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Transmission of Longitudinal Forces on Railroad Bridges

Transmission des forces longitudinales dans les ponts-rails

Tragsysteme zur Abtragung von Längskräften auf Eisenbahnbrücken

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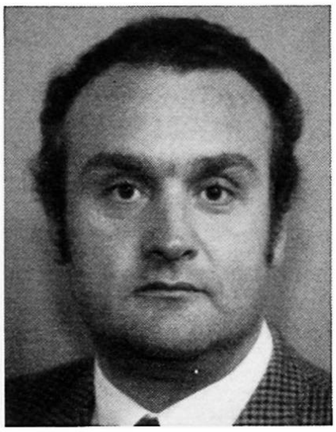

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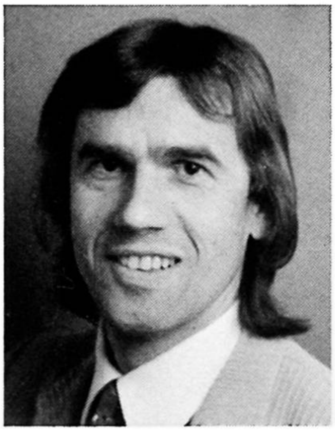

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SUMMARY
The paper outlines which structural systems are possible and suitable for the transmission of longitudinal forces on railroad bridges. Their dependence on other design and construction parameters is shown. The different systems are defined by significant limiting forces and are evaluated.

RESUME
La contribution met l'accent sur les systèmes structuraux appropriés pour la transmission des forces longitudinales dans les ponts-rails. Ces systèmes dépendent aussi d'autres paramètres de projet et de construction. Ces divers systèmes sont décrits et discutés, notamment sur la base de valeurs significatives pour les charges.

ZUSAMMENFASSUNG
Mit dem vorliegenden Beitrag wird aufgezeigt, welche Tragsysteme für die Abtragung der Längskräfte auf Eisenbahnbrücken aus heutiger Sicht möglich und sinnvoll sind. Ihre Abhängigkeit von anderen Planungs- und Konstruktionsparametern wird aufgezeigt. Die verschiedenartigen Systeme werden unter anderem durch signifikante Lastgrössen abgegrenzt und bewertet.

1. INTRODUCTION

When planning for the new railroad lines of the German Federal Railway (Deutsche Bundesbahn, DB) started in 1975, the design for bridges with respect to vertical loads could be based on the state-of-the-art of road bridges although these loads are significantly larger. The design for longitudinal forces, however, could be based only to a very limited extent on the experiences from road bridges. The reasons are on one hand that the size of the braking and acceleration forces are considerably different, and on the other hand that the rails of railroad bridges are unintentionally cooperating load-bearing members for longitudinal forces or temperature restraints.

2. LONGITUDINAL FORCES ON RAILROAD BRIDGES

2.1 Braking and Acceleration Forces

New measurements have indicated that friction factors in excess of 40 % may be activitated shortly before a train stops. Consequently, the German Railway Codes require for the new railroad lines to take into account a braking force of 20 kN per m of track (friction factor 25 %) over a maximum length of 312,5 m. Onto these braking forces an acceleration force of 1000 kN has to be superimposed for double-track bridges over a loaded length not exceeding 30 m. For long bridges the biggest braking and accelerating forces are consequently

$$L_B + L_A = 6.25 + 1.0 = 7.25 \text{ MN.}$$

The corresponding longitudinal force for road bridges in Germany always remains smaller than 0.9 MN due to the individual traffic units.

2.2 Movement Resistance of Bearings

Changes in length of structures created by changes of temperature activate resistance in the bearings. In order to quantify the corresponding loads, a double track single cell prestressed concrete box on sliding bearings with a span of 44 m is used as a typical example: the rails with ballast and the structural concrete have a dead weight of about 190 + 260 = 450 kN/m. Assuming a friction factor of 3.5 % - the actual friction factors are given by the bearing manufacturers - each meter of superstructure supported on sliding bearings creates a friction force of 0.035 x 450 = 16 kN/m. For a 44 m span we reach

$$L_{F,S} = 0.035 \times 0.45 \times 44 = 0.7 \text{ MN.}$$

2.3 Restraint Forces from Continuously Welded Rails

As the structure transmits forces through the ballast to the rails, changes in length of the structure may cause restraint forces in the longitudinal direction. Depending on the bridge system the overall change of temperature in the structure or the temperature differential between structure and rails may be governing. The size of these restraint forces depends heavily on the structural system and will be discussed later. The changes in overall temperature given by the codes are -30 K and +20 K against the average temperature. As the temperature variations in the rails have to be accounted for with 50 K, the temperature differentials between structure and rails come to -20 K and +30 K.

2.4 Other Longitudinal Forces

Other longitudinal forces may be created by the fixed support of a superstructure on two or more piers, by unsymmetric sun radiation, rotations of foundations and wind loads on piers rigidly connected with the superstructure. These longitudinal forces are, however, small when compared to those mentioned earlier. Earthquake loadings are also neglected here due to their extremely low probability in Germany.

3. STRUCTURAL SYSTEMS WITH SINGLE SPAN GIRDERS

Experience indicates that for concrete bridge girders fixed at one end with spans up to about 90 m the continuity of the rails has not to be interrupted in order to cater for the changes in length of the superstructure. From this knowledge the structural concept was derived to design longer bridges as a sequence of single spans over which the ballast with the rails is carried continuously. This basic concept developed from the point of view that the railway tracks and the vertical load path offer various possibilities for carrying the longitudinal forces which are determined mainly by the topography (pier heights) and the stiffness of the foundations depending on the soils characteristics.

3.1 The Elevated Superstructure

The elevated superstructure should be understood with the idea that the transmission of longitudinal forces in simple span bridges have such a small influence on substructure deformations, that the structure can be regarded as nearly rigid. Under this condition the braking and acceleration forces acting on a span are completely transmitted to its fixed point pier. This load case does not cause any considerable stresses and strains in the rails. Each pier has to carry the same longitudinal force which for single and double track bridges with the often chosen spans of 25 m comes to

$$L_B + L_A = (20 + 33.3)\cdot 25\cdot 10^{-3} = 1.3 \text{ MN}.$$

As the continuously welded rails cannot follow the changes in length of the beams they are restrained by temperature changes in addition to directly applied temperature loadings. The maximum values always occur at the beam joints and amount normally to

$$R_R = q\ell/4$$

with q meaning the resistance against sliding of the complete railtie grid or of the rails alone, see Fig. 1.

Special situations are created at the bridge ends: at the abutment with sliding bearings the restraint is about double its regular value, while at the abutment with the fixed beam connection the restraint decreases accordingly. On the abutment with the fixed point and on the first pier in front of the abutment with sliding bearings act - as a consequence - also longitudinal forces of the size R_R, which means for two rails and a span of 25 m

$$L_R \sim 2 \times 0.02 \times 25/4 = 0.25 \text{ MN}.$$

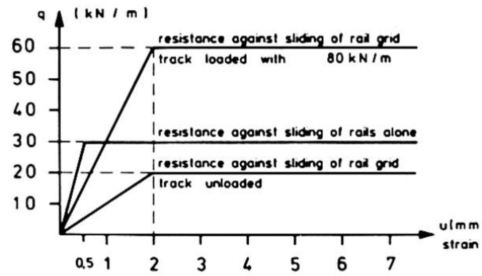

Fig. 1 Stress-strain diagram of ballast

The resistance forces from bearings have even smaller values for this system and may be neglected in accordance with the German Railway Code. Using the former calculation examples and taking into account the slightly smaller structural dead weight we reach

$$L_F \sim 0.035 \times 0.40 \times 25/2 = 0.18 \text{ MN}.$$

The loads to be applied to the structure are given in Fig. 2.

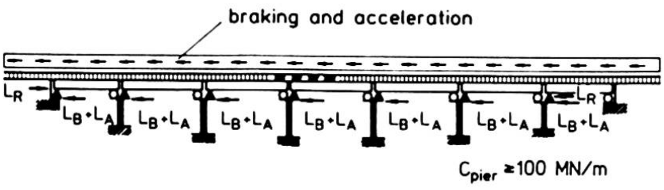

Fig. 2 Statical system and longitudinal forces of the elevated superstructure.

3.2 Valley Bridges without Special Devices

The formerly assumed simplification to neglect the pier deformations due to longitudinal forces is only valid for pier stiffnesses in excess of 100 NM/m which vary by not more than 10 % in between them. This condition can actually be fulfilled for pier heights up to about 15 m.

Higher piers and, more importantly, variations in stiffness between adjacent piers and abutments result in a statically highly redundant system. The continuously welded rails act unintendedly as a link between all the substructures of different stiffness and are, in addition, strongly anchored to the regular track continuations beyond both bridge ends.

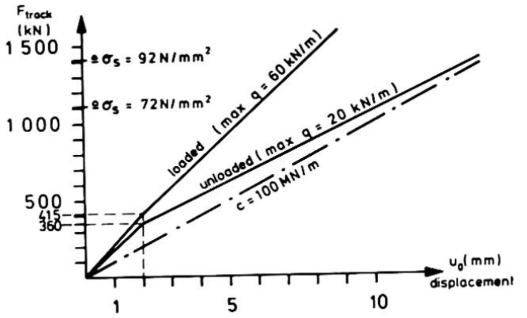

Fig. 3 Force-displacement characteristics of rail grids

The stiffness of this anchorage comes to 100 MN/m per rail grid for the range of permissible rail stresses as shown in Fig. 3. For pier heights up to 30 m the substructure stiffness is significantly influenced by the soils characteristics, which means that the structural assumptions for the proof of safety may suffer certain uncertainties.

Statical models become especially complicated and time-consuming because of the non-linear stress-strain characteristic of the ballast connecting the rails with the structure, see Fig. 1.

The load distribution by the tracks creates additional rail stresses which, if superimposed onto those due to change of temperature in the structure, must not exceed 72 MN/mm^2 in compression (safety against buckling of the rail grid) or 92 N/mm^2 in tension (safety against rail failure). At the same time it has to be demonstrated that the relative movements between rails and structure do not exceed 4 mm in order to ascertain a reliable support of the rails by the ballast.

The load-bearing system causes bigger loads at the stiffer abutments and piers than those loads actually acting on the corresponding spans.

The upper limit of the share of forces from braking and acceleration for very stiff abutments is obtained by adding to the load acting on the adjacent span that share of braking forces which the rails may carry across the next joint within the limit of permissible stresses. With a stress of about 70 N/mm^2 (excluding temperature influences), a rail area of about 2x77 cm^2 and 44(58) m spans, thus leads to

$$\max L_{B+A} = (1.0+0.02\text{x}44) + 70\text{x}154\text{x}10^{-4} \sim 3.0\ (3.24)\ \text{MN}.$$

The less stiff piers have to be loaded with at least one half of their direct load, i.e. for a 44 m span

$$\min L_{B+A} = 0.5\cdot(1.0 + 0.02 \text{ x } 44) = 0.94 \text{ MN}.$$

The pier stiffness influences only very little the restraint due to changes in temperature. With similar support conditions as those for the elevated superstructure, similar strains are to be expected here also.

For small pier heights (up to about 20 m) and favorable soil conditions it is appropriate to provide fixed-sliding support conditions similar to those of the elevated superstructure as shown in Fig. 4a. For this system the extreme rail stresses and track movements will occur at the joint at the abutments. If the permissible values cannot be adhered to at this location it is advisable to choose the support conditions for longitudinal forces in such way that both abutments with their generally relatively large stiffnesses take part in carrying the longitudinal forces. For these systems - compare Fig. 4b - where one intermediate span with small substructure stiffnesses will be fixed longitudinally to both corresponsing piers. One pier with only longitudinally sliding bearings may also be considered.

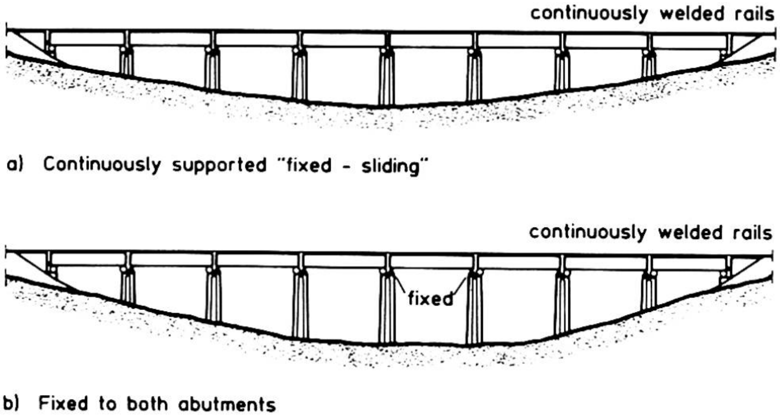

Fig. 4 Simple span valley bridges without special devices

Generally valid limiting values for the applicability of the systems outlined here cannot be given due to their complexity. Calculated examples did show, however, that the permissible stresses and movements can still be adhered to if individual pier stiffnesses amount to only about 40 MN/m which correspond to pier heights of about 30 m.

3.3 Valley Bridge with Special Devices

If the rail stresses or relative movements would become too big for a sequence of simple span girders with continuously welded rails, special devices can be applied. These should permit, if possible, the use of continuously welded rails, as is the case e.g. for the inclusion of creep couplers. These structural elements are located between all simple spans and act only for short-time longitudinal forces. As a consequence they do not react to the changes in length of the structure due to temperature, but carry significant amounts of the braking and acceleration forces to the abutments. Prototypes for hydraulically acting creep couplers are currently in the testing stage.

Another special device connects all single beams by longitudinal force couplers. Hereby, the longitudinal forces are carried similarly as for continuous girders.

4. STRUCTURAL SYSTEMS WITH CONTINUOUS GIRDERS

For continuous girders continuously welded rails are only feasible for short girder lengths so that a rail expension jointing with a discontinuous ballast is the standard detail. By this the participation of the rails in carrying longitudinal forces is strongly reduced. The overall changes in temperature of the structure do not create restraints in the rails as these can only be caused by relative changes in temperature between structure and rails.

Due to the discontinuous rails continuous girders form a straightforward static system for carrying longitudinal forces, which is only very little influenced by the soil stiffness.

The superposition of braking and acceleration forces with the friction forces from the bearings leads to considerably bigger longitudinal forces for continuous girders than for simple span girders. Possible limiting maximum values are:

By full exploitation of the permissible changes in length for the statical rail system, continuous girders with a length of up to 940 m can be built. At a fixed point located at one end, for assumed 58 m spans, friction forces corresponding to Section 2.2 of

$$\max L_F \triangleq 0.035 \times 0.5 \times 915 = 16.0 \text{ MN}$$

may act, onto which braking and acceleration forces of 7.25 MN have to be superimposed. Taking into account the favorable influence of the horizontal bearing stiffness of the abutment and the elastic superstructure elongation, the upper limit reaches in any case a value of

$$\max \sum L > 20 \text{ MN}.$$

This force acts in tension or compression onto the box beam and has to be withstood by the abutment. As these forces are imposed onto other additional loads the tensile forces have to be overcome by prestressing so that from these loads alone compression forces in the range of 40 MN are created.

In order to illustrate the influence of these action forces on a structure they are compared with the prestressing forces required for bending in a 44 m span: at midspan about 32 MN are necessary and at the piers about 45 MN, i.e. an average prestress force of 38 MN. An increase of this prestress by about 50 % would require considerable additional structural measures not compatible with the aim of durable structures. The same holds true for the corresponding abutments. For this reason it is advisable to locate joints and fixed points in such a way that the calculated longitudinal forces do not exceed 12 MN. From this condition the following structural systems offer themselves for carrying the longitudinal forces.

4.1 The Continuous Structure Fixed at one End

In order to permit a relatively simple and quick replacement of a beam by transverse shifting, the lengths of continuous girders for valley bridges of the new DB railway lines are limited to 440 m. Superstructures up to this length are expediently provided with a fixed point at one abutment as shown in Fig. 5. Their longitudinal forces remain within the limits given above and can be carried easily by the abutment. The rail expansion jointing is located at the other abutment, well accessible and hardly influenced by beam deflections in plan and elevation.

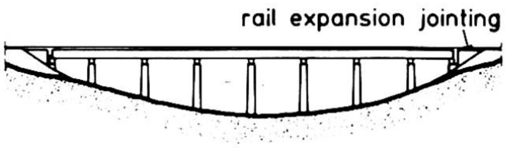

Fig. 5 Continuous girder bridge fixed at one end

4.2 The Continuous Structure with one Central Joint

For bridge lengths between 440 m and 880 m it is advisable to repeat the continuous structure mirror image as shown in Fig. 6. The same conditions for each part are created in this way with regard to longitudinal forces and their transmission. The rail expansion jointing for movements up to 800 mm is now located at the transition pier.

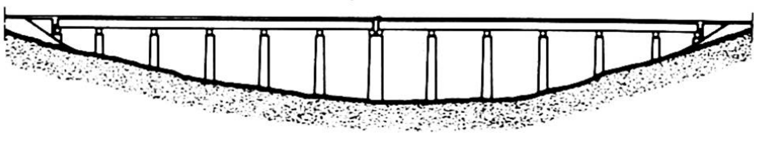

Fig. 6 Continuous girder bridge with one central joint

4.3 Structures with Hydraulic Longitudinal Bearings

For bridges longer than discussed above it is not possible within the framework of the given parameters to use exclusively the abutments as fixed points. Traditional piers are only in a very limited way suitable for the location of fixed points of railway bridges because the large braking and acceleration loads act partially dynamically. In the most unfavorable case the braking jolt may cause uncomfortable oscillations. Such a load transmission would in any case cause additional deformations and movements. Special devices between piers for carrying the longitudinal forces with small deformations require generally quite an effort.

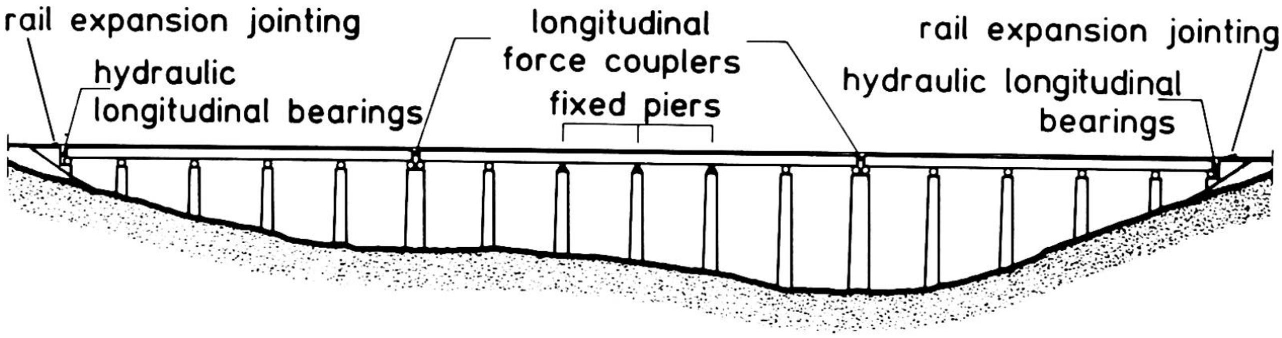

Fig. 7 Continuous girder bridge with hydraulic longitudinal bearings

On this background and in order to achieve as small as possible longitudinal forces acting in tension the system shown in Fig. 7 proves especially advantageous: Joints are located at both abutments, and in the bridge center one or more piers are fixed to the superstructure. This group of fixed points has the sole purpose of carrying the differntial friction forces from both bridge halves. Braking and acceleration loads are transmitted to the abutments by means of hydraulic bumpers acting in compression. With two rail expansion joints for movements up to 800 mm on the abutments this system can be built for bridge lengths up to 1600 m. In this case the biggest tensile force still remains below 12 MN and compression forces only up to 7.25 MN act on each abutment. This system offers itself, of course, for bridge lengths below 880 m also.

In order to permit replacement of the structure sectionswise, joints with longitudinal force couplers are provided.

The hydraulic longitudinal bearings are very similar to the hydraulic creep couplers in design and action. In difference to those, however, they do not have to fulfill special deformation conditions. Maintenance requirements and lifetime are estimated to be similar as for other bearings. Positive experiences are available from various 10 to 15 year old bridges.

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