RAILWAY BRIDGE SERVICEABILITY LIMIT STATES AND RAIL STRESSES FOR TRACK-BRIDGE INTERACTION – OVERVIEW AND CASE STUDY

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Abstract

Railway bridges carry rail tracks, which are typically not discontinued between embankments and the bridge structure. This continuity introduces additional forces in the rail track and its fastenings, as well as in the bridge bearings and substructure. Alternatively, rail expansion devices can be installed near the bridge joints, typically at one or both abutments, which reduce or eliminate the influence of the bridge on the tracks. However, expansion joint devices are expensive, require maintenance, introduce additional impact forces, and increase noise, especially on high-speed railways. Therefore, whenever possible, they should be avoided in favor of additional design verifications to allow for continuous welded rail (CWR) track [1]. Design criteria are adopted within EN 1990-A2 [2] and HRN EN 1991-2 [3] to limit displacements and stresses, preventing unacceptable deformations or rail failure. Serviceability limit state criteria, including limits on deck uplift, twist, displacements, and changes in vertical or horizontal curvature for both traffic safety and passenger comfort, must be satisfied according to EN1990-A2 [2]. To determine if the additional rail stresses are within permissible limits, to prevent tension failure or buckling of the rail, the combined response of the bridge structure and railway track must be considered. Simplified calculation methods to account for this interaction are provided in EN1991-2 [3], but they can only be used for bridges with expansion lengths less than 60 m (steel superstructure) or 90 m (concrete and composite superstructure). For larger expansion lengths, a more complex analysis must be carried out, including an integral model of the bridge, embankment, track, and the non-linear connection between them. This paper gives an overview of the Eurocode design verifications for CWR regarding deformations and rail stresses, and its application to a case study of a two-track railway bridge. Special emphasis is given to a calculation model for deriving the influences of track-bridge interaction.

Keywords: railway bridge, continuous welded rail, expansion device, track bridge interaction, steel stress, serviceability

1 Introduction and theoretical background

Continuous welded rails (CWR) is a todays standard for railways, especially for high-speed tracks, where the rail is continuous over very long distances (tens or hundreds of kilometers). Rails are fixed to the sleepers by a clamping fastener, with an adequate force so that the rail movement is transmitted to the sleepers, and then further to the ballast [4]. Rail movements are therefore restricted by the ballast stiffness, causing additional stress in the rail steel due to thermal and traffic actions. This stress is anticipated in relation to the rail profile, sleeper spacing and track configuration [1]. Permissible stress limit is influenced by rail fatigue strength and maintenance level (appropriate safety factor), which is determined by Goodman-Smith diagram.

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There a maximum permissible stress limit is given at 470 MPa (for UIC60 rail with 900 MPa strength) [5, 6]. In Goodman-Smith diagram total stress in CWR comprises of contributions from residual stress due to rail steel rolling (80 MPa), temperature stress (120 MPa), and bending stress from wheel force between the sleepers (158 MPa) [5].Taking into account all these effects, for a permissible stress limit of 470 MPa to be reached, only 112 MPa is freely available [1]. When the rail ballast bed is replaced with a bridge structure, additional stresses can occur in the track due to changes in the longitudinal stiffness of the ballast support (bridge substructure stiffness), as well as the deformations and movements in the bridge superstructure (vertical deflections and rotations of the bridge deck supporting the rail – bridge superstructure stiffness). These additional stresses need to be accounted for and calculated, so that the total stress is not exceeding the permissible limit, which can lead to tensile failure or lateral buckling of the rail [1]. To do so, track-bridge interaction must be considered since any forces applied to CWR also induce additional forces and movements in the bridge deck and bearings, and vice-versa, any movements of the bridge deck induce additional forces in the CWR [4]. This interaction can also cause the increase of the bridge bearing forces and/or uplift of the track elements.

Track-bridge interaction can be avoided by installing rail expansion devices (RED) on either just one or both deck ends (usually only on the deck end where the bridge movable joint is located). REDs are considered only when the track-bridge interaction leads to exceeding permissible stress in the rail or excessive forces in bridge bearings, or when bridge movements exceed limits set by rail serviceability limit state (traffic safety and traffic comfort conditions). Whenever possible, RED are to be avoided due to their cost, high maintenance requirements, introduction of additional impact forces, and increase of noise, especially in urban areas [1]. Thus, verifications must be made to ensure the integrity of CWR (rail stress), but also traffic safety and traffic comfort verifications, which consist of serviceability limit states conditions for maximum allowable movements and accelerations in vertical and horizontal direction. Criteria for performing such verifications are given in Design codes (Eurocode EN 1990A2 [2] and EN 1991-2 [3]), which rely heavily on the code of practice UIC 774-3 [4]. All of these are integrated and condensed in the technical report CEN/TR 17231 [1]. In the following sections an overview will be given of all the relevant verifications applied to a case study of a two-track railway bridge with a 125 m expansion length, due to which a detailed model and calculation of track-bridge interaction is asked for.

2 Case study bridge

A case study bridge has been selected for SLS deformations and vibrations verifications, as well as track-bridge interaction analysis. Bridge is a two-track continuous girder over 7 spans, 126 m in total length (figure 1). Superstructure cross section comprises two edge 2200 mm high I girders (flanges 600 × 50, web 2100 × 20) distanced at 9.2 m, connected by 1100 mm high cross girders, distanced at 1.74 m to 2.5 m. Bridge deck is a 14 mm thick steel orthotropic slab with open sectioned 250 × 20 mm stiffeners distanced at 400 mm. 60E1 track is laid on ballast, up to 65 cm thick. Main girders are supported by discrete columns (cross section Φ80 cm, height up to 8 m) which are distanced differently for each girder, causing skew support conditions for superstructure girders. Span lengths are ranging from 15 m to 21 m. Each column is extended to a single pile foundation (Φ130 cm, up to 11 m in depth). Abutments are massive reinforced concrete structures 10 m wide and up to 13 m long, founded on 9 piles and a pile plate 150 cm thick. Each girder is supported by 8 pot bearings, 4 of which are fixed for all movements (figure 1). Since all columns are very slender (pendl-like columns), a longitudinally fixed point is considered only at abutment U8, and transversally fixed points are at both abutments for just one girder.

This yields an effective expansion length L_T equaling the bridge length of 126 m, starting from abutment U8, towards abutment U1, where a single rail expansion device is installed for the whole bridge (as recommended in [1, 4] for expansion lengths between 60 and 140 m). Numerical model of a bridge was made, consisting of beam elements (main girders, cross girders, deck stiffeners, columns, piles), plate elements (abutments, deck slab) and spring elements (bearings, foundation supports matching soil conditions) (figure 1).

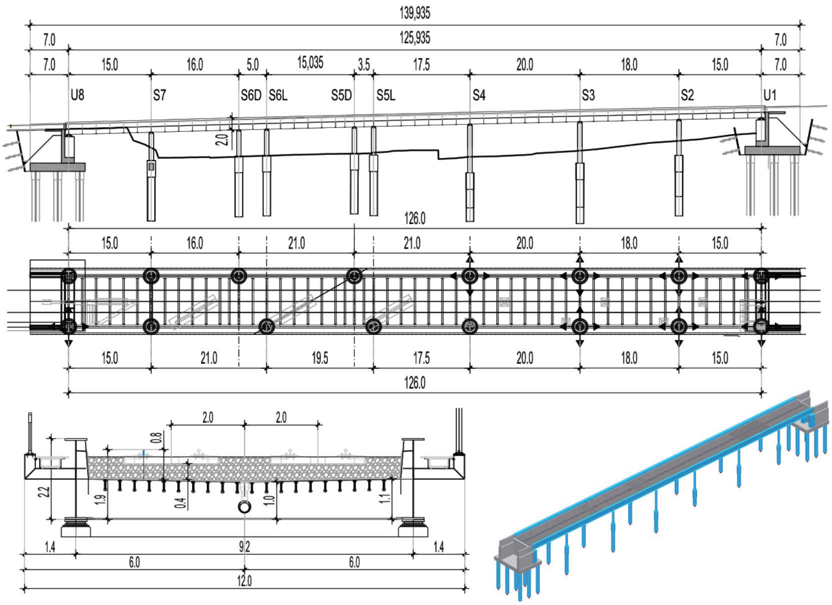

Figure 1 Longitudinal section, cross section, and layout of the case study bridge

3 Deformations and vibrations verifications (SLS)

Verifications regarding deformations and vibrations for railway bridges ensure that excessive deformations will not endanger traffic by creating unacceptable changes in vertical and horizontal track geometry, nor will they cause excessive rail stresses and vibrations in bridge structure [2]. Forces lifting the rail fastening system must be minimized by limiting vertical displacements at deck ends. Horizontal displacements can cause weakening of the ballast and destabilization of the track [2]. Risk of derailment at bridge expansion joints near the abutments must be minimized by limiting any angular discontinuity of the track [7]. Excessive vibrations can cause ballast instability and loss of wheel to rail contact surface. All these changes can reduce traffic safety and passenger comfort [2]. They are to be calculated as serviceability limit state (SLS) regarding bridge elements (traffic comfort) and ultimate limit state (ULS) regarding the rail (traffic safety) [7]. However, displacement and stress limit values given in UIC 774-3 were calibrated with the old permissible stress method, so no additional safety factors are to be applied [7]. Figure 2 shows an overview of the relevant Eurocode sections for conducting deformations and vibrations verifications, either for traffic safety (red boxes) or passenger comfort criteria (green boxes). Both criteria are primarily given in EN 1990 A2.4.4 [2], but most of them are then further cross referenced with EN 1991-2 [3] in sections dealing with bridge dynamic effects (EN 1991-2 6.4.4) or combined response of the structure and the track (EN 1991-2 6.5.4).

Excessive deformations can affect the loads imposed on the track–bridge system and thus create dynamic effects (resonance, excessive accelerations, ballast instability, dynamic factor amplification…) which demand a dedicated dynamic analysis considering bridge-vehicle interaction (real high speed load model train- HSLM). Conditions which must be satisfied to avoid dynamic analysis are given in section 6.4.4 of EN 1991-2 [3]. For train speeds less than 200 km/h and continuous girder structural systems no dynamic analysis is required. For most other simple structures (beam or plate structures with rigid supports and no skew effects) upper and lower limits for first natural frequency according to chart in EN 1991-2 6.4.4 must be met (for span lengths from 2 m to 100 m).

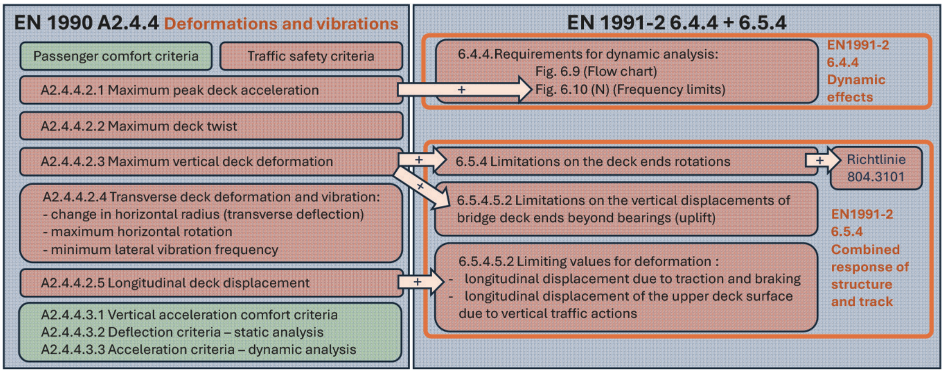

Figure 2 Eurocode railway bridge deformations and vibrations verification outline

3.1 Traffic safety criteria

Table 1 shows all the traffic safety conditions and how they apply to the case study bridge. Dynamic analysis is not needed because the bridge is a continuous girder for train speeds less than 200 km/h. Vertical peak acceleration of the deck, limited to 3.5 m/s^2 for ballasted track, is satisfied by checking the range condition for first vertical frequency of the bridge. Further conditions for deck deformations are all checked for traffic loads LM71, SW/0 (and additionally SW/2 for vertical deformations), multiplied by appropriate Φ and α factors as noted for each condition in table 1. Longitudinal deformations are checked for traction and braking forces, and transverse for temperature difference and wind. A2.4.4.2.3 from EN 1990 [2] states that vertical deck deformation should not exceed L/600. However, to avoid excessive track maintenance, a more conservative condition of L/800 for v ‹ 80 km/h is adopted [7]. For angular rotations of deck ends no explicit condition is given in Eurocode, so a condition from Richtlinie 804.3101 [8] is adopted. Longitudinal deck displacement conditions (5 mm and 8 mm for horizontal and vertical loads, respectively) are set to reduce ballast degradation [1]. All the conditions for a continuous two track case study bridge are met (table 1).

Table 1 Conditions for traffic safety in a case study bridge

Condition	Demand	Case study bridge	
Peak deck acceleration for ballasted track $\gamma_{bt} = 3.5\ m/s^2$	Dynamic analysis?	Not needed: v ‹ 200 km/h.continuous bridge	✓
	$n_{o.inf} < n_o < n_{o.sup}$ $n_{o.sup} = 94.76 L_\Phi^{-0.748}$ $n_{o.inf} = 23.58 L_\Phi^{-0.592}$	L = 27 m 3.35 Hz ‹ f_{v1} ‹ 8.05 Hz f_{v1} = 4.37 Hz	✓
Deck twist (Φ≠1.0; α=1.21)	v ≤ 120km/h ⇢ t ≤ 4.5mm	t = 1.0mm	✓
Vertical deck deformation (Φ=1.0; α=1.0)	L / 800 = 15 / 800 = 19 mm L / 800 = 21 / 800 = 26 mm	11 mm 17 mm	✓
Deck ends rotations (Φ≠1.0; α=1.21)	θ_{lim} = 3.5 mrad [8] (for two track bridge)	θ = 3.38 mrad	✓
Deck ends uplift (Φ≠1.0; α=1.21)	3 mm (for v ≤ 160 km/h)	0.5 mm	✓
Change in horizontal radius (Φ≠1.0; α=1.21.wind.temp)	Radius change r › r_1=1700m (for v ≤ 120 km/h)	$r = \frac{L^2}{8\delta_H} = \frac{125{,}9^2}{8 \cdot 0{,}038} = 52.140m$	✓
Horizontal rotation (Φ≠1.0; α=1.21.wind.temp)	α ≤ α_1 = 0.0035 rad	α = 0.0019 rad	✓
Lateral deck frequency	f_{ho} ≥ 1.2 Hz (for one track bridges)	Not applicable	
Longitudinal deck displacement (traction and braking loads)	δ_B ≤ 5 mm (for RED at one deck end)	δ_B = 1.7 mm	✓
Longitudinal deck displacement (vertical loads)	δ_H ≤ 8 mm (for RED at one deck end)	δ_H = 1.9 mm	✓

3.2 Passenger comfort criteria

Passenger comfort level criteria is set by defining a limiting value for vertical acceleration b_v. Recommended levels of comfort are given in A2.4.4.3.1 as very good (b_v = 1.0 m/s²), good (b_v = 1.3 m/s²) and acceptable (b_v = 2.0 m/s²). These levels can be verified by adopting maximum vertical deflection under LM71 (Φ ≠ 1.0; α = 1.0; only one track is loaded). Maximum vertical deflection is given as a L/δ ratio dependant on the vehicle speed, span length, span number and structural system (figure A2.3 from [2]). Alternatively, acceleration can be determined from a dynamic analysis. Table 2 shows passenger comfort verification for a case study bridge. It is worth noting that in most cases passenger comfort criteria has no significance when vertical deformations are already satisfied according to traffic safety criteria adopting the permissible deflections to avoid excessive track maintenance (given as formula δ ≤ L/(15v[km/h]-400); L/800 for v ‹ 80 km/h; L/2600 for v › 200 km/h) [7].

Table 2 Conditions for passenger comfort in a case study bridge

Condition	Demand	Case study bridge	
Vertical deflection for LM71 (Φ≠1.0; α=1.0; one track loaded) L/δ = 900 · 0.9 = 810 (v ≤ 120km/h.b_v=1.0 m/s². continuous girder.7 spans 15-21 m)	L / 810 = 15 / 810 = 18 mm L / 810 = 21 / 810 = 26 mm	4 mm 10 mm	✓

4 Track-bridge interaction

Track-bridge interaction can be accounted for by several calculation methods, depending on the expansion length, track construction and material of the bridge superstructure. The simplest method, given in section 6.5.4.6.1 of EN 1991-2 [3], is valid only for expansion lengths up to 40 m. Second, seldomly used method utilizing graphical data given in annex G of EN 1991-2 [3], is valid for expansion lengths up to 60 m (steel superstructure) or 90 m (concrete superstructure). Both methods originate from [4]. Since the case study bridge has a larger expansion length of 125 m, a detailed numerical analysis is needed, in compliance with sections 6.5.4.2 to 6.5.4.5 of EN 1991-2 [3]. This analysis is performed using nonlinear FE models including bridge structure, embankments and rail, with various ballast stiffnesses (figure 3).

4.1 Considered load effects and combinations

For assessing additional stress in the rail, following actions need to be considered: traction and braking longitudinal forces (applied only on rails above the bridge); classified vertical traffic loads (considered load cases from LM1, SW/0 and SW/2, with $\Phi = 1.0$; $\alpha = 1.21$); and different thermal effects between structure and track (temperature variation in the bridge $\Delta TD = \pm 35°C$, temperature variation in the track $\Delta TN = \pm 50°C$, $|\Delta T\_D-\Delta T\_R| \leq 20$). It should be noted that for vertical loads, only the axial force is contributing to the additional rail stresses calculation, while bending stress is neglected. Bending stress in the rail is already accounted for as 20 MPa when the "reserve" for additional stress was set (difference between freely available 112 MPa from Goodman-Smith diagram and finally allowed 92 MPa of additional tension stress) [1]. If the stress due to bending is calculated explicitly, a limit of 112 MPa can be used [1]. Each action is to be calculated nonlinearly according to the different bilinear stiffness of the springs representing the ballast (figure 3). Forces in the rails from each action may be combined using linear superposition [3].

4.2 Detailed modelling and stress analysis

Finite element model is constructed according to provisions given in EN 1991-2 and UIC 774-3R [3, 4]. Experience and recommendations gathered from similar models shown in [9, 10] was considered. Validity of the model was tested against the results from "test case" of a similar bridge type (example C.4 presented in [1]). According to [4], the model includes 300 m of embankment on each side of the bridge (figure 3a). Rails (two tracks with UIC 60E1 beam sections) are supported by longitudinally bilinear springs which are then either fixed at the bottom (embankment part) or connected to bridge deck (figure 3c). Longitudinal stiffness of the springs is dependent on the ballast conditions (chart numbering from figure 3c – (3): loaded/frozen 60kN/m/0.5mm; (4): loaded/unfrozen 60kN/m/2mm; (5): unloaded /frozen 30kN/m/0.5mm; (6): unloaded/unfrozen 20kN/m/2mm). Unloaded ballast stiffness conditions are used for all the springs on the embankment. Frozen ballast stiffness conditions are only used for calculation of negative temperature difference (winter). Vertical spring stiffness is set to 24 MN/m. For each action two calculations were made, one with the active bridge elements supporting the tracks, and the other where the bridge elements were fixed (to simulate the embankment conditions, as if the bridge was not present). Stress difference between both cases was made to show what additional stresses arise due to bridge structure in place. They were than compared to the maximum limit of 92 MPa for tension (potential fatigue failure) and 72 MPa for compression (potential buckling failure). The largest additional stresses are present where the tracks are entering the bridge superstructure with no rail expansion device (U8) (figure 4c). Temperature difference yields no additional stresses. Combined stresses from all actions are within the limit of additional stresses (figure 4c) (max 47 MPa compression, max 61 MPa tension).

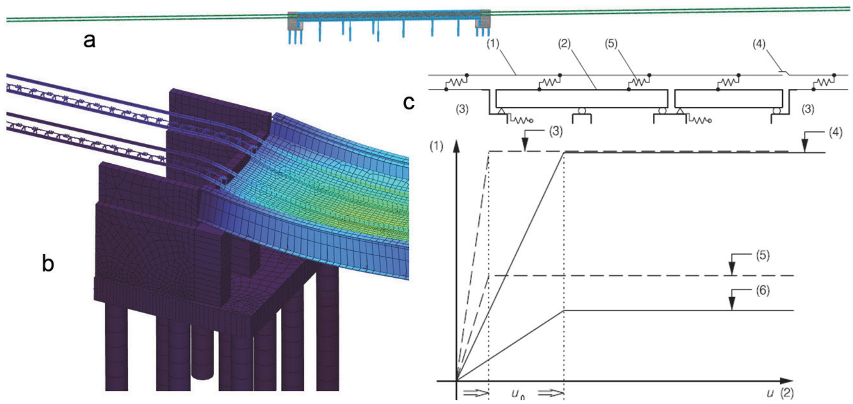

Figure 3 a) track-bridge model, b) vertical load deformation, c) ballast springs

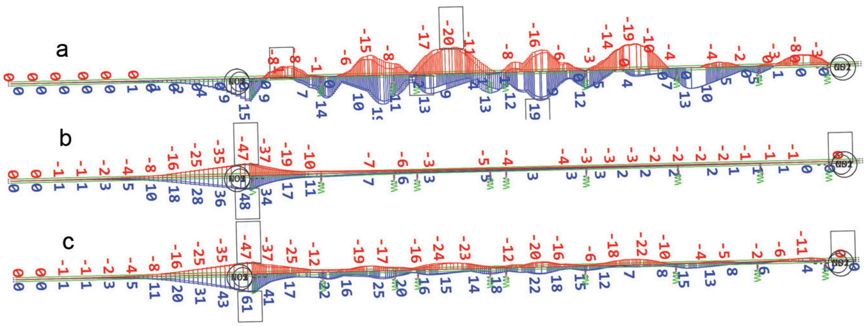

Figure 4 Additional tension and compression stress in the rail due to vertical traffic (a), traction and breaking (b), and their combination (c)

5 Conclusion

Bridge length, substructure and bearing horizontal stiffness, superstructure vertical stiffness and ballast stiffness all have a major impact on SLS verifications and additional track stresses, when considering a CWR track. All these conditions, including a track-bridge interaction, were calculated and deformations and stresses verified according to design code, for a two-track railway bridge with CWR track.

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