LWR ON BRIDGES

-

IR Vs UIC APPROACH

JC Chaurasia Sr DEN/Co/SEE/ECR
Ganesh Kumar XEN/Br/II/CR

INDEX

Sl.No	**Description**	Page No.
1.	Introductory	
2.	Provisions in LWR manual	
3.	Use of UIC 774-3R	
4.	Parameters affecting Phenomenon-Bridge	
5.	Parameters affecting Phenomenon-Track	
6.	Case study	
7.	**Conclusion**	

1.0 INTRODUCTION

The above topic has been under discussion for quiet a long time the problem in continuing LWR/CWR over bridges has been a long debated subject. The problems are due to the interaction of the forces in the rail and the bridge as well as displacement of the various elements of the bridge and track. Thus the authors have tried with the help of a case study to explain the actual behavior of the track & bridge w.r.t each other. Following are the things which are to kept in mind from safety considerations

1.1 The track structure has to be safe against buckling at the highest temperatures

1.2 The maximum rail stresses in the rail under the worst condition including live loads should not exceed the yield limit of rail steel.

1.3 The gap arising from the fracture of the rail at the lowest temperature should not exceed a pre-determined limit.

1.4 The stresses in the girder as well as in the substructure of the bridge should not exceed safe limits.

2.0 LWR ON BRIDGE AS PER LWR MANUAL

Para -4.5.6:- Bridges with ballasted deck (without bearing):

LWR/CWR can be continued over bridges without bearings like slabs, box culverts and arches.

Para –4.5.7:- Bridges with/without ballasted deck (with bearings):

i) LWR/CWR shall not be continued over bridges with overall length as specified in Para 4.5.7.1 for BG and not more than 20 metre for MG.

i i) Bridges on which LWR/CWR is not permitted/provided shall be isolated by a minimum length of 36 meter well anchored track on either sides.

Para-4.5.7.1:- i) Bridges provided with rail-free fastenings (single span not Exceeding 30.5 metre and having sliding bearings on both ends)

Overall length of the bridge should not exceed the maximum as provided in Table-1 with following stipulations:-

a) Rail-free fastenings shall be provided throughout the length of the bridge between abutments.

b) The approach track up to 50 m on both sides shall be wel anchored by providing any one of the following:-

i) ST sleepers with elastic fastening

ii) PRC sleepers with elastic rail clips with fair ‘T’ or similar type creep anchors.

c) The ballast section of approach track up to 50 metre shall be heaped up to the foot of the rail on the shoulders and kept in well compacted and consolidated condition during the months of extreme summer and winter.

Para-4.5.7.1:- ii) Bridges provided with rail-free fastenings and partly box-anchored (with single span not exceeding 30.5 metre and having sliding bearings at both ends)

Overall length of the bridge should not exceed the maximum as provided in Table-1 with following stipulations:-

a) On each span, 4 central sleepers shall be box-anchored with fair ‘V’ or similar type creep anchors and the remaining sleepers shall be provided with rail-free Fastenings.

b) The bridge timbers laid on girders shall not be provided with through notch but shall be notched to accommodate individual rivet heads.

c) The track structure in the approaches shall be laid and maintained to the Standards as stated in item 4.5.7.1 (i) (b) and (c) above.

d) The girders shall be centralized with reference to the location strips on the bearing, before laying LWR/CWR.

e) The sliding bearings shall be inspected during the months of March and October each year and cleared of al foreign materials. Lubrication of the bearings shal be done once in two years.

TABLE - 1

Maximum overall length of bridges permitted on LWR/CWR on BG (in metre)

(Para - 4.5.7.1 (i) & (ii))

Temperature zones	Rail section used	Rail-free fastenings on bridges Para 4.5.7.1 (i)	Rail-free fastenings on bridges and partly box-anchored Para 4.5.7.1 (ii)
		Type of sleeper used in approaches	Type of sleeper used in approaches
		PRC/ST	PRC/ST
I	60kg	30	77
	52kg/90R	45	90
II	60kg	11	42
	52kg/90R	27	58
III	60kg	11	23
	52kg/90R	27	43
IV	60kg	11	23
	52kg/90R	27	43

Para-4.5.7.1 iii)

Welded rails may be provided from pier to pier with rail-free fastenings and with SEJ on each pier. The rail shall be box-anchored on four sleepers at the fixed end of the girder if the girder is supported on rollers on one side and rockers on other side. In case of girder supported on sliding bearings on both sides, the central portion of the welded rails over each span shall be box- anchored on four sleepers. See Fig.4.5.7.1(i i) below.

Para-4.5.7.1 iv)

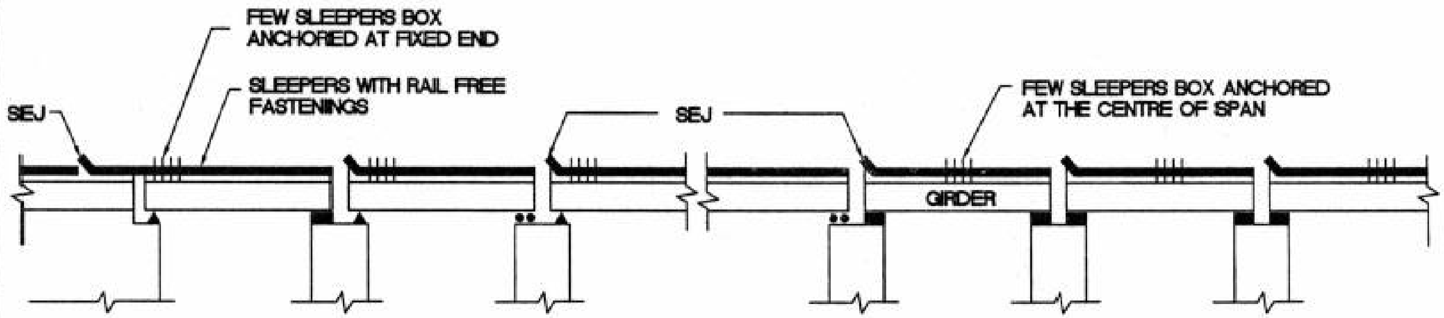
LWR/CWR may also be continued over a bridge with the provision of SEJ at the far end approach of the bridge using rail-free fastenings over the girder bridge (Fig. 4.5.7.1 (iv)). The length of the bridge in this case, however, wil be restricted by the capacity of the SEJ to absorb expansion, contraction and creep, if any, of the rails. The length of the bridges with the above arrangement that can be permitted in various rail temperature zones for LWR/CWR with SEJs having maximum movement of 120 mm and 190 mm are as follows:-

Rail temp. zone	Max. move ment of SEJ used (mm)	Max. length of bridge with SEJ		Initial gap to be provided at t_d	
		With ST/PRC approach sleepers	With CST-9 approach sleepers	With ST/PRC approach sleepers	With CST-9 approach sleepers
IV	190	55 m	45 m	7.0 cm	6.5 cm
III	190	70 m	70 m	7.0 cm	6.5 cm
II	190	110 m	100 m	6.5 cm	6.5 cm
I	190	160 m	150 m	6.5 cm	6.0 cm
II	120	20 m	15 m	4.0 cm	4.0 cm
I	120	50 m	50 m	4.0 cm	4.0 cm

Note: *SEJ is to be installed 10 metre away from the abutments.*

WELDED RAILS ON BRIDGE

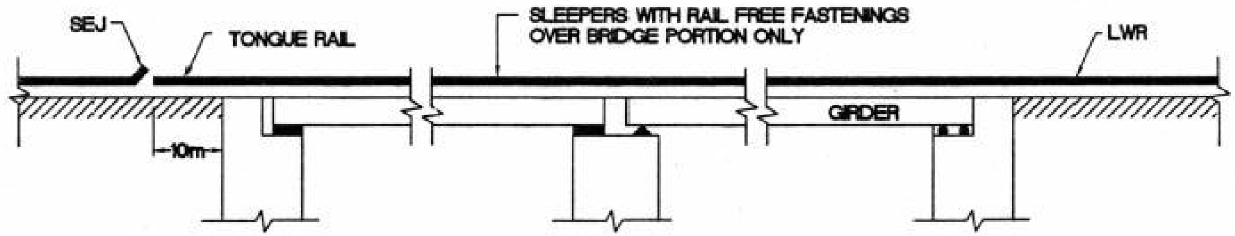
Fig: 4.5.7.1(iii+iv)



NOTE:
TONGUE RAIL SHALL BE IN CONTINUATION TO THE FREE END OF THE GIRDER

WELDED RAILS ON BRIDGE (PIER TO PIER)

Fig. 4.5.7.1(iii)



LWR/CWR ON BRIDGE WITH SEJ AT THE FAR END APPROACH OF THE BRIDGE

Fig. 4.5.7.1 (iv)

LEGEND

- ▲ ROCKER BEARING
- ● ● ROLLER BEARING
- ▬ SLIDING BEARING

NOTE:
SEJ TO BE INSTALLED 10 m AWAY FROM ABUTMENTS

2.1.1 LWR ON BRIDGES AS PER UIC MANUAL :

As per the report no. 774-3R of UIC following are the observations:

4.1 General:-

- The UIC report 774-3R, the checks required with regard to interaction phenomenon only have been considered, other checks with regard to problems of comfort, dynamic behavior or simply strength, have not been covered here.
- Further, the report is applicable for ballasted decks, thus the future design/constructions of the new bridges can be done based upon this report. However, it can be adopted for unballasted decks also by taking the values of track resistance 'k' according to the type of fastenings arrangement and making other substitutions / assumptions, wherever applicable.
- Though the report is applicable for deck arrangement, it can be suitably modified for plate/open web girders by adopting the plane structure analysis ather than the space structure analysis, which is applicable for deck slab behavior.
- Interaction must be taken into account as a serviceability limit state as regards the bridge, as well as being an ultimate limit state as regards the rail. Forces and displacements must therefore be calculated using partial safety factors for the loads concerned.

The relevant factors are applied to the forces according to checks required at ultimate limit state as regards the strength of the bearings and the substructure.

4.2 Assumptions:-

- In the case of CWR, the temperature variation in track may be assumed to be zero, as it does not affect the interaction effects (support reactions, additional rail stresses, absolute and relative displacements of track and deck), while the maximum and minimum values relevant to the deck should be considered. However, when the expansion devices are there, the temperature variation in the track should be considered, and the most unfavorable conditions for the interaction effects should be sought.
- The design curves and formulae are valid for single track bridges carrying CWR or with an expansion device in the track.

- The friction at the movable bearings has been considered to be zero.

PARAMETERS AFFECTING THE PHENOMENON:

The predominant forces generated due to interaction between track and bridges are dependent on a number of parameters of bridge and track or both:

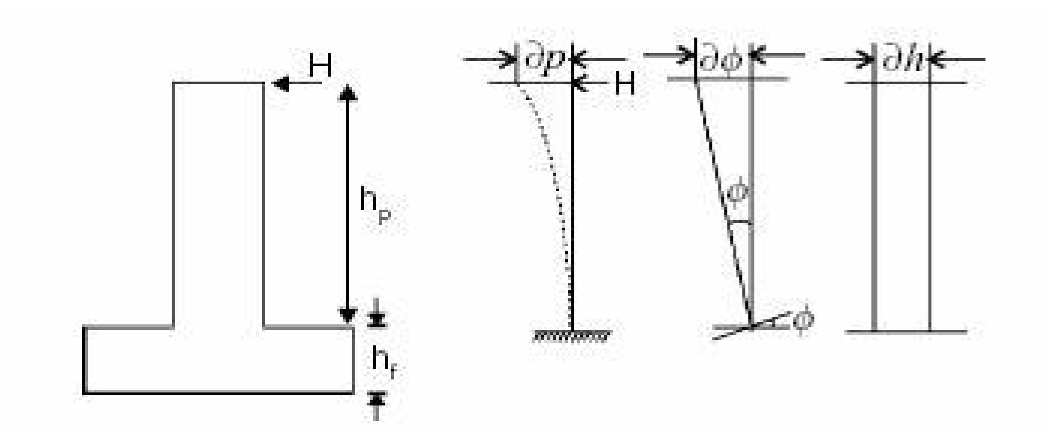
4.1 Bridge parameters

4.1(1) Expansion length of the bridge (L):

For a single span simply supported bridge, the expansion length is the span length. For a continuous bridge with a fixed support at the end, it is the total length of the deck. If the fixed elastic support is located at some intermediate point, the deck is considered to have two expansion lengths on either side of fixed elastic support.

4.1(2) Support stiffness:

The resistance of the deck to horizontal displacement is a fundamental parameter as it affects al interaction phenomena. This factor is determined primarily by the total stiffness of the supports. The total support stiffness is composed of the stiffness of each support. The stiffness of each support is in turn composed of the stiffness of the bearing, pier, base, foundation and soil.



The stiffness K of the support including its foundation to displacement along the longitudinal axis of the bridge is given by

$$K = \frac{H(KN)}{\sum \partial i(cm)}$$

with $\partial i = \partial p + \partial \phi + \partial h + \partial a$

where, ∂p= displacement at the head of the support due to deck's deformation (this could be calculated assuming the pier to be a cantilever fixed at the base)

∂ = displacement at the head of the support due to foundation rotation.

∂h = displacement due to horizontal movement of the foundation.

∂a = relative displacement between upper and lower parts of the bearing

The value of the displacement component is determined at the level of the bearing as shown in the above figure.

4.1(3) Bending stiffness of the Deck:

As a result of bending of the deck, the upper edge of the deck is displaced in the horizontal direction. This deformation also generates interaction forces.

4.1. (4) Height of the Deck:

The distance of the upper surface of the of the deck slab from the neutral axis of the deck and the distance of the neutral axis from the center of rotation of the bearing affect the interaction phenomena due to the bending of the deck.

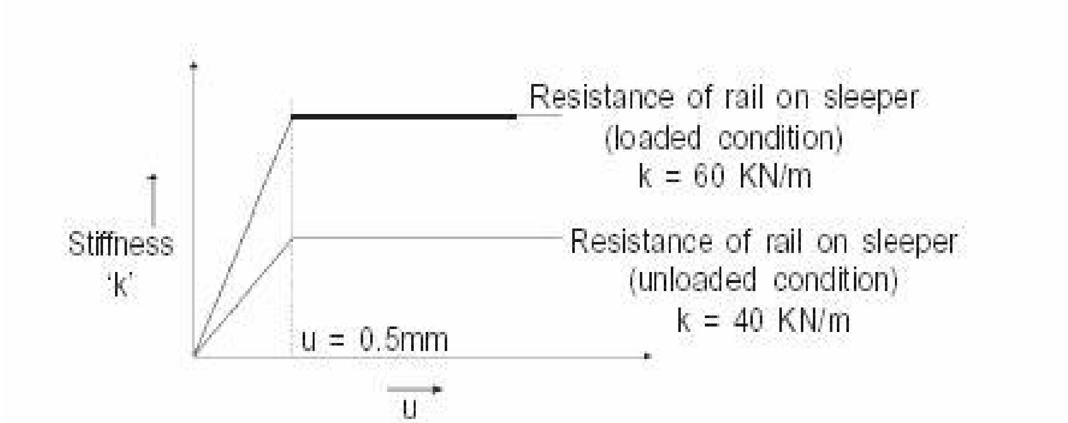
4.2 Track parameters:

4.2(1) Cross sectional area of the Rail :

The Cross sectional area of the Rail is also an important track parameter.

4.2(2) Track resistance:

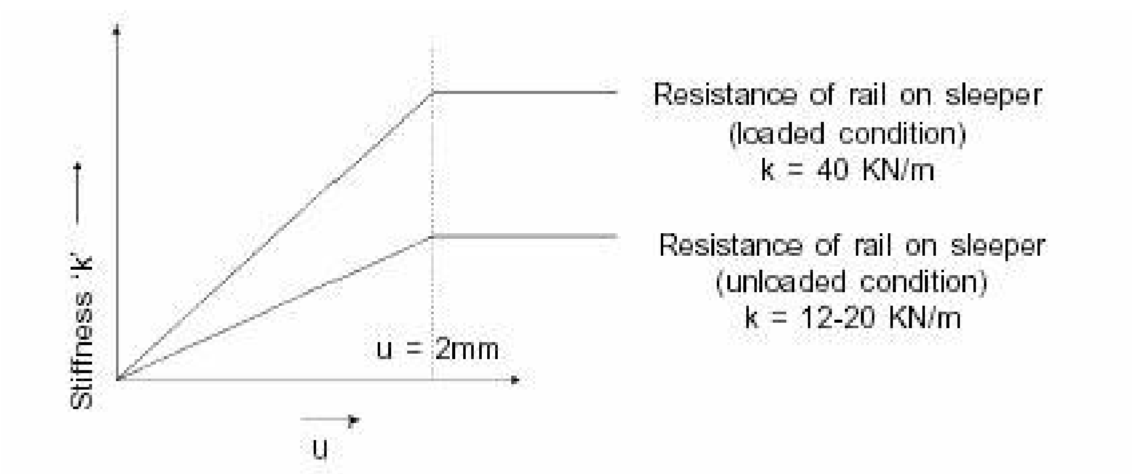
The resistance 'k' of the track per unit length to longitudinal displacement 'u' is an important parameter. This parameter in turn depends on a large number of factors such as whether the track is loaded or unloaded, ballasted or caked, standard of maintenance etc. The resistance to longitudinal displacement is higher on loaded track than on unloaded track as can be seen from the figure below. The value of k has to be established by each railway system as per its track structure.



TRACK STIFFNESS PARAMETERS (FROZEN BALLAST)

Once the values of K, the stiffness of the bridge structure and k, the stiffness of the track have been evaluated, use can be made of the interaction diagrams given in UIC774-3R for calculation of the additional stresses in the rail and additional forces at the bridge support due to each of the actions causing interaction effects: viz.,

(1) change of temperature (2) acceleration and braking forces (3) deck deformation.



TRACK STIFFNESS PARAMETERS (NORMAL BALLAST)

5.0 COMBINATIONS OF EFFECTS:

In view of the above, the consequence for the bridge laid with LWR track, the different criteria to be satisfied are as given below :

a) The permissible rail stresses in LWR should be within limits.

b) Limits have to be placed on the absolute and relative displacements of the deck and the track

c) Limits are to be placed on the permissible end rotations of the bridge.

d) The bridge elements should be designed for the additional reactions due to the bridge-track interaction.

Based on the above theoretical analysis of the bridge and track, the LWR can be continued safely over the bridges. But, for doing this, each individual bridge requires a detailed analysis. Utilizing the interactive design graphs available in UIC report 774-3R, this can be done. In this report, it has also been indicted that a computer program has been developed for track-bridge analysis and field tests have validated the results of the theoretical analysis.

However, for the utilization of the above UIC report, large number of bridge and track parameters along with the structural arrangement with load disposition and permitted displacements is required.

It is because of the difficulty in obtaining the above data for each and every bridge and the rigorous analysis to be done, that the LWR manual has prescribed the locations where LWR can be provided with a simple consideration of temperature variation alone.

CASE STUDY:

1. INTRODUCTION

The bridge, which is being built as part of the high-speed line Brussels - Lille (junction for Paris/London),has a length of 438 m and consists of 7 spans, which are supported by 6 piers and 2 abutments (Fig. 1).The main span, i.e. span no. 6, which crosses the river Scheldt, is 120 m long and is reinforced by two ballasted tracks with UIC 60 rails laid on concrete sleepers. Throughout this article the following values apply:

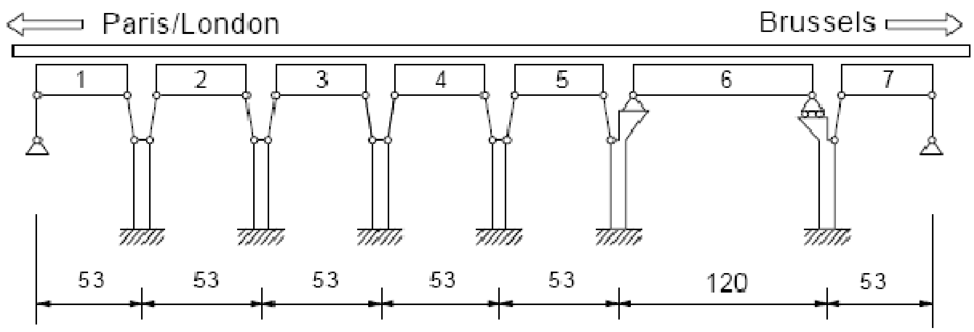


Figure 1 Schematic overview of the bridge configuration

Rails/track:	A = 15372 mm² (2 x UIC60), E = 210 kN/mm². α = 1.2 E10^{-5} °C^{-1}, ΔT = +40°C;
Span 1-5,7:	A = 26 E10^{6} mm², E = 34.75 kN/mm², α = 1.2 E10^{-5} °C^{-1}, ΔT = +30°C, longitudinal support stiffness 15 kN/mm;
Span 6:	A = 10.13 E10^{6} mm², E = 34.75 kN/mm², α = 1.2 E10^{-5} °C^{-1}, ΔT = +30°C, longitudinal concentrated load = -1350 kN (acting at the location of the roller support , due to friction);
Ballast/track:	F_{max} = 20 kN/mm, u_0 = 2 mm (bilinear characteristic);
Piers:	longitudinal support stiffness = 15000 kN/mm;
Braking load:	8 kN/m/track over 400 m track including span 6.

2. COMPUTER MODELLING OF CASE WITH FULL CWR TRACK

Calculations were made using the computer programme PROLIS20 developed at Delft University of Technology. The complete track and bridge configuration was modeled in a discrete system consisting of 263 nodes and 416 elements assuming construction symmetry over both tracks. The study started with looking at the standard case, i.e. with full CWR track, in which both tracks are subjected to temperature loading. The results are shown in Fig. 2, which consists of three graphs respectively referring to (a) the track (rail) displacement, (b) the relative displacement between bridge and track, and (c) the compressive internal track force.

Graph (a) shows a practically free expansion of the 7 bridge spans, as was to be expected then regarding the huge difference in normal stiffness between bridge and track. The maximum track displacement is 18.4 mm. The maximum relative displacement (graph (b)), however, amounts to 28.6 mm at the location of the roller support of the main span, i.e.span 6. As the longitudinal restraint between sleeper and ballast is usually lower than between rail and sleeper and the elastic part of the displacement is limited, most of the relative displacements are due to shifting of the sleepers in the ballast. As depicted in graph (c), the maximum compressive internal track force amounts to -2300 kN (-1150 kN per rail). For comparison, in the undisturbed track, built on subgrade, it is -1550 kN (- 775 kN per rail). These values remain within the limits set out in UIC leaflet 774/3. The maximum bridge force appears to be -4320 kN, which includes the friction force acting at the roller support.

3. MEASURES

In the following the effect of two constructional measures is assessed, i.e. the installation of a conventional expansion device at the location of the maximum relative displacement and, alternatively, the installation of a number of fastenings with sliding facilities, so-called zero longitudinal restraint (ZLR) fastenings. As shown in Fig. 3, this type of fastening - successfully applied at the 'Olifants River Bridge' in South Africa - consists of a special steel baseplate which is fastened to the sleeper with a Pandrol rail clip. Under normal circumstances there are small openings between the baseplate and the top side of the rail foot. In case of large lateral forces, the baseplate prevents turning over of the rail. The rail pad under the rail, is made of a low friction material like Teflon, provides an almost zero friction movement between rail and sleeper when train loading is absent. When train loading is present, it offers some resistance to possible braking forces.

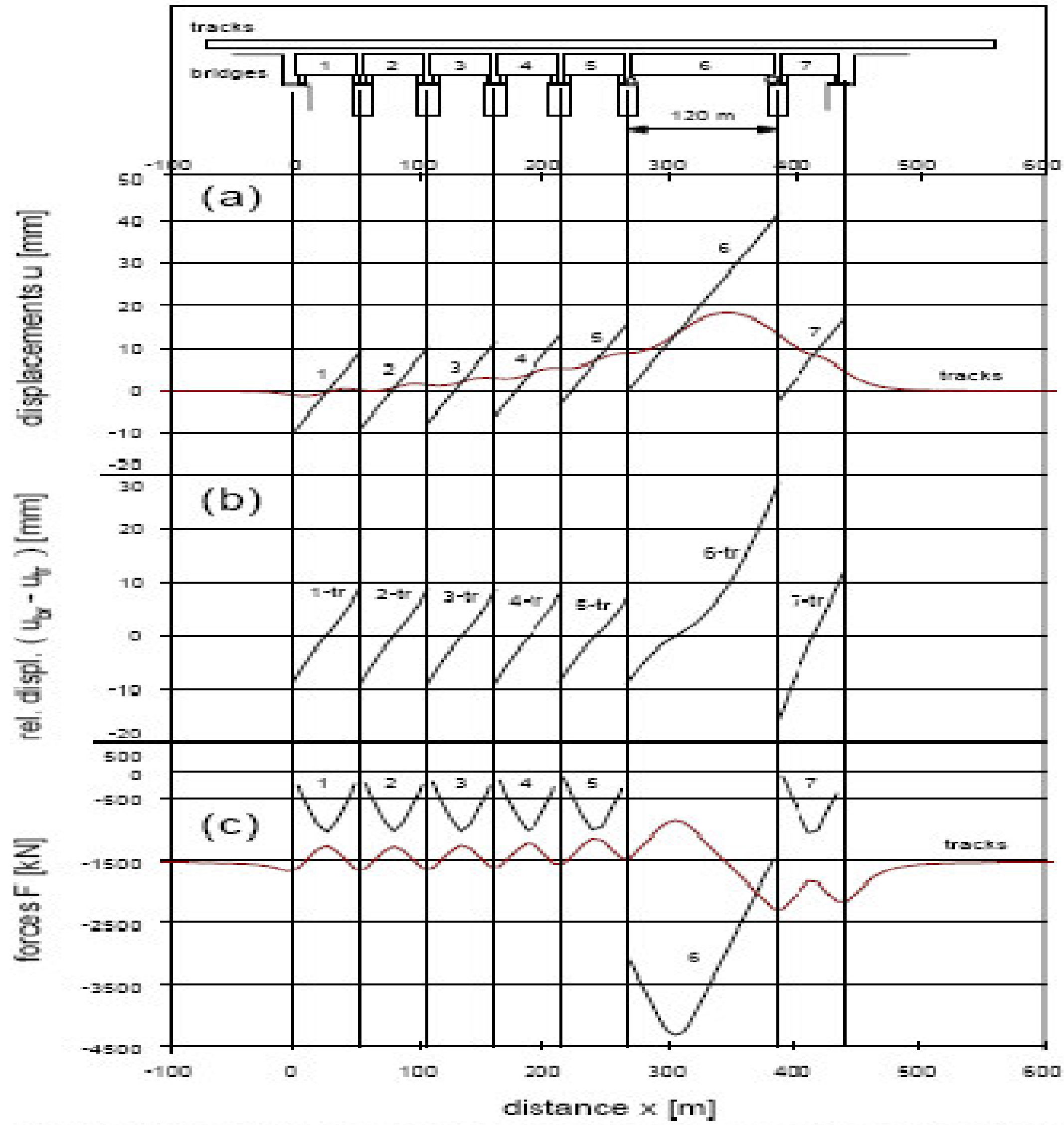


Figure 2 Longitudinal displacements and forces in CWR track on a bridge featuring a large

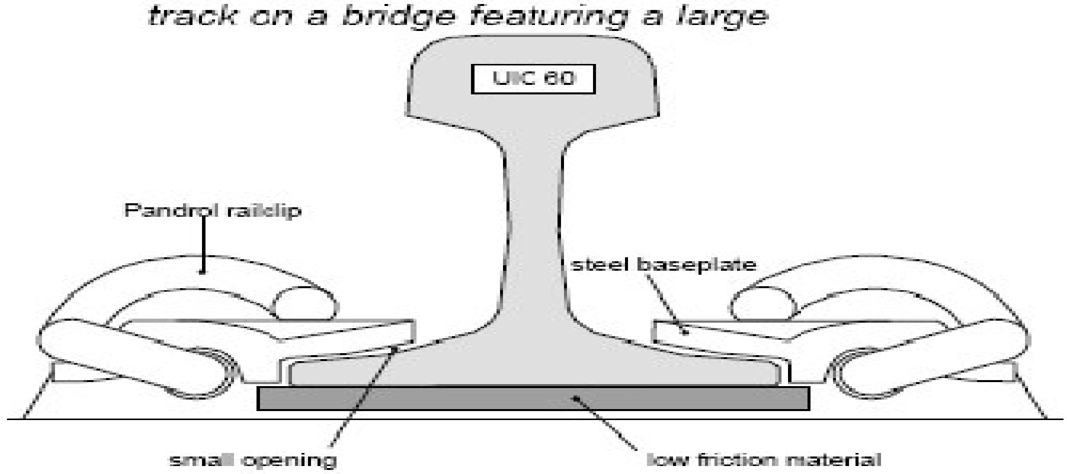


Figure 3 Principle of ZLR rail fastening

3.1 COMPARISON OF RESULTS

In order to facilitate comparison, the calculation results of four different cases, IIV, are given in a single illustration, i.e. Fig. 4, which consists of three graphs (a), (b) and (c). The insensitive bridge displacements and bridge forces are not shown in graph (c).

Case I, the standard case with full CWR track, has already been described in great detail in Fig. 2. In Case II an expansion device is present, located above the roller support of span 6. As shown in graph (a), the rail displacement - i.e. between the two rails - at the location of the expansion device reaches 79.3 mm (special expansion devices can accommodate rail displacements of up to 220 mm). The maximum relative displacement between track and bridge, shown in graph (b), is rather small (10.5 mm absolute value). According to graph (c), the maximum track force has decreased to the undisturbed value and is, of course, zero at the location of the expansion device. The maximum force in span 6 is 1450 kN.

Cases III and IV illustrate the application of ZLR fastenings over 39 m and 75 m, respectively. The maximum track displacements, shown in graph (a), has decreased to 12.6 mm and 7.0 mm respectively. The relative displacement, graph (b), has increased to 37.0 mm and 41.7 mm, which seems to deteriorate the situation. However, when omitting the relative displacements over the ZLR part of the track, i.e. the dotted lines, the maximum relative displacements in the non-ZLR part, where the frictional forces between track and bridge fully develop, is reduced to 17.6 mm in Case III and 9.3 mm in Case IV (respectively 63% and 33% of the value in Case I). Thus in case IV an even better reduction, with regard to ballast movement, has been achieved than in Case II. The maximum track compressive force and bridge compressive force have also decreased more or less proportionally.

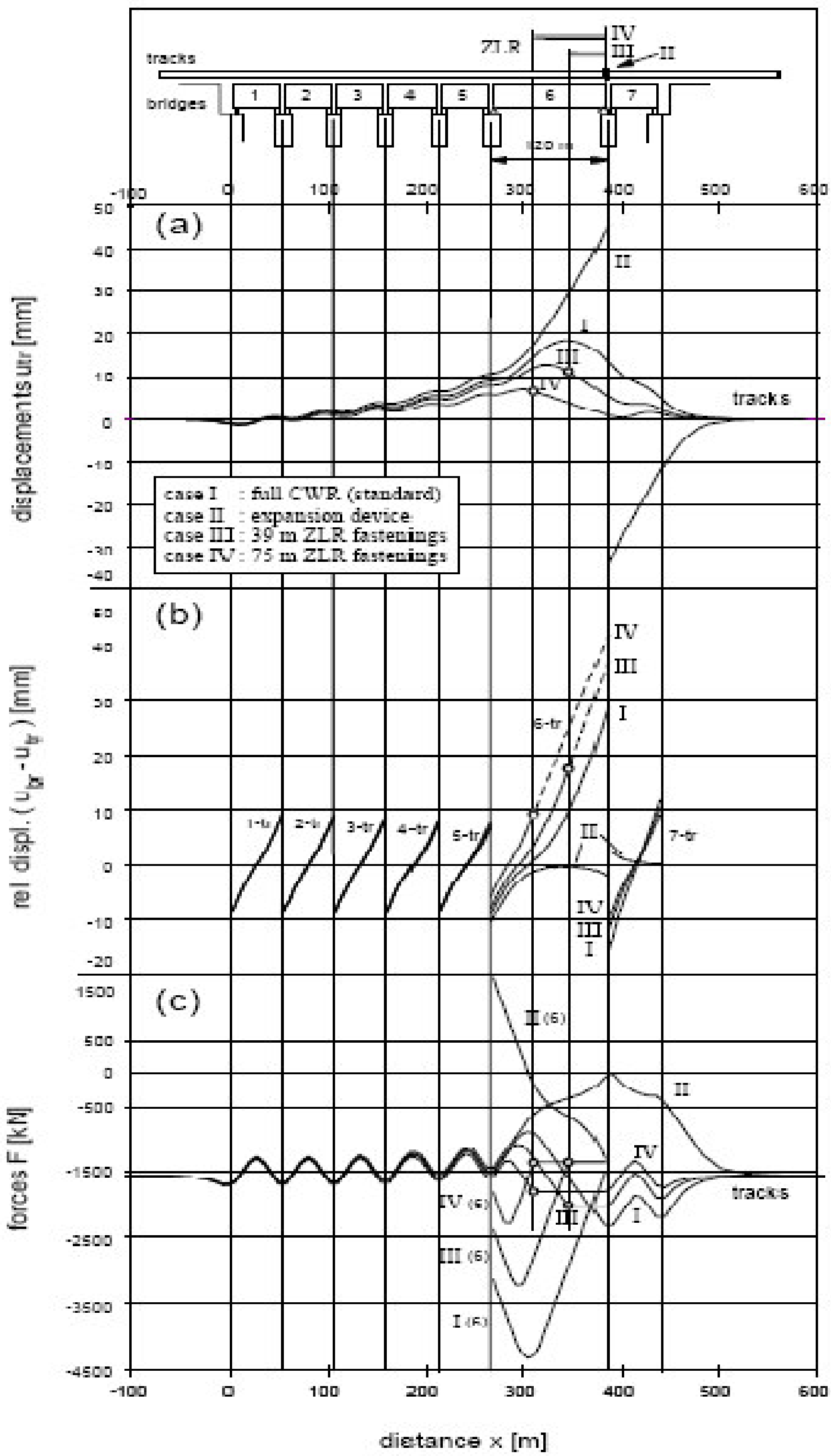


Figure 4 Comparison of possible solutions to alleviate excessive displacements and forces in CWR track on long bridges

3.2 BRAKING LOAD

An interesting situation arises when the track with 75 m ZLR fastenings, which is subjected to the temperature loading (Case IV), is also loaded by braking forces exerted by a TGV train. After applying the temperature load, a distributed braking force of 8 kN/m was exerted on track over a length of 400 m, i.e. the length of a TGV train A situation outline and respective results are given in Fig. 5, graphs (a), (b) and (c).

Graph (a) shows the maximum track displacement to be 11.4 mm, when braking is applied in the direction of the bridge expansion. The maximum relative displacement (graph (b)) of the non-ZLR part, obtained with braking in the direction contrary to the bridge expansion, has increased to 16.7 mm (80% higher compared to the situation without breaking forces). It is instructive to see what the result would be when, instead of ZLR fastenings, expansion devices are used (Case II). The result is also shown in Fig. 5 (b). Apparently the absolute value of the relative displacement (16.2 mm) is of the same order as the corresponding value of the ZLR case.

As mentioned before, some part of the relative displacement is elastic due to the longitudinal elasticity of the rail fastenings and ballast. The unfavorable assumption of zero longitudinal restraint is also not realistic in this case of a vertically loaded track. Based on these considerations it may be expected that the plastic part of the relative displacement, causing sleeper sliding in the ballast, will be less in practice.

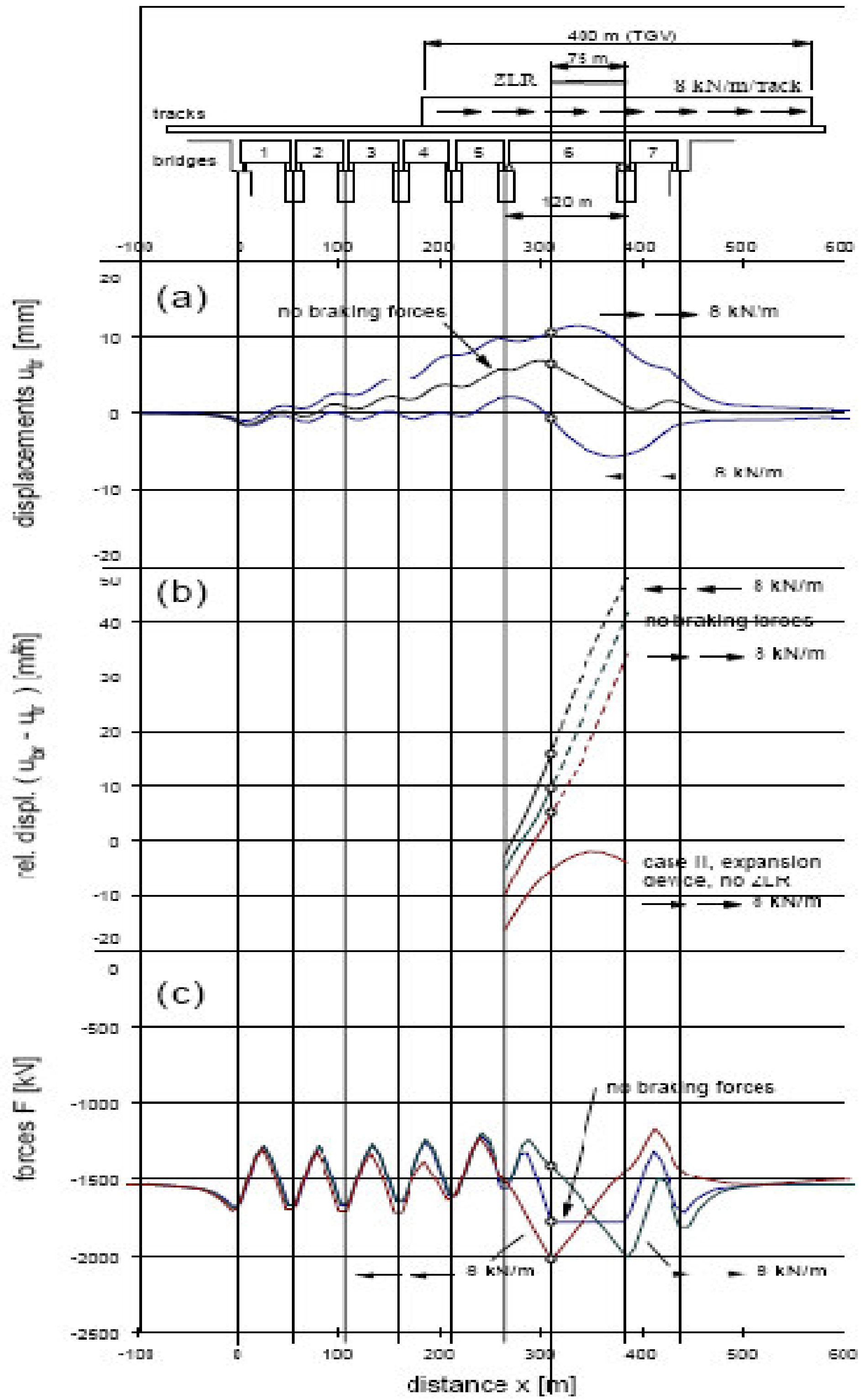


Figure 5 Effect of braking forces acting on one track of which 75 m is equipped with ZLR fastenings

4. FIELD MEASUREMENT RESULTS

For the computer modelling of the bridge assumptions were made with regard to temperature variations of the bridge spans and the rails, based on the available UICleaflets. These values are overall maxima for all types of bridges. For this particular application, i.e. a concrete bridge, these values were assumed to be severe. Therefore, it was decided to install measuring gauges at the bridge itself so that the actual values of the bridge temperature and expansion for span 6 could be recorded.

The measurement data presented in Fig. 6 was obtained at the end of July/-beginning of August 1995 during a period with extremely high temperatures. Despite this heat spell, the maximum temperature of the span did not exceed 28 °C. As an initial result it can be concluded that the assumptions made with regard to the temperature of the span were very conservative. It is also remarkable that these initial measurement results do not show the assumed linearity between temperature and expansion. Various reasons may be responsible for this behaviour, such as the influence of the arch temperature movement on expansion and the non-uniformity of the temperature over the cross-section of the bridge span.

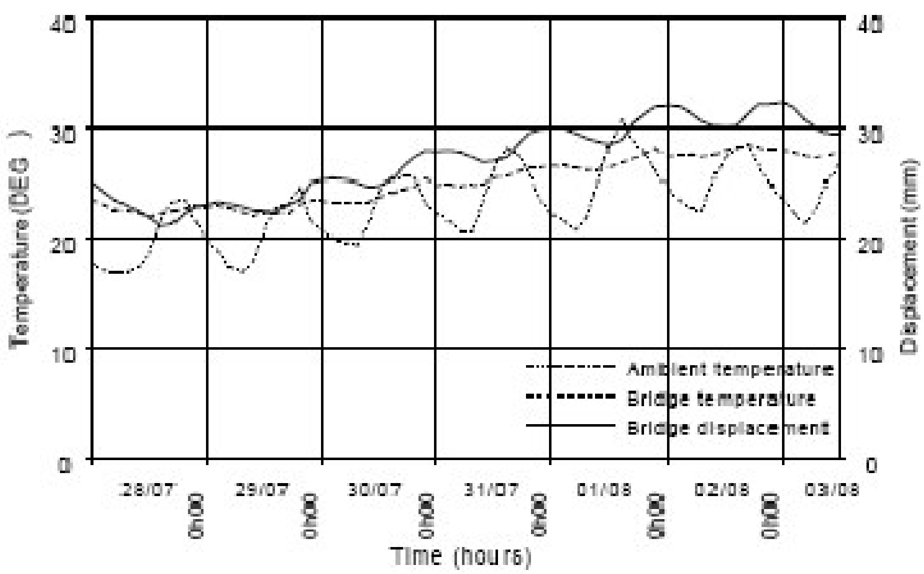


Figure 6 Measured temperature and displacement of span 6 as a function of time

5. CONCLUSIONS

The application of expansion devices in high-speed tracks on existing bridges, as a means to prevent excessive longitudinal displacements and forces, is not attractive due to comfort, safety and maintenance aspects. An alternative and very effective solution is possibly the use of so-called zero longitudinal restraint (ZLR) fastenings over some length of the track. The calculations, carried out in this respect, show a considerable reduction of track displacements, track forces, and the relative sleeper/ballast displacements. This reduction depends on the length over which these fastenings are installed. The use of ZLR fastenings, though not widely accepted yet and the construction perhaps requiring some further development, should be given more attention considering the favourable theoretical results achieved. In cases where the temperature limits, or the constructional parameters, are not known very well in advance it is advised to postpone the decision whether or not to install ZLR fastenings until sufficient measurement results are available.

As the measurements carried out to date have given very interesting results, they were continued for another year. In the meantime, no rail expansion devices were installed in the track mainly for two reasons:

1. the calculations have shown that the supplementary rail stresses are acceptable according to the UIC leaflet 774/3;
2. referring to the initial measurement results, the assumptions made for the calculations seem to be safe.

Therefore, this is an appropriate decision, which at the same time represents a saving in investment costs.