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WORKSHOP 1

Posters

Tests on Steel Railway Bridges over the River Tisza

Essais des ponts ferroviaires métalliques sur la Tisza

Experimentelle Untersuchungen der Eisenbahn-Stahlbrücken über die Tisza

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INTRODUCTION

The crossing of the River Tisza by two continuous 3+4 span heavy-duty, high-speed railway bridges (Fig. 1.) offered an excellent chance to carry out delicate investigations relating to the special erection technology and the general behaviour of these type of structures. Some details worth of interest are presented in the poster-contribution of the authors.

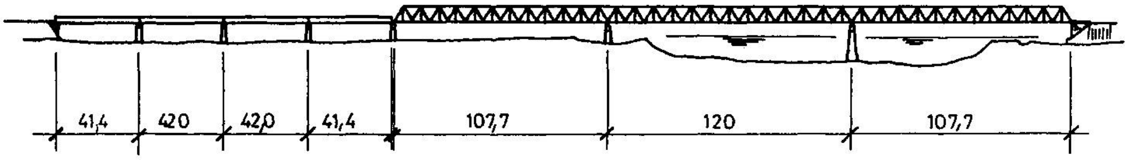

Fig. 1.

The design and arrangement of the steel superstructure were governed by the effort to make the erection operations as simple as possible. The roughly 18 m long plant made welded structural steel units were connected on the site by HSFG bolts.

Both continuous superstructures were pre-assembled on the banks of the river, in the line of the bridge axis. The completely assembled units were then gradually pushed to their place. Temporary trestles were needed only for the truss bridge. Chrom-nickel plated stools were placed atop the supports to distribute the reaction forces during the pushing process and PTFE coated plates were inserted continuously along the slide-track.

THREE-SPAN CONTINUOUS TRUSS BRIDGE

Over the river, the first superstructure is a three span continuous truss bridge with constant heigth of 9000 mm.

Actual measurements reveal the nature of semi-rigid connections of the complex structural system. The obtained normal stresses are confronted to easy to handle calculations of the floor system.

Components of normal stresses near the intersections of diagonals chords of the main are analysed for further refinement of fatigue design. (Fig.2)

Measured deflections show the effect of the rigid connections of bars in the main and the contribution of floor system as well. Special problems, such as effect of lack of fitness are analysed to focus attention to crucial details of bracing system.

Horizontal and vertical natural frequencies, maximum lateral displacements and dynamic coefficients had also been determined and are presented.

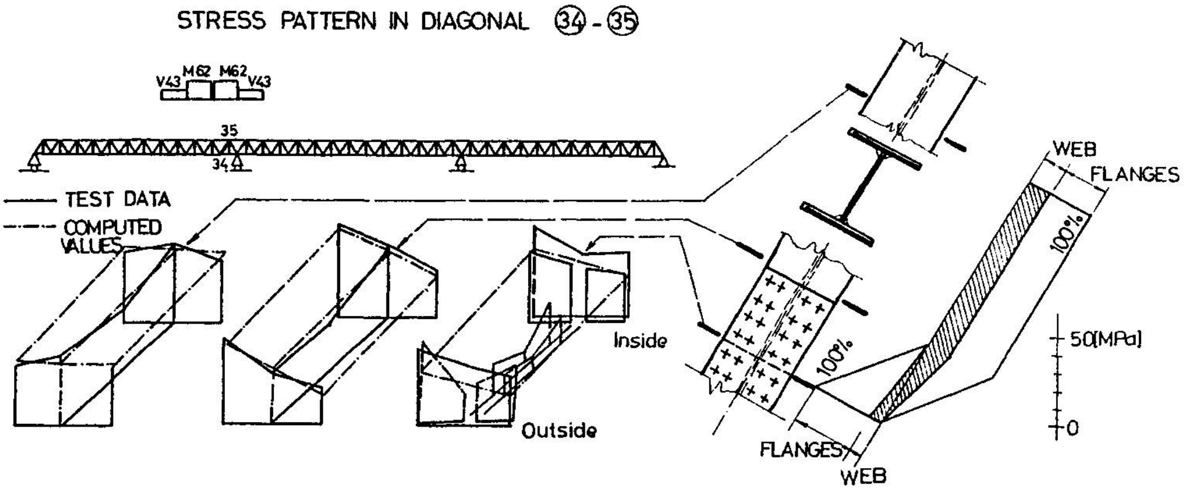

Fig. 2.

FOUR-SPAN CONTINUOUS PLATE GIRDER

The second structure of this bridge system is a four-span continuous plate girder with stringers, cross beams and bracing system similar to the trussed one.

Static measurements showed, that because of the elastic connections among the stringers and main girders, stringers behave as parts of the main girder, but not always with full intensity depending on the position of the loading along the bridge.

The horizontal stiffeners of the web can be taken into consideration totally while during calculation of cross sectional properties of main girders, stringers partially (0,6- 0,8) which are subjected to biaxial bending and warping.

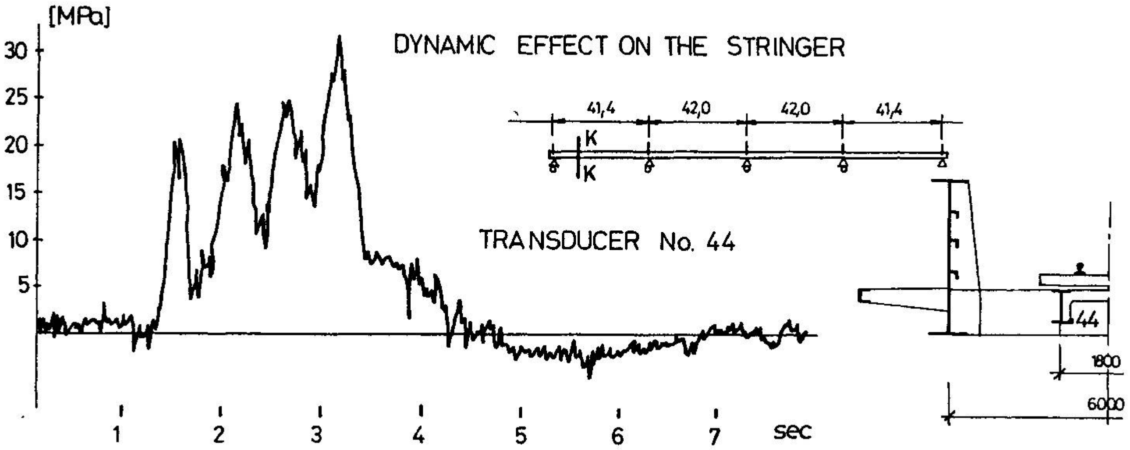

Fig. 3.

Dynamic measurements are illustrated by Fig. 3. Analysis of measured strains on stringers at a train velocity of 60 km/h resulted appr. 15% stress increase because of dynamic effect.

CONCLUSION

Test data prove, that special attention has to be paid to the interaction of different structural components so as to achieve a good agreement at the reality and the results of numerical approaches.

Veitshöchheim-Viaduct: a Concrete Arch Bridge with 162 m Main Span

Viaduc de Veitshöchheim: un pont en arc de 162 m d'ouverture

Talbrücke Veitshöchheim: eine 162 m weit gespannte Bogenbrücke in Beton

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DESIGN

The New Railroad Line from Hannover to Würzburg crosses the valley of the river Main about 10 km north of Würzburg. The main span (arch) is 162 m. This is the longest span of all concrete railroad bridges in Germany. Over the arch and the piers, a 1262 m long prestressed concrete box girder has been incrementally launched. This has been the longest launch from one end ever done. After launching the continuous girder was made discontinuous at piers 5, 10, 14 and 17, thus dividing the girder into five parts. The reason for this separation is to provide the possibility to replace the superstructure in short pieces of 237.o; 369.5; 214.o; 160.5 or 299.o m, should this ever be necessary. It is assumed that the not prestressed piers and the solid arch never has to be replaced.

Over piers No.5, 10 and 17 is a so called "longitudinal force coupler" and over pier 14 is an expansion joint. The bridge has two fixed points where longitudinal forces (e.g. braking forces) may be transmitted to ground. This is
a) at the crest of the arch, where all longitudinal forces acting onto the superstructure from axis 0 to 14 and
b) at piers 16, 17, 18, where all longitudinal forces acting onto the superstructure from axis 14 to 23
are transmitted to the soil. Rail expansion joints are in the axes 0, 14 and 23. The span between axes 22 and 23 consists of a solid slab 1.4o m deep and built on a scaffolding after the main bridge was launched.

CONSTRUCTION

The arch was built by free cantilevering with the aid of auxiliary cables, which are anchored in auxiliary towers, built of precast concrete segments. The arch is solid and 1.8o to 1.5o m (at crest) thick. Its fresh concrete was chilled from 28° to 10°C by liquified nitrogen. In consequence of the chilling the strength of the concrete could be increased from 55 to 65 N/mm^2. The reinforcement of the arch consisted of not prestressed Dywidag bars with a rolled thread. When these bars met in the closure joint in the crest of the arch, the ribs of the thread did not fit together. The free ends of the bars had to be elasticly twisted in order to be able to turn the coupler nut from one end of a bar onto the other bar. There was not sufficient space to join the two ends of bars by overlapping.

The incremental launching of the heavy concrete superstructure over the slender arch has been an engineering challenge. An arch of 162 m span and a rise of 25 m only, is sensitive, if a beam, three times as deep as the arch, is launched from one side over the arch. The arch was supported by additional cables on the heavily loaded side and it was ballasted by hanged up concrete blocks on the other side.

ACKNOWLEDGEMENT

The preliminary design, the tender documents, the checking of the detailed design and the permanent site inspection has been done by Leonhardt, Andrä und Partner, Consulting Engineers Ltd., Stuttgart, in close collaboration with the client, the German Federal Railway Administration, Nürnberg Division. The detailed design has been made by Obermeyer, Munich. Concrete specialist advisor was Prof. R.Springenschmid, Technical University, Munich. The bridge has been built by the joint venture of the contractors Strabag (Cologne/Würzburg) and WTB (Walter, Thosti, Boswau; Augsburg/Aschaffenburg).

The bridge is a milestone in the development of railroad bridge design and construction and it is a landmark in the beautiful Main valley. The viaduct was opened for traffic on May 29, 1989.

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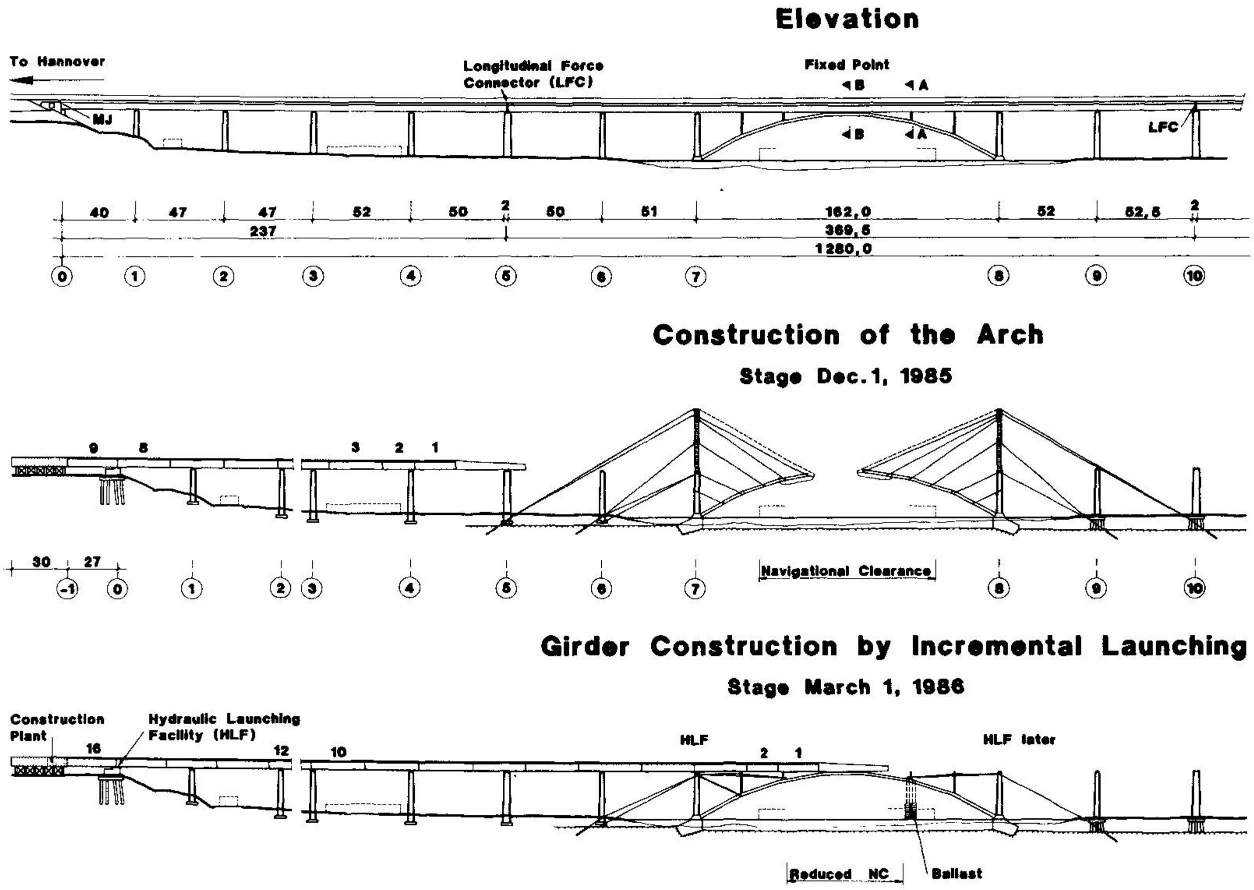

Veitshöchheim-Viaduct

Enz-Viaduct: a 1044 m Long Prestressed Concrete Box Girder Bridge

Viaduc de l'Enz: un pont en poutre-caisson en béton précontraint

Enztalbrücke: eine 1044 m lange Spannbeton-Hohlkastenbrücke

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DESIGN

The New Railroad Line from Mannheim to Stuttgart crosses the valley of the river Enz about 25 km before Stuttgart. This is the longest viaduct on this new line. It consists of 18 spans of 58 m each, that means a bridge length of 1044 m. The maximum height of the rails above the valley is 47 m.
The bridge is straight on its total length in plan view. The gradient has a change of longitudinal inclination from -0.15 % to +1.2434 %. The radius of the sag-curvature is 60,000 m. This curvature comprises half of the bridge length and is so flat that it can hardly be recognized in the elevation view.
The "General Design" for viaducts of the New Railroad Lines provides for simple supported box girders with a distance of the piers of 44 m. In order not to impair the free view through the scenic valley, the distance between piers has been chosen with 58 m. Additional aesthetical advantages are obtained from the choice of a continuous girder. With a depth of 4.75 m only (compared to 5.3o m required for a simple supported girder) the structure can be designed more slender (piers 3.o m wide instead of 3.5 m).

The total length of the box girder of 1044 m is devided into three equal parts of 348 m each (6 spans of 58 m). Should it ever be necessary, the bridge can be replaced in parts of 348 m in length. A new 348 m long bridge would be built on auxiliary piers aside of the old bridge. By lateral shifting, the old bridge could be replaced by the new one within a few days. The three parts are connected by longitudinal couplers at the piers No.6 and 12 (see poster).
The horizontal forces from braking and accelerating of the trains are transmitted to the abutments (axis 0 and 18). There, the forces are carried onto the ground via hydraulic dampers. The dampers follow slow changes in the width of the rail expansion joints from temperature changes. The dampers react as stiff members in case of the occurance of sudden forces. The bridge is fixed to the piers Nos.7 through 11. This group of five piers holds the bridge in position even if braking should occur many times to the same direction. The group of fixed piers is also able to take the whole braking force, should the hydraulic dampers ever fail.

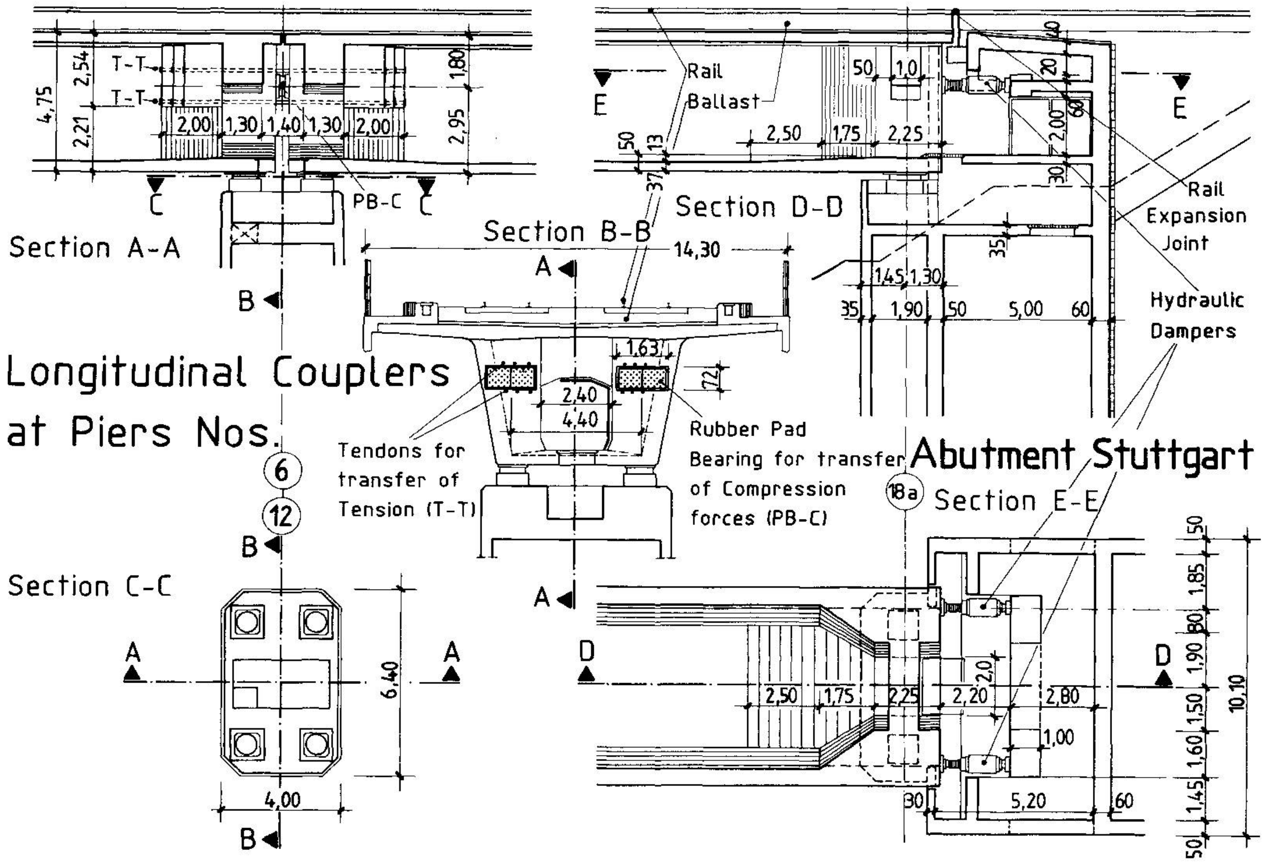
CONSTRUCTION

The bridge has been built by the incremental launching method on its total length of 1044 m, straight, also in elevation view. Half of its length has been bent into the radius of 60,000 m during launching. Additional prestressing has been provided for these additional moments of constraint.
During launching the three continuous girders have been fixed together to one continuous box girder. After launching the bending stiffness has been released above piers Nos.6 and 12.

The envelopes of the bending moments show maximum moments near the front end of the girder during launching (see poster). The auxiliary steel launching nose has been 36 m long. The length of a casting element was 29 m, that means half a span length. The trough has been cast on Wednesday, the deck slab on Friday, prestressing and launching has been done on Monday, for a mid span element. The pier elements required two weeks of construction time.
The fabrication plant behind the abutment consisted of three parts of 29 m length each:
a) preassembly of reinforcement and tendons,
b) casting chamber with movable roof and
c) curing chamber with heating in winter.
The concrete cover has been measured by magnetic devices (Proceq Profometer) immediately after removal of formwork. About 40,000 measurements have been made. Also cracks in the concrete have been carefully searched. Improvements in the reinforcement (and in its fixing in the formwork) have been made as a result of the careful inspection during construction works.

ACKNOWLEDGEMENT

The preliminary and detailed design, the preparation of tender documents and the permanent site inspection has been done by Leonhardt, Andrä und Partner, Consulting Engineers, Stuttgart in close collaboration with the client, The German Federal Railway Administration, Karlsruhe Division. The soil and foundation specialist was Dr.-Ing. Christow, Karlsruhe. The checking of the design and the inspection of the prestressing works has been done by Dr.-Ing.Kiefer, Darmstadt. The supervising soil expert was the soil institute of Prof.Smoltczyck and Partners, Stuttgart. The bridge has been built by the joint venture of the contractors Dyckerhoff & Widmann (Munich/Stuttgart), Stumpf (Bruchsal) and C.Baresel (Stuttgart). Due to the very cooperative and open-minded client a most progressive railroad bridge of high quality and durability could be realized. The bridge was opened for traffic on June 2, 1991.



Enz-Viaduct: Coupling devices, hydraulic dampers and rail expansion joint

Railway Bridge with Double Composite Action across River Main

Pont-rail en structure mixte sur le Main

Eisenbahnbrücke mit Doppelverbund über den Main

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Senior Supervising Engineer

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DESIGN

The double track railway bridge across the River Main at Nantenbach will link the new highspeed railway line Hannover-Würzburg to the existing trunk line Würzburg-Aschaffenburg. Due to local conditions, the bridge has a main span of 208 m, a slope of 12.5 ‰ and a radius of 2650 m.

Based on the investigation of numerous alternates, a continuous truss girder with spans of 83.2 - 208 - 83.2 = 374.4 m was found to be the best solution from economic, ecologic and aesthetic point of view, Fig.1, 2. The construction depth varies between 8.5 m at ℄ and the abutments and 16.5 m at the main piers, corresponding to slenderness ratios of 1:24 and 1:13, respectively.

The cross-section, s.Fig.3, consists of
- the truss girders in a mutual distance of 6.o m and with a spacing of 10.4 m;
- the top slab of reinforced concrete, which corresponds to the »Rahmenplanung für Talbrücken« (Masterplanning for valley bridges);
- the bottom chord, of steel in the center of the mid span and of concrete at the piers and in the side spans. The concrete bottom chord limits economically the deformations and makes that fatigue considerations do not govern the dimensioning.

The steel weight is 3300 t or 620 kg/m^2.

CONSTRUCTION

The construction started in early 1991 and is scheduled to be finished in late 1993. The site spans will be erected on auxiliary piers. After pouring of the bottom chord concrete, the center part of the main span, with a length of 120 m and a weight of about 1100 t, will be floated in and lifted. After closure of the center joint, the top slab is poured from ℄ towards the abutments.

ACKNOWLEDGEMENT

Owner is the German Federal Railway Administration, Nürnberg Division (Deutsche Bundesbahn, Direktion Nürnberg). The design and the tender documents were prepared by Leonhardt, Andrä & Partner GmbH, Stuttgart, Germany. The construction consortium is formed by Noell, DSD and Buyck for the steel structure; Strabag and Hochtief for the concrete slabs; and Max Streicher for the foundations and approach bridges.

Fig. 1: General Layout

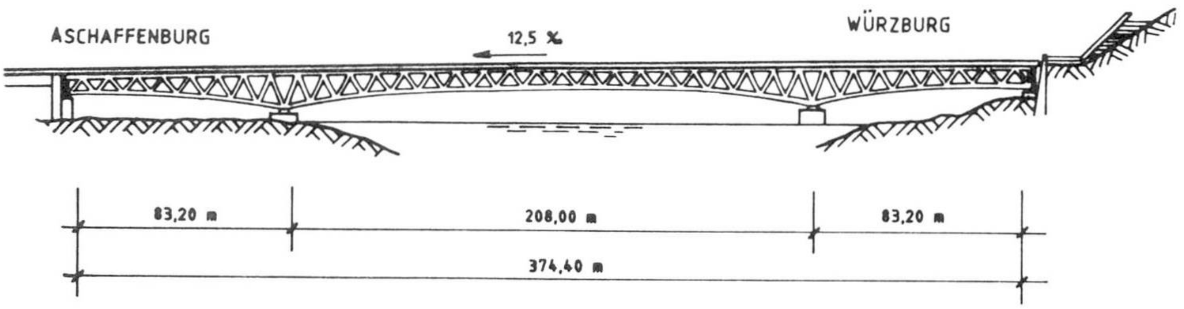
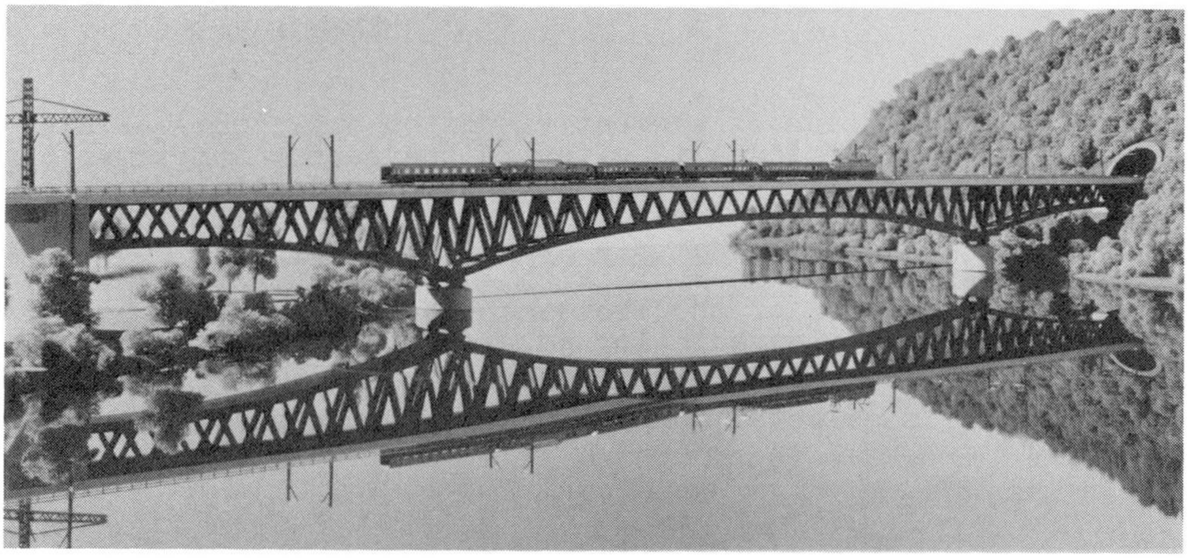


Fig. 2: Foto of model



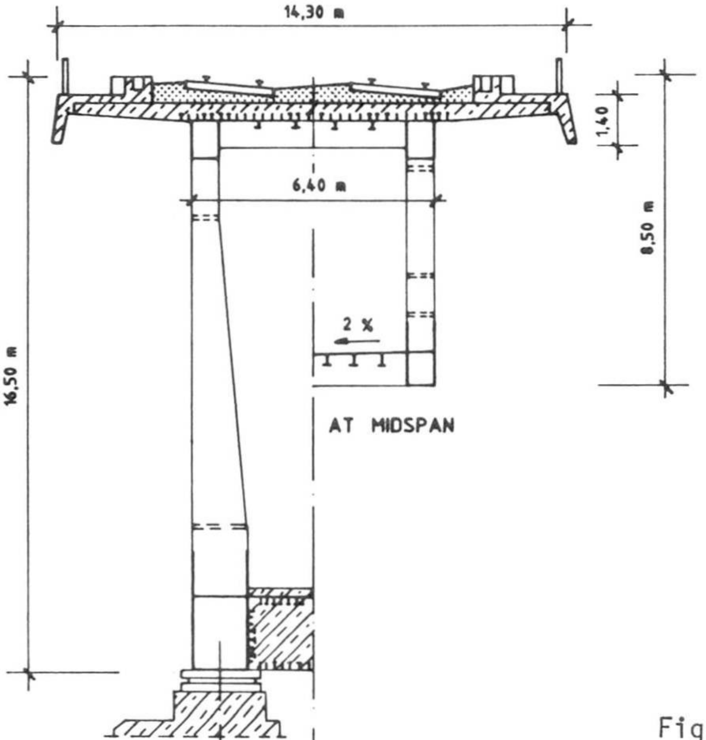


Fig. 3: Cross-section at ℄

Vibration Amplitudes of Steel Railway Bridges

Amplitude des vibrations de ponts ferroviaires métalliques

Schwingungsamplituden in Eisenbahn-Stahlbrücken

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A method of calculation of element vibration amplitudes of high-speed railway metal bridges is given in the paper. Sometimes high-frequency stresses, caused by vibrations, can significantly reduce the durability of metal bridge elements [1]. The operation of such bridge structures proves that vibrations of separate elements of lattice span structures increase with the growth of the trains' speeds. The main cause of the above-mentioned phenomenon seems to be the high-frequency dynamic forces, resulting from the "carriage wheel-bridge railway track" interaction.

Structural models selected for the problem in question are given in Fig.1 and 2.

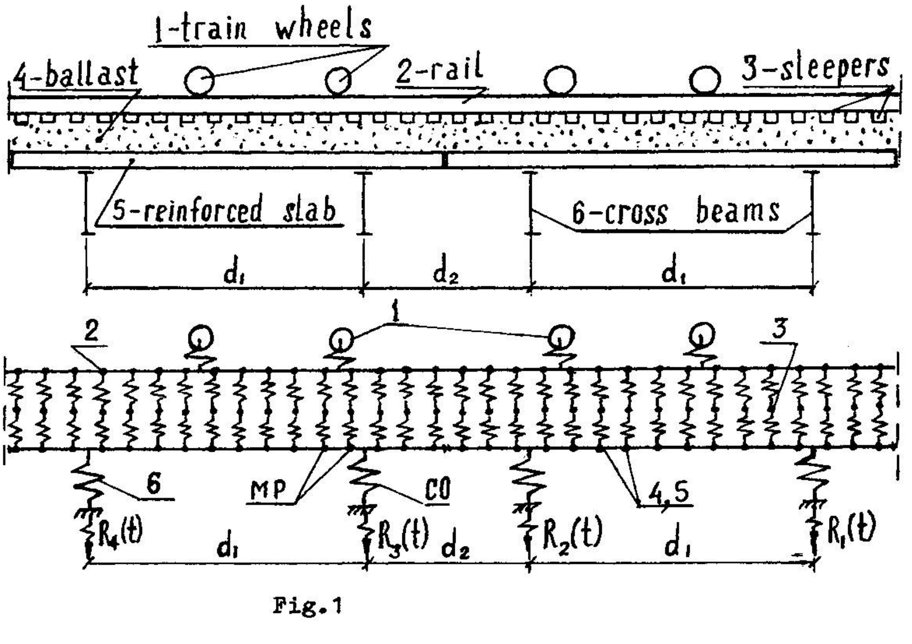

Fig.1

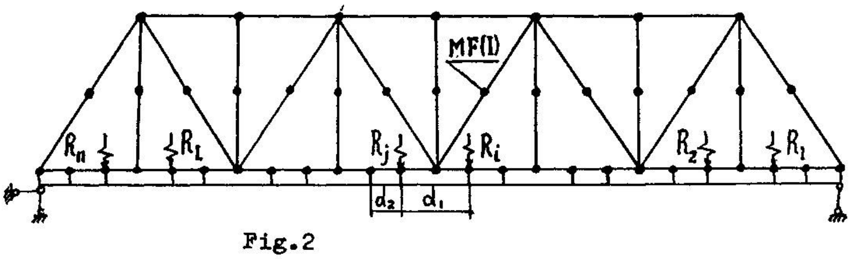

Fig.2

Metal span structures for the high-speed railway bridges are supposed to have rigid bottom chord and the continuous welded track placed on the ballast. The problem is solved in two stages. At the first stage on the basis of both the adopted structural model (see Fig.1), and mathematical model of "carriage wheels - bridge floor - bridge roadway" system variable reactions $R_i(t)$ at the elastic supports with CO stiffness are estimated. These elastic supports model the cross-beams of the span structure. At the second stage accelerations $\ddot{YF}(I)$ and displacements YF (I) of masses which model the elements of the main trusses are determined (see Fig.2). Oscillations of these masses are caused by dynamic forces $R_i(t)$ and are described by the equations as

$$MF(I) \cdot \ddot{YF}(I) = R(I,t) - \Sigma K(I, J) \cdot YF(J) - \Sigma \dot{YF}(J) \cdot B(I,J), \qquad (1)$$

where MF(I), YF(I) are mass and displacement of the first unit of the truss; K(I,J), B(I,J) are coefficients of rigidity and resistance of the system.

$R_i(t)$ - force, transferred to the first mass of the bottom chord of a truss from the bridge roadway, is determined by the following expression

$$R(I,t) = MP(YP(J) + YP(J+1))/2 , \qquad (2)$$

where YP(J) and YP(J+1) are accelerations of the roadway slab masses MP, between which the elastic support CO is placed (see Fig.1).

The estimation of YP(J) and YF(J) values is based on the numerical integration of the movement of masses, shown in Fig.1 and 2, by Predicator-Corrector Method [2]. The problem statement allows to take into account one-sided ties at the "wheel-rail" contact and during the "sleeper-ballast" interaction. Integration step T depends on dynamic parameters of the system in question. Generally T is adopted by an order of magnitude less than the minimum period of natural oscillations of the system ($T = 10^{-3} - 10^{-5}$ s.)

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