ends of the bridge. In case of bridge with jointed track/ expansion devices, the maximum permissible absolute horizontal displacement of deck under tractive/ braking loads is 30 mm.

3.3.2.2 Longitudinal displacement of Deck due to rotation of deck

Due to vertical loads, the longitudinal displacement of the upper surface of the deck end needs to be limited. this value should not exceed 8mm.

3.3.2.3 Relative displacement between rail and deck or between rail and embankment

The relative displacement between the rail and deck or between rail and embankment under tractive / braking forces needs to be checked. This value should not exceed 4mm.

COMMENTARY

C 3.3.2

The ballast packing can get loose or the entire track assembly can get unstable if too much displacement is there in bridge elements. These checks also control passenger comfort and, indirectly, the additional stresses in rails.

C 3.3.2.1

The limits are as per clause 1.5.4 of UIC 774-3R 2001. Excessive movements of decks can result in deconsolidation of ballast / deformation in the track plinth due to which proper performance of track cannot be ensured. This limit also indirectly controls the rail stresses.

If the deck movement is more than permissible, the options are either to not run LWR/CWR through the bridge (and provide jointed track), or to provide switch expansion joint at one or both ends. If the deck displacement is beyond the capacity of normal expansion devices, special arrangements for expansion of rails and supporting track structure may be required.

C 3.3.2.2

The limit is as per clause 1.5.4 of UIC 774-3R 2001. This check is also to ensure ballast stability.

C 3.3.2.3

The limit is as per clause 1.5.4 of UIC 774-3R 2001. This relative displacement determines the stability of track structure. Para 2.1.2.1 of UIC 774-3R mentions that "….relative rail displacement is not needed for verifying the effects of temperature variation and always lies within the limit value for the effects due to braking as long as absolute displacement of the deck stays within the limit value of 5 mm."

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3.3.2.4 Vertical displacement of upper surface of deck with respect to adjoining structure

The vertical displacement of the upper surface of deck in relation to the adjacent structural elements also needs to be checked. The maximum vertical displacement shall be 3 mm for maximum speeds on site of 160 km/h, and 2 mm for maximum speeds greater than 160 km/h.

COMMENTARY

C 3.3.2.4

Vertical displacement of the deck is a source of discomfort for the passengers and this has to be limited based on the speed of train operations. UIC 774-3R 2001 does not specify any limits for this and leaves the same to the concerned authorities to decide. The limits given are as per clause 6.5.4.5.2 of EN 1991 – 2003 part 2.

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4.0 STEPS IN CHECKING A STRUCTURE UNDER RSI

4.1 CHOOSE REPRESENTATIVE STRETCH FOR RSI STUDY

Representative stretches for RSI studies shall be identified along the viaduct by studying the following :

- Change in Pier Stiffness
  - Integral Spans.
  - Change in Bearing Arrangement.
  - Extended Pier Caps.
  - Change in soil conditions.
- Change in Span Stiffness
  - Change in span length.
  - Change in girder type.
  - Steel Bridges
  - Composite Girders.
- Change in Span Arrangement
  - Stations.
  - Cross Over Locations.

The RSI studies for a metro viaduct shall be done for stretches. A stretch is defined as viaduct from station to station or from its start to next station. The evaluation shall be done as follows:

- Small stretches of viaducts upto 20 spans can be fully modeled and studied under RSI.
- For longer stretches, RSI studies shall be conducted on representative stretches of the viaduct such as to cover the worst combinations of long spans/ flexible sub structure/ curved alignment/ stations/ any other special features of alignment. If a clear cut representative section is not identifiable, or there are multiple critical locations, multiple representative sections shall be chosen for analysis.
- In addition to complete stretch evaluation, any special spans (special from span length, sub structure height, span type etc considerations) should be evaluated in a standalone analysis.
- The stretches taken up for RSI study must

COMMENTARY

C 4.0

C 4.1

Metros with long stretches of viaducts pose the issue of choosing stretches on which RSI study is to be done. The results of RSI are dependent on the stiffness of different elements and it is quite clear that any stiff element will attract more force. Choosing representative section is very important to get the worst scenario possible. The changes in the lateral stiffness of span supporting elements and bending stiffness of the deck have a major impact on the vertical deck deformation and lateral deck movement.

The RSI evaluation for metros can be done for the stretches and in case the stresses are above the safe value, additional analysis shall be undertaken to identify the trends of stresses/ deformations and possible remedial actions. Based on these analyses, remedial action to alter the viaduct design or track arrangement shall be taken up and RSI studies carried out again.

100 m track length (whether on viaduct or embankment) on either side of the viaduct/

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include minimum 100 m track (whether on viaduct or embankment) beyond the stretch/span/location of interest.

4.2 VERTICAL TRAIN LOADS

The vertical train load shall be as per design loading or the heaviest trains actually running on the route, depending on the type of analysis being done. For initial design, the design loading shall be used and for subsequent checking, the details of actual trains running/ likely to run may be used. The placement of load shall be done such as to create maximum rotation at the ends. The loads shall be enhanced by the actual Coefficient of Dynamic Augment (CDA) specified in the IRS Bridge Rules or otherwise laid down for that particular metro.

4.3 BRAKING AND TRACTIVE LOADS

The braking and tractive (acceleration) forces from vehicles are longitudinal forces applied parallel to the path on top of rails, uniformly distributed along the train length. The loads shall be taken from the design loading or the heaviest trains actually running on the route similar to the vertical loads. These loads shall be applied such as to create the most adverse effect on the structure. Corresponding vertical train loads are to be combined with these loads.

For multiple tracks, the tractive and braking loads shall be applied as per normal traffic operations with appropriate mode i.e. braking or traction such as to produce worst effect on the substructure. For more than two tracks, only two tracks shall be considered loaded.

For girders at gradients, the live load has a component which is applied as longitudinal load on the bearings/ sub structure. This load shall be

COMMENTARY

stretch/span of interest has been specified since this length is required to anchor the rails and to dissipate the longitudinal forces. As per para 1.7.3 of UIC 774-3R “The model shall also include a part of the track on the adjacent embankments over at least 100m.”

C 4.2

C 4.3

The braking forces are applied along the direction of train movement and the tractive forces are applied reverse to the direction of movement of train. Normal train operations in double line are in opposite directions and the braking forces from one track are in the direction of tractive forces from the other track and their effect is additive. However in yards and other locations, train movements might be occurring in same direction and in this case, simultaneous braking (or tractive, whichever is more critical) forces on both tracks can be considered.

Clause 1.4.3 of UIC 774-3R provides that load from two tracks only need to be considered. Since the longitudinal loads are not always applied at the full level by all the trains, this is reasonable. The same clause provides that tractive load on one track and braking load on other shall be considered. However, if regular operation conditions are not like this, the actual loads for these conditions shall be applied.

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applied along with the tractive/braking loads, as per the direction of movement of trains.

4.4 TEMPERATURE VARIATIONS TO BE CONSIDERED

For the interaction analysis, without considering expansion devices, the stresses in the rail due to variation of temperature of the deck are considered as “additional stress”, to be added to the stresses eventually due to the variation of temperature of the confined rail.

When an expansion device has been fitted in the area being assessed, the variation of temperature of the deck and the variation of temperature of the rail shall be taken into account.

C 4.4

If the mean temperature of deck/ rails is to be changed during service for any reason, RSI studies shall be done afresh to verify that all parameters are within limits. The vertical thermal gradient is required to be considered for design of cross-section of the girder separately, which is not part of RSI study.

4.5 LOAD FACTORS

The load factors for the vertical, longitudinal and thermal loads shall be taken as 1 for simply supported as well as continuous spans.

C 4.5

Clause 1.5.1 of UIC 774-3R specifies the load factors to be used.

4.6 STIFFNESS PARAMETERS OF STRUCTURE

C 4.6

4.6.1 Pre-dimensioning of structure

To start with, the structure has to be given certain dimensions. These can be assumed through the experience of the designer, from other similar structures already constructed in the past or guidance can be taken from the pre-dimensioning method specified in clause 1.6.2 of UIC 774-3R 2001.

C 4.6.1

The pre-dimensioning allows the designer to assume structural stiffness and run an RSI before the actual design is taken up. After getting the idea of stresses/ displacements for the assumed stiffnesses, the designer can optimize the design and run RSI again to verify if the results are acceptable. The procedure is iterative till the desired level of optimization is achieved.

4.6.2 Determining stiffness of sub structure

The stiffness of sub-structure has to be determined using the principles of structural analysis. The deflection of foundation depends on the soil stiffness, If computer program is used, soil has to be modeled as springs and for this soil spring stiffness needs to be worked out.

C 4.6.2

Soil behavior under different conditions is quite complex and working out soil stiffness/ soil spring stiffness is quite a difficult task and requires understanding of the engineering properties of soils in subgrade and their behaviour under loads.

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4.6.3 Determining stiffness of bearings.

The bearings can be fixed/free type or elastic bearings. The fixed bearings can be considered as rigid permitting no movement and the free end can be modeled as free, neglecting the friction.

C 4.6.3

The friction in bearings shall be assumed realistically. Para 2.1.3.5 of UIC 774-3R states that "The effects of friction on rail stresses and displacements are always favourable especially when the support stiffness is low, so that ignoring friction is in general conservative for safety."

4.7 ANALYSIS METHODOLOGY

C 4.7

4.7.1 Analysis methodology using graphs given in UIC 774-3R

Annexure A and B of UIC 774-3R have graphs which have been plotted for single 60 Kg track bridges with fixed bearing at one end having single span less than 110 m. These can be modified for multiple track, different rail section, different temperature variations, single deck with multiple spans etc. The method shall be used as given in UIC 774-3R. For succession of spans/ decks, suitable FEM based computer program shall be used.

C 4.7.1

The graphs are applicable for single span/ single deck only. However, metro systems rarely have single span/ single deck in a stretch. Therefore, this method is not discussed in detail in these guidelines. For succession of spans/ decks, computer program has been recommended by UIC 774-3R in para 3.2 even though simplified rules for analysis of bridges with succession of decks have been given in para 3.3. These rules are applicable for succession of decks subject to certain conditions, however the results obtained using these rules are generally conservative.

4.7.2 Choosing computer program for carrying out RSI

RSI studies can be done using computer program or can be done using graphs given in UIC 774-3R. Due to several limitations of the graphs, computer programs are generally used. Any computer program which has the capability to model the actual behaviour of the bridge and track elements can be used. The computer program shall, however, be validated before being permitted for use. The validation shall be done using the test cases given in Appendix D of UIC 774-3R. A computer program shall be considered validated when the error on the single effects as well as on overall effect is less than 10% with respect to corresponding type of analysis (sum of effects or

C 4.7.2

The use of an FEM based computer program for numerical simulation of RSI is allowed as per para 3.4 of UIC 774-3R. Validation of software with the test cases given in UIC 774-3R, or better still, against other such software also is required before starting to rely on the results given by a particular software. The validation of software is covered in para 1.7.1 of UIC 774-3R.

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global effect). Larger tolerances, upto 20%, can be accepted if error is on safe side.

4.7.3 Analysis methodology using FEM

This section describes the analysis methodology that shall be followed to obtain prudent results in a numerical simulation for RSI analysis.

4.7.3.1 Recommendations for FEM Modeling

The study of the track-deck interaction involves the implementation of numerical models that captures the actual configuration and properties of the structure and the track. The model should be able to adequately represent the structural behaviour under different loads. Few important aspects are as follows:

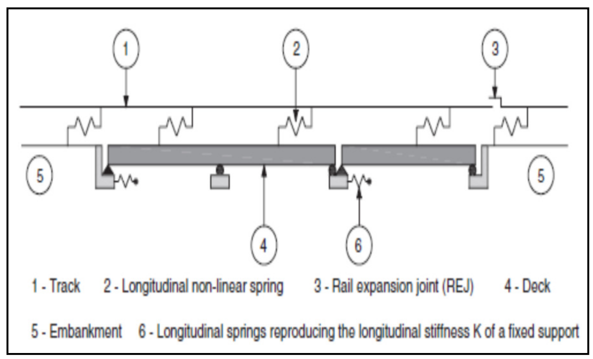
a) The elements corresponding to the rails and the deck should be located at the level of the respective centers of gravity. Likewise, the connections corresponding to support devices should be placed at the level of their centers of rotation.
b) The longitudinal behaviour of the track-deck connection shall be modelled as a bi-linear spring which can capture load/displacement relation similar to that illustrated in C 3.2.6. Separate springs shall be used for loaded and unloaded elements.
c) In some cases, it is possible to replace the mentioned elements by a connection of equivalent stiffness to that of the foundation/column/support group.
d) The maximum element length shall not exceed 2 m.

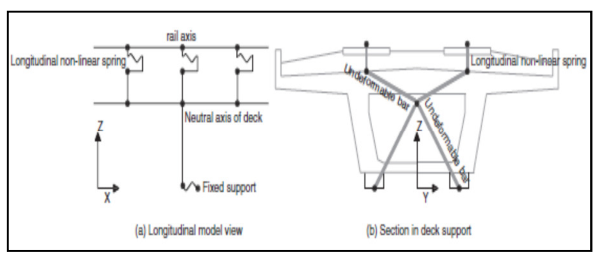
COMMENTARY

C 4.7.3

C 4.7.3.1

These recommendations are given in para 1.7.3 of UIC 774-3R. More recommendations are given in UIC 774-3R and all the recommendations have not been reproduced for the sake of brevity and for these, actual leaflet may be referred.





Elements of a Typical Modelling

Capturing the non-linear behaviour of connection between rail elements and deck elements is the most important and involved part of modeling for RSI and deserves close attention from the design engineer.

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4.7.3.2 Recommendations for FEM Analysis

The analysis using a computer assisted modeling can be achieved by two methodologies as follows:

- **Simplified Analysis:** A simplified analysis calls for running each action individually and then arithmetically combining them using factors.
- **Complete Analysis:** A complete analysis calls for applying the thermal loads and then on the deformed stiffnesses of bilinear springs running an additional live load analysis.

In the simplified analysis, first step is application of thermal loading. The longitudinal resistance of ballast is taken from **Unloaded stiffness curve** and is limited by the Limit of resistance of **unloaded track**.

Separately, train loading is applied and analyzed. In this case, longitudinal resistance of ballast is taken from **Loaded stiffness curve** and is limited by the Limit of resistance of **loaded track**.

The results are then combined by superposing the results of train load case on the results from thermal load case.

The error arises in this case because there is an apparent increase in the resistance of the ballast due to ignoring the resistance already mobilized by the track for the thermal loading while considering the train loading. **This results in an assumption of greater yielding load for track than the actual curve. This approach leads to increase in stresses in the rails and slightly lower substructure deflections and reactions.**

In complete analysis, first step is application of thermal loading similar to simplified analysis using the **Unloaded stiffness curve**. In second step, however, train loading is applied on the results obtained from the first step. In this case, longitudinal resistance of ballast is taken from

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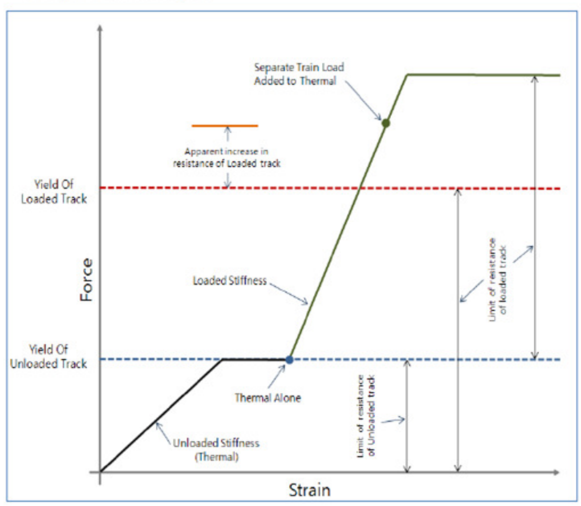
C 4.7.3.2

Both type of analyses are allowed as per para 1.7.3 of UIC 774-3R. The choice of analysis option is largely dependent on the situation being evaluated. In case of assessing the phenomenon in simply supported spans, simplified analysis will provide reasonable results. For optimization of design and in case of special spans such as arch bridges, cable stayed bridges and truss bridges etc, use of complete analysis will be required.

Difference in approach of the two types of analysis is illustrated graphically as below:

Simplified Analysis



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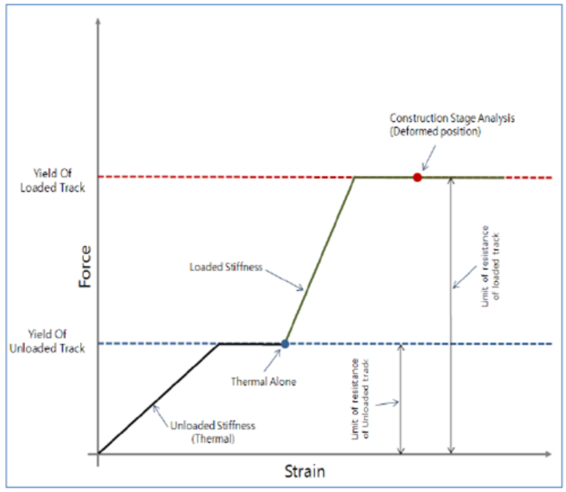
Loaded stiffness curve and is limited by the Limit of resistance of **loaded track**. In this case, there is no overestimation of the track resistance.

Complete Analysis



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4.8 RAIL GAP ANALYSIS

An important analysis which shall be performed as part of RSI is rail-gap analysis. A rail break occurs if tensile force (due to decrease in rail temperature below the stress-free temperature, or other reasons) exceeds ultimate tensile strength of the rail. In the broken rail condition analysis the following two aspects shall be studied:

C 4.8

The rail break is likely to occur at highly stressed locations (for example near an expansion joint in superstructure) or at locations where rail is weak (such as bad weld or rail flaw locations).

4.8.1 Gap at Rail-Break

Rail break means the wheel has to jump over a gap. This gap has to be limited to ensure that the wheel does not jump out. Limits for gap at the time of break in rail shall be specified by the track design engineer.

C 4.8.1

Looking at the probability of occurrence, while studying the broken rail condition, it is generally assumed that, at a location, only one rail breaks out of the two rails at a time.

4.8.2 Distribution of unbalanced forces

As the rail breaks suddenly, the force in the rail at its break point drops to zero and generates a set of unbalanced forces in the system. This suddenly released potential strain energy is imparted to the structure in the form of an equal and opposite axial force which was earlier being applied to the ends of the broken rail at the break point. In order to bring the structure to a new equilibrium state, the redistribution of forces takes place among the remaining unbroken rail and rest of the structure. **The stability of track structure under the forces generated by the**

C 4.8.2

Rails are also required to be opened for maintenance purposes. This provision is important for such cases also.

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interaction phenomenon after rail break is important and needs to be studied.

4.8.3 Broken rail scenario analysis

Forces released by the rail break has the following force components:

- Gradually released force.
- Impulse force.

In order to carry out the broken rail scenario analysis, first the potential strain energy built up in the rail due to temperature loads, creep and shrinkage of deck(s) and tractive and braking forces need to be worked out. For this, a non-linear incremental stage analysis shall be carried out as described below:

Step 1 – The model created for RSI studies shall be used without any loads for this study.
Step 2 – Build up the thermal stresses in the rails by loading the model by a thermal cycle which gives the maximum tensile force in the rail at joint in superstructure location.
Step 3 – Add Creep and shrinkage strains on deck elements in the model.
Step 4 – Add tractive/ braking loads on rails in the model.
Step 5 – Introduce Axial restraints at the extreme left and right ends of the rails. (This locks the deformation already undergone by elements in the model)
Step 6 – Change the rail temperature as per the thermal loading cycle such as to create maximum tensile stresses in rails.

Rail-Gap Analysis: After the stress build up in rails is completed, create a very small gap at critical location (over joint in super-structure). Deconstruct the rail length of the most critical rail (which create maximum imbalanced force on the bearings) to simulate the sudden release of locked-in potential strain energy.

The final gap at the rail break location and the reactions in the sub structure from the above study are outputs to be used for further study/ checks.

COMMENTARY

C 4.8.3

Movement of rail after rail break causes the track resistance to build up. The same elasto-plastic curves as described in para 3.2.6 of these guidelines come into play. The length of track which moves after rail break depends on the stiffness of track which will balance the built up potential energy in rails.

For getting an idea of the Rail Gap likely to be created as a result of break, the following formula given in TCRP light rail handbook (Chapter 7) may be used:

$$\mathrm{R_{Gap}} = \frac{(\alpha\Delta\mathrm{T})^2\mathrm{E_R A_R}}{\mathrm{R_F}}$$

$$\mathrm{R_F} = \frac{\mathrm{P_{fns} n_{ns} + P_{fs} n_s}}{\mathrm{n_{ns} + n_s}}$$

$\mathrm{R_{Gap}}$ = Rail Gap
α = Coefficent of Thermal Expansion
$\Delta\mathrm{T}$ = Temperature Change
$\mathrm{E_R}$ = Elastic Modulus of Rail
$\mathrm{A_R}$ = Area of Rail
$\mathrm{R_F}$ = Longitudinal Restrain (force /length)
$\mathrm{P_{fns}}$ =Minimum longitudinal restraint force in non-slip fastener
$\mathrm{P_{fs}}$ =Minimum longitudinal restraint force in slip fastener
$\mathrm{n_{ns}}$ = Number of non – slip fastener
$\mathrm{n_s}$ = Number of Slip Fastener

The analysis for broken rail is non-linear and time history analysis is required at the last stage when small gap is introduced in the rail.

If rail gap is more than that permitted for the metro, fasteners with relatively higher longitudinal restraint should be used. To address the structural issues,

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fasteners with a relatively lower longitudinal restraint should be used. The track and structural design engineers must coordinate and optimize the opposing design requirements for the metro system.

4.8.4 Care to be taken in maintenance

The viaduct designer must specify the conditions under which the rail gap analysis has been performed. The repairs after any fracture or opening of rails for maintenance etc shall be done as per these assumptions.

C 4.8.3

It is especially important that the stresses transferred to the structure be 'pulled back' by opening sufficient number of fasteners and 'pulling' the rails back to their original length or by other means. Suitable instructions as per advice of viaduct designer shall be made a part of the track maintenance manual.

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5.0 SPECIAL CASES IN RSI

Following special cases in RSI need more involved studies:

C 5.0

These are conditions which occur in typical metro systems but are not covered by UIC 774-3R.

5.1 HORIZONTAL CURVATURE ANALYSIS

Due to the horizontal alignment of track on curves, axial forces in the rail and superstructure have an outward component, resulting in radial forces on bearings and sub-structure. The track structure interaction analysis in case of horizontal curvature, consequently, is more involved. For such cases, the analysis for thermal case and tractive/braking loads has to be carried out separately.

Following forces are recommended to be considered in the two cases:

C 5.1

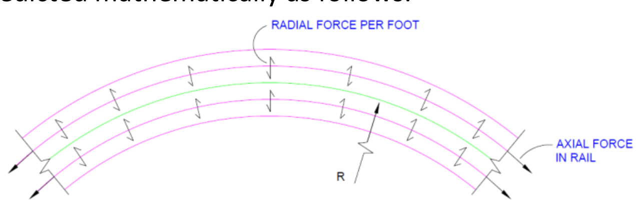
The magnitude of the radial force is a function of rail temperature, rail size, curve radius, and longitudinal fastener restraint.

5.1.1 Thermal Analysis

This analysis shall consider the following effects:

- Temperature Gradient
- Tangential Expansion

The rail forces due to the temperature can be predicted mathematically as follows:



$$\text{Radial Force Per Length of Rail} = \frac{E\alpha\Delta T}{R} + \frac{K_f L_{Radial}}{n_{track}}$$

E = Modulus of Elasticity of Rail Section
α = Coefficent of Thermal Expansion
ΔT = Temperature Gradient
K_f = Fastner Slip value divided by pitch
L_{Radial} = Radial Length
n_{track} = number of tracks
R = Radius of Rail Curve

C 5.1.1 The radial interaction of the rails in curved portion both for the thermal based analysis and braking analysis have a very different response owing to the radial redistribution of the stresses / forces in the track and plinths.

5.1.2 Braking / Tractive Analysis

This analysis shall consider the following effects:

- Braking / Tractive forces
- Nosing Force on Rail

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- Lurching (Vertical Bending)

The effect of tractive/ braking forces has to be studied through modeling duly considering the effects of curvature.

The additional stresses as per RSI studies shall be compared with the permissible additional compressive/tensile stresses specified by the track design engineers looking at the curvature, other track features and forces on track etc.

5.1.3 Allowable additional stresses in rails

The allowable additional stresses in rails for curved track cannot be the same as those for straight track. These have to be separately studied and specified by the track design engineers.

5.2 RSI FOR TURNOUTS ON VIADUCT

5.2.1 Introduction

The presence of turnout in track affects the distribution of stresses in rails in RSI studies as the stiffness turnout structure is much more as compared with the normal track. When CWR (continuous welded rail) is continued through a turnout on viaduct, RSI effects can cause movements/thermal stresses which

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C 5.1.3 Guidance for allowable additional stresses in rails for curved alignment may be taken from the Korean Design Standard: Railway Design Manual (Volume Track) provisions enumerated below:
For Ballasted Track: To allow for the lower stability of track on curved alignment which is subject to lateral loads from trains:
The permissible additional Compressive stresses on account of RSI shall not exceed:

For $R \geq 1500$: 72N/mm^2
For $1500 > R \geq 700$: 58N/mm^2
For $700 > R \geq 600$: 54N/mm^2
For $600 > R \geq 300$: 27N/mm^2

The permissible additional Tensile stresses on account of RSI shall not exceed: 92 N/mm^2
For Ballastless Track: Since the load is taken by fasteners, which can be designed for the load actually coming and there is no problem of stability, the permissible additional Tensile as well as Compressive stresses on account of RSI shall not exceed: 92 N/mm^2. However, the fasteners in this case need to be checked for additional stresses.

C 5.2

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may cause damage to anti-creep arrangement between the straight tongue rail and stock rail.

5.2.2 FEM Modeling of Turnouts

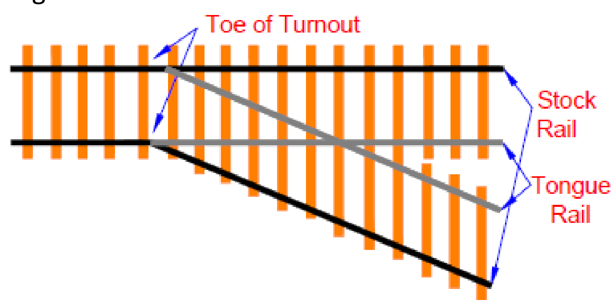
The connecting element system of stock rail/lead rail should be modelled with spring to model the interaction behavior. The track resistance for the turnout portion shall be modelled as bi-linear curve similar to the normal track, with appropriate values.

Appropriate braking loads can be considered in the turnout analysis as train speeds are operating at reduced speeds. The crossing train loading shall be idealized as EUDL applied on both the tracks.

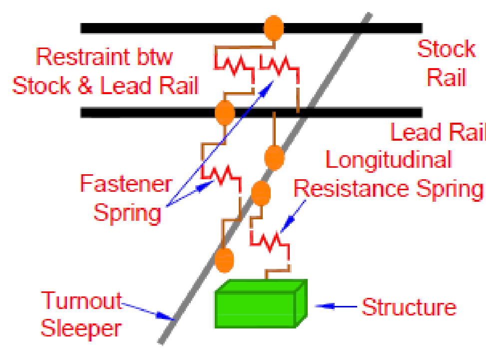
C 5.2.2

The curvature of the stock rail / lead rails can be ignored owing to the close spacing of fasteners/ sleepers. The close fastener spacing provides enough radial restraint to prevent any instantaneous buckling.

The heel joint are recommended to be connected using spring with a linear stiffness curve depicting its longitudinal resistance.



A Typical idealized turnout



Modeling of turnout to capture its behaviour

5.3 RSI ON FLOATING TRACKS / AT GRADE SLAB TRACK

5.3.1 Introduction

The floating tracks are used extensively in metro sections where the ballast maintenance is critically difficult. Floating tracks are also being used in UG sections of metro rails.

C 5.3

C 5.3.1

Trains rolling on imperfect wheels and tracks are a source of dynamic loads that act on the track and excite waves propagating from the track to nearby building. These railway induced vibrations can be a nuisance for people who live or work in buildings

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near railway lines. Measures have been invented to reduce the railway vibration. Generally, elastic elements or additional masses are inserted to the railway tracks for attenuation.

5.3.2 Modeling of Slab Track

The track structure on the floating tracks should be treated as a beam on elastic foundation.

The track elements should be made of beam elements, slab track should be modelled as surface elements and the sleepers should be modelled as solid elements

The interaction between track and sleepers should be done using bi-linear spring for ballast less conditions. The sleeper interaction with the slab should be modelled using joint element fixed in rotations in all directions.

The slab track shall be modelled as plate elements with soil springs.

The additional stresses due to the vertical bending can be neglected as the slab tracks have high stiffness and do not deform excessively in the vertical direction. Though the vertical bending does not exist, a vertical deformation in slab track owing to the soil failure can be modelled.

The slab track also has to model for a sudden change in stiffness in transition areas. In the case of floating tracks on rocks, the induced vibration in tracks due to increased sub-grade stiffness may also be required to be checked.

C 5.3.2

The ballastless track can be considered according to two different approaches:

- Consideration of the track by means of linear distributed forces;
- Consideration of the track by means of a finite element model, which reproduces the characteristics of several components of the track.

The ballastless track bed can be modelled in a three-dimensional configuration with shell finite elements with 4 nodes per element. The beams are represented making use of two longitudinal alignments of shell finite elements classified as “thick” and with a non-constant thickness in order to considerer the differences between the bottom and the top of the beam. The top slab between beams and the cantilevers were modelled in a similar way using shell finite elements classified as “thin” and with a non-constant thickness. The diaphragms are represented with shell elements classified as “thin” and with a constant thickness.

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6.0 CONTROLLING RSI EFFECTS

The control of RSI effects viz, stresses in rails and/or deflection of bridge components beyond the limits laid down in UIC 774-3R is the next question which arises after the RSI analysis is completed.

Besides the obvious option of redesigning the bridge elements to make the girders and/or sub-structure stiffer, RSI effects can also be controlled by adopting any of the following measures individually or in combination. The decision in this regard shall be taken on techno-economic considerations, which shall be a joint decision of track and viaduct design engineers.

6.1 MODIFICATION OF BEARING ARRANGEMENT

If additional rail stresses due to RSI exceed the limits, changing the expansion length can be an option to reduce these. By choosing different locations of fixed bearing in case of continuous spans, the expansion length can be changed. Changing the bearing type and their stiffnesses is another option which helps in controlling bridge deflections as well as track stresses. Replacing flexible bearings like neoprene pads with fixed-free type bearings can provide relief in many cases.

6.2 PROVIDING SWITCH EXPANSION JOINT (SEJ)

SEJs are devices provided at the ends of LWR/CWR which permit longitudinal movement of rails and at the same time maintaining correct guidance/ support to the wheels. Allowing LWR to move will reduce the stresses in rails and can be a solution in locations with longer spans/ taller piers where the rail-stresses are beyond permissible limits. Due to the SEJ(s), the horizontal deck forces are not transferred to approaches but to the fixed bearings, alleviating the effects on the rail. These also provide relief in the desired stiffness of the sub-structure as the allowable movement of decks for locations where SEJ is provided is also 30 mm as against 5 mm where LWR/CWR is continued through.

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C 6.0

The options available are either to modify the stiffness of structural elements, including change of bearing arrangements, or modifications in the track resistance/ arrangement.

C 6.1

Changing bearing configuration is a much cheaper option as compared to making the structure stiffer. This is an important parameter for optimization of design.

C 6.2

SEJs are generally undesirable from the point of view of track maintenance. These shall be provided only where unavoidable, and after consultation with the track design engineers. Generally speaking, bridges with expansion lengths of the order of 100m may generally be accommodated without resorting to rail expansion devices. Expansion lengths of the order of 300m to 400m will very probably necessitate at least one rail expansion device. Expansion lengths greater that this may necessitate at least two expansion devices or different track arrangement to cater to the large movements of deck end.

While deciding the location of SEJ, it shall be ensured that the rail expansion devices are not adversely affected by bending effects in the rail due to the close proximity of end of bridge deck etc.

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6.3 LOW TOE LOAD FASTENERS

Low toe load fasteners are special fasteners having reduced slip resistance on rails. This alters the track stiffness curve. With lesser stiffness of track, the movement of rails is more which can relieve stresses in these. The toe load of fasteners can be reduced in certain stretch(es) of track for the same. However, adequate care needs to be exercised that the reduction in clamping force on rail does not jeopardize the stability of rail.

COMMENTARY

C 6.3

The locations where low toe load fasteners are to be provided shall be clearly identifiable in field and appropriate maintenance instructions shall be issued for ensuring that these are not disturbed/ replaced by improper fasteners during maintenance activities.

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