



BSI Standards Publication

Eurocode 1: Actions on Structures - Traffic Loads on Bridges - Track-Bridge Interaction

National foreword

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Eurocode 1 : Actions sur les structures - Actions sur les
ponts, dues au trafic - Interaction voie-pont

Eurocode 1: Einwirkungen auf Tragwerke -
Verkehrslasten auf Brücken - Gleis-Brücken
Interaktion

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European foreword

This document (CEN/TR 17231:2018) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI.

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Introduction

The subject of Track-Bridge Interaction has become particularly important with respect to longer span bridges and viaducts supporting tracks, especially for those carrying high speed trains. However, investigations which have been undertaken in order to address that specific issue have raised points which are relevant to all types of railway bridge. Consequently, the content of this Technical Report is intended to be applicable to all types of railway bridge, for both ballasted and ballastless track, and for all types of railway (e.g. conventional railways, metro and light rail systems, and high speed railways).

It is also clear that the increased availability of computational methods of analysis, since the basis for existing codes was laid down in the 1990s, has made it possible to consider approaches to calculation of Track-Bridge Interaction effects which could not be expected to be used in routine procedures in the past.

There are three principal 'outputs' set out in the final sections of this Technical Report. They are as follows:

- 1) Guidance for designers and maintainers of railway track and structures to assist them in adopting current best practice in taking Track-Bridge Interaction effects into account (Clause 12 of this report).
- 2) Recommendations for future development of standards, especially the revision of the relevant section of the Eurocode EN 1991-2:2003 6.5.4 (Clause 13 and Annex E of this report).
- 3) Identification of areas in which new research and development is needed to make further improvements in approaches to Track-Bridge Interaction (Clause 14 of this report).

1 Scope

This document reviews current practice with regard to designing, constructing and maintaining the parts of bridges and tracks where railway rails are installed across discontinuities in supporting structures. Current Standards and Codes of Practice are examined and some particular case histories are reviewed.

The document gives guidance with respect to current best practice and makes recommendations for future standards development and also identifies areas in which further research and development is needed.

2 Normative references

There are no normative references in this document.

3 Terms and definitions

For the purposes of this document, the following terms and definitions apply.

ISO and IEC maintain terminological databases for use in standardization at the following addresses:

- IEC Electropedia: available at <http://www.electropedia.org/>
- ISO Online browsing platform: available at <http://www.iso.org/obp>

3.1

track-bridge interaction

conditions under which forces and/or displacements in a railway track and its supporting bridge structure are influenced by the fact that rails span discontinuities in a bridge structure e.g. structural movement joints or bridge deck ends

3.2

additional load

load in an element of the track, (e.g. rail and rail fixing) on a bridge compared with what is expected in that element if the same track system were to be installed with the same loading actions away from any bridge

Note 1 to entry The word 'additional' is used in the same sense to describe additional stresses, additional forces and additional deformations.

3.3

thermal fixed point

point in the structure of the bridge, without the track, which is assumed not to be displaced when there is a change in temperature. (Otherwise known as the “centre of thermal displacement” or “thermal centre”)

3.4

deck length

L_D

distance between structural movement joints in the bridge deck

3.5

span length

L_S

distance between vertical supports, e.g. piers and abutments

3.6 expansion length, L_T of a deck

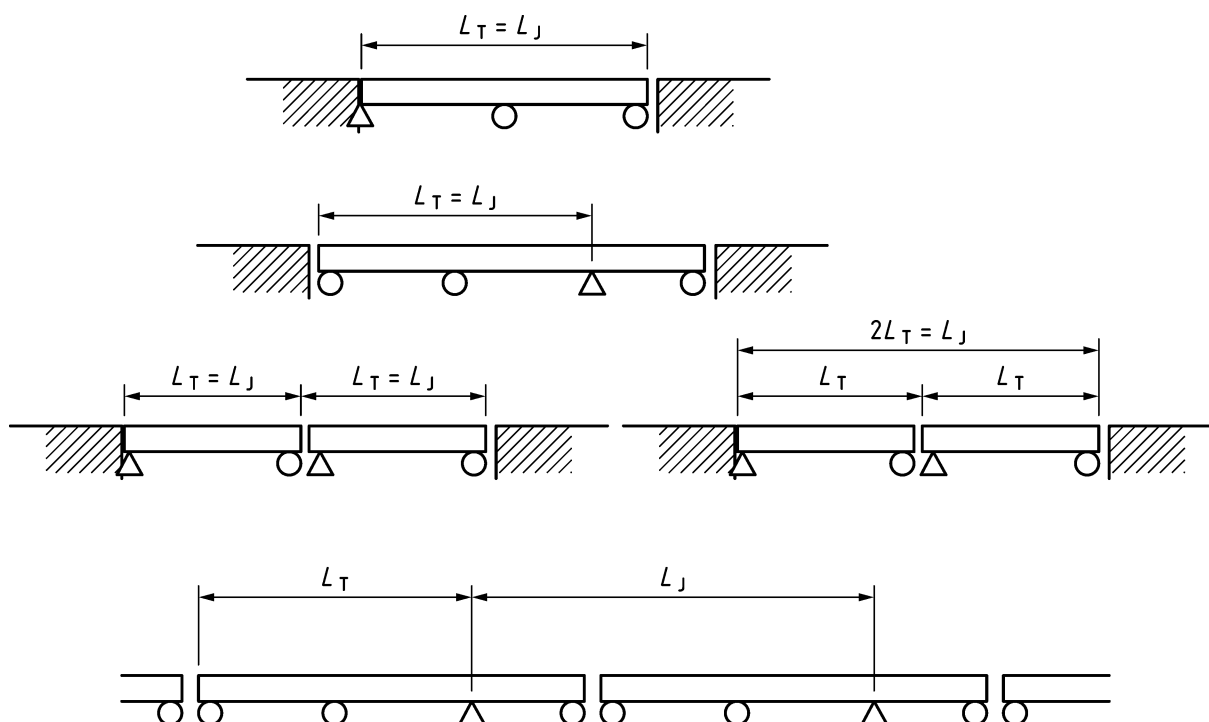
distance between the thermal fixed point and the free end of the deck

Note 1 to entry: For bridge designs in which the thermal fixed point is neither at one end nor at the mid-point of the deck, the distance from the thermal fixed point to the further free end is taken to be L_T . (See Figure 1.)

3.7 effective expansion length, L_J at a joint

total of the distances from the joint to the thermal fixed point for the two bridge decks adjacent to the joint

Note 1 to entry: See Figure 1.



Key

- △ represents a 'fixed' support
- represents a 'free' support

Figure 1 — Examples of expansion lengths L_J and L_T

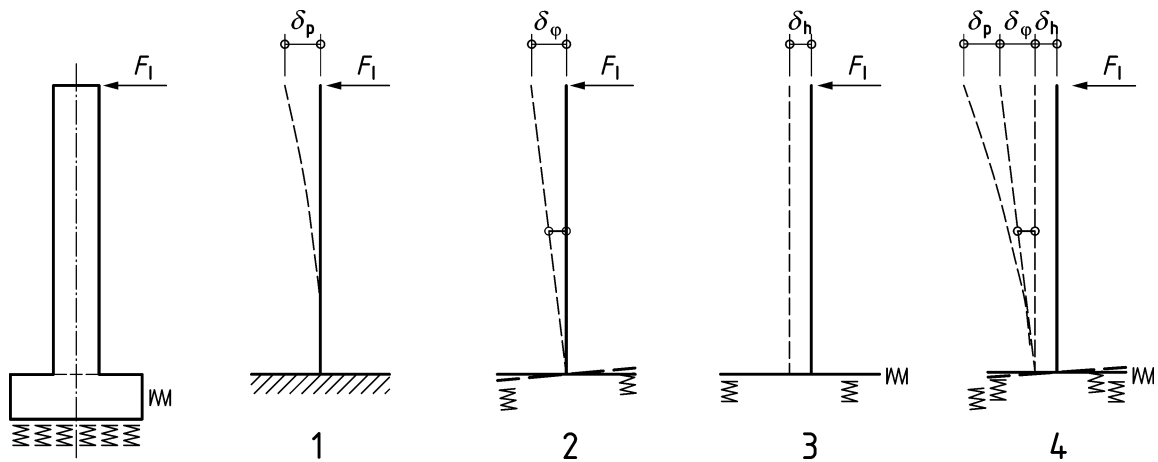
3.8 support stiffness

longitudinal stiffness of a single pier given by

$$K = \frac{F}{\delta_p + \delta_\varphi + \delta_h}$$

Note 1 to entry: Depending on the type of bearings used, the tolerance of the bearing and the shear stiffness may have to be considered by calculating the longitudinal stiffness.

Note 2 to entry: For the case represented in Figure 2 as an example.

**Key**

- (1) bending of the pier
- (2) rotation of the foundation
- (3) displacement of the foundation
- (4) total displacement of the pier head

Figure 2 — Example of the determination of equivalent longitudinal stiffness at bearings

4 Symbols and abbreviations

For the purposes of this document, the following symbols and abbreviations apply.

E	elastic ("Young's") modulus. For rails, it is assumed that $E = 210 \text{ GN/m}$.
F	longitudinal force
K	longitudinal stiffness at a single pier (see Clause 3 definition 7)
L_D	deck length (see Clause 3 definition 4)
L_j	effective expansion length at a joint (see Clause 3 definition 6)
L_S	span length (see Clause 3 definition 5)
L_T	expansion length (see Clause 3 definition 4)
SFT	Stress Free Temperature. (Temperature at which the axial stress in the rail is zero for unloaded track)
SLS	Serviceability Limit State (see definition in EN 1990:2002 ¹ , 1.5.2.14)
ULS	Ultimate Limit State (see definition in EN 1990:2002 ² , 1.5.2.13)
$\alpha, \alpha^{\text{th}}$	coefficient of thermal expansion. For rails, it is assumed that $\alpha = 1,2 \times 10^{-5} \text{ K}^{-1}$
δ_B	axial displacement of the bridge deck due to traction or braking forces
δ_h	longitudinal displacement due to rigid body translation of the pier (see Figure 2)
δ_p	longitudinal displacement due to bending of the pier (see Figure 2)

¹ As impacted by EN 1990:2002/A1:2005 and EN 1990:2002/A1:2005/AC:2010.

² As impacted by EN 1990:2002/A1:2005 and EN 1990:2002/A1:2005/AC:2010.

δ_V	axial displacement of the bridge deck due to vertical loading
$\delta_{\theta D}, \delta_{\theta R}$	displacements due to rotation of the free end of the deck
δ_φ	longitudinal displacement due to rotation of the foundation of the pier (see Figure 2)
λ	ratio of span length (L_S) to depth of bridge deck structure
θ_{TD}	angle of rotation of the free end of the bridge deck due to temperature difference

5 Description of the Technical Issue

5.1 General

Interaction between the track structure and a bridge structure (i.e. the consequences of the behaviour of one of those structures on the other) occurs because there is a physical connection between them, whether the rails are directly fixed or there is a ballast bed in between the track and the bridge. The interaction results in forces being applied to the track (rails, fastenings and ballast) and the bridge substructure (foundations, piers, abutments, bearings). These forces are in addition to those which would be expected if the track and bridge were analysed separately.

If these additional forces are too high this may lead to failure modes including tensile failure of the rail, lateral buckling of the track, shear failure of bridge bearings, longitudinal failure of the bridge substructure or uplift of track elements. These forces shall be taken into account in assessing both serviceability limit state (SLS) and ultimate limit state (ULS) conditions of the structure, although only SLS conditions should be taken into account for calculating stresses in the track (see 9.3 and 9.4),

As a general principle, track engineers prefer to have bridges which are designed to reduce the influence of the bridge on the track to a minimum. Existing and proposed standards and codes set maximum limiting values of stresses, forces and deformations. For specific projects the preferred practice at the design stage may be to aim to achieve values well below those limits.

However, historically the problem of Track-Bridge Interaction has been solved by installing rail expansion devices close to structure movement joints on longer bridges. Rail expansion devices are expensive to install and to maintain, especially on high speed lines where impact forces arising from imperfect joints in the rail cause deterioration of the track and the supporting structure. On many urban railways there is a need to reduce the number of rail expansion devices to remove a source of noise, even if the train speeds are lower.

The resolution of this apparent conflict between the interests of bridge designers and track designers shall be based on an understanding of the economic implications of different solutions. At the simplest level, this requires an understanding of the relative construction and maintenance costs of, for example, installing rail expansion devices compared to modifying the bridge design (e.g. longitudinal stiffness of sub-structures) and/or accepting higher operational stresses in the rails. However, even more significant economic benefits may accrue from changes to the detailed design or even the overall configuration of bridge structures.

In order to reduce maintenance costs, tracks are now designed to work with lower stresses in the rails, and some other components, than they were some years ago. For example, in most European main line tracks fifty years ago it was common to use rails of 50 kg/m to 56 kg/m with sleepers 650 mm to 700 mm apart. With the same maximum axle loads, the same railways now use rails of 60 kg/m with higher strength steel and with sleepers 600 mm to 650 mm apart. This means that there is a greater margin between the operational rail stresses and the ultimate failure conditions.

The underlying principle in many of the cases described in this report is that in some critical locations it may be desirable to use this margin to allow higher operational stresses in continuous rails (implying an acceptance of a shorter rail life) in order to avoid the use of rail expansion devices.

5.2 Axial effects

5.2.1 Origin of axial forces and displacements

It is assumed throughout this Technical Report that track configurations which are to be considered can also be used in applications where there is no bridge e.g. on earthworks, etc. and that in those situations the track performs in an acceptable way. There is also an underlying understanding of the fact that changes in temperature do not cause continuous welded rail (CWR) to expand or to contract. The length of CWR remains the same as the temperature changes because the longitudinal and lateral restraint provided by the ballast or slab, prevent movement of the track. Of course, the state of stress in the rail does change with temperature.

The main purpose of this report is to consider additional forces and deformations which arise on bridges, in particular the longitudinal forces which arise as a result of any actions which tend to open or close the gap at structure movement joints. These may be due to:

- Changes in temperature (Mean temperature and temperature gradients).
- Application of traction and braking forces.
- Application of vertical forces.
- Creep and shrinkage of concrete elements.
- Movement of the thermal fixed point due to, e.g. Rotation of pier foundations due to settlement.

5.2.2 Force transfer between track and deck ends

Figure E.4, within the text for the proposed revision of EN 1991-2, shows plots of the k value, which is the longitudinal shear resistance of the track (force/m), which is transferred from the track to the deck or track formation, plotted against u , the relative longitudinal displacement between the track and the deck or track formation. It is shown as an elastic-plastic function but it shall be recognized that, for calculation purposes, this is greatly simplified from the measured plots of k versus the relative displacement u . Note that in the recent past the parameter k is sometimes used for the generic force (as in UIC 774-3R, Figure 5) and more often is used specifically for the plateau value of the force. The term ‘ k -function’ has therefore been adopted to represent the longitudinal shear resistance of the track over the elastic-plastic range and k is used for the plateau value.

Values of k and u_0 are set out in Table E.1. Although the values for ballasted and ballastless track are included in the same table, this obscures the very different behaviour between the two cases. With (unloaded) ballasted track u_0 marks the relative movement when the sleepers start to move through the ballast. The ballast shears at the level of the underside of the sleeper, and the ballast between the sleepers is moved with the sleepers. With ballastless track, the relative movement occurs in the fastenings and with larger movements the rail slides through the fastenings. Sliding in the fastenings can occur for loaded ballasted track where the shear resistance of the ballast is greater than the frictional resistance of the fastening.

5.2.3 Rail stresses

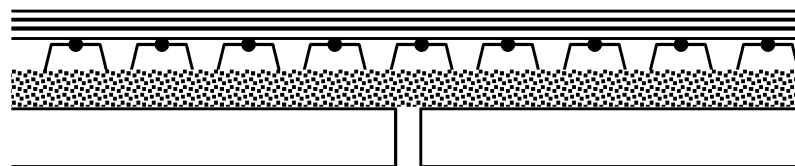


Figure 3 — Typical configuration of track across a structural joint

Figure 3 shows schematically a typical situation in which a continuous rail is fastened across a discontinuity in the structure. High local rail stresses may be generated, principally by two mechanisms:

- Opening and closing of the joint by axial displacement of the bridge deck ends, or
- movement of the joint due to rotation of the bridge deck ends.

Thermal expansion or contraction of the structure leads, principally, to longitudinal displacements of the free ends of the structural elements, opening or closing the gap at the joint.

The relationship between the movement of the joint and the stress in the rail depends on the elastic or non-elastic (e.g. slip) shear stiffness of the components between the rail and the structure. In qualitative terms, this behaviour is quite well understood by track engineers as it is fundamental to the design of track with continuous welded rail when considering lateral track stability (i.e. resistance to buckling). It is less familiar to bridge engineers.

If there is enough provision for relative movement between the bridge and the rail, the bridge will expand and contract as the temperature changes, and the rail will simply remain in position with no displacement and no change in stress. Where there is not “enough” provision for relative movement between the bridge and the rail, the rail will be dragged along by the moving bridge deck. In these circumstances the relative displacement between the rail and bridge deck is reduced but the absolute displacement of the rail is increased. Additional strain due to the movement of the joint is imposed on the rail (equal to the slope of the curve of absolute rail displacement v. position along the rail) and that results in an additional axial stress in the rail. This mechanism is illustrated in Figure 4.

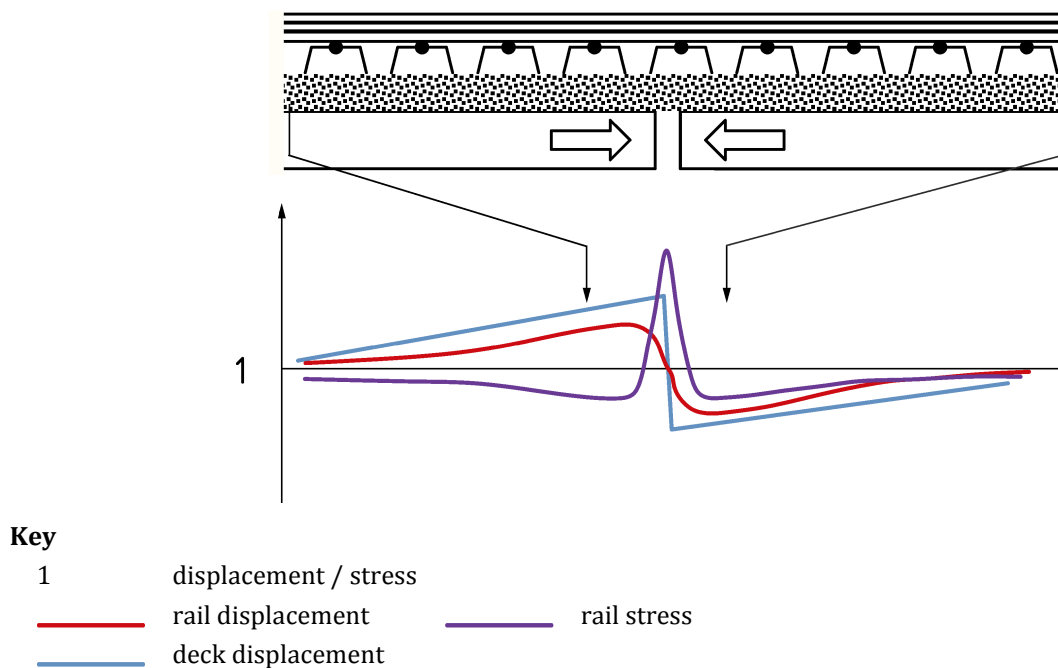


Figure 4 — Effect of longitudinal joint movement on rail stress

In order to create any kind of a model for calculation, it is necessary to characterize this behaviour mathematically. Most models represent the behaviour with a simple bi-linear (elastic-plastic) force-displacement curve as described in 8.1 i.e. it is assumed that the longitudinal displacement of the rail increases in a linear, elastic response to the applied force until it reaches a critical relative displacement, u_0 , beyond which the rail slips through the fastening system with a constant restraining force, k . The implications for structural design calculations are considered in more detail in 10.2 of this report.

The bi-linear representation is a useful approximation but the true behaviour of the track components is much more complex. The components which can affect the behaviour include rail fastening systems, under-sleeper or under-ballast mats and the ballast itself.

For a real bridge structure the rail stress distribution may be calculated from first principles, treating the track, the bridge and the connection between them as a single structure for numerical analysis. Figure 5 shows a typical form of axial rail stress distribution for a simple, single span bridge. A point to note is that although the deck displacements are limited to the length of the bridge, the rail displacements and stresses are affected for some distance beyond the ends of the bridge.

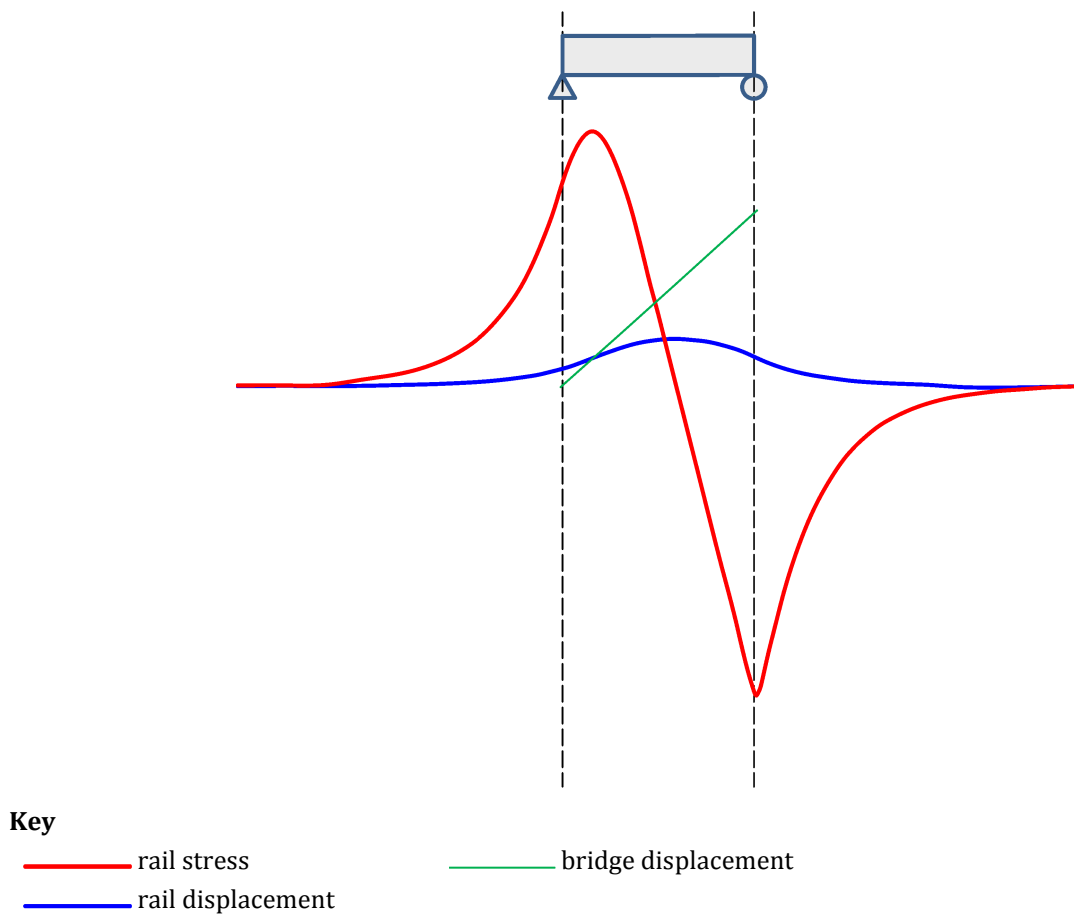


Figure 5 — Rail stress due to bridge temperature variation across a simple, single span bridge

5.2.4 Forces acting on the fixed point (e.g. Bearing forces)

The longitudinal forces transmitted to the bridge deck through the bearings depend on the detail design of the bearing, the shear stiffness of the bearings and the supporting substructure. In general terms the shear stiffness of bearings which permit longitudinal movement is taken to be zero, but for more accurate calculations the elastic stiffness of some types of moving bearings (e.g. elastomeric bearings) should be taken into account.

The stiffness of the support, K , including its foundation, with respect to displacement along the longitudinal axis of the bridge caused by a horizontal force, F , is given by

$$K = \frac{F}{\sum \delta_i}$$

In which $\sum \delta_i$ represents the sum of the horizontal displacements due to elastic shear of the bearing, displacement and bending of the pier (see 3.7).

5.2.5 Interaction with sub-structure

Forces on the sub-structure are influenced by the presence or absence of rail expansion devices, the stiffness of the thermal fixed point and a number of other parameters. In order to comply with the requirements for longitudinal force capacity the sub-structure has to be very stiff longitudinally. For the abutments, much of the stiffness is in the backfill (particularly where the longitudinal movement is in the direction of the backfill) and this shall be quantified, taking into account the difference between tension

and compression. For intermediate deck spans away from the abutments, the anchor pier and its foundations shall be designed around the need for stiffness and this may require the use of A-frames or longitudinal shear walls.

The stiffness of the soil may also depend on the loading rate and may be higher in dynamic loading cases e.g. train braking cases. For larger bridges, this factor should be determined by geotechnical engineers and taken into account in any analysis.

5.3 Vertical effects

5.3.1 Effect of vertical forces and displacements

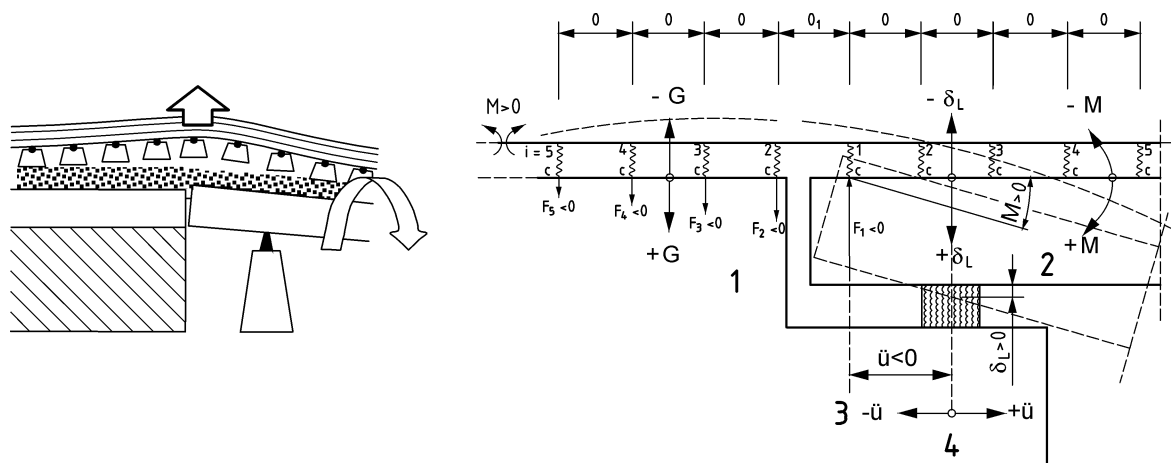
The axial forces and stresses in the track are affected by the vertical loads applied to the track, and the resulting forces from the track onto the deck. The effects caused by loads applied to the track are the same as those away from the bridge, so they do not contribute to the additional stress as defined in 3.2. However, bridge design for vertical loads is generally based on specific load models defined in the Eurocode (e.g. “Load Model 71” as a model representing European main line traffic conditions) whereas track strength away from bridges may be determined using other assumptions about applied vertical loads (e.g. “Rail Fastening Categories” in the EN 13481 series of standards). The two approaches give similar, but not necessarily identical, results.

The effects of the forces from the track onto the deck shall be considered. In particular the end-rotations of the decks need to be calculated to evaluate $\delta_{\theta D}$ and $\delta_{\theta R}$. These effects may be observed anywhere on the bridge. The effects are most significant at the free ends of the deck.

5.3.2 Bridge deck end rotation

At the free ends of the deck, bending of the bridge under vertical train loading and under vertical temperature gradient results in relative rotational movement of the structural elements. The rotational movement of the bridge deck end leads to vertical and horizontal movement. (See Figure 6). In this case, if the rail is continuous across the structural joint the stress in the rail and other track components depends on the resistance of the track to uplift and the bending stiffness of the rails. The additional bending stresses in the rail are neglected, but on ballastless track the effects of uplift forces on the rail fastening system shall be taken into account. It is important to note that the bending stress ahead of the first wheel due to this effect is in the opposite direction to the bending due to the weight of the train. If the track is ballasted, the resistance to uplift is simply the force required to lift the track (rails, sleepers, etc.) out of the ballast. This action may lead to degradation of the ballast. For ballastless track, it depends on the uplift characteristics of the rail fastening system. The uplift mechanism is illustrated in Figure 6.

In principle, bridge deck end rotation effects should be calculated using the vertical load case applied in the general bridge design calculations, positioned in the worst possible location. In practice, sufficiently accurate analysis can usually be carried out using a uniformly distributed load e.g. a UDL of 80 kN/mm in place of Load Model 71.



Key

- 1 abutment
- 2 bridge deck
- 3 last rail fastening position on deck
- 4 axis of bridge bearing

$$\delta = \delta_L + \varphi \cdot \ddot{u}$$

Figure 6 — Effect of bridge deck end rotation

5.4 Limits to the need for detailed calculations

It is the differential movement between the track and the structure which creates additional stresses. For short expansion lengths on stiff bridges, movement at structure movement joints is very small and therefore it is not necessary to provide any mitigation measures in the track to reduce rail stresses. The definitions of a “short” and “stiff” bridge are not universally agreed but on the basis of many years of experience most railway administrations do not require the installation of rail expansion devices on bridges with expansion lengths less than a specified value – typically around 30 m. It is suggested, therefore, that detailed calculations should not be required for ballasted deck bridges with effective expansion lengths, L_j , less than 30 m. For these cases the bearing forces can be calculated with sufficient accuracy by using the “simplified method” in 6.5.4.6.1 of EN 1991-2:2003.

Recent experience suggests that even if the expansion length is more than 30 m, continuous rail can be used safely without installing rail expansion devices but that as expansion lengths increase, more care shall be taken at the design stage. For ballasted deck bridges with continuous rail on a single deck section and with an effective expansion length, L_j , greater than 30 m but still less than 60 m for steel bridges or 90 m for a concrete bridge, it is still not necessary to make detailed calculations for rail stresses but more detailed structural analysis is required to determine the bearing forces and stiffness requirements.

For longer bridge decks it is necessary to make detailed calculations for rail stresses and bearing forces and to consider the use of detail features that can be utilized in design to mitigate the effects of Track-Bridge Interaction.

For very long expansion lengths there is no alternative to the use of rail expansion devices. Calculations based on UIC774-3R indicate that the stresses in continuous rails are likely to exceed the maximum limit values, regardless of the use of mitigating measures, if the effective expansion length, L_j , is greater than about 140 m. In this case, rail expansion joints become necessary at least at one structure movement joint. If L_j is greater than 140 m for each structure movement joint, REJs are necessary at each bridge joint and detailed analysis of the rail stress is unnecessary (see 8.3.2).

For ballastless tracks, the same principles apply but there is less experience available to provide a basis for recommending particular values of expansion length above which more detailed calculation is required. Some of the case studies in Clause 7 of this Technical Report suggest that limiting expansion lengths for concrete bridges with ballastless tracks may be similar to those for ballasted track on steel bridges.

Bridge deck end rotation effects can be neglected for short, stiff spans. In general, if the mid-span vertical deflection is less than $L_s/3\,000$, as required for high speed bridges (see 5.6.1), the angle of rotation at the deck end is very small and if the bearing position is close to the free end of the deck (i.e. there is very little “oversail”) the resulting displacements can be ignored.

5.5 Calculation of multiple loading conditions

Within the codes for analysis of track-bridge interaction there are several load cases (temperature effects, traction and braking effects, deck bending effects, etc.) which shall be combined to assess the overall performance. One alternative is to analyse each case as an independent effect and then to use simple linear superposition to combine the results. This approach is endorsed in EN 1991-2:2003.

Another approach is to adopt a more rigorous analysis which takes account of nonlinearity. In this approach the effects are considered in the sequence in which they occur and the end result of the analysis of one effect becomes the starting condition for the analysis of the next effect. These analyses are usually performed with specialist, nonlinear software, but good results can be achieved with spreadsheets (see Annex B).

Comparison of the two approaches on viaducts on German high speed lines has indicated that this second method predicts less severe peak stress conditions than the standard approach involving linear superposition. The difference arises because of nonlinearities in the response of the structure to the applied load. In general the use of nonlinear analysis results in lower calculated rail stresses but higher bearing forces.

5.6 Effect of bridge deformations

5.6.1 Effect on track geometry

Care should be taken to check the possible effects of bridge deformation on track geometry, especially the possibility of track twist and cross-level faults on skew bridges and bridges supporting curved track. Acceptable tolerances for new (or recently renewed) loaded ballasted track are given in Table 1 of EN 13231-1:2013 and in EN 1990:2002³, A2.4.4.2.2. By inference, if a part of the tolerance is needed to accommodate bridge deformations it is necessary to ensure that other effects, such as construction tolerances and deformation of the track itself, are reduced to achieve an acceptable combined system performance.

It is worth noting that within the Eurocode (EN 1990:2002⁴) the maximum vertical displacement of the bridge deck at mid-span is limited to $L_s/600$ as a general requirement but that Annex A2 of EN 1990:2002⁵ includes tighter limits e.g. $L_s/3\,000$ for high speed lines, mainly on the basis of passenger comfort criteria. These values are currently under review and future standards may require tighter restrictions on maximum deflection, especially for high speed lines.

³ As impacted by EN 1990:2002/A1:2005 and EN 1990:2002/A1:2005/AC:2010.

⁴ As impacted by EN 1990:2002/A1:2005 and EN 1990:2002/A1:2005/AC:2010.

⁵ As impacted by EN 1990:2002/A1:2005 and EN 1990:2002/A1:2005/AC:2010.

5.6.2 Effect on stability of ballasted track

In the case of ballasted track, bridge deformations may also affect the risk of lateral track instability. There is no specific requirement in the Eurocode to take this risk into account, but it is considered in some national rules. For example, in Germany the longitudinal movement of a ballasted bridge under braking loads is limited to 4 mm.

This is a particular problem at discontinuities of the deck. Further information may be found in the leaflet UIC776-2.

It is also important to note that changes in the track configuration may be important not only when they are on the bridge but also if they are close to the bridge. For example, if a double track railway is reduced to a single track across a bridge, the position of the turnouts may be significant: At the turnout, the axial force in two tracks (four rails) is transferred into a single track (two rails). If there is a sufficient length of plain line on the single track side of the turnout, that extra force is dissipated by shear transfer through the ballast, but if the turnout is too close to the bridge the higher force in the rail may still be significant over the bridge end structure movement joint.

5.6.3 Effect of ballast degradation over structural joints.

Repeated movement of structural joints and bridge deck ends can also cause degradation or migration of ballast which will result in long term loss of vertical and lateral track geometry if remedial maintenance work is not carried out. If it is not possible to avoid large joint movements special measures may be required to ensure that ballast is retained (see 8.2.2).

Within the Eurocode EN 1991-2:2003, there is a requirement to ensure that the longitudinal movement across a structure joint of the bridge due to traction and braking forces, δ_B , does not exceed 5mm. This figure is included to limit the effect of bridge deck end movement on ballast degradation. For the same reason, there is a requirement to limit movement across a structure joint resulting from vertical bending of the bridge deck, δ_V , to 8 mm. These figures become significant restraints on the design of long – span bridges for high speed railways. In particular, two questions arise:

Firstly, if vertical bending is limited on high speed railways for reasons such as passenger comfort (see 5.6.1 above) it is unlikely that the 8mm limit on δ_H will ever be reached. There is, therefore, a strong argument to support the idea that instead of requiring that $\delta_B < 5$ mm and $\delta_H < 8$ mm there could be a requirement that $(\delta_B + \delta_H) < 13$ mm and $\delta_H < 8$ mm.

Secondly, there is an argument for changing these requirements on bridges designed for ballastless tracks where ballast degradation is not an issue. The current limits are based on acceptable movements in ballast. Logically, there are limits to the amount of movement which is acceptable on fastening systems for ballastless tracks but these have not been quantified. For standard fastenings the limits may be more severe than for ballasted track but special fastening systems may be used to accommodate much larger displacements.

However, at the time of writing this report, there is insufficient evidence to confirm that either of these suggested changes would result in the design of safe structures and maintainable track, and in addition there is no data available to provide a basis for applying different conditions for ballastless track. Consequently, in this respect, no changes are included in the recommendations made for the next revision of the Eurocode in Annex E. This remains an area in which there is scope for further revision of the Eurocode, or for applications for derogation from the Eurocode, as and when better evidence becomes available in the future.

5.7 Effects on track construction and maintenance activities

When the track is first laid on the bridge, the rails should be installed with a stress-free temperature which takes into account the temperature of the bridge structure at the time, especially for ballastless tracks. Ideally the rail stress should be neutralized to the usual (national) stress free temperature when

the bridge is in its zero position (e.g. SFT of 23 °C to 27 °C when the bridge temperature is about 10 °C in northern Europe).

Steps should be taken to avoid locating rail welds in the area around structure movement joints where rail stresses due to track-bridge interaction forces are expected to be high.

Some track maintenance activities may result in temporary changes to track-bridge interaction forces. For example if the rails are cut prior to re-railing or track renewal, bridge bearing forces may be affected.

6 History and background

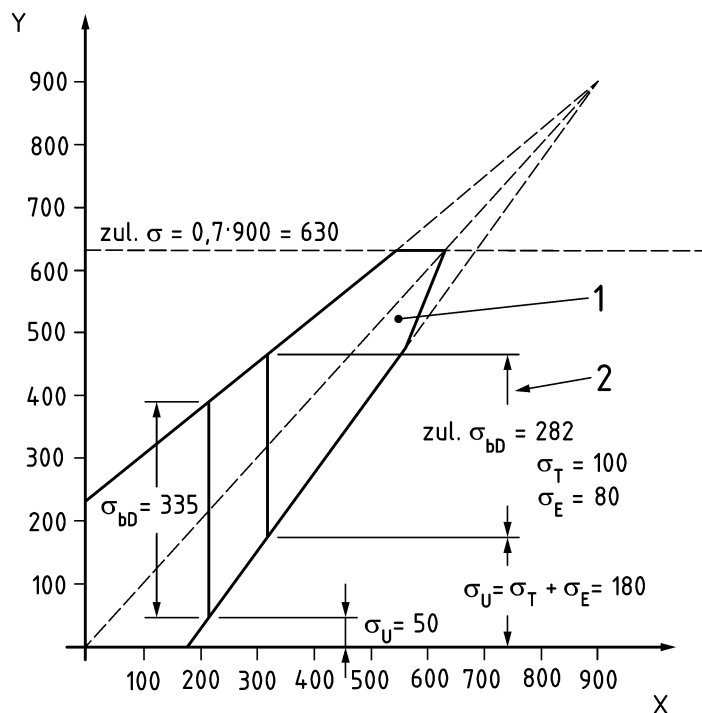
6.1 Existing codes and standards

In principle, current practice in Europe is driven by 6.5.4 of EN 1991-2:2003 which is based, in turn, on the Leaflet UIC774-3R last revised by UIC in 2001. Some insight into the background of the development of these design documents is given in a paper published by researchers at TU Delft in 1990, see [1] and in the (now superseded) Deutsche Bahn bridge design code DS 804, see [2].

The design standards (EN 1991-2 and DS 804) and code of practice (UIC 774-3R) were based on conventional bridge and track configurations i.e. bridges with “fixed” or “free” bearings on stiff piers carrying ballasted track with concrete sleepers. When more recent developments, such as the use of integral bridges and ballastless track, are used designers shall take into account the various ways in which they are moving away from the concepts upon which these documents are based.

These documents all include the requirement that the additional axial stress in the rail which results from the Track-Bridge Interaction effects should not exceed 92 MPa in tension or 72 MPa in compression. These limits apply to the very specific case of ballasted track with 'heavy' concrete sleepers at 650 mm spacing and UIC60 rail with a strength of at least 900 MPa (assumed to represent the 'standard' track configuration). However, no guidance is given on how to deal with other track configurations. It is therefore important to understand something of the origin of these figures. Some of the original data used has now been put into the public domain, see [3].

The starting point for the estimation of these allowable additional stresses was the Goodman-Smith diagram for the relevant type of rail. As an illustration, Figure 7 shows the diagram for new (as-rolled) rail with 60E1 profile and grade R260 steel. In [3] data are also included for a lighter rail section and for corroded and worn rails. All of these factors are taken into account in selecting recommended limit values for track and bridge design.

**Key**

- X σ_u, σ_o (alternating stress) (MPa)
- Y σ_m (mean stress) (MPa)
- 1 allowable stress zone
- 2 allowable bending fatigue stress at centre of rail foot (2×10^6 cycles)

Figure 7 — Goodman-Smith diagram for 60E1 rail grade R260

An estimate was made of the rail stresses expected in track away from any bridge for this 'standard' configuration and the results were subtracted from the allowable stresses shown on the Goodman-Smith diagram. Just to emphasize this point, the Goodman-Smith diagram is related to the material properties of the rail steel. The stress anticipated in track away from any bridge is related to the rail profile, sleeper spacing and general configuration of the track. Consequently, the allowable additional stress estimated in this way is only valid for that particular rail steel grade and track configuration.

For the standard track configuration, the estimated value of the allowable additional stress is 112 MPa in tension, where the potential failure mode is rail fatigue, and 72 MPa in compression, where the potential failure mode is track buckling. From the tension figure a further 20 MPa was subtracted, being the expected 'worst case' for the increase in axial stress in the rail due to (vertical) deflection of the bridge deck. Thus, this leads to the widely used figures of 92 MPa in tension and 72 MPa in compression as the allowable additional axial stress in the rails due to longitudinal Track-Bridge Interaction effects^[4].

The use of the 20 MPa approximation to the effects of bending was important when designers relied on simple 1-dimensional models, but with 2- and 3-dimensional modelling becoming widely available it is generally better to carry out a more detailed analysis. It should be noted that for bridges with long, flexible spans and for "bowstring" bridges the increase in axial rail stress due to bridge bending can be more than 20 MPa.

It is noted that the use of a limit of 112 MPa when bridge deck bending is calculated explicitly does conflict with the recommendation of UIC774-3R, 17.2 but is consistent with the analysis in [4].

6.2 Differences between national rules

National rules for track design tend to be based on the assumption that the only possible mitigating measure to prevent track failures over structure joints is the installation of a rail expansion device. In order to take into account all possible track design and maintenance conditions, such rules are generally extremely conservative. In the United Kingdom, for example, there is a requirement in Network Rail's standards, see [5] for all track on bridges with expansion lengths of more than 30 m to be fitted with rail expansion devices.

7 Case studies

7.1 Scheldt River Bridge (Belgium)

The bridge which carries the high speed railway line over the River Scheldt near Antwerp in Belgium has ballasted track with continuous welded rails. The longest span of the bridge has an expansion length of 120 m. Calculations were made using the methods of UIC774-3R for three cases: For track with continuous rails and no mitigating measures, for track with continuous rails and zero longitudinal restraint (ZLR) rail fasteners near to the structural movement joints and for track with rail expansion devices. The calculations were published in 1995, see [6] and showed clearly that these mitigating measures would be effective in reducing rail stresses to maintain a position well within the UIC774-3R limits. However, field measurements indicated that real track forces were much lower than those predicted. The measurements also indicated that the linear relationship between track forces and temperature, assumed in the UIC774-3R modelling, is over conservative. In the event, the track was installed on the bridge with no mitigating devices. During the first 20 years of operation there have been no unusual maintenance problems on this bridge.

7.2 Dedicated high speed lines in France and Spain

The high speed railway networks in France and Spain have a significant number of long bridges with ballasted track. Because of other requirements in the design of high speed railways, such bridges have only very large radius curves and also have a well consolidated, well maintained track. It has been demonstrated that bridges with expansion lengths of as much as 200 m without rail expansion devices have performed satisfactorily without exceeding the rail stress limits of UIC774-3R, see [7].

On very long viaducts, Spanish high speed lines have generally been designed with a small number of very large rail expansion devices, see [8], each allowing up to 1,2 m of longitudinal rail movement.

7.3 Olifants River Bridge (South Africa)

The bridge over the Olifants River on the iron ore export ('OrEx') line in South Africa carries a single track heavy haul railway with ballasted track with expansion lengths of almost 500 m. In 1988, a train was derailed on the bridge when the track buckled in hot weather, see [9]. This case serves to demonstrate that this type of track failure is more than a theoretical condition and that the consequences of such a failure can be very serious.

7.4 Bridges on Denver RTD (USA)

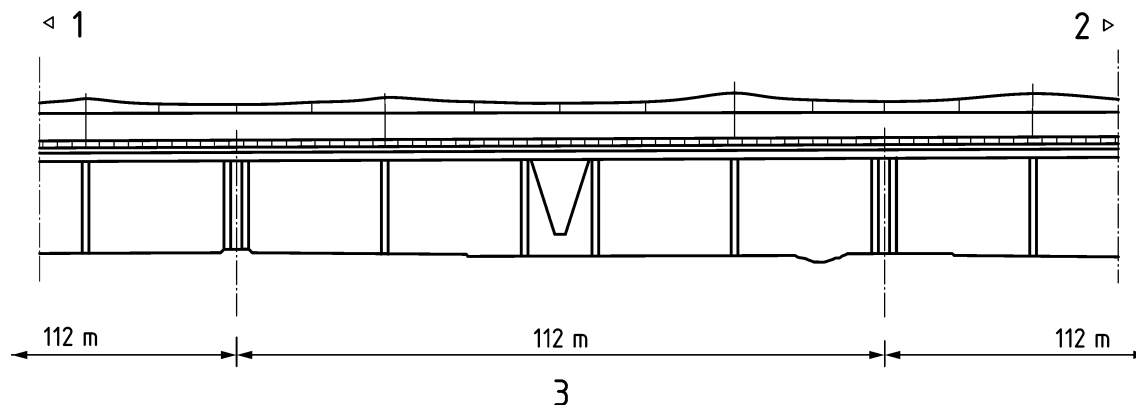
The details of two long, light rail viaducts (the 'Median Flyover Bridge' and the 'Colfax Bridge') on the Denver RTD system have been published, see [10] and provide a useful insight into the approach to structural design in the United States. The structures which are described have expansion lengths up to 220 m. The bearing forces and bridge loading were calculated in detail and many comments were made about the length of gap in the rail that would result from a fracture of the rail (limited by the specification to 75 mm). There does not appear to have been any requirement to limit the rail stress and no calculations of rail stress are presented i.e. the consequences of a rail failure are considered in detail but there does not appear to be any process by which the risk of such a failure is to be minimized. However, the rail used

for this project is “115RE” (57 kg/m) and the maximum axle load is less than 100 kN, so there is significant capacity for the rail to sustain higher stresses at the bridge joints.

7.5 Historic bridges in central Europe

Work carried out in modernising track on historic bridges in the Czech Republic, see [11] has been based on a range of limit values for different types of bridge allowed without rail expansion devices ranges from 20 m for directly fastened track to 180 m for 65 kg/m rail for ballasted track on concrete bridges.

7.6 Semi-integral bridges on German high speed lines



Key

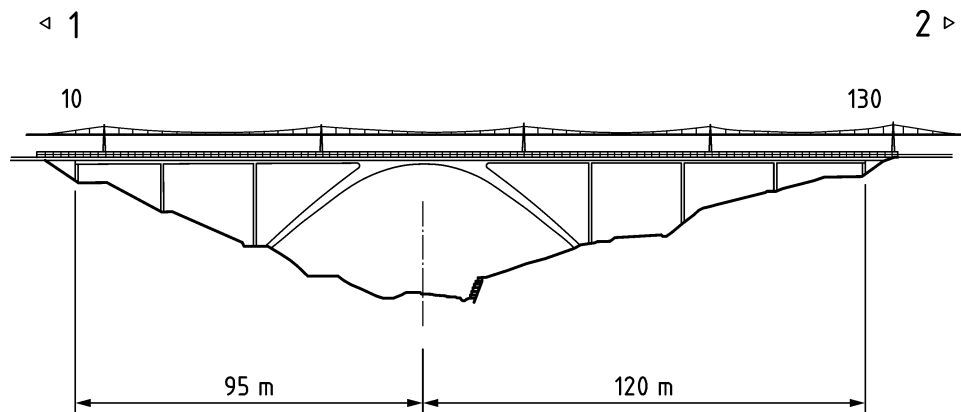
- 1 Erfurt
- 2 Leipzig
- 3 middle block

Figure 8 — Gänsebachthalbrücke

Full details of these cases have been published recently, see [12]. The Gänsebachthalbrücke is a 1 001 m long railway viaduct on the new high-speed line between Erfurt and Leipzig. The Gänsebachthalbrücke is designed as a semi-integral frame bridge with ballastless track. The topic of track-bridge interaction was one of the most important issues when choosing the static system in the design phase of the bridge. The expansion length of the continuous deck segments and the longitudinal stiffness of the substructure have been optimized to reduce the additional rail stresses to a level below the allowable stress limit of 92 N/mm².

The result of the design process was a series of eight “middle blocks” of each 112 m and two “border blocks” of 52,5 m. With this design, the 1 001 m long structure has been constructed without any bearings and without rail expansion joints.

This bridge with its very innovative design and low maintenance costs was the winner of the 2014 German Bridge Construction Prize.

**Key**

- 1 Nürnberg
- 2 Erfurt

Figure 9 — Grubentalbrücke

The Grubentalbrücke is a railway viaduct which supports ballastless track to carry the new high-speed line between Nürnberg and Erfurt. The bridge spans over a 215 m wide valley. A particularity of this line is the ambition of the Deutsche Bahn to build innovative bridges to improve the durability, decrease the maintenance effort and allow designs with high aesthetic quality.

The Grubentalbrücke is a semi integral structure. An arch and slender piers support the continuous superstructure. Both are connected monolithically to the superstructure, so that bearings are only needed at the abutments.

Because of the relevant expansion length of 120 m, the calculated additional rail stresses exceed the allowable limit of 92 N/mm² and therefore a rail expansion joint would be necessary at axis 130. To avoid this expensive device and to reduce disturbance to the continuity of the rail, special measures to compensate for this exceedance have been carried out:

- control of the bridge temperature by connecting the rail to the bridge;
- decrease the neutral temperature of the rail in order to reduce the tension stresses in the rail (rail buckling risk is not existent because of its connection to the slab track);
- long term monitoring of the rail stresses to determine the actual stresses in the rail over time;
- Possibility to decouple the rail from the bridge in the future.

8 Design considerations for track.

8.1 Representation of axial behaviour of track.

The transfer of axial forces between the rail and the bridge deck depends on the overall behaviour of the track structure when loaded in vertical shear. If a test is carried out on a section of track, it is found that the load-displacement relationship is, typically, as shown in Figure 20. For ballasted tracks, this relationship is dominated by the behaviour of the ballast itself. For ballastless tracks, the relationship is almost entirely governed by the behaviour of the rail fastening system.

In order to carry out any analysis, the load-displacement relationship is usually represented by a simple bi-linear diagram. The initial “elastic” displacement takes place up to a limiting value of u_0 . If the displacement is greater than u_0 it is assumed that the force between the rail and the bridge deck has a

constant value of k . Throughout this report, k is taken to be the total force in two rails, per metre length of track i.e. it has the units of force per unit length, but it is NOT a stiffness.

For long, multi-span bridges with track forms which have low values of u_0 (e.g. conventional ballastless tracks) a further simplifying assumption may be made, using a fully plastic model i.e. in effect, setting $u_0 = 0$. This makes it possible to carry out very quick calculations with minimum amounts of computing power (see Annex B) but should be used with caution and should only be used for initial calculations and not be used for final design assessment.

8.2 Understanding of ballast behaviour

8.2.1 Ballast properties

In contrast to rails, sleepers or fastenings, the ballast is not a material with well-defined properties.

The ballast properties vary considerably and even change over time and with the humidity and temperature. For instance, the lateral track resistance (LTR) of freshly tamped ballast is reduced by 50 % compared to consolidated ballast. Therefore exact calculations based on one single value of the input parameter are not generally applicable. A reasonable range of possible values has to be considered and the sensitivity of the results has to be analysed. The results are to be used as an input parameter for further calculations (such as bridge design) where the whole range has to be taken into account.

The ballast comprises an assembly of individual granular particles with multiple contact points which rely on the normal and tangential resistance at the inter-particle contacts. The ballast can therefore only transmit compressive forces which are resisted at the contact points but cannot transmit tensile forces which tend to reduce the inter-particle resistance. Under a static compressive load the ballast behaves like a solid body. If the ballast formation is stiff then the ballast is also stiff. If the formation behaves in an elastic way then the ballast is also elastic. The principles of soil mechanics have to be applied and take account of the ballast and formation soil properties. Under a dynamic load, vibrations may lead to ballast flowing and destabilization so that the track stability is at risk and also the cant in curves can reduce.

If there is little or no vertical load, and sufficient axial force is applied either to the bridge deck (e.g. thermal effects) or the rail (e.g. traction and braking effects) the sleepers will begin to slip in the ballast, i.e. if $u > u_0$ for unloaded ballasted track, the most likely factor determining the force, k , is the ability of the ballast to restrain the sleeper position. If a significant vertical load is applied, the ballast stones “lock together” and the value of k is increased. In this case it is more likely that the rail will slip relative to the sleeper, i.e. if $u > u_0$ for loaded ballasted track, the most likely factor determining the force, k , is the ability of the rail fastening system to restrain the rail.

8.2.2 Importance of effective ballast retention

In order to ensure that there is sufficient and consistent support for the track on ballasted deck bridges, measures should be taken to ensure proper retention of the ballast and the ballast shoulders at structure movement joints and bridge deck ends. In 2012, SNCF initiated a survey of European railways undertaken by UIC, see [13] to confirm that such measures were being used where appropriate and that they were effective. The survey was initiated by SNCF and the questionnaire was mainly targetted at railway bridge owners with large railway viaducts carrying high speed lines with ballasted track and continuously welded rails (CWR).

8.3 Description/ limitations of available track devices for mitigation of effects

8.3.1 Principles

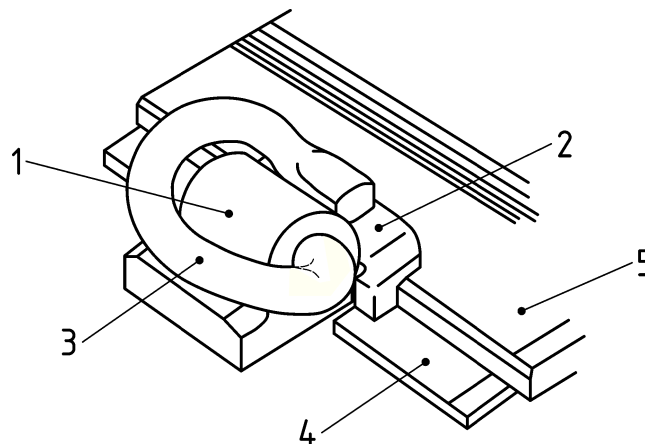
The rail fastening system is defined as the assembly of all of the components which together fix the rail to the sleeper or the rail to any other supports (e.g. baseplate or track slab). A typical assembly of components for an elastic rail fastening system is shown in Figure 10.

The purposes of the rail fastening systems are:

- to absorb the vertical and longitudinal rail forces elastically and to transfer them from the support;
- to isolate or damp vibrations caused by traffic;
- to maintain the track gauge and rail inclination within certain tolerances;
- to provide electrical insulation between the rail and the support.

The rail fastening system is generally made of:

- 2 elastic components (clips) which apply a vertical downward load on the rail foot. The clamping force provided by the clips is about 20kN per rail seat. The clamping force depends mainly on the clip design.
- 2 side insulators placed on either side of the rail foot. The side insulators are used for electrical insulation of the rail and for gauge adjustment.
- 1 rail pad placed between the rail foot and the support. The functions of the rail pad are to insulate the rail electrically from the support, to provide vertical resilience to the track and to increase the longitudinal creep resistance of the rail relative to the support.



Key

- 1 shoulder or anchor
- 2 insulator
- 3 spring clip
- 4 pad
- 5 rail foot

Figure 10 — Components of a typical elastic rail fastening system

The combination of the clamping force and the rail pad allow the longitudinal resistance to be obtained relative to the longitudinal stiffness of the support. The longitudinal restraint of the fastening can be measured following the test method described in EN 13146-1. By using a clip design and/or sliding interface between the rail foot and the rail seat the longitudinal resistance of the fastening system can be reduced. Such a solution can be used to reduce the rail stress on bridges due to Track-Bridge Interaction.

In cases where longitudinal Track-Bridge Interaction calculations are based on finite element analysis (FEA) the vertical stiffness of the rail fastening has no influence on the results of the calculation. However,

the vertical and uplift stiffness values are significant in calculating the effects of bridge deck end rotation on the fastening system.

A structural assessment should be undertaken as part of the analysis and design procedure for track on bridges to demonstrate that the complete system complies with performance specifications.

Compliance with Track-Bridge Interaction design limits is achieved preferably by the use of the basic, unmodified track structure applied on the bridge or, if necessary, the use of track with additional devices that offer mitigation to those effects that are responsible for the non-compliant situation.

Track devices that offer mitigation principally consist of (a combination of):

- components to reduce rail stress;
- components to enable differential longitudinal movement to occur;
- (for ballastless tracks) Components to enable lateral movement to occur.

8.3.2 Practical solutions

Reduction of rail stress can be achieved by the use of reduced- or zero-longitudinal restraint (RLR or ZLR) rail fastenings on sleepers in ballast or ballastless track. This is delivered by a combination of low-friction rail pads and low clamping force and/or low friction insulators that replace the conventional rail pads, clips and insulators.

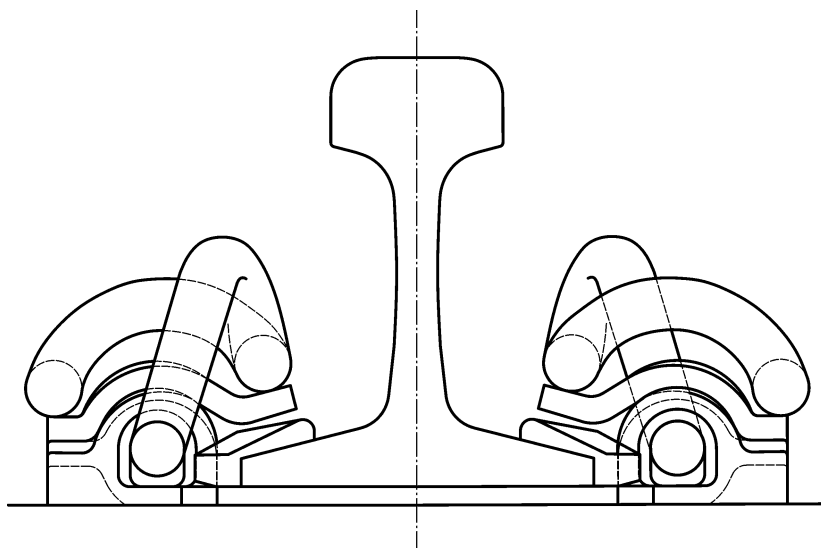


Figure 11 — Zero longitudinal restraint (ZLR) fastening for ballasted track

For ballastless track, reduced or zero longitudinal restraint (RLR or ZLR) rail fastening systems use a combination of low friction rail pads and low toe load clips and/or low friction insulators.



Figure 12 — Zero longitudinal restraint (ZLR) fastening for ballastless track

Alternatively, and in cases where the methods described above cannot provide sufficient mitigation of the rail stresses, eliminating the axial rail stress is an option. This can be achieved by the provision of a rail expansion joint (REJ) or other type of rail expansion device. In practice, the longitudinal force cannot be reduced to zero, because there are always some friction effects, but it can be reduced to a level so small that it can be ignored in the analysis.

Rail expansion devices shall not be used on lines with heavy traffic or with high cant (super-elevation) without the approval of the responsible track engineer. The track engineer may also restrict (or even prohibit) the use of rail expansion devices on very high speed lines or in noise-critical locations on urban railways.

Standard REJs have an expansion capacity of up to 120 mm and usually extend over a distance of about 4,8 m, i.e. up to 8 rail support points.



Figure 13 — Rail expansion joint (REJ) in ballasted track



Figure 14 — Rail expansion joint (REJ) in embedded rail



Figure 15 — Rail expansion joint (REJ) in ballastless track

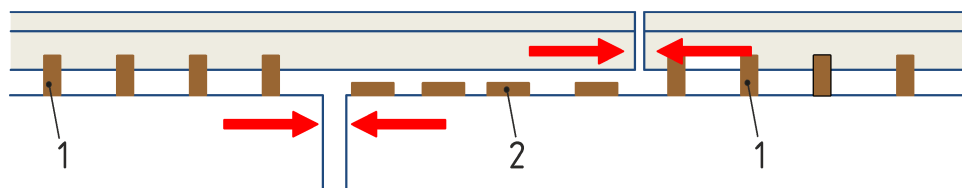


Figure 16 — Special rail expansion device in ballastless track

Special rail expansion devices may be provided with greater expansion capacity. An important difference between standard REJs and special rail expansion devices is the capacity of the latter to accommodate differential displacement of both rail ends. A rail expansion device requires a longer length to accommodate the increased movement capacity. Details of REJs and other rail expansion devices are covered by EN 13232-8.

When any of the above methods is used, the input parameters used for the calculation of rail stresses and bearing forces should be carried out using characteristic values measured for the components in question. It is also advisable to carry out additional calculations to ensure that, in the event of a rail break the gap which opens up is not excessive (see 8.5.2).

Combinations of devices may also be used. There are a number of advantages in placing REJs a few metres away from structure movement joints. This provides better support for the rail expansion device and also prevents impact loads arising at the rail joint from causing damage to the structure joint. In such cases the rail shall be free to slide between the two joints, e.g. by use of ZLR fastenings (see Figure 15).

**Key**

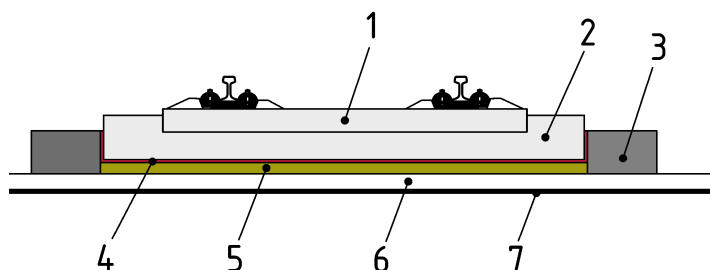
- 1 conventional rail fastenings
- 2 ZLR rail fastenings

Figure 17 — Example of a combination of REJ and ZLR fastenings

Finally, a solution which may be adopted in some extreme cases is the concept of “sliding slab track”, in which a short section of concrete slab track is de-coupled from the bridge structure.

The principle of the sliding slab track consists of uncoupling the track from the bridge superstructure in longitudinal direction [14]. The effects of track bridge interaction can almost be switched off. The principle is detailed in the patent [15] and in [14]. This track solution is used in Germany for ballastless track on some short bridges (length < 25 m).

The slab track is jointless over the entire bridge length and anchored behind the abutments. Lateral forces are taken by concrete humps to the side of the slab (see Figure 18). To uncouple the track and the bridge, a sliding layer (generally 2 plastic foils and 2 bituminous paper layers) is interposed.

**Key**

- 1 embedded track element
- 2 track slab
- 3 concrete humps
- 4 sliding layer
- 5 rigid foam layer
- 6 protective concrete
- 7 bridge sealing

Figure 18 — Longitudinal sliding slab track on a short bridge (DB AG)

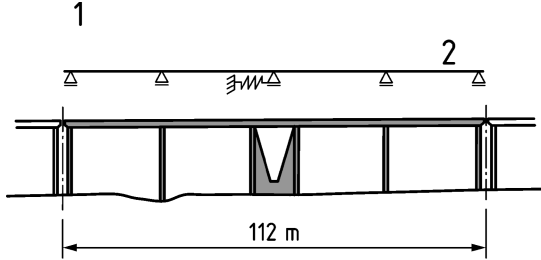
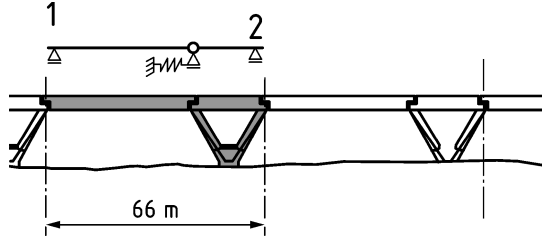
The principle of sliding slab track has been built on long bridges in China on the High-Speed Line Beijing-Tianjin (Bögl integral). A similar technique was adopted on the approach to St. Pancras station in London when new tracks were built to connect to the high speed line towards France. In that case the restricted space available for the junction layout made it necessary to install switches and crossings which were partly on a bridge and partly on earthworks. A short section of slab track was provided under the special trackwork so that, once again, it could be de-coupled from the bridge.

NOTE For long bridges, some sliding slab track solutions are subject of patents.

8.4 Description/ limitations of bridge design for mitigation of effects

8.4.1 General

As shown in the case study in 7.6, the static system of the bridge, the position of the fixed point and the length of the bridge are determinant parameters for the distribution and the value of rail stresses. The topic of track-bridge interaction shall be taken into account in a very early phase of the bridge design. By choosing a bridge structure which has been adapted to reduce rail stresses, it is possible to avoid rail expansion joints or to minimize their number [16].

Example 1: Gänsebachtalbrücke HSL Erfurt-Leipzig/Halle (Germany)	Example 2: Viaduc de la savoureuse HSL Rhin-Rhône (France)
 Photograph © MarxKrontal	 Photograph © SNCF
 1 static system 2 concrete deck	 1 static system 2 steel deck

8.4.2 "Steering bars" and virtual fixed points.

The principle of "steering bars" is to define the position of bridge fixed points, independently from the bearing layout of the bridge [14], [17].

The virtual fixed point is achieved by utilizing a kinematic bar design which can absorb temperature displacements without the development of significant restraining forces and the braking forces are distributed to both abutments.