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Railway Installation on the Tagus Suspension Bridge in Lisbon, Portugal

Trafic ferroviaire sur le pont sur le Tage à Lisbonne, Portugal

Eisenbahnverkehr über die Tagus-Brücke in Lissabon, Portugal

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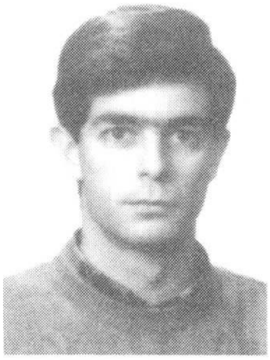

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SUMMARY

Design solutions are presented for the railway installation on the Tagus suspension bridge in Lisbon, in particular the strengthening of its northern access viaduct. The roadway deck, presently with mid-span hinges requiring frequent maintenance, is transformed into a continuous deck by the use of external prestressing. The design solutions to install the railway deck at the viaduct are discussed. Typical results of the structural models developed and in-situ measurements are referred to. The construction works on the suspension bridge and the northern access viaduct are expected to start in 1995 and are outlined in this paper.

RÉSUMÉ

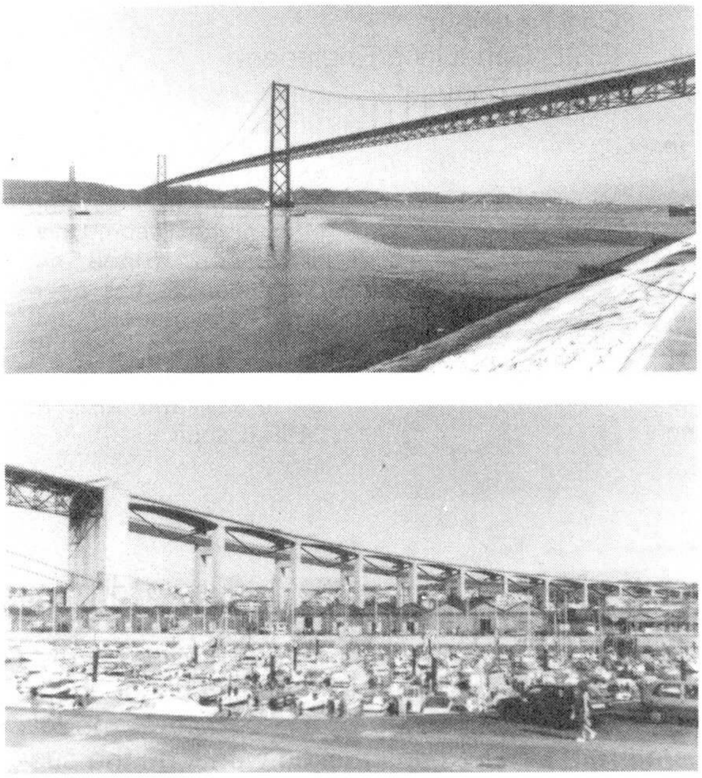
Cet article décrit le projet pour le passage du chemin de fer sur le pont suspendu sur le Tage à Lisbonne. Le renforcement du viaduc nord est présenté. Le tablier est actuellement une suite de poutres simples, qui nécessite un entretien fréquent. Il sera transformé en travée continue par application de précontrainte extérieure. Les solutions pour l'installation du tablier ferroviaire dans le viaduc sont discutés. Les résultats des modèles structuraux développés, de même que les mesures "in-situ" sont présentés. Les travaux du pont suspendu et du viaduc nord commenceront en 1995 et sont ici résumés.

ZUSAMMENFASSUNG

Dieser Artikel beschreibt das Projekt, den Eisenbahnverkehr über die Tagus-Hängebrücke in Lissabon zu führen, insbesondere die Verstärkung des nördlichen Viaduktes. Die Fahrbahnplatte ist gegenwärtig mit Zwischengelenken ausgebildet, die einen ständigen Unterhalt benötigen. Das System soll mit Hilfe der Vorspannung in eine durchlaufende Platte umgewandelt werden. Einige entwickelte konstruktive Modelle und Resultate der Messungen werden beschrieben. Die Bauarbeiten an der Hängebrücke und am nördlichen Viadukt dürften 1995 beginnen und werden in diesem Artikel erwähnt.

1. INTRODUCTION

Tagus Suspension Bridge (Fig.1) and its Northern Access Viaduct was built almost 30 years ago and originally designed for highway and railway traffic. The bridge was built with a roadway deck only, leaving the railway installation as a 2nd phase project. Three decades later the Government decided to install the railway deck, however for train loads much higher (about 2.5 times more) than originally envisaged. The original design for the railway installation was based on strengthening of the suspended spans through a set of cable stays anchored on the existing towers.



On the northern side, a long access viaduct (about 1 km) required also the installation of the railway deck as also the strengthening of the roadway deck. A brief description of the design studies and the solutions achieved for this project is presented in this paper with particular relevance for the design aspects where the authors were involved, namely the Basic Studies for the suspension bridge and the Rehabilitation and Railway Installation on the Northern Access Viaduct.

Fig.1 Tagus Suspension Bridge and Northern Access Viaduct with a perspective of the railway deck.

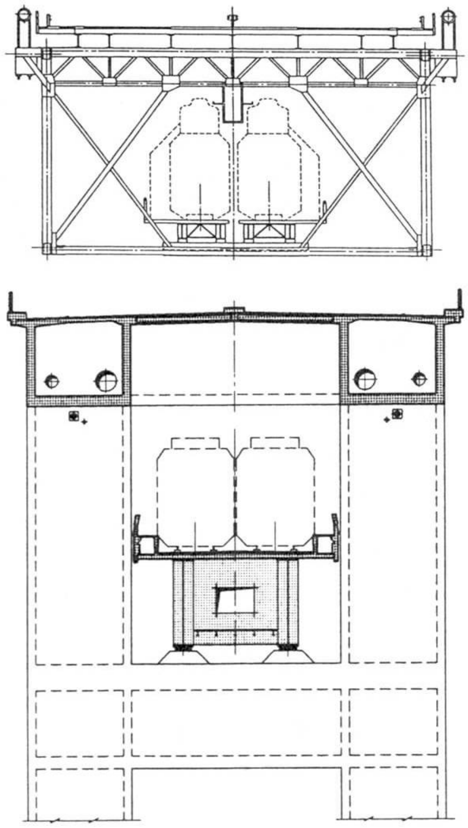


Fig.2 Location of the new railway deck on the bridge and access viaduct

2. PROJECT DESCRIPTION

The railway installation on the Tagus Suspension Bridge, with a main span of about 1000m, requires the installation of a secondary cable on the bridge, the strengthening of the main truss and the execution of new structural system under the roadway deck to support the new railway deck (Fig.2).

At the Northern Access Viaduct (Fig.3) the existing structure is a prestressed concrete bridge 937m long with typical spans of 76,0m and 74,2m. The roadway superstructure was built also in the sixties by a balanced cantilever scheme with expansion joints at every mid span. These joints allow relative longitudinal movements and rotations between the two extreme sections of the cantilevers but restrict vertical and transversal horizontal displacements. The existing hinge devices are made of high strength steel and are shown in Fig.4. Due to a deficient performance of these hinges, relative vertical displacements at the end sections of the cantilever produce deterioration of the expansion joints requiring frequent maintenance. Presently 95% of the original expansion joints were

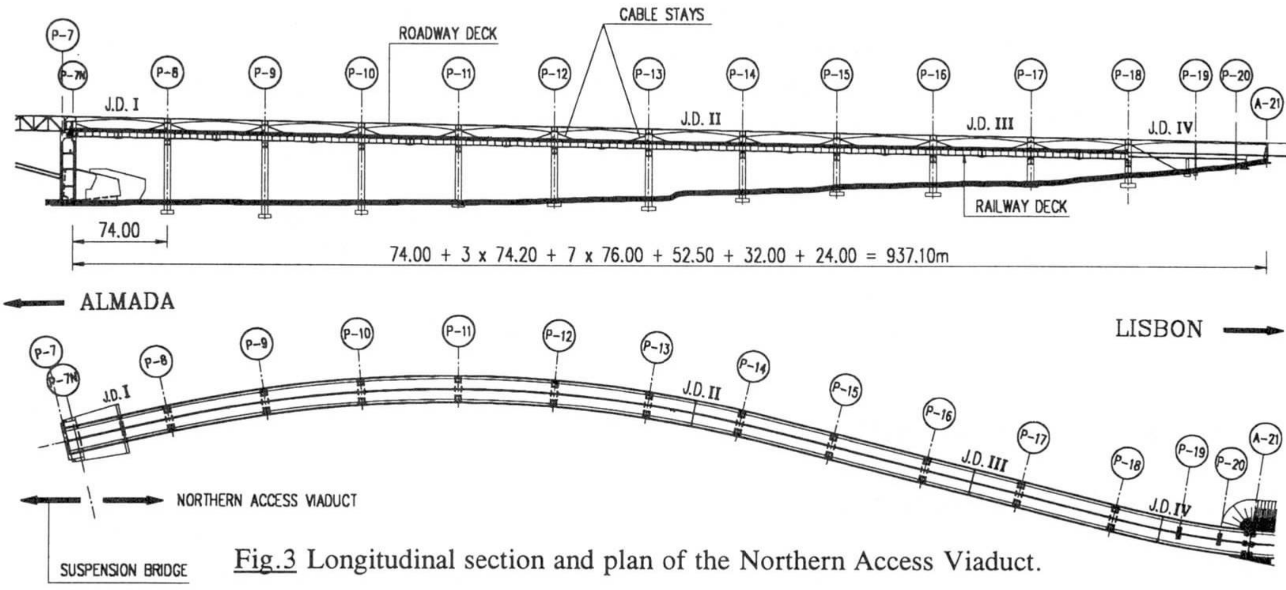


Fig.3 Longitudinal section and plan of the Northern Access Viaduct.

replaced by neoprene Transflex type joints requiring also considerable maintenance (including replacement by new ones) due to the dynamic effects of the relative displacements refered to above.

A design to eliminate the expansion joints was then envisaged since creep and shrinkage deformations are stabilized. Besides, due to the dynamic load effects at the joints, extensive cracking of the cantilever segments, adjacent to the joints, were detected by the inside of the box girders. So, a rehabilitation design of the roadway deck had also to be develloped.

The railway deck shall be installed between the two legs of the existing piers. A continuous composite steel-concrete bridge superstructure described in section 5 was designed. This structure is supported at the cross beams between the two pier legs (Fig.2).

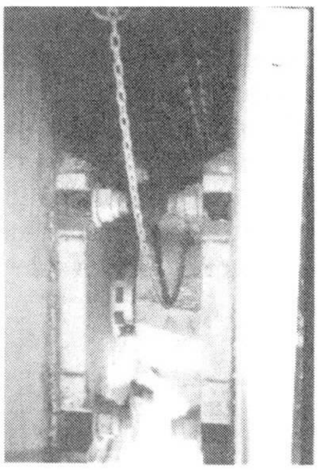


Fig.4 Existing hinge devices.

3. BASIC DESIGN CRITERIA

Prior to the preliminary design phase, a set of basic studies were develloped to establish the design criteria for the Suspension Bridge. That was decided due to the singularity of the project namely by the lack of national or european codes to be directly applied to a project like the one under discussion. The bridge was originally designed for light trains and the standard UIC ("Union Internationale de Chemins de Fer") railway loads, adopted in the portuguese code, would cause a too severe design criterion taking into consideration the type of trains expected to run on the bridge. The avaiable load capacity of the piers, towers and foundations as also the limits for deformability required for the suspended spans, were taken into consideration. On the basis of several studies, it was decided to design, the strengthening of the bridge for a set of train loads the most severe ones are freight trains with maximum load of 13800kN, passenger trains with a maximum load of 7232kN for the case of 4UTE type and double deck trains with 7400kN.

It was decided by economical reasons to establish the following traffic constraints:

- only one freight train on the suspension bridge;
- maximum of two passenger trains simultaneously on the suspension bridge;
- maximum speed of the train - 60 Km/h;
- maximum expected number of trains on the bridge - 250 per day in each direction.

The structural design was based on a set of limit state design conditions for rare, frequent and "quasi-permanent" load combinations. At the ultimate limit state (ULS) the train loads, the highway loading and other variable loads were combined according to standard ULS format adopted in the new Eurocodes 1 and 3 as also in the Portuguese Code for Actions.

The design of steel components were based on British Standard for Steel Bridges BS5400 Part 3 and UIC recommendations for railway bridges. The maximum slopes allowed under railway loading were limited to 2.5%. In the Viaduct, the longitudinal profile has already 2.0% and so the maximum allowable slopes due to structural deformations are 0.5%.

4. REHABILITATION OF THE ROADWAY DECK

4.1 Presente Structural Performance

Presently two types of anomalies are observed in the roadway deck - relative vertical displacements between the cantilever end sections at some of the expansion joints and extensive cracking at the box girder segments adjacent to the expansion joints. Typical field observations are shown in Fig.5. The refered relative vertical displacements are due to anormal deterioration of the steel hinge (Fig.4) and are responsable for the high maintenance that had been required at the expansion joints. In what concerns cracking of the concrete superstructure it was concluded, from field observations and from the analysis of the original design, it is due to the impact load effects induced by the relative vertical displacement at the expansion joints combined with a very low level of ordinary steel reinforcement and complete lack of prestressing (Fig.6) at the end segment of each cantilever.

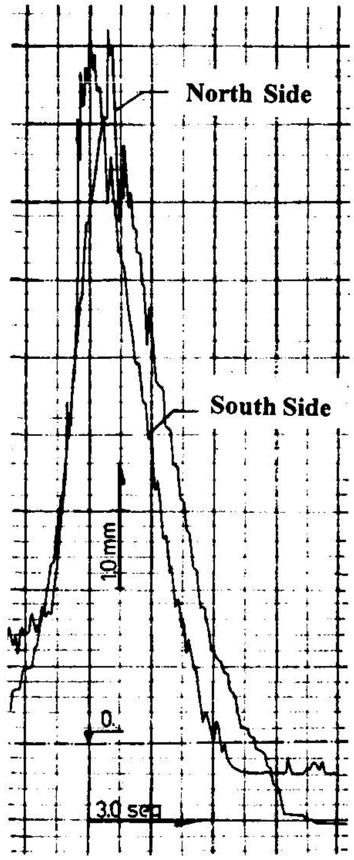


Fig.5 Displacements of the two adjacent sections to hinge 17-18 duo to load test vehicle, CMEST Report 3/94.

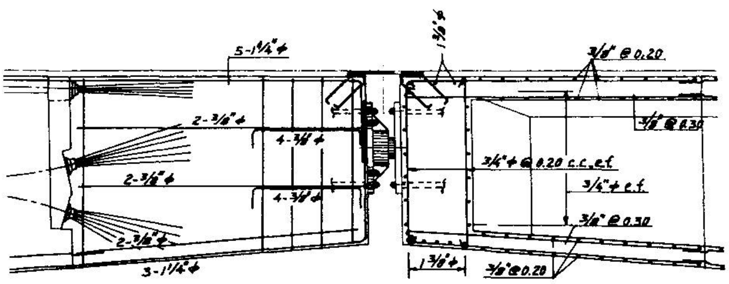


Fig.6 Typical cantilever joint - prestressing and reinforcement.

4.2 Improving Structural Continuity

As previously refered to, it was decided to eliminate as much as possible, the expansion joints at the roadway deck. The main structural effects are increased bending moments in the piers and foundations due to thermal actions in the deck and induced positive bending moments at the span segments due to traffic loads and positive thermal gradients at the deck sections. The former effect, has a direct implication on the number of expansion joints that can be eliminated. The last effect requires the design of a strengthening system for the deck.

From a detailed structural study it was concluded about the possibility of eliminating all the expansion joints except at the deck end sections (EJ I and EJ IV in Fig.3) and at the two intermediate sections (EJ II and EJ III). This solution was adopted through an optimization process where the number of expansion joints was maximized under the constraint of the design resistent bending moments of the piers. The existent foundations were checked for the structure obtained after the continuity introduced at the expansion joint deck sections. The resulting structure after elimination of 67% of the present expansion joints has a continuous deck along 452m, 225m and 152m as may be seen in Fig.3.

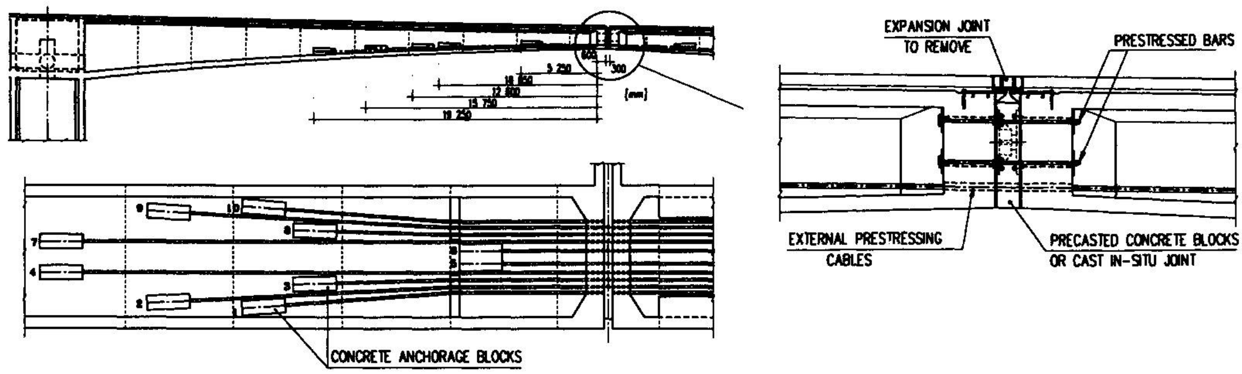


Fig.7 Design solution for a continuos joint: external cables layout and prestressing bars.

The design solution to obtain continuity requires:
- to insert precasted concrete blocks or alternatively to cast in-situ a transversal segment;
- to introduce a continuity prestressing at the joint and at the span sections through prestressing bars and external prestressing cables (Fig.7).

One typical layout of the cables arrangement is shown in Fig.7 which were introduced to rehabilitate also the box girder segments adjacent to the original expansion joint. Since no complete interruption of traffic is allowed during the execution of the works, a temporary prestressing is introduced through two of the cables in order to have minimum compressive stresses of about 1 MPa at the bottom flange box girder sections before permanent continuity is obtained. So, the temporary prestressing is stressed before continuity is introduced at the expansion joints, being the temporary cables replaced by permanent ones at the final stage of the closing operations.

4.3 Structural Behaviour

The bridge structure as modified by the continuity introduced at the span joints referred to in the previous section, was analysed with a 3D frame model. Longitudinal 3D beam elements at each segment of the two box girders, rigidly connected to transverse beam elements modelling the bridge deck, were considered. A linear structural analysis was carried out, since creep and shrinkage effects are no longer present. The ageing effects on the concrete modulus of elasticity and on its compressive strength was taken into consideration.

The structural design for the strengthening of the bridge deck is governed by load combinations of "permanent actions + highway loads + differential thermal actions" where internal and external prestressing load effects are included in the permanent actions. One of the most interesting challenges was to design the external prestressing to garantee no tensile stresses at the joints where continuity was introduced (i.e. the decompression limit state) for the "rare combination". This load combination includes the characteristic value of the highway loads (4 kN/m^2 uniform live load + 50 kN/m uniform transverse load acting at any cross section to produce the most severe load effects) together with the "frequent value" of the thermal gradient along the depth of the bridge deck (7.5°C between the top and bottom fibers). Local load effects under the "truck" live load (600 kN in 3 axes of 200 kN each, 1.5m distance between axles) were investigated on the basis of a finite element shell model of a typical viaduct span. However, for the normal stresses (bending+axial forces) at the joints where continuity was introduced, the "rare combination" referred to above is the governing design condition. Fig.8 shows the variation of the normal stresses along the extreme fibers of the bridge deck, obtained after the external prestressing. At each mid span sections the maximum compressive stress is about 6 MPa while the stress at the bottom fiber is between -1 MPa and zero. One shall note a permanent compressive stress at the two joint adjacent sections (-1 MPa at the top and bottom fibers) due to the almost axial external prestressing introduced at the span where the expansion joints were kept.

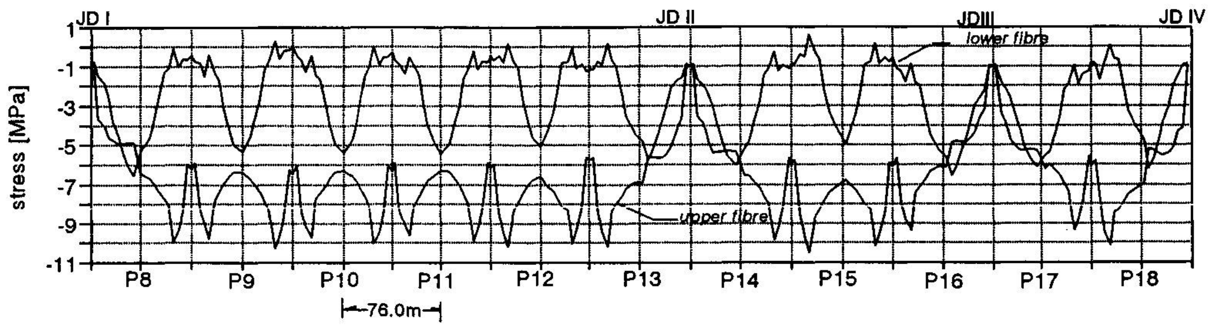


Fig.8 Normal stresses [MPa] for the rare combination at the extreme fibers of the deck.

Finally one notes the local effects introduced at the box girder flanges by the anchorages blocks (Fig.8) of the external prestressing. A detailled finite element analysis was required to predict increased craking effects under transverse tensile stresses introduced by the anchorages.

5. INSTALLATION OF THE RAILWAY DECK

Taking as a constraint, the avaiable load carrying capacity of the piers and foundations, structural solutions for the railway deck were investigated. The aim was to design a light railway deck, because earthquake forces tend to be the governing design actions for the piers. Then a steel structure was prefered to a prestressed concrete one. However, if a full steel deck was selected, increased difficulties had to be faced to reduce noise impact, in particular at the northern spans where the railway is only a few meters above residential and office buildings. So, a composite concrete and steel superstructure, with side noise barriers (Fig.9), was the most convenient. Several structural composite solutions, were investigated namely: a superstructure with two plate girders, a single cell box girder solution,a superstructure with 3 truss girders and a cable stayed plate girder solution. This last solution was selected for the final design. It consists on two plate girders, 4200mm deep, with transverse diaphragms. A cable stayed scheme is introduced at every span, with a slidding deviation cell under the mid span section of the deck (Fig.9), with a spacial configuration due to several geometrical constraints. The stays, anchored at the piers, are independent at every span and induce upward(prestressing) forces at mid span in order to reduce the permanent bending moments in the girder. A meaningful reduction in the plate thicknesses of the flanges is achieved by this scheme.

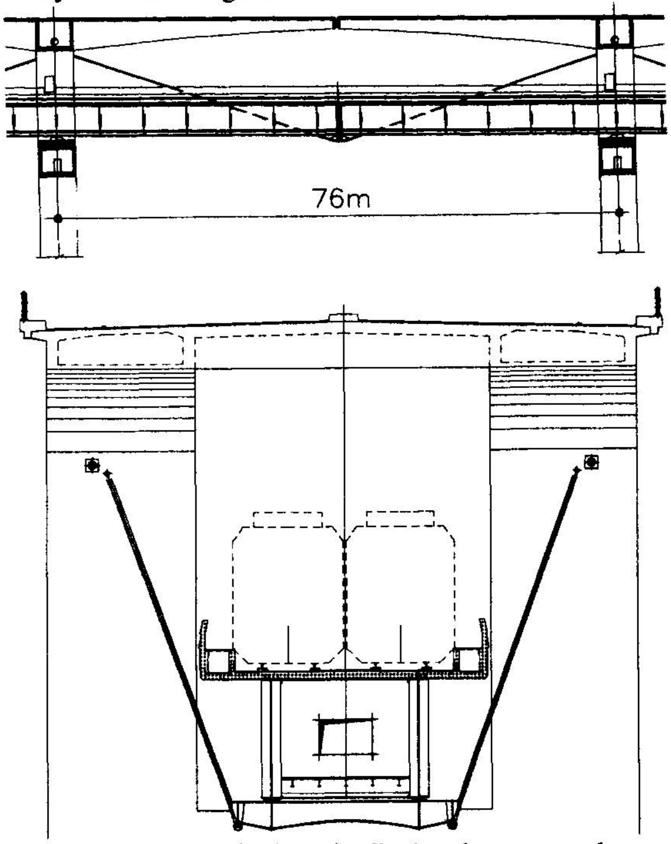


Fig.9 Railway deck. Longitudinal and transversal section.

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