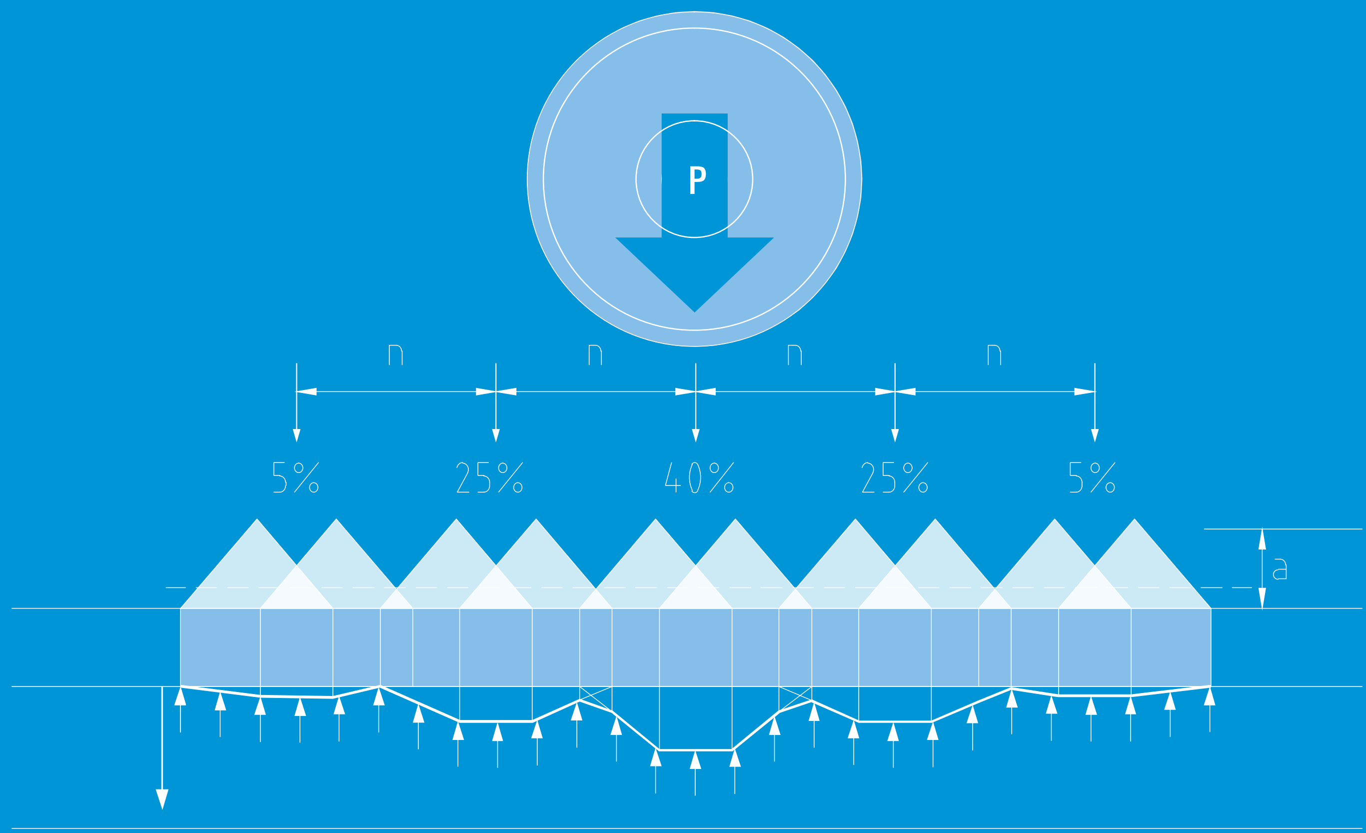
SSF Ingenieure

service

SUPERSTRUCTURE FOR WHEEL-RAIL-TRAFFIC

INTERACTION BETWEEN RAIL AND BRIDGE



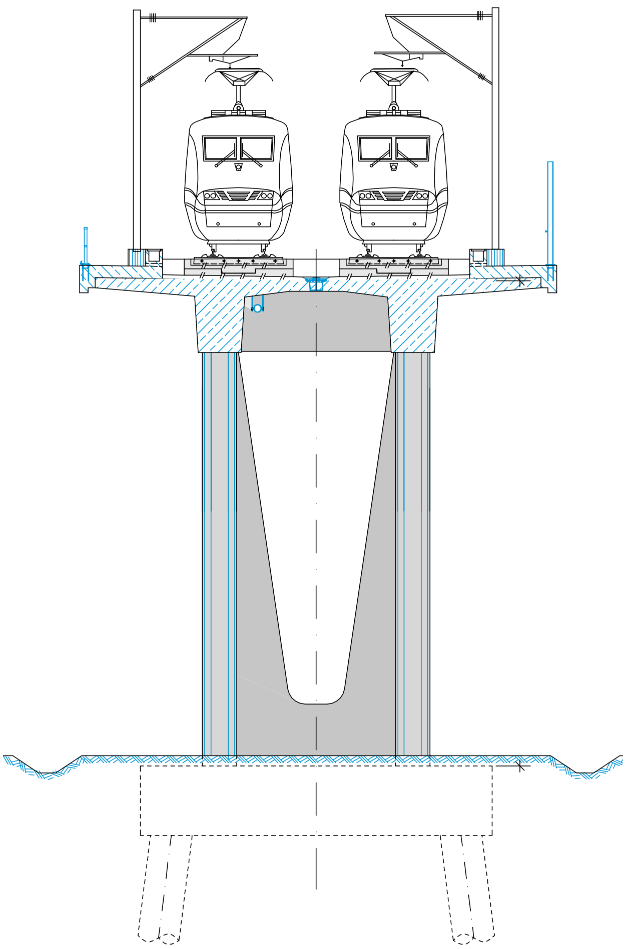
Track/Bridge Interaction – State of the art and examples

Track/Bridge Interaction significantly influences the choice of bridge systems, construction and calculation because of actions caused by rail traffic (braking and acceleration, LM 71) and temperature changes. Main parameters with regard to the construction are the track superstructure, the expansion length of the bridge superstructure, the support stiffness, the transition structures for bridges and the bridge design. Whereas in the majority of cases simplified calculation methods are sufficient, in certain difficult cases special and complex calculation methods are necessary. This article demonstrates the technical background and the state of the art with a prospect of further developments.

System observations

The track/bridge interaction plays a significant role when planning and dimensioning railway bridges. Due to acceleration and braking forces, temperature variations and resulting longitudinal extension of the superstructure – especially in the rail –, constraining forces occur in the track. In some load-bearing structures and for some lengths, these forces are only manageable by structural solutions such as expansion joints or by exact verification through calculation.

The course of additional rail stresses (see figure 3) due to temperature in case of a beam on two supports can be derived from the substructures stiffness.



LM 71: loadmodel acc. to DIN-Fb 101 (german annex to the Eurocode)

Figure 1+2 New railway line Erfurt-Leipzig/Halle - Gänsebach Viaduct

The image shows the following characteristics of interaction between track and bridge:

- The substructure's stiffness plays a decisive role in distributing additional rail stresses .
- The load-bearing behaviour of a bridge with elastic support (substructure stiffness ≈ 0 kN/cm) demonstrates that displacement in longitudinal direction can be reduced. When the thermal fixe point of the superstructure is shifted to the bridge centre, rail stresses decrease. The shift of the fixe point to the bridge centre leads thus directly to an elongation of the admissible extension length of the rail (without installation of a maintenance-intensive expansion joint).

Parameters of influences of track/bridge interaction

Admissible compensation and expansion lengths of the superstructure

In the German railway technical regulations maximum expansion lengths, for single- or multi-tracked railway bridges with ballasted and ballastless track as well as continuously welded rail, are determined as follows:

- 60 m for steel bridge
- 90 m for solid bridges and composite bridges

These values vary as steel bridges, in contrast to solid or composite bridges, react more sensitive to temperature changes.

Influences of acceleration and braking

The forces exerted on the rail track and the superstructure caused by acceleration and braking are only briefly acting forces compared to temperature changes, which are absorbed by the bridge's load-bearing structure, transferred via the track and dependent on the rail resistances as well as the horizontal stiffness of the bearings including the substructure (abutments, piers). The forces are limited by the maximum possible friction between wheel and rail (steel on steel) and lead to tensile stresses in the rail directly behind the braking train and to compressive stresses in front of the braking train.

The friction brakes are mainly actuated by compressed air. Because when the brake is actuated by the operators brake valve (in the traction vehicle) the pressure wave in the main line only spreads with 250 to 280 m/s, the train is slowed down with a time delay, the so called breakdown time. This delayed braking effect entails longitudinal dynamic forces, especially in long train convoys, i.e. the rear end, slowed down belatedly, runs into the front of the train convoy. This causes compressive stresses between the train coaches. As a result the individual, block-braked vehicle shows constant friction values, but the long, block-braked convoy shows a linear increase of braking jerks due to the slow emptying of the main air line.

The braking jerk, immediately before the train comes to a halt, which is relevant for the dimensioning of railway bridges, occurs within 0.04 to 0.54 seconds. Freight trains present the highest braking forces on the rail during braking jerks due to their high dead weight and braking system. In DIN-Fb 101 characteristic values for acceleration and braking are determined:

- Acceleration force: $Q_{lak} = 33$ kN/m · $L_{a,b}$ [m] ≤ 1000 kN for load models 71, SW/0, SW/2 and HSLM
- Braking force: $Q_{lbk} = 20$ kN/m · $L_{a,b}$ [m] ≤ 6000 kN for load models 71, SW/0 and HSLM
- $Q_{lbk} = 35$ kN/m · $L_{a,b}$ [m] for load model SW/2

In the load model 'trains without cargo' forces are negligible. The characteristic value of 20 kN/m braking force corresponds to ¼ of the line load of 80 kN/m of the load pattern LM 71.

The maximum length of action is chosen at 300 m so that in general no braking forces exceeding 6000 kN (600 to) occur. It has to be taken into consideration that heavy freight trains (2000 to and more) are usually not longer than 300 to 400 m because of the limited draw hook load (at acceleration).

Stiffness of substructure

The relation of load induction into the fixed bearings or the rail depends largely on the stiffness of the bridge's substructure. Piers of high viaducts are sometimes yielding. During short-term acting loads due to braking and accelerating, the superstructure does not participate in distribution of longitudinal forces leading to additional rail stresses (see figure 4).

The substructure's stiffness is composed of

- bending stiffness of pier shafts δ_p
- resistance of the ground δ_ϕ underneath the foundations against tilting of the foundations and footings
- resistance δ_h of the piers as a result of displacement of pile cap

Expansion joints, functionality and dimensioning

When for reason of the terrains topography or other constraints (e.g. rivers etc.), the maximum expansion length of the rails of

DIN-Fb 101: german annex to the Eurocode

Figure 3 Additional rail stresse due to support stiffness

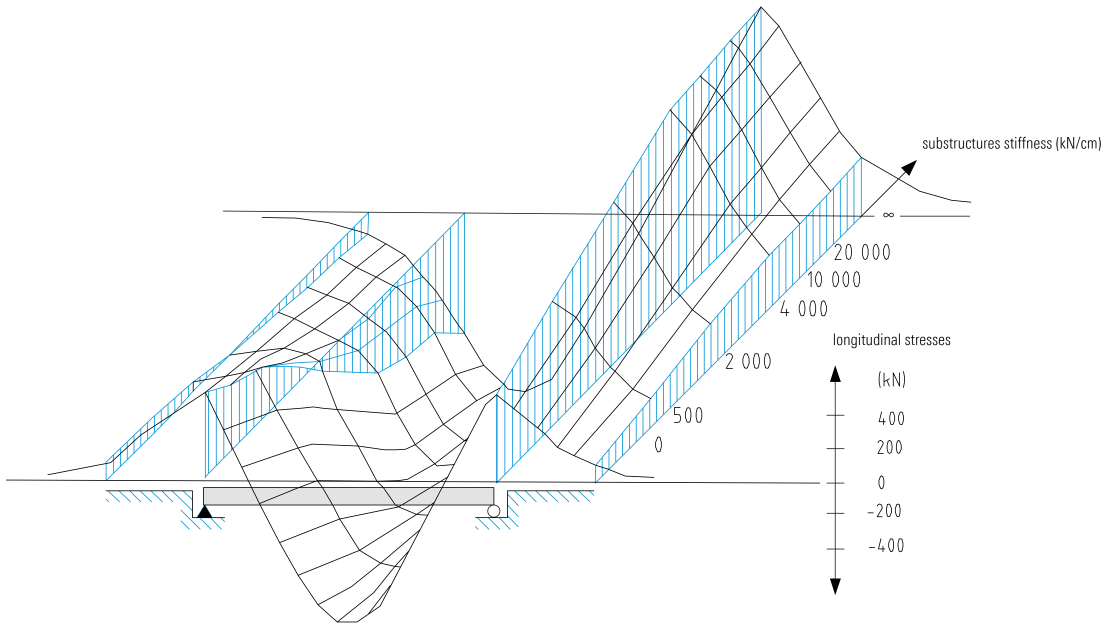
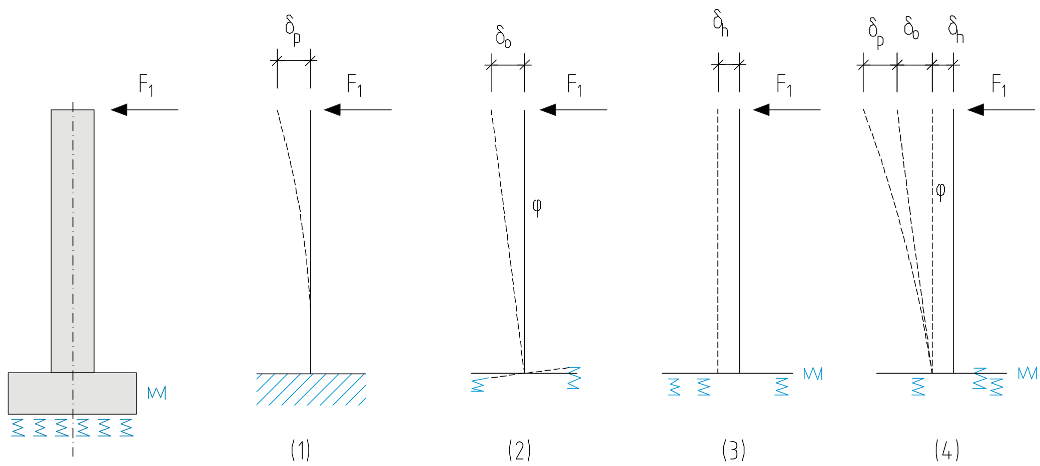


Figure 4 Factors influencing support stiffness



60 m/90 m cannot be maintained, the additional rail stresses are to be reduced in another way and the expansion joints to be assembled above the bearings in order to reduce rail stresses due to elongation of the superstructure (see figure 5).
So theoretically, infinitely long superstructures can be implemented if the rails bending stiffness would not have a restricting effect. Expansion joints, however, should be avoided because of the first initial investment and long-term maintenance costs.

Carriageway joints
To bridge expansion joints at the superstructure end in the area of the bearings and of fixed bearings to compensate the rotation angle of the final tangent, closed or open joint constructions can be implemented.

Closed joints
Closed joints are preferred as they are easier to assemble and to drain. They are mostly used above the fixed bearings. On top of movable bearings their use is limited to bridges with small expansion, as already displacements of 1 to 2 cm under frequent load change lead to loosening of the ballast in the space between the sleepers.
- Installation width 260 to 310 mm
- Acceptable joint movement $\Delta x \leq \pm 65$ mm

Open joints
When using expansion joints of rail UIC 60, open joints (see figure 7/8) only allow displacements of the superstructure of at maximum 200 mm at the movable bearing because of the limited distance to the neighbouring sleepers of 65 cm.

- Open joints with fixed rail furrow underneath the separation joint for large joint movements: opening of joint (underneath the system's level): minimum 200 mm, maximum 600 mm acceptable joint movement: $\Delta x \leq \pm 200$ mm

- Open joints with furrow and movable fastening which fixes the furrow centrically underneath the separation joint in case of large joint movements Opening of joint (underneath the system's level): minimum 200 mm, maximum 1000 mm acceptable joint movement: $\Delta x \leq \pm 200$ mm

For larger expansions, the use of full web rails type UIC 60/Vo 1-60 is recommended. They allow a maximum distance of 110 cm between the sleepers. Displacements of up to 66 cm can be absorbed (110 cm minus double the required distance between sleeper and track edge (2 × ½ x 44 cm). This represents an expansion length of a single-piece steel superstructure of up to 714 m under consideration of the temperature difference $\Delta T = 77$ k (acc. to DIN-Fb 101).

$$L = 714 \text{ m } \left(L \leq \frac{\Lambda L}{\text{at x } T\Lambda} = \frac{0,66}{1,2 \times 10^{-5} \times 77} = 714 \right)$$

Expansion joints
A standard design of expansion joints is fabricated and assembled. In Germany, expansion joints are chosen and dimensioned according to RiL 820.2040 (guidelines of the DB AG). Influencing effects are shown in figure 9. During installation, high quality and precision requirements are to be met regarding adjustment criteria.

Figure 5 Rail stresses when expansion joint on one end

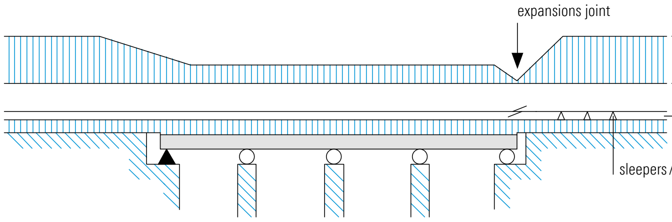


Figure 6 Detail of a closed joint

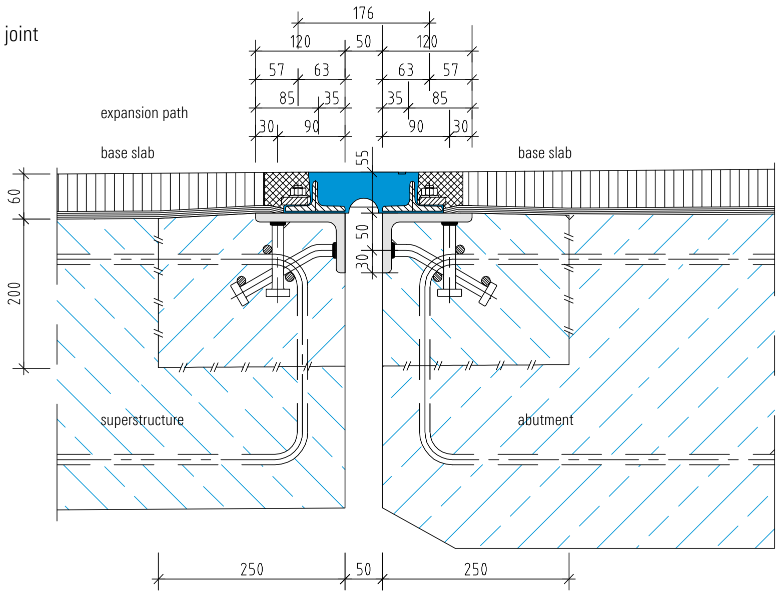


Figure 7 Detail of a open joint

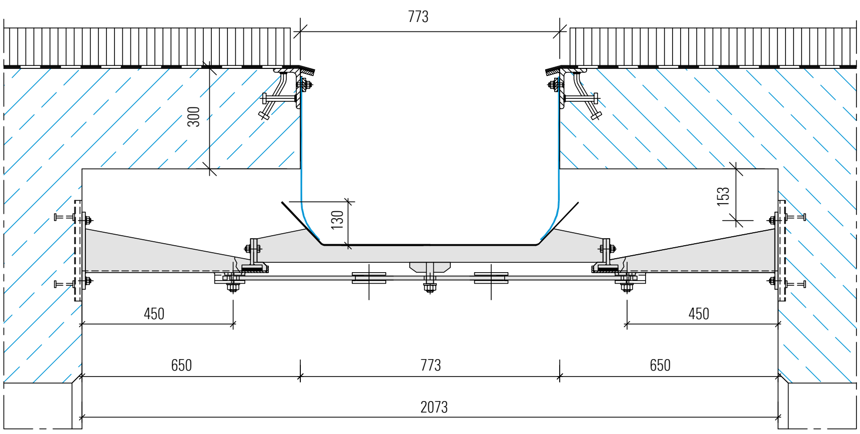
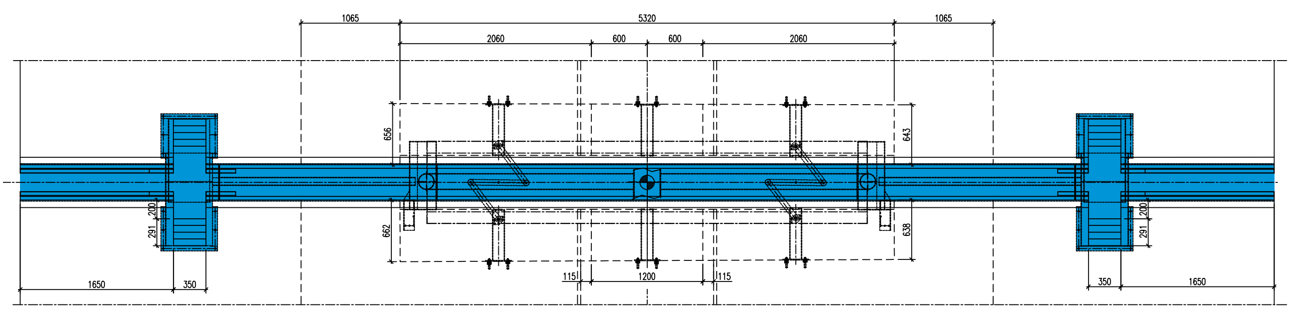


Figure 8 Top view on a section pier with an open joint



Superstructure height, projection, gravity centre of superstructure

Not only in longitudinal direction, can forces cause additional rail stresses. Occasionally an unfavourable formation of the final cross girder with large projection or a high superstructure (e.g. composite truss bridge with carriageway on top) causes longitudinal displacement as well as height displacement due to bending from vertical traffic influences (φ LM71).

It results thereof:

- a longitudinal displacement at the movable bearing (see figure 10 a) compared to the difference δH LM71 between longitudinal displacement due to movement of the superstructure's gravity centre caused by traffic action and displacement of the superstructure's edge resulting from torsion of the superstructure
- a vertical displacement due to the superstructure's torsion caused by traffic action and by the projection of the superstructure behind the bearing axis (see figure 10 b).

Rack behaviour against displacement of ballasted and ballastless track

The behaviour of the rail in longitudinal direction is divided in resistance to longitudinal displacement of the track (in ballast) and the longitudinal rail restraint in the rail fasteners (decisive for ballasted track in the winter (frozen rail bed) as well as ballastless track). The displacement behaviour is non-linear and was simplified to a bi-linear behaviour to facilitate verification procedures (see figure 11). The resistance factor depends on whether the track is loaded or unloaded.

Admissible additional rail stresses

Admissible additional rail stresses are to be verified for critical conditions. The limit values of traction and compression result from different physical backgrounds and are briefly described in the following.

Free stress portion (tensile stresses) of track/bridge interaction

Of additional rail stresses (traction) on the bridge for displacements from

- Φ LM71 (due to torsion and displacement due to the height difference between bridge bearing and gravity centre of the superstructure)
- temperature expansion of the superstructure and
- longitudinal displacement due to acceleration and braking

Thereof results a free stress portion of 112 N/mm^2 for additional rail stresses. The bridge's bending due to traffic loads results in additional nominal stress in the rail. This effect is taken into consideration with reduction by a general free stress portion of 20 N/mm^2. Under consideration of influences from bridge bending, admissible additional stresses of σ= 92 N/mm^2 result.

Figure 9 Calculation of rail expansion joint lenght

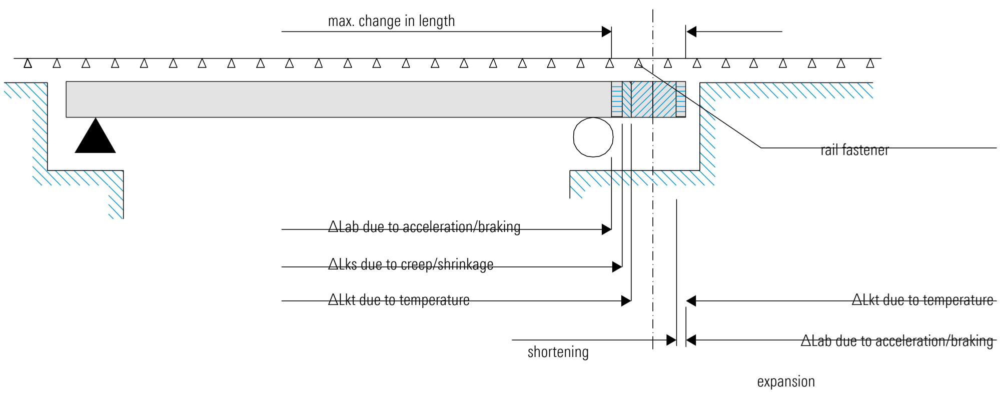


Figure 10 Effect of deck bending on the end sections

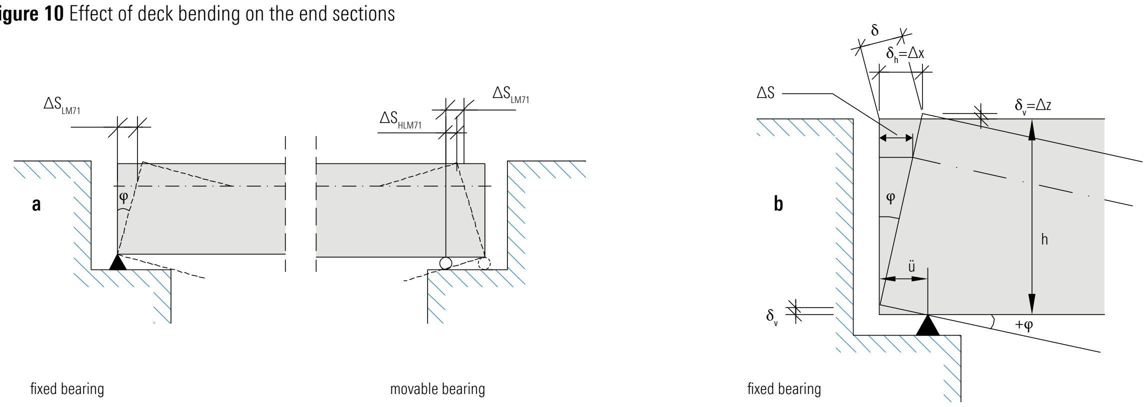
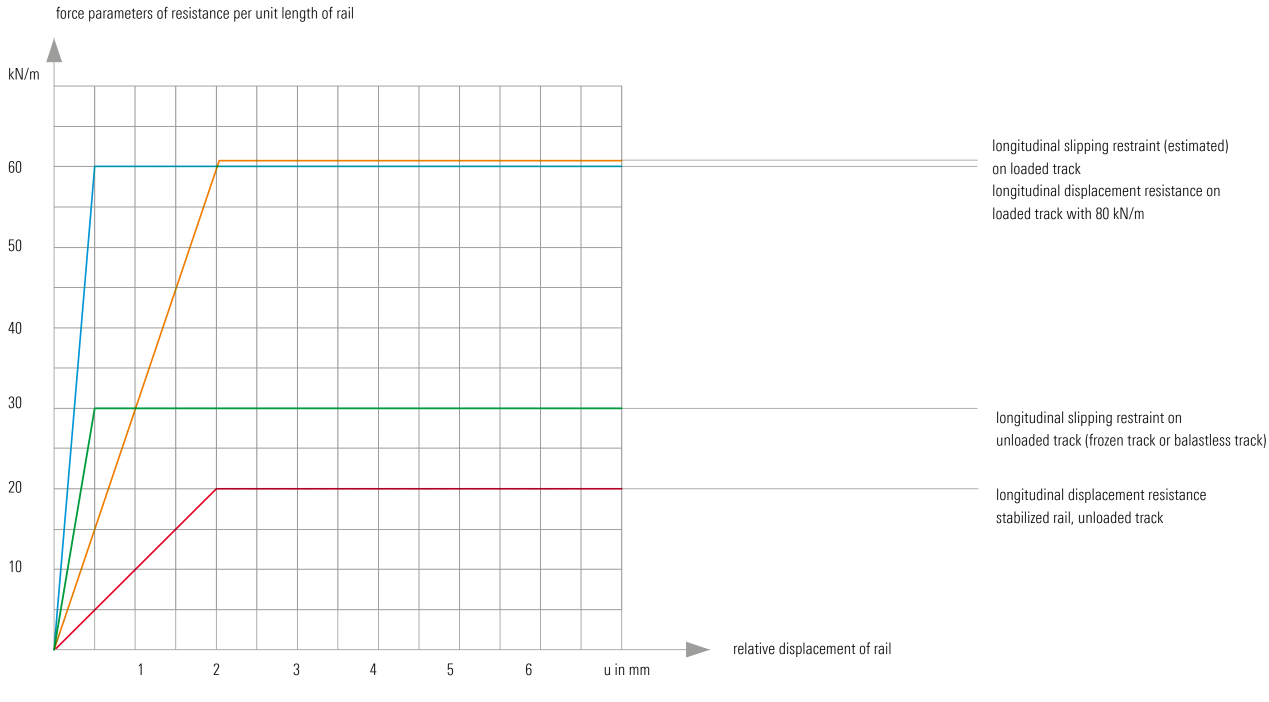


Figure 11 Rack behaviour of track against displacements



Free stress portion (compressive stresses) for track/bridge interaction

The verification of admissible additional rail stresses (compression) is derived from the warping criterion of the continuously welded track. Reliable and scientifically proven is the verification of possible additional rail stresses by Leykauf/Eisenmann in the context of introduction of the linear eddy current retarder for the ICE This verification is demonstrated in the following in abridged version. (see figure 12).

- **Critical temperature rise acc. to Chatkeo/Meier:** For a critical temperature rise, uniform assumptions of approximately 122 K established themselves as reference value.
- **Maximum rail temperature:** The rail temperature increases to a maximum of 18 to 20 K above the outside temperature whereas the highest external air temperature is assumed at 38°C. Unlike the habitually supposed maximum temperature in the rail of 65°C – also anchored in the regulations –,track verifications can be carried out with 58°C. Hence, an equivalent temperature rise of 38 K (= 58 K – 20 K) can be assumed relative to the lowest buckling temperature of 20°C.
- **Safety margin:** To take into consideration the differences of the buckling temperature occurring in-situ, the influences of braking forces and force build-up as well as the effects of higher lateral forces a safety margin depending on running speed acc. to table 1 has to be included in the verifications.

- Analysis of bridges:

ΔTvorh = 122 K– (38 K+ 50 K+ 3K) = 31 K

with

- 38 K rail temperature relative to buckling temperature
- 50 K safety margin for high-speed traffic v ≥ 230 km/h
- 3 K elongation of the rail under railway operation

The difference of 31 K (more or less 30 K) can be calculated to 72 N/mm² in the values of admissible additional rail stresses.

As a result of the above considerations, DIN-Fb 101 admits the following additional rail stresses:

- ballasted track:
  +92 N/mm² for tensile forces
  –72 N/mm² for compressive forces
- ballastless track:
  +/– 92 N/mm²

(The increase of compressive stresses in case of ballastless track to 92 N/mm² is possible because of the much more advantageous behaviour of ballastless track towards distortion).

Selection of adequate bridge systems

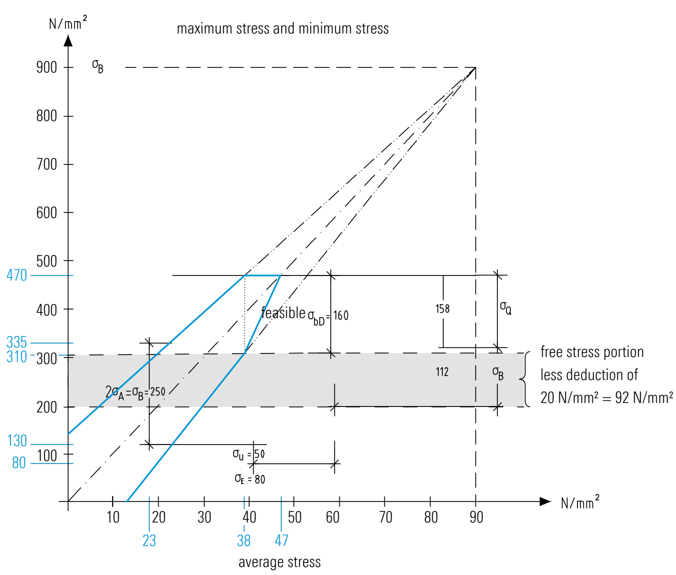
When designing a bridge, an appropriate bridge system has to be chosen early on. Bridge systems are divided into single- or

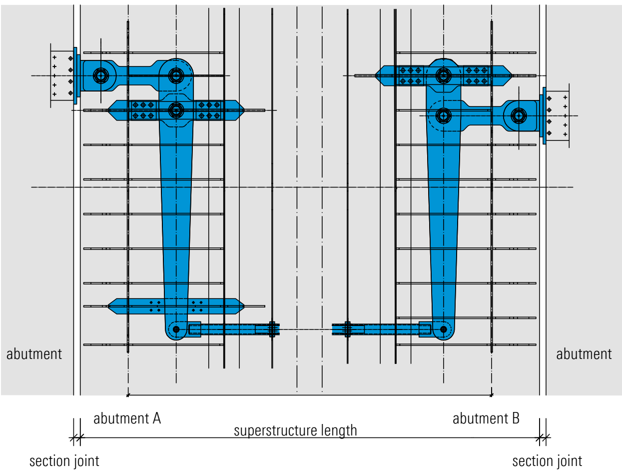
Table 1 – Safety margin due to running speed

V in km/h	< 80	100	120	140	160	> 230	> 230
ΔT_{Sich}	10	20	25	30	40	50	60

Explanation:
admissible σ 470 N/mm2, admissible rail stresses (yield point at 90 % static safety)
2 σA = σbD 205 N/mm2, rigidity of corroded rails acc. to test results of the Technical University Munich
admissible σbD 160 N/mm2, admissible maximum tensile bending strength
σQ calculated tensile bending stress at the rail foot from wheel force Q due to axle weights (rail UIC 60 coefficient of subgrade c = 100 N/cm³, σ= 0,2 ϕ) z. B. 158 N/mm² at 21 t with v = 200 km/h)
σT stress from temperature variations in the rail, T = 50 K
σE internal stress (due to rolling)
σU low stress of endurance test (dynamic fatigue test)

Figure 12 Stresses at rail flange (UIC 60, 900 N/mm²)





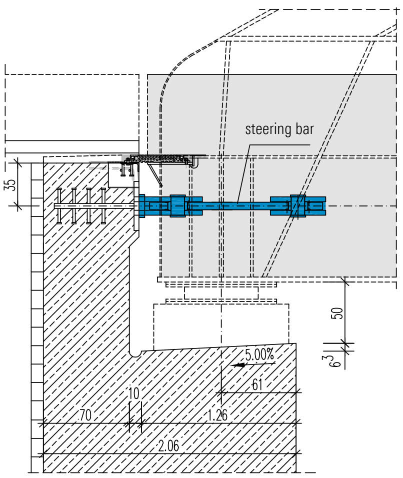


Figure 13 Details of the Meyer-Wunsdorf steering bar

multi-piece superstructures with individual or continuous girder systems. In general, the length of the superstructure is chosen in such way that no expansion joints are required if possible. It is evident that coordination with the sector of track construction is already recommendable at the very beginning. For arrangement of the expansion joints, the bridge's expansion length LT is decisive. The expansion length of single-piece bridges are the section lengths of the bridge superstructure, measured from the fixe point or the point of zero motion to a movable bridge end. The consideration of the bridge length and the decision to use single- or multi-piece bridges lead to standardized solutions which don't require supplementary calculations regarding track/bridge interaction.

Special constructions

To avoid expansion joints in the 70s and 80s, when numerous long bridges were built in the context of widening navigation channels which met the limits of expansion lengths (60 m to 90 m), special constructions were conceived allowing larger expansion lengths. The most frequent is the so called Meyer-Wunstorf steering bar (figure13), a simple, mechanical solution. By follower pins, oscillating links/levers and centring rods the centring beam is coupled at the underside to the superstructure and the abutment. The simple lever principle centres the superstructure in the middle in such way that the steel bridge can be implemented with an expansion length of up to 120 m instead of 60 m.

Calculation methods

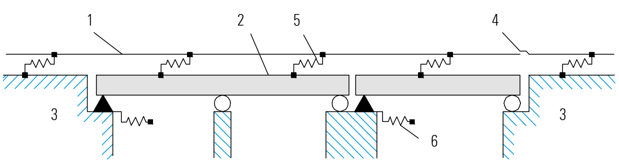
Simplified calculation method

For single-piece superstructures under the following conditions:

- rail UIC 60 with tensile strength of minimum 900 N/mm^2
- straight rail or rail radii r ≥ 1500 m
- for ballasted track concrete sleepers B 70 W at a distance of at maximum 65 cm or similar sleeper types with at least similar weight
- for ballasted track of at least 30 cm compacted ballast under the sleepers
- expansion length are 60 m/90 m or use of expansion joints

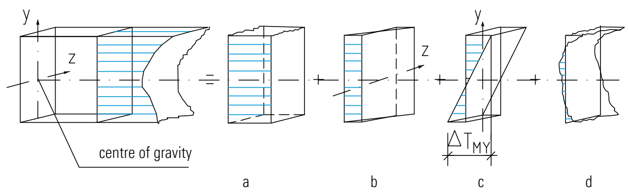
calculations can be carried out according to the simplified procedure described in DIN-Fb 101, annex K, chapter 2, without verifying additional rail stresses. In railway construction this applies to 90% of cases. Bearing forces are calculated according to DIN-Fb 101, annex K. Displacements at the movable bridge end caused by traffic loads and acceleration/braking are to be calculated according to DIN-Fb 101, annex K, chapter 2.1(P).

Figure 14 Simplified structural model for interaction analyses



1 Track
2 Superstructure (the image shows a superstructure with two spans and a single-span girder)
3 Earthwork construction
4 Expansion joint (if existing)
5 Non-linear longitudinal springs demonstrate the longitudinal loading/displacement behaviour of the track
6 Longitudinal springs demonstrate stiffness K in longitudinal direction of a fixed bearing under consideration of stiffness of foundation, columns and bearings etc.

Figure 15 Actions due to temperature



Universally applicable verification of track/bridge interaction

In all other cases, exact analyses of a simplified bridge system (see figure 14) have to be undertaken. System data:

- The calculation is to be carried out on the embankment area up to 90 m behind the superstructure end.

- For longitudinal rail restraint and rail resistance to longitudinal displacement, the relation has to be represented by a bilinear spring acc. to figure 9. In case of loaded rail, q is to be assumed for the loaded rail and in case of unloaded rail for the unloaded rail. The number of springs should be set at 10 per section on the bridge and double the amount in the embankment area. Alternatively the springs can be simulated by expansion-rigid bars between the rail and the superstructure. Their bending stiffness is determined by iteration methods so that from a displacement larger than 0.5 mm or 2 mm the bending stiffness is set to zero.

The substructures stiffness is to be calculated acc. to figure 4 and to be considered in a simplified way as springs at bearing height. Longitudinal forces of the rail and the bearings resulting from individual influences, are only to be superposed linearly, this estimation is however conservative. The discretization of the system poses the question if it suffices to take into account eccentricities between rail/sleeper, the gravity centre of the superstructure and the bearing axis or even more accurate spring models with consideration of eccentricities between rail, bottom sleeper edge and upper edge of carriageway as well as torsion possibilities of the sleeper on the ballast and changes of ballast characteristics. Influences are analyzed and show the result that eccentricity between rail and gravity centre as well as bearing axis of the superstructure made of box sections lead to marginal differences and are thus negligible.

The consideration of eccentricity for ballasted track, e.g. the distance of the force application point of the ballast onto the sleeper and the distance of the gravity centre of the rail to the force application point of the ballast onto the sleeper are, within calculation accuracy and can also be neglected.

Influences

- Temperature: For the analyses, temperature variations are decisive (figure 15a). Considerable influences (up to 6 % in case of box sections) result from the temperature gradient in vertical direction of the superstructure (figure 15c), the temperature gradient in horizontal direction (figure 15b) as well as the temperature gradient in the pier in longitudinal direction due to sun incidence.
- Acceleration and braking: a) load assumptions see DIN Fb 101 b) on multi-tracked load-bearing structures, at the same time

Figure 16 Limitation of deflections at the end of bridge deck

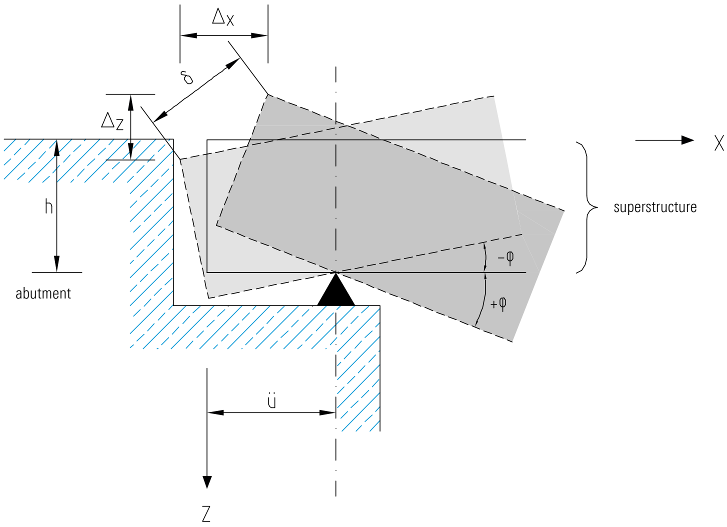


Table 2 – Limit values of deformation paths at superstructure and due to traffic action		
Span width L of final span	Design speed ve	Limit value δ
	v_e < 160 km/h	δ3 = 5 mm
< 3 m	160 km/h < ve < 230 km/h	δ3 = 4 mm
	v_e > 230 km/h	δ3 = 3 mm
> 25 m	for all v_e	δ3 = 9 mm
3 m < L < 25 m	intermediate values are to be interpolated linearly $\delta_L = \delta_3 + (L-3)*(\delta_{25} - \delta_3)/22$, L(m)	

Table 3 – Slab track – area of application

ballastless track type / bridge	**ATD**	**Züblin or Rheda simplified**	**Rheda on bridges**	**Bögl on bridges**
Up to 10 m and earth-covered bridges	X	X	X	X
Frame up to 20 m	X	X	X	X
Up to 25 m and earth-covered bridges	–	X	X	X
Vault and rows of vaults	X	X	X	X
Over 25 m	–	–	X	X

acting braking on one track and acceleration on the other (on double- and multi-tracked bridges of lengths up to > 100 m the load case braking/braking can be decisive)
- loading due to Φ · LM71 (on multi-tracked bridge on both tracks)
- shrinkage and creeping acc. to DIN Fb 102
- load group: Lgr 11 or. Lgr 23 including temperature acc. to DIN Fb 101, table 6.6

Calculation method (admissible rail stresses, deformation, bearing, ballastless track)

The following calculations have to be carried out:
- Maximum additional longitudinal rail stress in the area of the bridge and the abutments
- Deformation verification
  a) admissible displacement of the movable bearing caused by acceleration and braking
  < 4 mm for continuously welded rail and expansion joints on one side < 30 mm for expansions joints on both sides
  b) displacement in longitudinal and height direction has to be limited acc. to figure 16 (Lgr 11 only on one track)
- Depending on the above mentioned influences (temperature, traffic, acceleration and braking) the longitudinal bearing forces are to be calculated. Relevant verifications (DIN-Fb 101, annex 0) are to be carried out. The bearings are to be verified in accordance with DIN Fb 101, annex 0 and relevant standards.
- For ballastless track, verifications are required

Ballastless track on bridges

The construction and dimensioning of ballastless track on bridges is essentially influenced by the type of ballastless track. For reasons of construction technology and maintenance, it is advisable to continue the same system of ballastless track on bridges as it is used on the rest of the line. In addition to longitudinal force distribution, the transition from bridge to the other sections as well as the bridge spans is fundamental for the use of ballastless track on bridges. Therefore, in general the length of the bridge is decisive for use of ballastless track on bridges. A difference is hence to be made between ballastless track on short and on long bridges. Whether ballastless track systems are used depends not so much on the distribution of longitudinal forces but on the limitation of tensile forces in the asphalt and concrete layers or the movements in the concrete slabs at the transitions between bridge and earthwork due to bending caused by traffic loads. The application limits seen in table 3 result thereof.

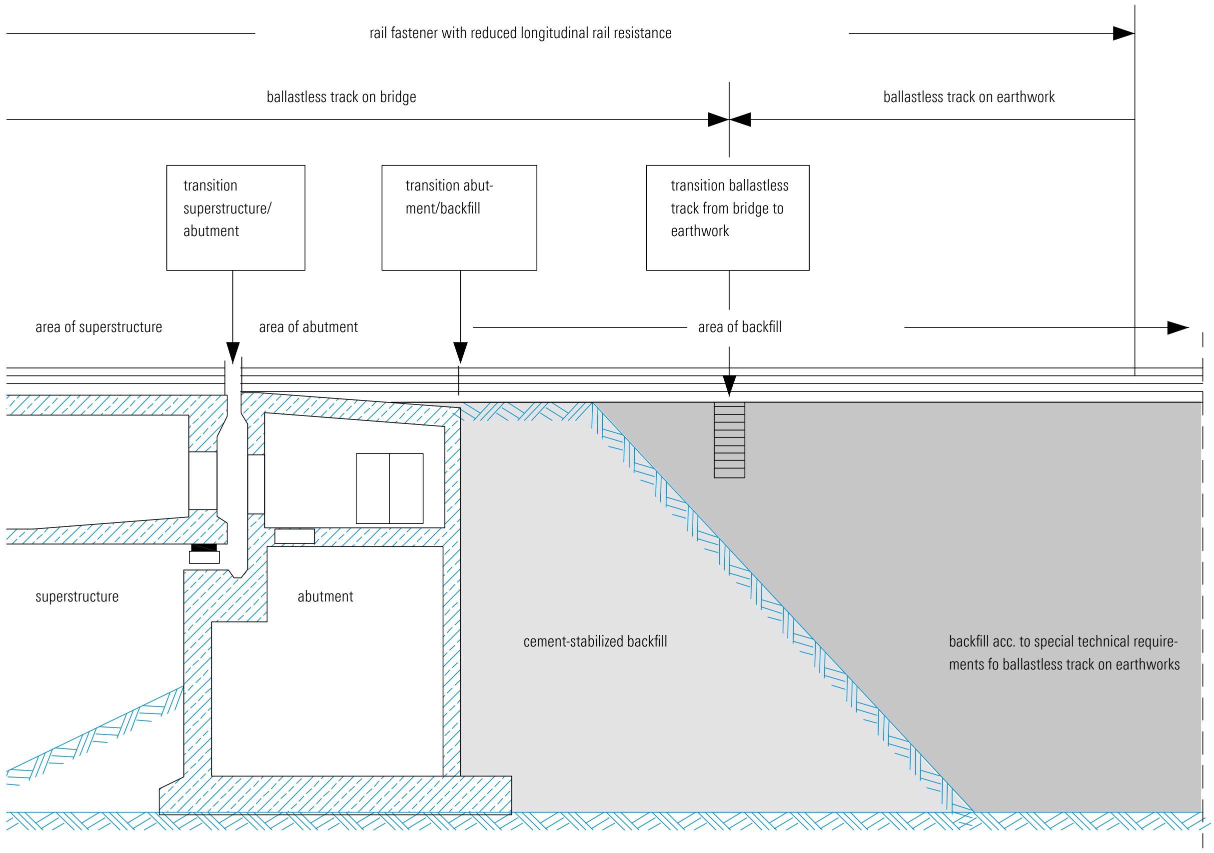
Ballastless track on short bridges

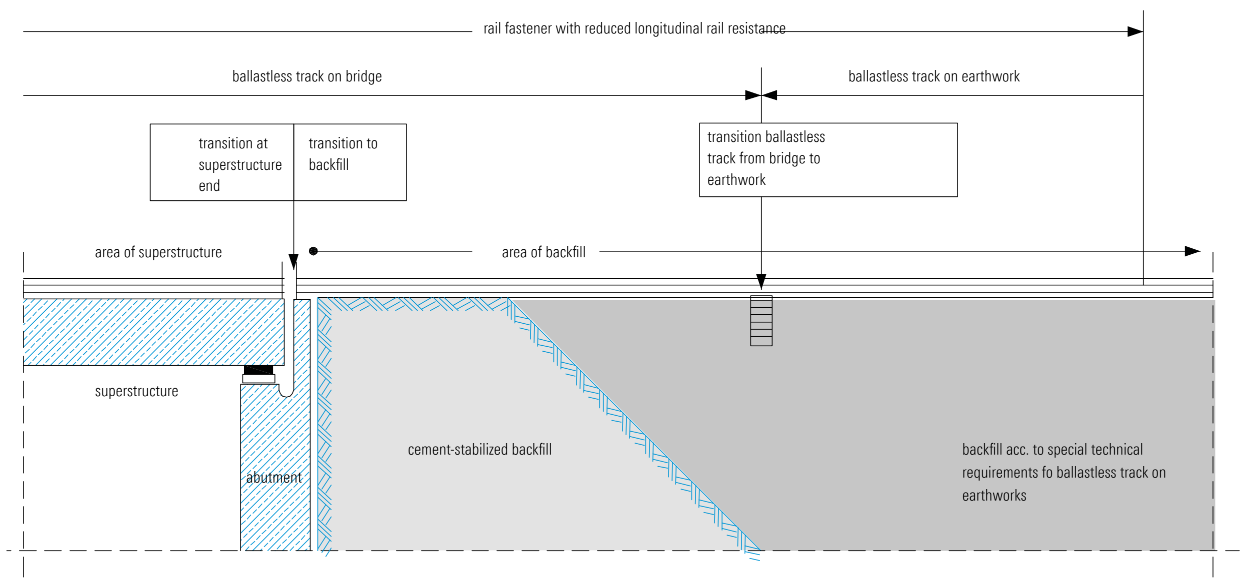
Bridges are considered to be short up to an expansion length of 25 m. By limiting this expansion length, horizontal forces in longitudinal direction caused by braking or acceleration can be distributed by the continuously welded track without exceeding the admissible rail stresses of 92 N/mm. The continuously welded track has to be continued up to 40 m over the bridge length. In general, the track slab has to be 'floating', i.e. movable in longitudinal direction achieved by lateral guiding blocks or longitudinal guiding. Guide bearings are to be assembled on the hump plate to absorb lateral forces.

Ballastless track on long bridges

Long bridges start at an expansion length of 25 m. On long bridges the track slabs have to be anchored to the superstructure in order to distribute the larger part of longitudinal forces caused by braking or acceleration via the bridge bearings so that the admissible rail stresses remaining in the track are not exceeded (coupled system bridge/track). To assure optimum maintenance, the track slabs are divided in short slabs. Their length should be between 4.00 and 5.50 m. The weight can be moved by regular cranes and drainage in the joints is assured. To distribute longitudinal forces, a force-fit connection between track slab and bridge is achieved by a hump structure. Usually, the track slab is fixed to the superstructure, i.e. unmovable in longitudinal and transversal direction. In case of continuous girders, the lengths of individual slabs are to be designed in such way that the transverse joints of the ballastless track are above the pile axes. Consequently, tensile forces in the support areas of the main girder are not transmitted into the track slab through participation.
The different systems of ballastless track respect especially the requirements related to bridge construction, replacement within a short amount of time, drainage and bending restrictions. Detailed

above: Figure 17 Ballastless track on long bridges
down: Figure 18 Ballastless track on short bridges


rail fastener with reduced longitudinal rail resistance
ballastless track on bridge
ballastless track on earthwork
transition
superstructure/
abutment
transition abut-
ment/backfill
transition ballastless
track from bridge to
earthwork
area of superstructure
area of abutment
area of backfill
superstructure
abutment
cement-stabilized backfill
backfill acc. to special technical require-
ments fo ballastless track on earthworks


rail fastener with reduced longitudinal rail resistance
ballastless track on bridge
ballastless track on earthwork
transition at
superstructure
end
transition to
backfill
transition ballastless
track from bridge to
earthwork
area of superstructure
area of backfill
superstructure
abutment
cement-stabilized backfill
backfill acc. to special technical
requirements fo ballastless track on
earthworks

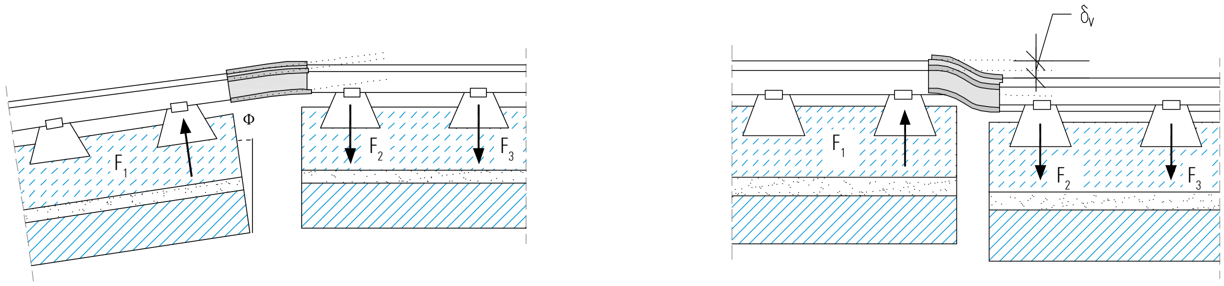


Figure 19 Forces of rail base for the slab track due to traffic actions

requirements are described in regulations Ril 804 and the catalogue of requirements for ballastless track (AKFF, issue 08/2002) of the German Railway Company DB AG.

Analyses of transition areas on bridges

The analysis of transitions between the superstructures as well as between superstructure and abutment are indispensible for application of ballastless track.

Situation for analyses

From figure 19 several factors of vertical displacement and torsion can be deduced. Vertical displacement and torsion cause vertical compressive forces on the neighbouring rail fasteners on the bridge as well as tensile forces at the rail fasteners on the abutments. It is evident that vertical displacement has a much larger influence onto fastening forces which can be directly deduced as the vertical displacement has essentially to be distributed by the neighbouring rail fasteners arising as force couple with a maximum distance of 650 mm, whereas in case of torsion the additional forces are distributed onto a greater length by the elastic embedding of the rail in the ballastless track and the more distanced rail fasteners participate to a larger extent in force distribution. The rail fastening forces result from spring stiffness of the fastening and are thus largely dependent on the stiffness of the intermediate layers. For normal rail fasteners (Ioarv 300 with Skl 15B and Zwp 104) the admissible tensile force is 12 kN.

At the bridge joints (joints between abutment/superstructure and superstructure/superstructure), track verifications are to be carried out according to German regulations (catalogue of requirements for ballastless track, Ril 804, supplements to Ril 804 for "Verification of superstructure ends of ballastless track" and "Indications for serviceability verification at superstructure ends of ballastless track"). This comprises the following verifications:

- rail stress calculations
- calculations of lifting forces at rail fastenings incl. verification of position permanence of track elements
- verification of maximum distances of rail fastenings at bridge joints
- calculation of lateral offset due to bearing clearance and temperature

Depending on the results of above listed verifications, special elements (special fastenings, compensating slabs, expansion joints, etc.) are to be implemented at bridge joints.

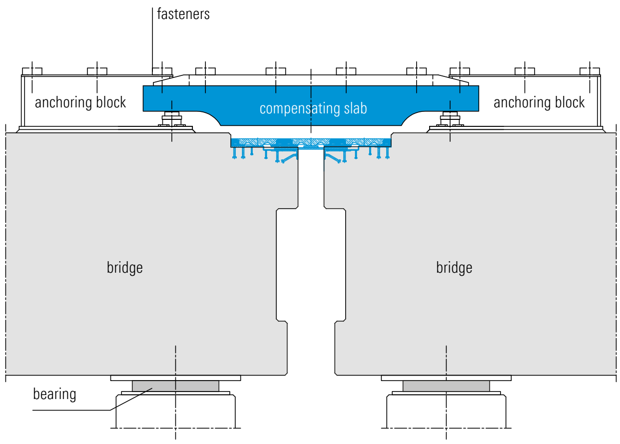


Figure 20 Detail of a compensating slab, Type Stog

Structural possibilities for limitation of rail fastening force

- **Installation of special rail fasteners:** By installing special rail fasteners with high flexibility, high tensile forces up to 27 kN can be absorbed. Disadvantages are high costs and difficult maintenance.
- **Installation of compensating slabs:** A significant contribution to the analysis is made by vertical displacement which can be restricted by compensating slabs between the superstructure's bearing axes between superstructure and abutment or between two superstructures. The compensating slabs are movable in longitudinal direction but are fixed in transverse direction by small bridges made of concrete, steel or steel composite with lengths of around 6 m. Vertical displacement is transformed into a longitudinal incline above the whole length of the compensating slabs. Only torsion remains whose influence is however manageable. The production of compensating slabs is rather complex and thus expensive as initial investment. Further costs arise for maintenance and repair and also the maintenance effort is much higher than compared to other common solutions. In general, nowadays solutions entail much more intensive maintenance.
- **Reduction of the projections**: The projection of the superstructure can be reduced when the abutment runs up to the bearing axis.
- **Design of integral and semi-integral railway bridges:** In case of integral bridges (figure 1+2), piles and abutments are connected to the superstructure monolithically and bending-resistant, without joints and bearings.
  In case of semi-integral structures, parts of the substructure take on the loads from the bridge deck through bearings. Preferably, such bridges are built when piles have to be high and settlement susceptibility of the ground is low. Integral structures are suited to considerably reduce investment costs as well as life cycle costs (inspection, maintenance, repair, life expectancy). Essential factors to this are the absence of maintenance-intensive and damage-sensitive joints and bearings, the use of complete cross-section instead of box section and the reduction of foundation elements. The nonexistence of transversal joints in the bridge deck decreases also the planning effort when constructing the bridge superstructure. Another advantage of these bridges is that, in relation to their stiffness, the monolithically connected substructures participate at the distribution of braking forces of railway traffic (figure 21).
  The transfer of horizontal loads in transversal direction follows a short path (monolithically) without bearings as intermediate structure.

Outlook and summary

Influence of the bilinear tension-expansion relation and the non-linear calculation on the results

Influence of the bilinear tension-expansion relation and the non-linear calculation on the results. The load cases temperature, braking etc. have to be superposed non-linearly. The linear addition of load cases leads in general to conservative results.

Change of the unloaded ballast bed to a loaded ballast bed

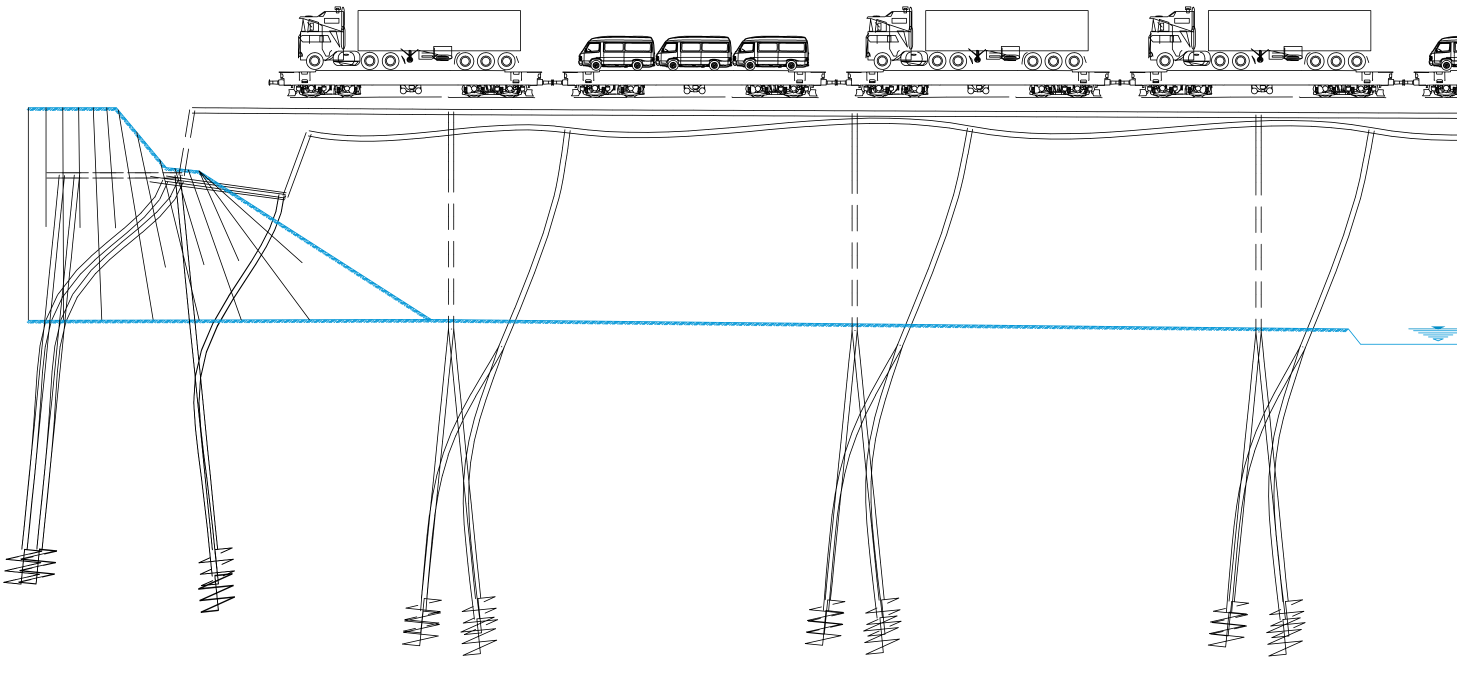
In the load case temperature, which in general causes large displacements (larger than 0.5 mm or 2 mm), the ballast bed on bridges slips in many areas. According to DIN-Fb load cases temperature, braking and bending are calculated individually with loaded and unloaded ballast bed. In reality, in load cases braking and bending under traffic influence, the loaded and unloaded state alter. Rail stresses are sometimes overestimated but bearing forces not always seized realistically. The track does hence not participate in force distribution and higher loads act on the bearings.

Steel and solid bridges without rail fasteners are also possible for larger expansion lengths

Indications of the maximum expansion length (60 m on steel bridges, 90 m on concrete or composite bridges) are proven values which take into consideration influences from temperature, acceleration and braking and of course the superstructure (construction height, projection) as well as track parameters to be on the safe side. By an accurate verification, expansion joints can be relinquished on larger bridges, too, in general a length increase of 12 to 20 % is achieved.

Use of sleeper anchors

By using protective caps or sleeper anchors, e.g. at the critical bridge transitions, the transverse displacement resistance can increase. The use of sleeper anchors at every third sleeper increase the buckling load and thus the admissible rail stresses by around 10 %. However, these supporting devices should only be applied



in some special cases as they interfere with mechanization of track maintenance.

Dynamic behaviour of the load-bearing structure in case of braking

Resulting from actual measures of braking carried out with ore trains on bridges of newly built lines of the first generation, differences between reality and theoretically calculated values of additional rail stresses of sometimes 50 % occurred. Variations result frequently from:

- modelling of the substructure's/pile's stiffness especially in case of deviating moments of inertia between bearing structures and feet of the piles.
- Young's modulus of piles and foundation
  Especially for the load case braking in-situ tests showed that the additional rail stresses are frequently overestimated. This is mainly due to subgrade characteristics evaluated by habitual investigation methods that are in fact higher in case of impulse-like braking jerks (< 0.5 s) and to the inability to activate the necessary displacements and torsion.
- Friction on bridge bearings
  The evaluation of braking test on bridges showed deficits of horizontal loads in the piles and the superstructures and a fast decrease of displacement amplitudes of the load-bearing structure in the swinging areas. This leads to the assumption of a dissipative influence of the bearing friction.

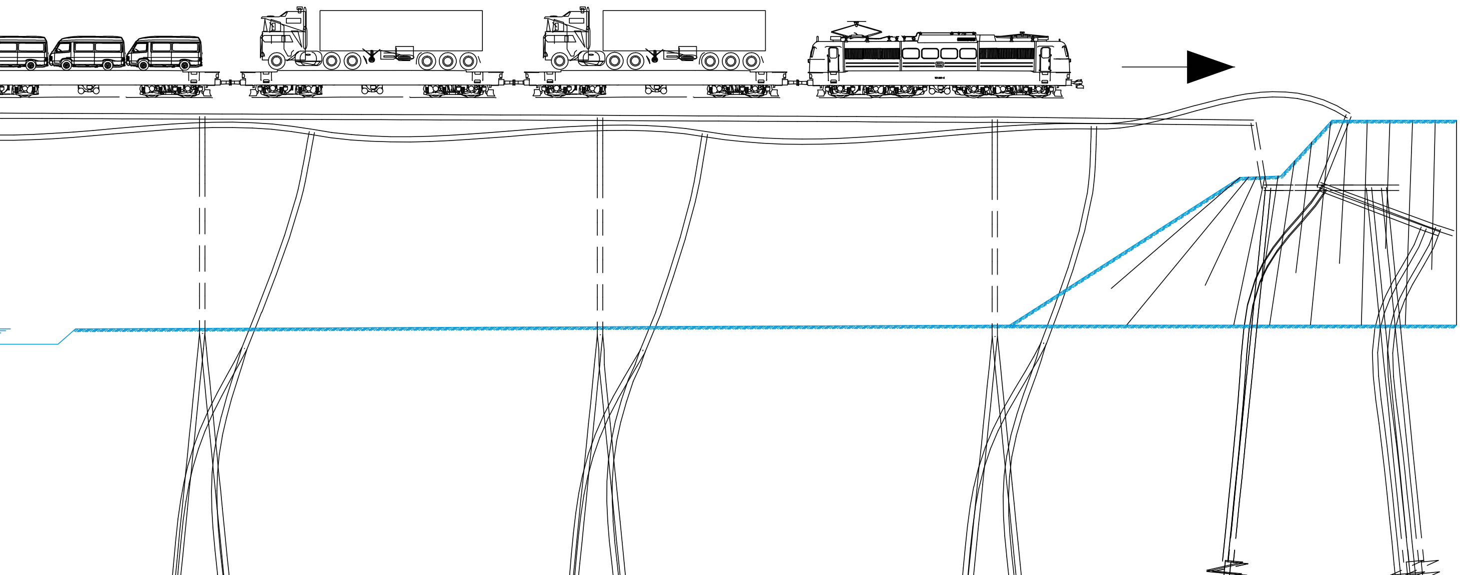


Figure 21 Forces caused by braking or acceleration on a long bridge

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