SLOVENSKI STANDARD
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Evrokod 1: Vplivi na konstrukcije - Prometna obtežba mostov - Medsebojni vpliv tračnice-most

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

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

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

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TECHNICAL REPORT

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

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

This draft Technical Report is submitted to CEN members for Vote. It has been drawn up by the Technical Committee CEN/TC 250.

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EUROPEAN COMMITTEE FOR STANDARDIZATION
COMITÉ EUROPÉEN DE NORMALISATION
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European foreword

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

This document is currently submitted to the CEN Enquiry.

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 11 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 12 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 13 of this report).

1 Scope

This Technical Report 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 Technical Report 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 effects
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")

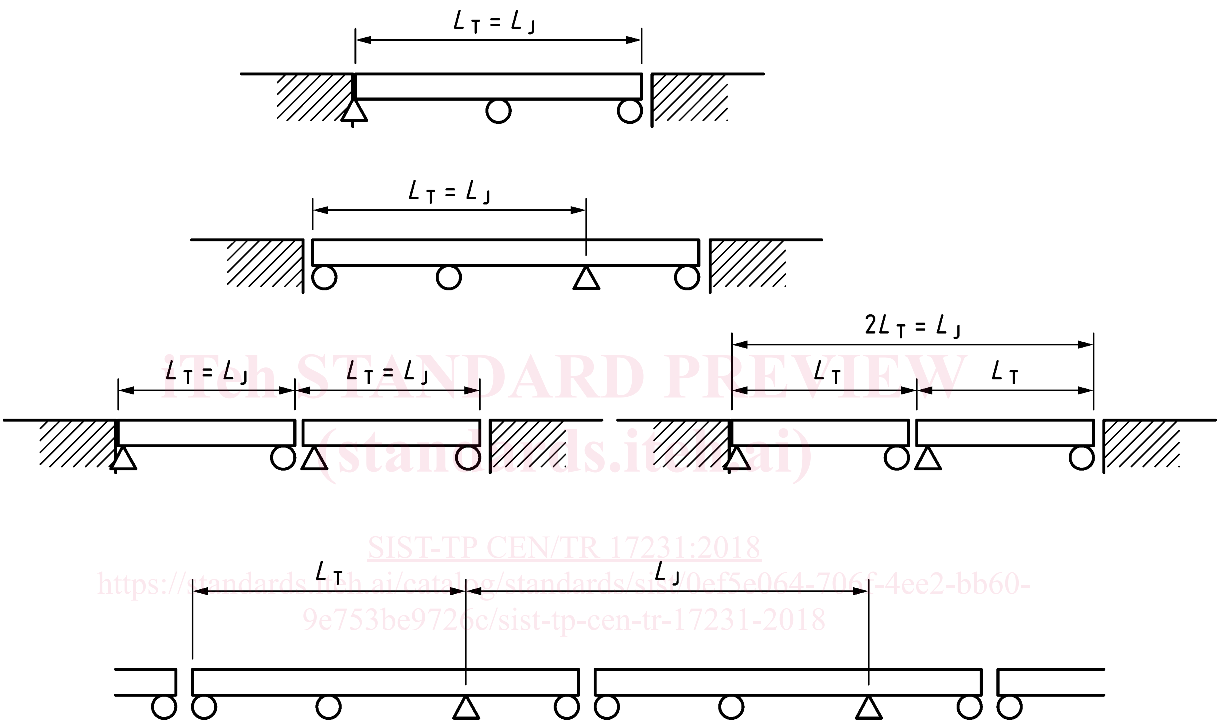
3.4
deck length
L_D
The distance between structural movement joints in the bridge deck

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 Fig.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. (See Fig. 1)



Key

△ represents a 'fixed' support

○ represents a 'free' support

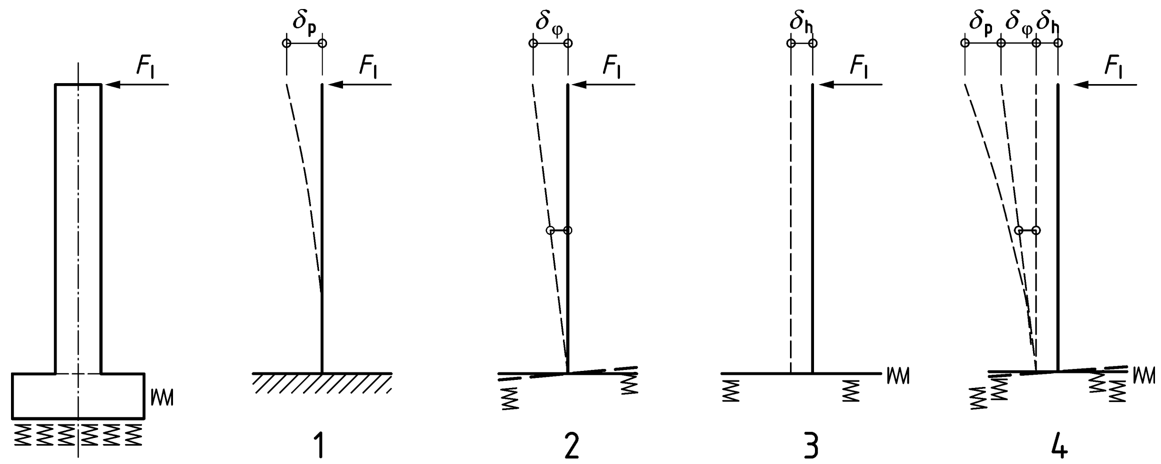
Figure 1 — Examples of expansion lengths L_J and L_T

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 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	Serviceabiity 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)
α, α^{th}	coefficient of thermal expansion. For rails, it is assumed that $\alpha = 1{,}2 \times 10^{-5}\ K^{-1}$
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 Clause 3 definition 7)

[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_{p}	longitudinal displacement due to bending of the pier (see Clause 3 definition 7)
δ_{V}	axial displacement of the bridge deck due to vertical loading
$\delta_{\theta\mathrm{D}}$, $\delta_{\theta\mathrm{R}}$	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 Clause 3 definition 7)
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 to 56 kg/m with sleepers 650 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 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

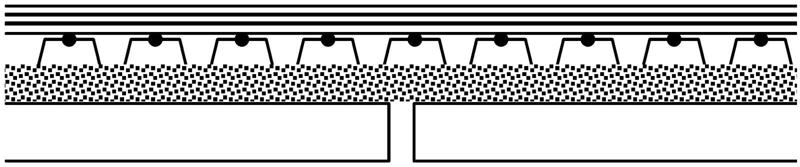


Figure 3 — Typical configuration of track across a structural joint

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 and 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.