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The bridge over the "La Fontaine" ravine, Réunion Island

This article deals with the design and construction of a bridge on Réunion Island. Its design results from a close cooperation between architect, landscape designer and engineers. It concerns a steel arch bridge with an unusual slenderness ratio of 1:7.5 guided by geotechnics, accessibility to the excavations and environmental and aesthetic considerations. Detailed studies of the wind (numerical calculation of the wind, wind tunnel studies, calculations with turbulent wind) allowed cyclone conditions to be taken into account in the design for top speeds, which can reach nearly 300 km/h. The structure was built using the cantilever method thanks to the addition of diagonals and temporary anchoring tie beams allowing a transitory lattice-type operation.

1 Introduction

1.1 Geographical context

Réunion Island is an overseas French département located in the Indian Ocean to the east of Madagascar. It is an island of volcanic origin with an almost circular form approx. 60 km in diameter. The island is composed primarily of the old volcano, now extinct and collapsed, which gave place to the creation of three "cirques" (calderas) and the active volcano, located in the east of the island.

In the west of the island, the sides of the old volcano consist of inclines at an angle of approx. 15° to the horizontal which since the collapse of the volcano are subjected to continuous erosion. This erosion has given and gives rise to the formation of valleys of variable width and depth. Because of their box-like character, they have been given the name "ravine".

1.2 Road transport on the island

Réunion Island had nearly 300 000 private vehicles in 2004, which means about one car for every two inhabitants. In that same year, 135 000 t of four-star petrol and 230 000 t of diesel were imported.

Réunion has a network of five main roads providing a route around the island and a crossing across the plains from Saint-Benoît to Saint-Pierre as well as the road leading to Cilaos.

The road network in the west of Réunion is composed of secondary roads and main roads, and therefore the characteristics are often reduced to 2×1 lanes.

In order to ensure a continuous motorway type of connection in the west of the island, work on a connection project with 2×2 lanes connecting Saint-Paul to the commune of Etang-Salé started in the mid-1990s. This future connection is called "La route des Tamarins" (Fig. 1).

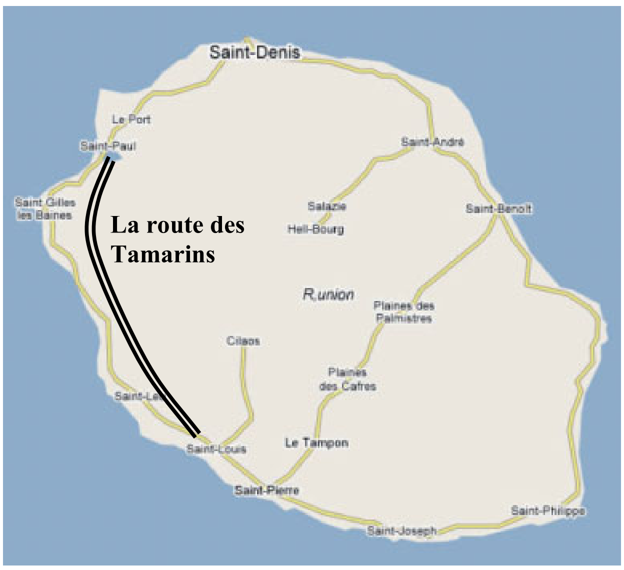


Fig. 1. The main road network of Réunion

The construction of this new connection is the largest construction project the island has ever known! More than 30 km of road are under construction, including 30 major civil engineering structures. Four of them can be regarded as exceptional because of their type, dimensions and/or location.

2 The bridge on the "La Fontaine" ravine – the functional programme

There are essentially five parameters that make up the programme of the structure:
- The situation and the lengthwise profile
- The functional transverse section
- The geological and geotechnical context
- The climatic conditions
- The respect for the environment.

2.1 Situation and lengthwise profile

The Tamarins road runs along the sides of the volcano, approx. 1 km from the

Fig. 2. *La Fontaine ravine seen from Saint-Leu*

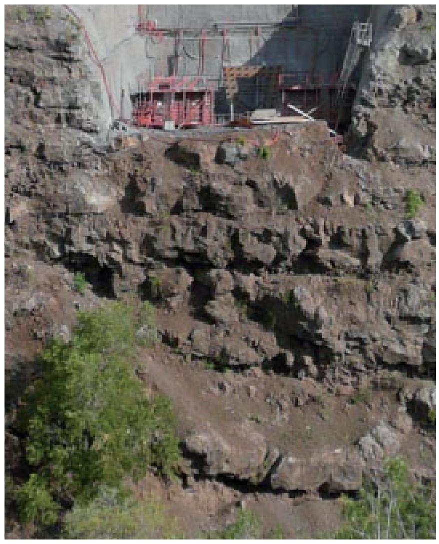


Fig. 5. *Alternating basalt scoria*

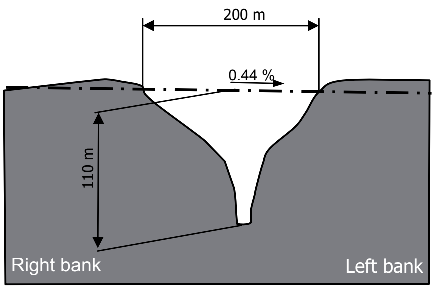


Fig. 3. *Lengthwise profile at the bridge*

coast and at an altitude close to 300 m. Consequently, it must cross innumerable ravines, which explains the large number of significant civil engineering structures. La Fontaine ravine (Fig. 2), located near Saint-Leu, has a width of 200 m and a depth of 110 m at the level of the road (Fig. 3).

2.2 The functional transverse section

As was mentioned above, the road has two traffic lanes in each direction (Fig. 4).

If we consider a 7 m carriageway in each traffic direction, to which we must add a central reservation of 2.60 m containing the "New Jersey" safety barrier, lateral clearance strips each 1 m wide and the space necessary to anchor the crash barrier (2 × 75 cm, standard French BN4 profile), the result is a minimum total width of structure of 20.10 m.

2.3 The geological and geotechnical contexts

Considering the volcanic origin of the island, the subsoil consists primarily of an alternation of basalt strata, with various degrees of fracture, and pockets of scoria (Fig. 5). Basalts generally show excellent mechanical characteristics, whereas scoria are pulverulent materials with a much lower stiffness. The whole forms a particularly heterogeneous matrix and the preliminary reconnaissance work generally faced many difficulties in predicting the true geological facies that would be met with during the excavation work.

It nevertheless proved to be possible to establish, from a great number of surveys, the possible support zones for the structure (Fig. 6), according to its design.

The young age of the ravines and the ongoing erosion mean that the profile of the cliffs is still undergoing development, through the undercutting of the scoria pockets and consequent slip of the basalt blocks. These considerations forced the designers to retain a certain setback relative to the edges of the ravine before installing any foundation. A setback of 15–20 m is generally considered as reasonable in order to get clear of the zones of decompression behind the cliffs, and a balancing gradient of 50° is regularly established.

As a first approach it proved reasonable to consider (Fig. 6) the following:

- For a bridge whose reactions are vertical, the support zone must be at the rear of a zone having a setback of 20 m relative to the edge of the cliffs and find a basalt slab sufficiently thick to allow diffusion of the support reactions.
- For a bridge with inclined reactions (arch or portal bridge) it was necessary to consider a zone following the natural balancing slope as unstable, but a support surface directly behind this zone was acceptable, as far as it was possible to find

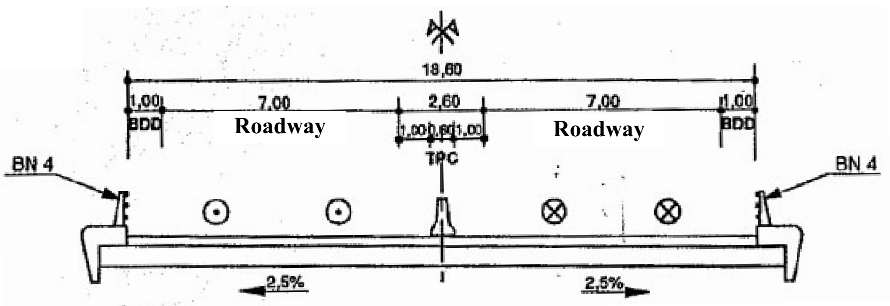


Fig. 4. *Functional transverse section*

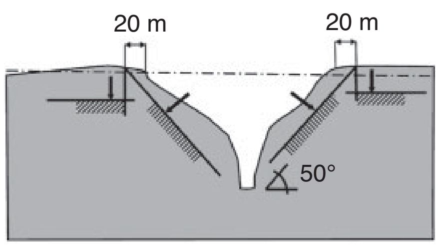


Fig. 6. *Possible support zones*

Fig. 7. View of the west and of “La Fontaine” from the Heights

one or more basalt strata that were in a position of being able to balance the horizontal component.

2.4 Climatic conditions

From December to April, the Indian Ocean is regularly subjected to tropical cyclones. These storms, which beat down in an unpredictable manner on Réunion, cause particularly violent winds, being able to reach gusts near to 300 km/h locally.

2.5 Environment

Réunion Island is the home of a very significant endemic flora and fauna of which many species are protected. In particular, certain birds such as the “puffin de Baillon” nest and reproduce in the cracks and fractures of the basalt strata. It was important to identify the nesting zones in order to preserve them during the construction phases.

3 Design of the bridge

3.1 Type of structure

The design of the structure is the result of the collaborative work between the architects and the engineers. In addition, at the site concerned, the environmental impact assumes a very large importance.

3.1.1 Environmental impact

Fig. 7 shows a view from the Heights. The general slope of the ground, sloping down at 15° towards the ocean, is clearly visible. On the other hand, the ravines appear small and “La Fontaine” ravine is only just visible in the middle distance in the centre of the photo. This perception encourages us not to emphasize the structure compared to its environment.

3.1.2 Choice of type of structure

Fig. 8 summarizes the two possible situations, as a function of the support system.

- The bridge is supported vertically on the ground. In this configuration, the minimum span (without support) of the structure is about 240 m, and
  - either the bridge is isostatic, and only a tied arch type of bridge makes it possible to cross this span,
  - or the bridge is hyperstatic, and in this case it is necessary to envisage balancing spans or counterweights if the span is suspended, which considerably increases the total length.
- A component inclined on the ground supports the bridge. Its typology is that of an arch bridge or a portal frame bridge. In this case
  - the length of the structure is strictly the one necessary for carrying out the crossing, i.e. 200 m, and
  - the span of the structural system (the arch or the portal) can even be reduced in relation to the total length, as a function of its own

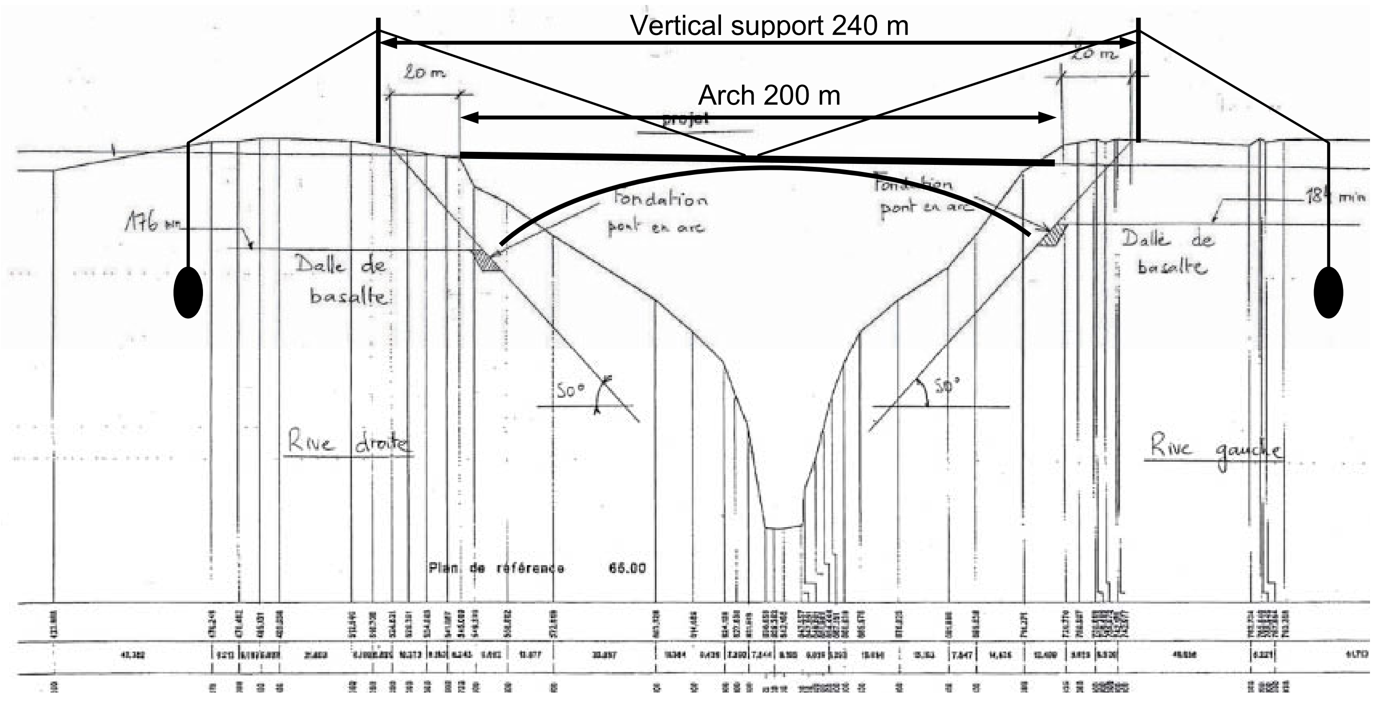


Fig. 8. Choice of type of structure

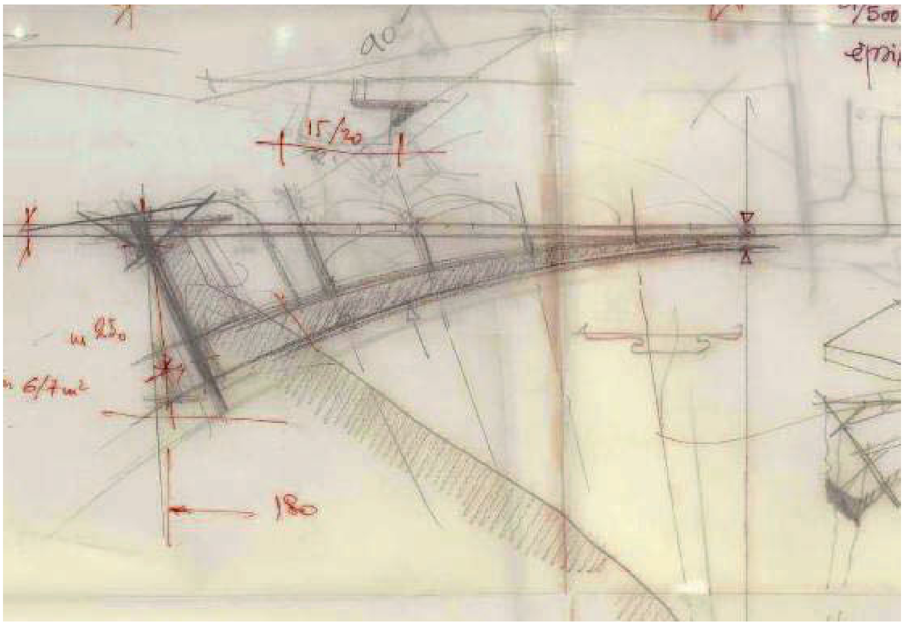


Fig. 9. Architectural principle of the elevation of the arch

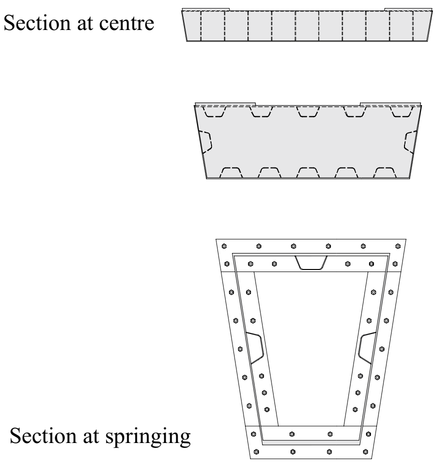


Fig. 10. Evolution of the transverse section

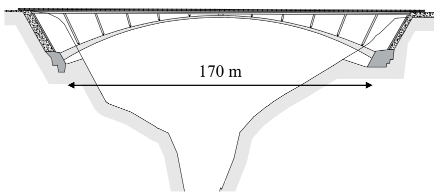


Fig. 11. Longitudinal section

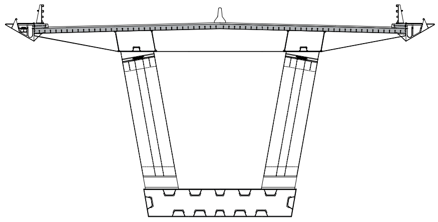


Fig. 12. Transverse section

slenderness ratio and the slopes of the cliffs.

It is obvious that if the geotechnical conditions allow the construction of a bridge of the second type (arch bridge), this solution proves from the start to be the most interesting, in terms of both landscape and technical plan, and the most economic. Within this framework, various solutions were subjected to a comparative analysis, but the form of the arch was the one finally retained, due to the purity and expressivity of its line.

3.2 Technical and architectural option

The principle of the operation of an arch bridge is such that it is mainly subjected to an almost constant compression force. It is therefore logical to design a constant section.

The design takes account of this principle. However, to benefit from the possibilities of providing fixity for the arch in its foundations and in the basalt strata, the height of the section was increased at the springing, which also made it possible to decrease it in the centre section and to introduce a large slenderness into the structure.

Transversely, the arch is mainly subjected to the forces of the wind. It behaves like a beam supported at its two ends, with the maximum bending moment at the centre. The width of its section develops proportionally to this bending moment (Figs. 9 and 10).

These principles have made it possible to fit the arch section between two inclined planes.

The geometry of the arch thus defined was used as the basis for the geometrical construction of the piers and the deck (Fig. 11).

The deck is composed of two small box girders 2 m high and 2 m wide. These two small box girders are braced every 4 m in order to support the reinforced concrete deck. They are in addition supported by the radiating piers, thus inclined. The piers as well as the box girders of the deck fit within the construction planes of the arch (Fig. 12).

3.3 The arch
3.3.1 Rise

The rise of the arch is usually guided by economic considerations. It is generally between 1/5 and 1/6 of the span. In comparison with the general case, other parameters have to be taken into account :

– The foundations must be supported on sufficiently rigid basalt strata in order to take the thrust of the arch.
– The abrupt nature of the cliffs makes the earthworks and access to the bottom of the excavation particularly awkward. It is necessary to reduce their depth as much as possible.
– Aesthetically, the economics reports led to an aspect lacking some dynamism.

These were the reasons behind the decision to decrease the slenderness ratio to 1:7.5, i.e. a rise of 22.50 m for a span of 170 m.

3.3.2 Structural composition

The arch is a steel box girder whose height varies from 5 m at the springing to 1 m in the centre. Its average width varies from 3 to 10.5 m respectively.

The four faces of the box girder consist of panels stiffened by plates. These panels were checked by means of Eurocode 3 Part 1–5 by taking account of the combined plate-column behaviour, the behaviour of the “column alone” and the mixed behaviour when the width of the panels becomes sufficiently significant (Fig. 13).

3.4 The bridge deck

The two small box girders of the deck are also of steel, just like the spacers laid out every 4 m.

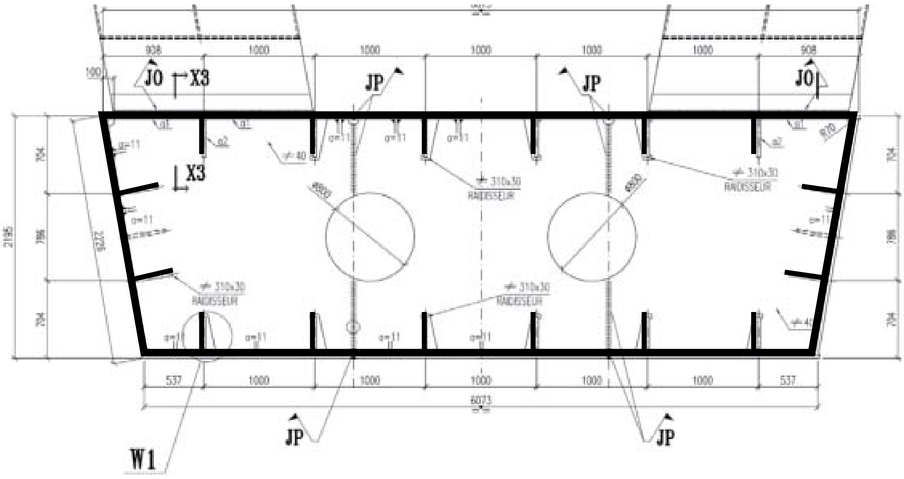


Fig. 13. Section through the arch

3.5 The piers

In pairs and positioned every 16 m, the piers transfer the loads on the deck to the arch. They are provided with hinges at both ends so that they are subjected to axial forces only. Indeed, under the effect of asymmetrical forces (a longitudinally loaded half-bridge), the arch works exclusively in bending and deforms significantly, inducing significant relative rotations, particularly at the bases of the piers. Fixity here would have imposed bending on the piers incompatible with their resistance to buckling.

Initially, elastomeric pot or spherical bearings were envisaged in order to ensure this function, but in order to limit the maintenance (inspection, replacement)

– the supports at the interface with the deck are in the form of steel roller bearings on steel (Fig. 14), and
– the supports at the interface with the arch are in the form of a welded assembly (Fig. 15).

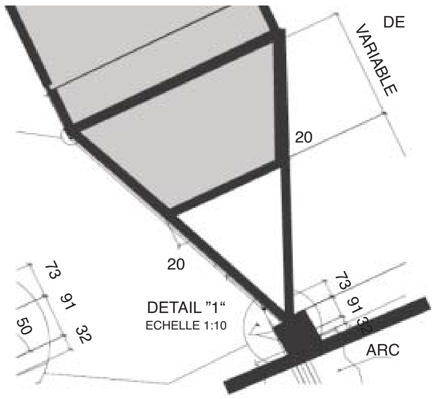


Fig. 15. Lower mechanical support at the arch

If the operation of the roller bearings is relatively obvious (bearing and rotation of one surface on the other), the operation of the mechanical support is based on the flexibility of the plates they are made of. The latter, without diaphragms, bend and absorb imposed rotations. The checks carried out relate to the inflection caused, taking account of the instability as well as fatigue.

4 The wind studies

Cyclonic phenomena are extremely frequent on Réunion. Consequently, great attention was paid to this subject from the very start of the design phases and various studies were undertaken by the "scientific and technical centre for building":

– Numerical calculations to determine the wind speed characteristics at the site of the structure characterized by the standards, in particular ENV 1991-2-4, by means of models taking account of the topography of the ravine.
– Experimental determination (by atmospheric wind tunnel tests) of the characteristics of the average winds and their turbulence.
– Wind tunnel measurement of the aerodynamic coefficients of the structure in various configurations (with/without equipment) (Fig. 16).

These preliminary studies and measurements enabled the establishment of a turbulent wind model (Fig. 17) which was then used in the dynamic calculation of the structure for the service and construction phases. Once in service, it is thus designed to resist wind damage for a return period of 50 years. During the construction phase, for a return period of 10 years.

Fig. 16. Relief scale model

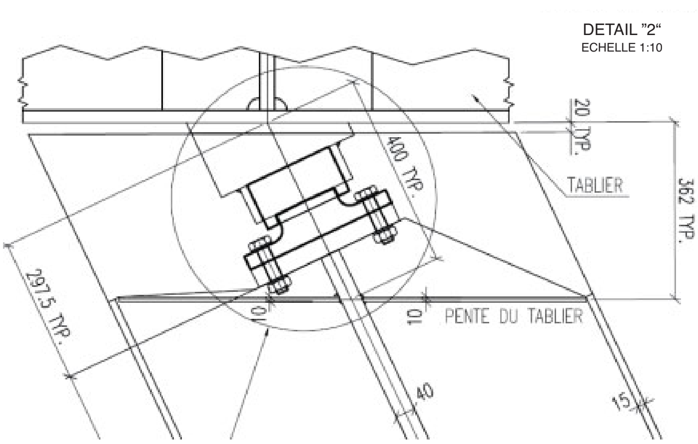


Fig. 14. Upper roller bearing (deck on pier)

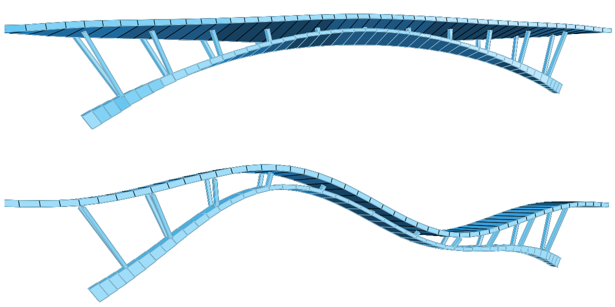


Fig. 17. Clean vibration modes

5 The construction

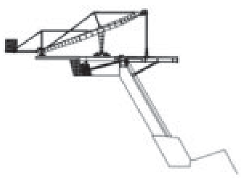
The development of the construction method for the structure was the responsibility of the contractor.

After construction of the earthworks, foundations and abutments, the assembly of the steel frame, with a total weight of 2000 t, was erected using the cantilever method (Fig. 18). Basic sections were manufactured in the Cimolai workshops in Italy before being conveyed by ship to Réunion. After assembly on site to make up elements with a maximum weight of 100 t, these were set up one after the other by means of a derrick crane – by side – built especially for this work.

Prior to keying-up, arch operation obviously cannot be assured. Consequently, the final structures are supplemented by provisional bars in order to make them work as a lattice. The arch constitutes the compressed rib of the lattice, whereas the deck constitutes the tension rib of it, anchored itself in the solid rock mass by means of strong active temporary ties (20 ties each 1150 kN per bank) making it possible to take up again not only the weight of all the structure as built, but also the weight of the derrick’s maximum cantilever during the installation of the last section. The diagonals and uprights of the lattice are made up of the temporary bars and the piers respectively.

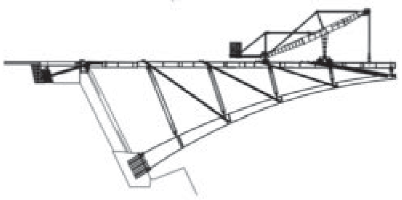
At the keying-up, the structural deformations are compensated thanks to a particular adjustment of the bars. This adjustment ensures that at the installation, the coherence of rotations allow the arch to be closed easily. After keying-up, fully releasing the ties and temporary diagonals restores the arch effect.

The construction ends with the realization of the concrete deck, the installation of the road surfacing, coatings and different equipment of the structure.

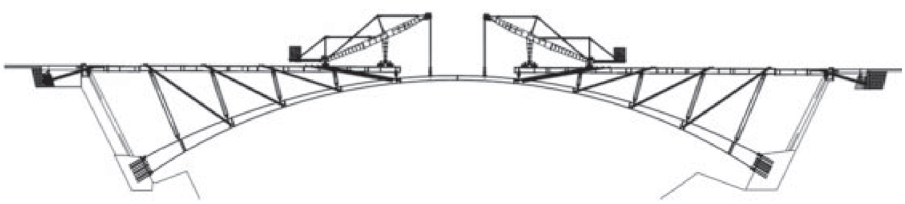


Erecting the 1st deck section

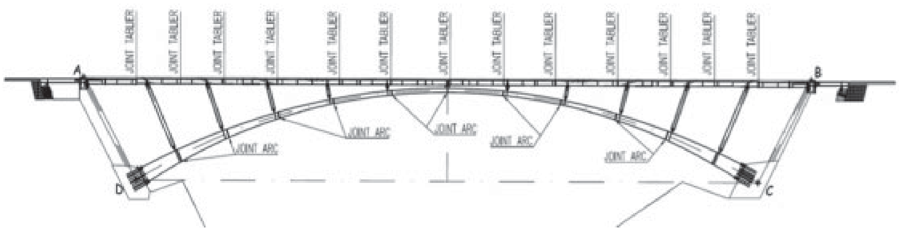
Erecting the 3rd arch section



Erecting the 6th deck section



Keying up the arch



Dismantling the temporary elements

Fig. 18. Some phases of construction

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Design of an electrified double-track railway swing bridge in Malaysia

Dedicated to Dipl. Ing. *Gerhard Seifried* on the occasion of his 70th birthday

A new swing bridge for an electrified double-track railway has been proposed in Malaysia to replace an existing 40-year-old single-track swing bridge. The proposed swing bridge has two equal cantilevers each 45 m long and features a steel "sail" on both sides as the main longitudinal structural element. The 9.1 m wide bridge deck is a composite design – steel structure with concrete slab. The swing span will be powered by means of an electrohydraulic drive situated in the centre pier and slews over the centre bearing located below the centre pivot shaft.

1 Introduction

MMC-Gamuda JV is currently constructing a new 328 km electrified double-track line from Padang Besar to Ipoh in Malaysia. The electrified railway line has been designed to accommodate the metre-gauge double track. A 282 m long crossing over the River Prai near Butterworth, which includes a movable bridge, forms part of the double-track work. Prai Swing Bridge will be completed on a design-and-build basis, and as such, MMC-Gamuda JV has appointed Leonhardt, Andrä & Partner (LAP) as its consultant for the tender design for the new movable bridge. LAP has broad experience in movable bridges such as the Shatt-al-Arab Bridge [1] designed and constructed more than 35 years ago.

The movable bridge is to be designed as a swing bridge with two equal spans of 45 m and for vessel navigation purposes opens to an angle of approx. 72°. The complete swinging operation is powered by an electrohydraulic drive supported by a centre bearing located below the centre pivot shaft in the central pier.

The new bridge will be built parallel to an existing 40-year-old bridge with a single track which includes a movable bridge of the swing type as well (Fig. 1).

2 Guidelines and design criteria

For the tender design, LAP has been advised to pay special attention to aesthetics, economy and constructability and to keep the bridge deck as narrow as possible. In addition, other aspects such as structural stability, durability, maintenance, serviceability and the navigation safety of vessels must be considered as well. Other features considered during the design stage are:

- A straight alignment on plan and a constant bridge width.
- A bridge design to accommodate two new tracks spaced 4.2 m apart.
- A design train speed of 60 km/h at the bridge.
- Provision for overhead electrification (contact wire 4.6 m above top of rail) with structurally integrated masts/gantries at intervals of approx. 15 m.
- Ballast-less track with composite sleepers fixed directly to the concrete deck.
- A closed deck and guard rails to prevent damage to structural members during possible train derailment.
- Railway loadings based on KTMB loading diagram A3 (locomotive with 12 axles each of 200 kN

Fig. 1. Existing river bridge

followed by a trailing load of 58.3 kN/m); dynamic effects considered in accordance with the requirements of BD37/01.
– Design wind speed of 35 m/s.
– Two navigational channels each with a minimum clear width of approx. 30 m required upon completion of the bridge, and during construction a single channel with a clear width of 20 m must be provided; the existing bridge will be demolished upon completion of the new bridge.
– An impact force of 5 MN due to aberrant vessels with a speed of 2 knots at piers adjacent to the navigational channels.
– Piers aligned at a skew angle (parallel to the current) to provide the least resistance against water flow.

3 Structure

3.1 Superstructure

The 45 m equally balanced cantilever spans are composed of a steel composite deck slab supported by a grillage of steel girders, two outer steel "sails" with a central opening and a central column. The deck is 9.1 m wide. Each steel sail is composed of an upper and lower chord and a deep web which is provided with stiffeners in the longitudinal and vertical directions. The web is stiffened horizontally by a series of L-shaped sections. Vertical T-shaped stiffeners every 4 m fixed to the regular cross-beams ensure transverse stiffness and provide lateral fixity for the top chord. An opening is provided in the centre of each sail for aesthetics reasons. This feature ensures a special design appearance and the uniqueness of this bridge. At the centre of the bridge there is a 1.0 × 2.0 m column (pylon) which tapers gently up to 2.4 m at the top in the longitudinal direction. The approx. 11 m high column is composed of a steel box section which is welded airtight (Fig. 3).

The deck consists of the concrete slab, cross-beams at 4.0 m intervals and two longitudinal stringers supporting the tracks. The regular cross-beams are of the I-girder type and have a depth of approx. 1.40 m to provide sufficient stiffness in the transverse direction (Fig. 4). At the top chord, shear stud connectors ensure composite action with the concrete slab. A 2 m wide central box-type cross-beam is provided at the centre of the bridge (Fig. 5). This beam is haunched transversely, vary-

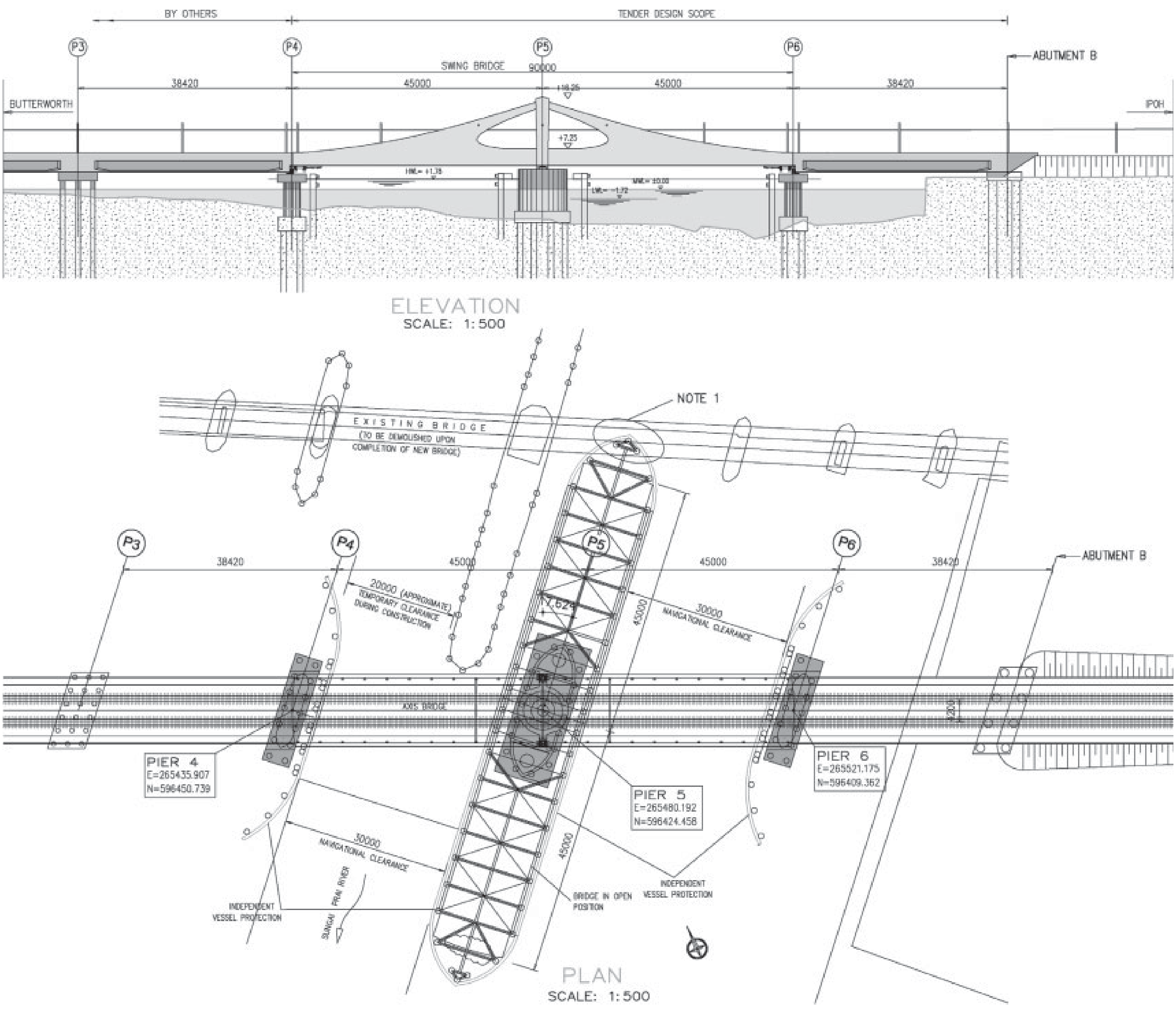

Fig. 2. General layout of new swing bridge

ing from approx. 1.4 to 2.75 m on the axis of the bridge.

During bridge slewing, this central cross-beam transfers the support reactions from the main trusses to the central pivot shaft. On the underside of the cross-beam, two steel support blocks are attached at a spacing of 3.9 m from the axis of the bridge in order to provide stable support for the bridge when it is open. The bottom of each steel support block is provided with a 30 mm thick elastomeric layer to compensate for any unevenness on top of the concrete ring wall. For maintenance and emergency operation, in case the "lift-and-turn cylinder" should fail, the bridge can be placed on the steel support blocks at any intermediate turning position and slewed back into the closed position. To do this, auxiliary jacks, each with a capacity of 2 × 200 t, are needed on each side of the steel support block. The longitudinal stringers below the tracks consist of a steel composite section. The steel I-girders are 825 mm deep. During normal service the bridge is supported on a single line bearing configuration

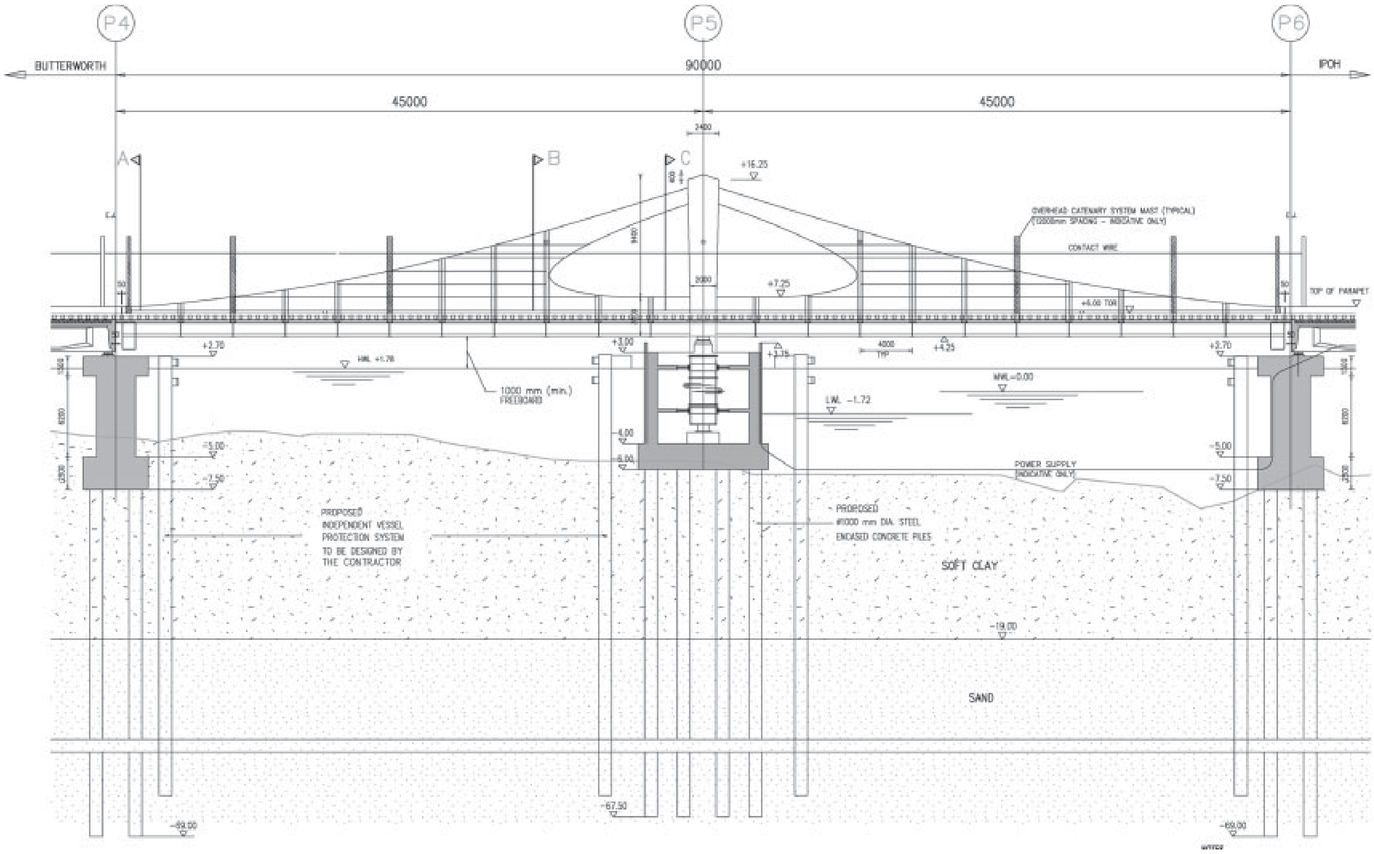

Fig. 3. Longitudinal section

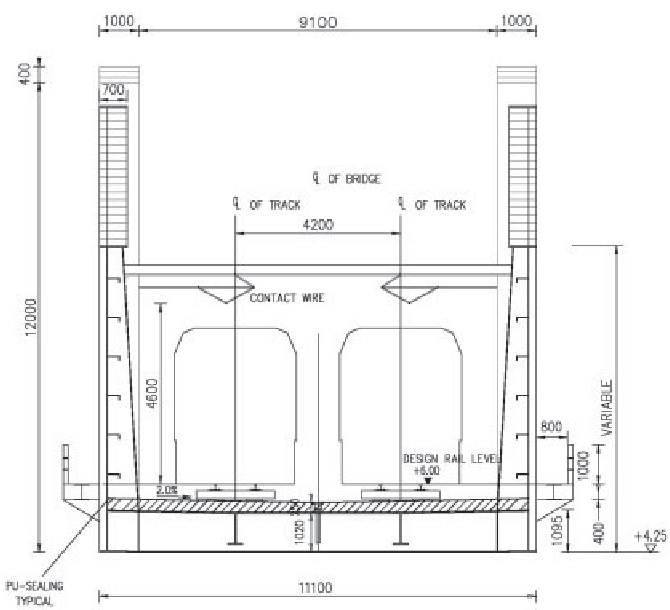

Fig. 4. Typical cross-section

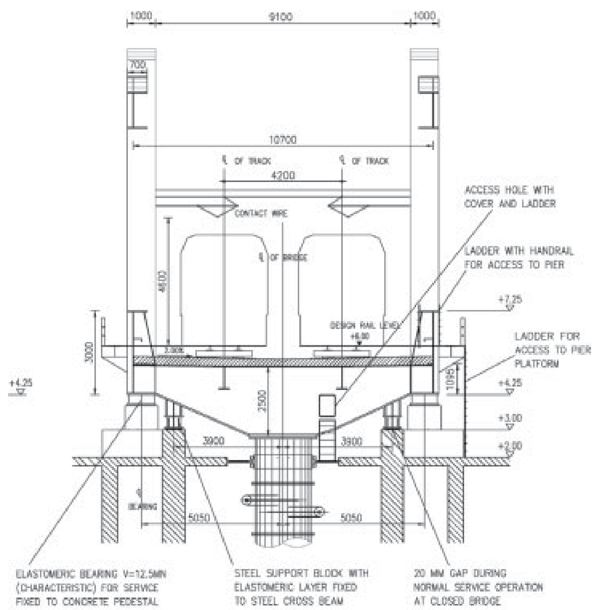

Fig. 5. Cross-section at central pier

situated on top of the central pier. The elastomeric bearings are located directly below the plane of each sail, thus enabling a direct load path for the railway traffic loads. The elastomeric bearings, each with a characteristic loading of 12.5 MN, are fixed on top of a concrete pedestal.

An end cross-beam, consisting of a steel box, is provided at the end of each cantilever (Fig. 6). The approx. 1.0 m wide end cross-beams are haunched transversely. The structural depth varies from approx. 1.4 to 2.25 m on the axis of the bridge, thus providing sufficient stiffness in the transverse direction, as required by the rail locking system. The steel structure is composed mainly of steel plates and built-up sections of steel grade 50B. The steel structure will be provided with a protective coating in accordance with EN ISO 12944. The concrete slab is generally 300 mm thick, except the inner 5 m, where the depth decreases to 250 mm on the axis of the bridge. Concrete grade C 40/20 has been chosen. The slab is supported by a grid of steel beams and connected to them with shear stud connectors.

The top of the concrete has a 2 % transverse fall; gravity drainage outlets are provided every 4.0 m on the axis of the bridge. Anchor bolts are embedded in the concrete slab for fixing the sleepers. End locks are provided at the bottom of the cantilever ends of each sail. The bottom flanges are widened locally and provided with stiffeners for fixing the end locks (Fig. 7). These end locks level and centre both ends of the bridge. Furthermore, the end locks also serve as bridge supports during normal service and hence are designed for passing trains and transverse wind, for example.

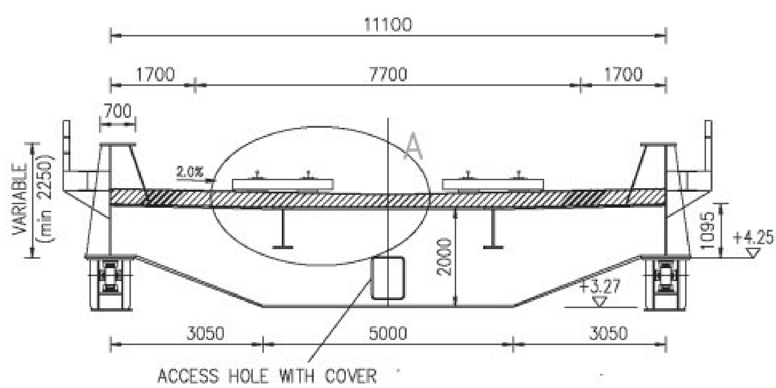

Fig. 6. Cross-section at end cross-beam

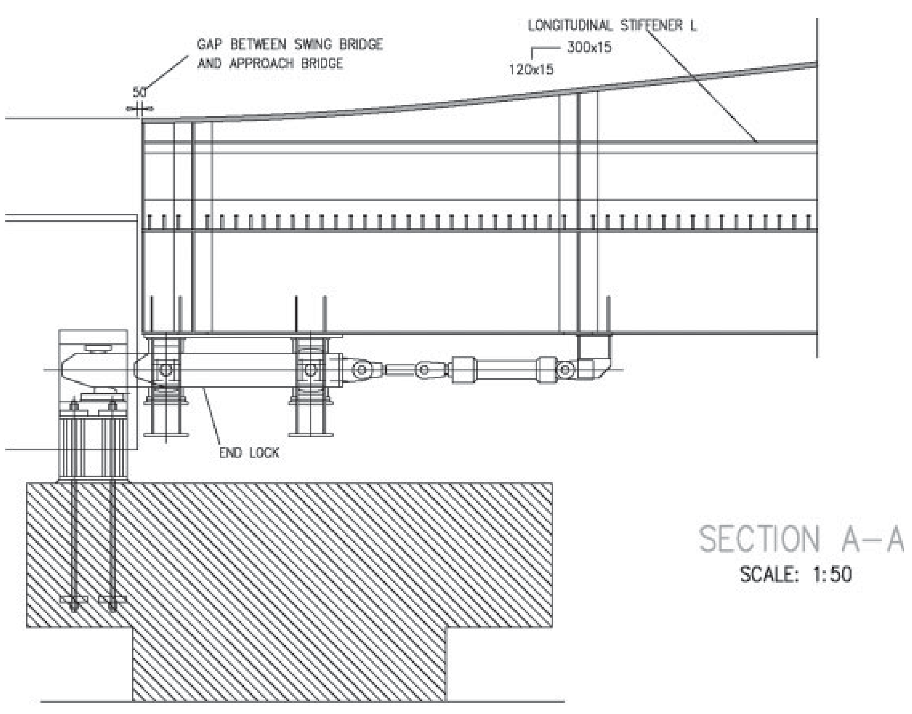

Fig. 7. End lock

3.2 Substructure

The 8 m wide central pier shaft fulfils structural demands and M&E-related space requirements, e. g. housing the machinery and equipment required. For aesthetics and hydraulic reasons, the pier shaft has a triangular shape at the front. With an overall pier shaft length of 25 m, there is sufficient space next to the bridge deck for the installation or replacement of equipment or machinery inside the pier (Figs. 8 and 9).

The central pier shaft is approx. 7.75 m high and provides a freeboard of about 2 m in relation to high water level. At the centre of the pier there is an inner ring wall with an average wall thickness of 80 cm. The top of the ring wall serves as a stable support for the bridge deck in the open position and as a sliding path for emergency situations. The concrete surface has a high degree of evenness and is exactly level in order to allow slewing of the bridge deck's steel support shoes on Teflon pads on top of the ring wall. Besides the ring wall, two 50 cm thick vertical walls are provided inside the shaft to support the upper cross-beams and the slabs and to stiffen the shaft. All inner walls and slabs include openings and ladders for access, inspection and maintenance, and there are large openings for replacing mechanical parts, equipment and materials. The concrete bottom slab (pile cap) measures 25×10 m and is 2.0 m thick. Inside the ring wall, the lift-and-turn cylinder is supported by means of a large concrete pedestal on top of the bottom slab. Piers P4 and P6 each consist of a 20×5 m pile cap, a solid 3.0 m thick shaft and the heading beam. These piers support the approach bridge viaducts as well as the swing bridge. The shaft is 14 m long and has triangular front ends similar to the central pier. Steel anchor blocks are provided on top of the 5.2 m wide skewed heading beam to support the end locks. All piers are supported by deep foundations using bored reinforced concrete piles 1.0 m in diameter. All piles are vertical and are approx. 61.5 m long.

4 Swing bridge drive system

4.1 Design features

The swing bridge has an electrohydraulic drive in the central pier for opening and closing the bridge for navigation and railway traffic. The slewing drive is powered by two electric motors and two hydraulic pumps acting on four hydraulic cylinders. It takes about 2 minutes to turn the swing span through the full 72° from open to closed or vice versa. The total time for opening (closing) including lifting (lowering) and unlocking (locking) is about 5 minutes – and this does not include the time needed for regulating the traffic.

4.2 Centre bearing

The centre pivot shaft provides the vertical and horizontal support for the swing bridge during the slewing operation only and transmits all vertical and horizontal forces due to wind and out-of-balance loads to the foundation. The centre pivot shaft is of structural steel. The hydraulic cylinders for the turning drive act on the centre pivot shaft during turning and transmit the torque due to friction, wind and inertia to the foundation structure at pier P5. For railway service the swing bridge is supported by two standard elastomeric bearings situated left and right of the centre pivot directly below the steel sails. This support system ensures that the centre pivot shaft is not loaded during railway service – in order to minimize wear and tear on the turning mechanism (low operating costs). For normal slewing purposes, the complete swing bridge – prior to unlocking the bridge ends – is lifted by approx. 50 mm by the lift-and-turn cylinder situated beneath the pivot

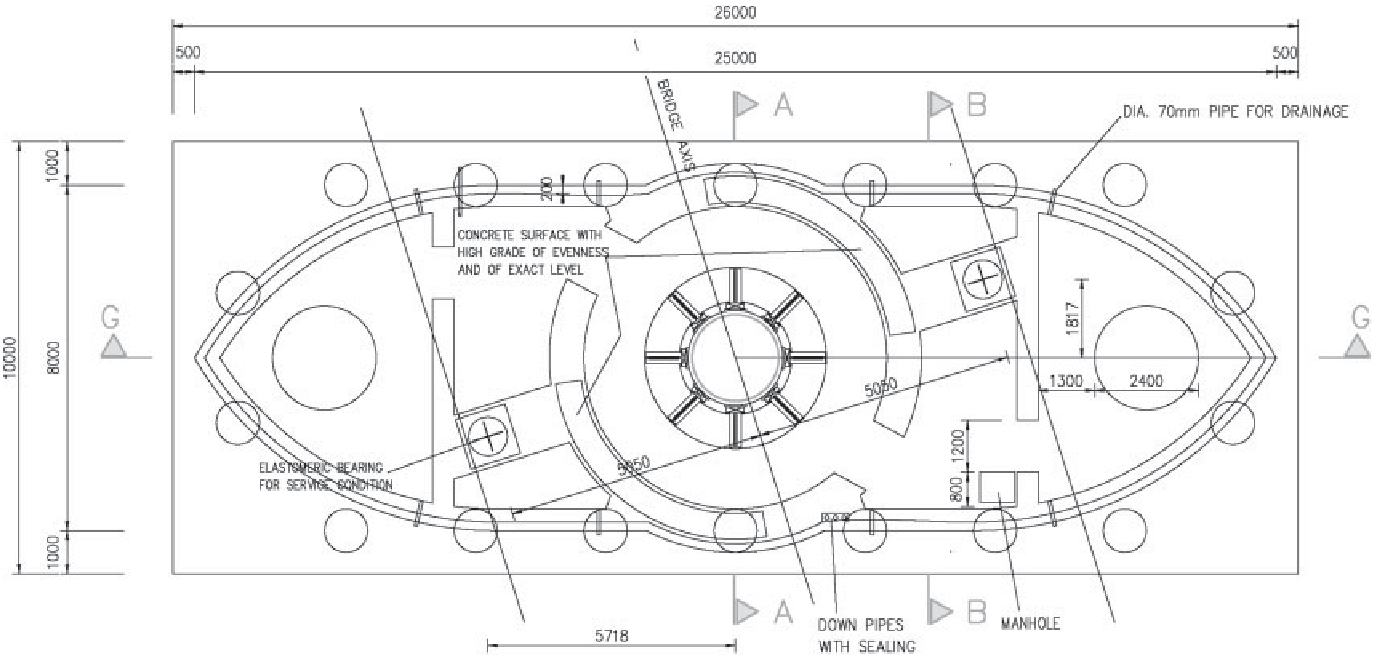

Fig. 8. Plan on central pier P5

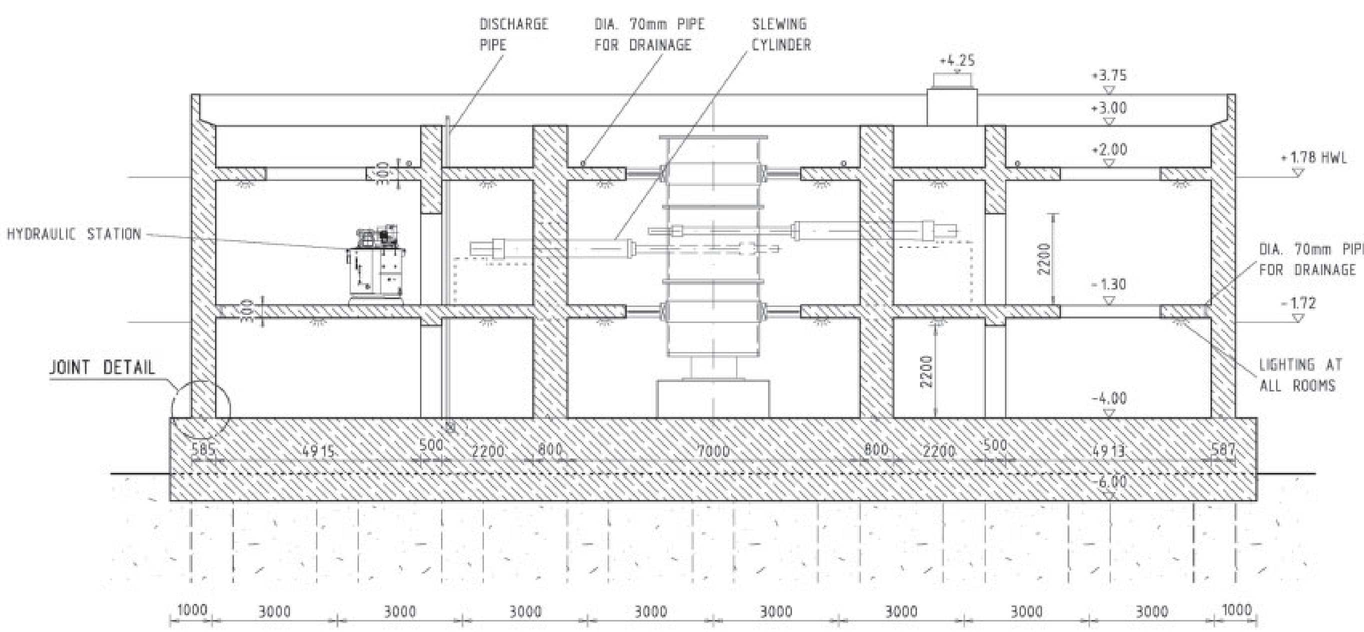

Fig. 9. Longitudinal section through central pier P5

shaft. This ensures that the friction forces at the end locks are minimized during locking and centring. During the turning operation the dead weight of the complete swing bridge is supported by the hydraulic fluid within the lift-and-turn cylinder. In order to maintain the stability of the bridge during turning, the centre pivot shaft is supported in the horizontal direction by means of guide bearings at two levels. During normal service operation, the upper guide bearings serve as horizontal supports for the bridge deck, e. g. for resisting wind loads or loads from braking and acceleration. After the bridge is turned to the open position, it is lowered onto two steel support blocks fixed to the main cross-beam. These two steel support blocks take the total vertical bridge loads, with the pivot shaft resisting any out-of-balance loads via its two horizontal guide bearing supports. No special locking device for the bridge is provided at this stage because any eccentric horizontal loadings (e. g. from wind) are resisted by the friction forces acting at the steel support blocks.

4.3 Hydraulics

The hydraulic system is located within the central pier P5 and facilitates the operation of the turning drive and the locking systems. Two motor pump units are provided for opening and closing the bridge and for operating the locking system. In an emergency (e. g. failure of the power supply or the electronic control) the swing bridge may be operated at a reduced speed by means of separate diesel-powered emergency pump units.

4.4 Turning machinery

Turning of the bridge is carried out with four double-acting hydraulic slewing cylinders which act with sufficient torque on the centre pivot shaft. The hydraulic slewing cylinders are connected to the pier by means of concrete pedestals. Lifting and turning simultaneously is not envisaged, although it is possible. Spherical bearings are attached to the ends of the turning cylinder (eye-bar) which allow for an oblique angle of the cylinder axis. After completion of each turning cycle, no special stopping or buffer device is required. The slewing motion is controlled precisely and stopped by means of the large slewing cylinders. The capacity of the centring lock is adequate to centre the swing span precisely at the closed bridge position.

4.5 Locking system

In order to achieve proper levelling and horizontal adjustment of the bridge deck for railway service, a locking system is provided which consists of four end locks, one at each deck corner, and four rail locks, one at each rail track end.

The end lock levels and centers both bridge ends. The lock capacity is adequate for lifting the bridge cantilevers by up to 80 mm. This is necessary to level the bridge deck precisely with the approach bridges after a slewing cycle is completed. The lock is wedge-shaped (both vertically and horizontally) for bridge lifting and is supported by sliding bearings situated within the steel anchor block. The locks are operated by means of hydraulic cylinders.

To allow slewing of the bridge, a small gap between the UIC rails needs to be provided at each end of the bridge. Therefore, rail locks are provided at both bridge ends to ensure rail continuity. The rail locks are designed to allow for expansion of the rails and minor misalignment of the bridge ends due to temperature effects. The rail locks are operated by means of hydraulic cylinders.

5 Protection against vessel impact

The purpose of protective works is to protect the bridge superstructure in its open position and its supporting piers from damage which may be caused by accidental vessel collision. Such protective works are designed to eliminate or reduce the impact forces to non-destructive levels by either redirecting the vessel or by dissipating and absorbing the vessel energy. In accordance with the AASHTO guidelines for vessel collision design, the piers, which house mechanical equipment or support movable machinery, should be fully protected from vessel contact by aberrant vessels. There should be no contact between the vessel and the pier or the bridge deck when the protection system is in its fully deformed position. Special consideration must be given to the overhang of the raked bows common among ships and barges.

For the tender design, a pile-supported fender system is indicated only in principle; the final solution will be developed by the design-and-build contractor. The fender system is independent of the piers and, in addition, serves for vessel guidance purposes. This system is a common means of protection in shipping channels with non-excessive water depth and is effective in low-energy vessel collision events where severe impacts are not anticipated. The pile-supported fender system indicated is composed of steel piles with horizontal walings and bracing to ensure that the fender system acts as one unit with increased stiffness. The horizontal walings will be equipped with an anti-sparking rubbing surface.

6 Final remark

The Malaysian contractor Ikhmas Jaya Sdn. Bhd. has been selected by MMC-Gamuda JV to design and build the swing bridge. Waagner-Biro from Austria is the supplier for M&E works. MMC-Gamuda JV appointed Leonhardt, Andrä & Partner to check the detail design. Start of the pile foundation is expected by April 2011.

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Keywords: railway; swing bridge; steel composite; steel sail structure; centre pivot bearing; end locks; hydraulic slewing cylinders; deep foundation; protection system

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Linz–Urfahr railway and road bridge – a 110-year-old structure severely damaged by corrosion

At the very end of the 1890s, the rural district of Urfahr was connected to the town of Linz, Austria, by means of a seven-span, 393.94 m long steel bridge. Serious problems regarding its durability arose from the use of the bridge for road traffic due to the application of de-icing salt, which began in the mid-1960s and caused severe corrosion damage. In 1981 the bridge was improved by restoring the anti-corrosion protection and by replacing the old deck construction with a new one. Since that time, restoration works and repairs have been carried out continuously. This paper deals with the present reliability of the main spans with a special focus on the structural behaviour under traffic and wind loads, quantified by means of modern standards.

1 The original bridge (1900)

Linz Railway Bridge, as it is called nowadays, was originally designed for railway and road traffic (carts, pedestrians, etc.). The bridge was completed in 1900. Fig. 1 shows the main spans crossing the River Danube. Without any doubt, with its "transparent" and "fancy" construction it must be considered a beautiful relic of a bygone age.

Fig. 1. Main spans crossing the River Danube

Later on we will see that the use of the railway bridge for road traffic is the main cause of the present, very poor condition of the bridge. From the mid-1960s onwards, de-icing salt was used to guarantee safe use of the bridge in winter, although this caused severe corrosion damage in the lower parts of the structure. At the beginning of the 1980s the situation was such that a comprehensive upgrading, even for a temporary use, had become necessary.

Fig. 2 shows the general arrangement of the seven spans of the bridge.

One of the three quasi identical main spans crossing the Danube is considered in this paper (see Figs. 3 and 4). Fig. 5 provides the reader with an idea of the construction of the main trusses.

The original structural analysis of this rather complicated structure is very simple: the normal forces in the main girders and bracing members were simply determined by graphical statics, and the deck construction was not analysed at all – the dimensions may have been chosen according to bibliographical references such as [1]–[9] and/or by experience. As was the rule at the time of the construction of the bridge, the deck was not considered to interact

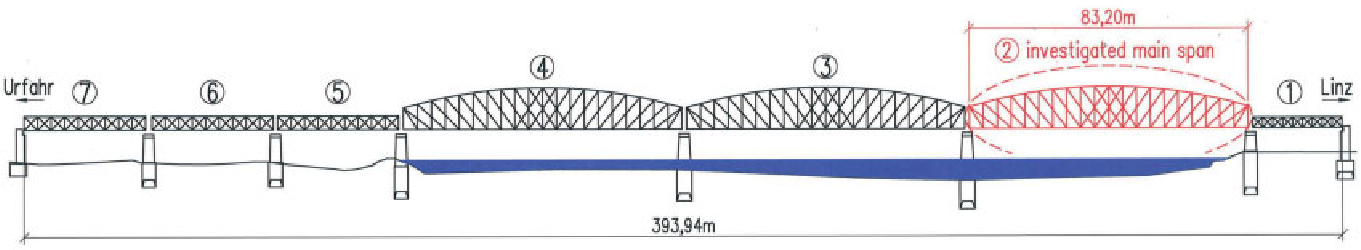

Fig. 2. General arrangement of the bridge

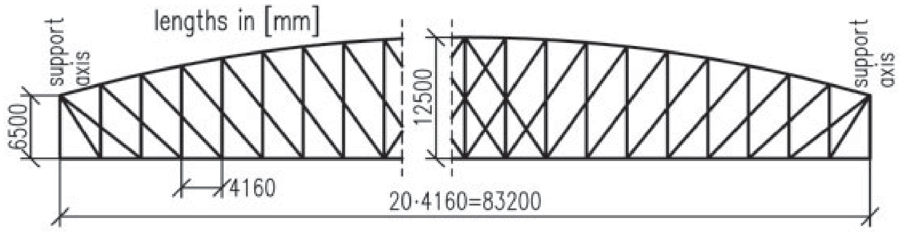

Fig. 3. Dimensions of a main span

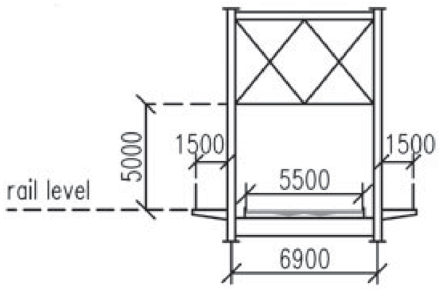

Fig. 4. Main span cross-section

with the main girders nor were the cross-beams considered to be restrained by the struts and posts of the main girders.

According to publications at the time of erection ([1]–[7]), the yield strength of the mild steels used in bridge construction amounted to 240–290 N/mm^2 and the permissible strength is given as 120 N/mm^2. Tests on the original material were performed in 1993 and these revealed a yield strength of 215 N/mm^2, which is very close to the value of 220 N/mm^2 according to [18], and a very high degree of ductility. Fatigue tests on specimens with good to medium surface conditions yielded a fatigue strength $\Delta\sigma$ of about 130 N/mm^2 = 0.4 R_2 for $2 \cdot 10^6$ load cycles (R_2 is the ultimate strength) which is standard for unnotched specimens. The insensitivity to notching is also confirmed by the fact that no cracks were detected in any of the inspections.

Chemical analyses on the unkilled material yielded a very low concentration of carbon (C), but a high concentration of phosphor (P) and sulphur (S) compounds, rendering the construction unsuitable for welding.

2 The situation by the end of the 1970s

Fig. 6 shows the deck of the original construction.

Due to the extensive use of deicing salt from the mid-1960s onwards, the condition of the bridge had deteriorated to an alarming degree by the end of the 1970s, i.e. over a period of some 15 years. The most severe damage had occurred in the deck construction, the lower parts of the main girders and the lower bracing members. For safety reasons and to prolong the life of the bridge it was decided to improve the most severely damaged part of the structure. The related works were performed in 1981 and 1982.

Because of the age of the bridge (80 years at that time) and its extremely poor condition, the whole deck was replaced by a new one and some of the badly damaged elements of the main girders were strengthened by additional plates and angle sections in the most affected areas. A detail of the new deck construction is given in Fig. 7. In addition, in 1982 a new corrosion protection system with a min. 15-year service life (which would last until about 1997) was ap-

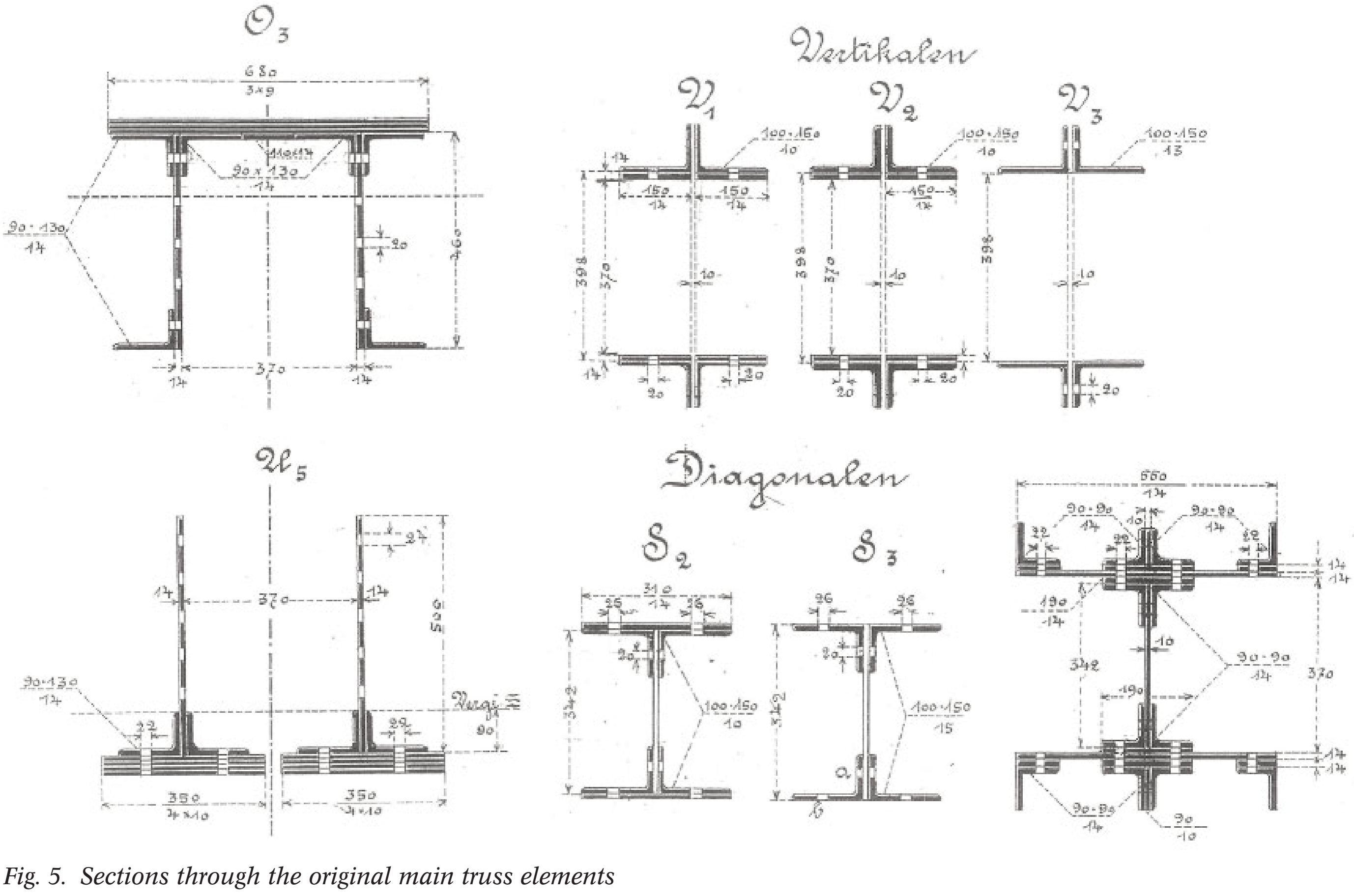

Fig. 5. Sections through the original main truss elements

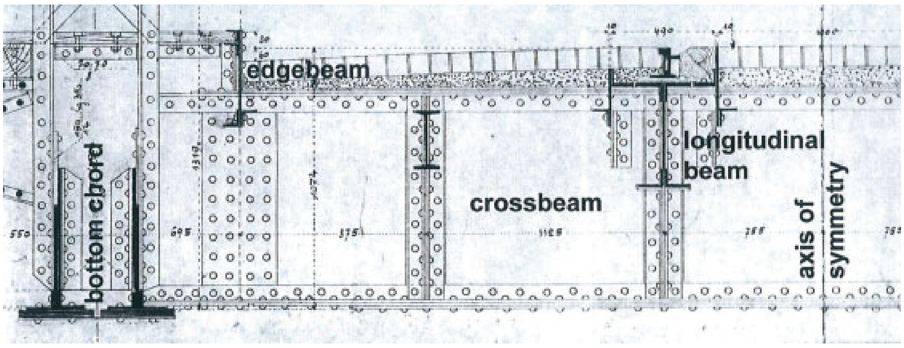

Fig. 6. Detail of deck of original construction

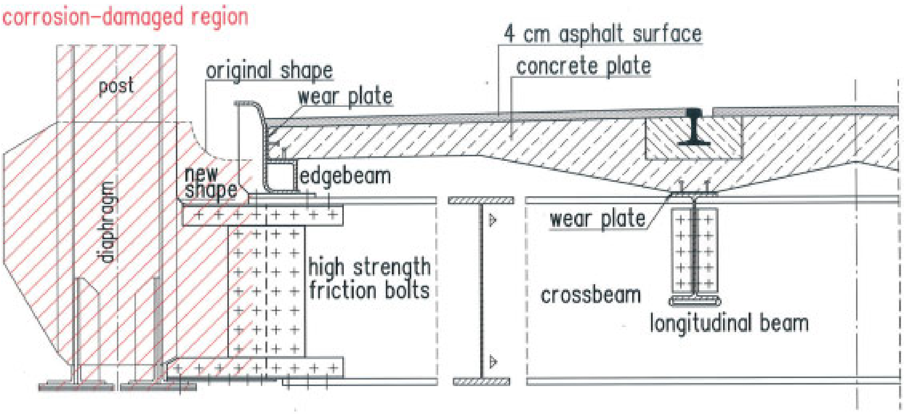

Fig. 7. Detail of the deck installed in 1981–1982

plied. The protective paint systems on the lower part of the bridge meet the medium-term protection required for severe corrosive attack in urban areas.

3 The situation by the mid-1990s

The aforementioned repair and upgrading works performed during 1981 and 1982 had been designed for a period of some 15 years, say up to 1995–2000. But owing to the expiry of this period and the evident and rapid progress of corrosion damage, comprehensive checks were performed as early as 1993. These involved materials tests and a qualification of the overall condition of the bridge (date of report: 2003).

4 The situation by 2008 and the current investigations

Continuous maintenance and repair work has been performed since 1981; Fig. 8 shows an example.

As, however, the corrosive attack had been neither eliminated nor mitigated, the corrosion-induced damage had proceeded and reached a degree that was definitely worse than only a few years earlier and which now is indeed critical. The current risk evaluation and the definition of the expiry date were performed as follows:

- Checking the damaged areas and quantifying the loss in cross-section and of the loadbearing capacity of the sections.
- Using analysis codes and guidelines for quantifying the load-carrying capacity of existing bridges ([10]–[19]), problem-related load assumptions and definition of the ULS and SLS as agreed with the client (ÖBB, Austrian Federal Railways).
- Problem-related structural analyses considering the structure as built and allowing for damaged sections; plastic models were considered where applicable.
- Defined activation of the concrete plate to lend a defined transverse stiffness to the deck instead of the lower bracing which is badly damaged in many regions (see Fig. 9); this is achieved by means of blocking devices installed to enable the end regions of the bracing (see Fig. 11 and section 5).
- Continuous monitoring works.

Fig. 8. Upgraded area within a strut

In an inspection performed in the years 2009–2010, major damage was detected at levels below approx. 2.5–3.0 m above top of rail, comprising the following elements: lower bracing (angle sections, gusset plates and some rivets; see Fig. 9), main girder bottom chords, posts and diagonal struts of main girders (see Fig. 10), intersection and nodal points of the original structure, and even elements of the "new" girder grid of the deck installed in 1981.

Flat material loss over a large area and – more critical in terms of reliability – deep grooves locally were detected in numerous instances with local material loss rates of up to 30 % of the original thicknesses. A report was prepared and taken as the basis for the calculation of the load factors for the structure in the current condi-

Fig. 9. Severely damaged gusset plate in lower bracing

Fig. 10. Corroded area within a strut

tion. Due to the atmospheric conditions, an advancing loss of material in the range of 0.1–0.3 mm per year is to be assumed.

To enhance the transverse stiffness in the bottom part of the structure, the end bays of the lower bracing have been activated by simple blocking devices welded to the related cross-beams (July 2010; see Fig. 11).

The purpose of this simple measure is to provide a clear and stiff support to the concrete plate in the end zones. This is important because the lower bracing is badly damaged in many regions (see Fig. 9) and therefore nearly the full transverse stiffness in the deck area is provided solely by the concrete plate.

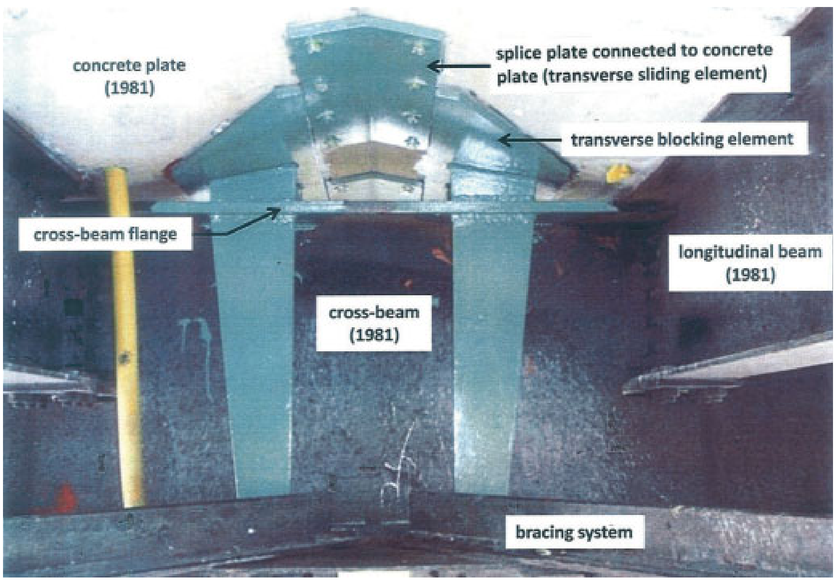

Fig. 11. Blocking device to enable the end regions of the lower bracing

5 Resisting horizontal forces

5.1 The structural system of the deck

The deck construction installed in 1981–1982 consists of a steel girder grid and a concrete plate placed on this. The wear plates to which the concrete plate is connected by small studs enable the plate to slide (see Fig. 7).

The grid itself is a Vierendeel girder and does not have much shear stiffness. Due to the intentional sliding of the concrete plate on the steel grid, the former does not actually work as a shear diaphragm. The horizontal stiffness of the deck construction is derived more from the interaction of the concrete plate with the steel structure by means of interface forces. These forces occur between the concrete plate and the edge beams and are transferred mainly by stiffeners located on the cross-beam axes (see Fig. 12). The maximum load that can be accommodated by the concrete plate/edge beam detail – welded and bolted joints between edge beams and cross-beams – amounts to 120 kN.

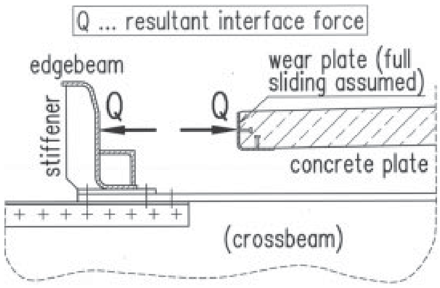

Fig. 12. Detail showing the absorption of interface forces (exploded view)

5.2 Resisting wind forces

Fig. 13 shows the deformations occurring on the upper bracing and on deck level due to wind actions and the correlated interface forces between the concrete plate and the edge beams.

The consideration of the interface forces occurring between the concrete plate and the edge beams to enable the transverse stiffness of the plate had not been included in the structural analysis performed in 1981 (see section 2). In combination with the blocking devices shown in Fig. 11, the concrete plate provides the deck with the required transverse stiffness instead of the lower bracing which is badly damaged in many places. The blocking devices shown in Fig. 11 act as quasi fixed supports for the concrete plate, transmitting the resultant forces occurring to the undamaged end bays of the bracing.

When the transverse deformations at deck level do not correspond to the deformations of the upper bracing, the posts and struts of the main girders are deformed transverse to the main girder planes and therefore are subjected to lateral forces and bending moments (see Fig. 13). The lateral forces are absorbed by the upper bracing which is supported by the end frames. Recent analyses have revealed that the actual degree of horizontal stiffening by means of the concrete plate depends on both the transferable contact forces between the plate edges and the edge beams and on the three-dimensional load-carrying effect of the structure as a whole (see section 9). The three-dimensional load transfer mechanism results in an embedding of the deck by the large number of posts and struts of the main girders of the main trusses. Due to this embedding effect, a considerable portion of the horizontal load acting in the concrete plate is transferred to the upper bracing. The three-dimensional transfer of lateral loads will be clarified in detail by the numeric results of global analyses.

6 The structural analyses

6.1 Design codes and guidelines

Modern codes and guidelines were used for the structural analyses, among them Eurocodes EN 1990, EN 1991,

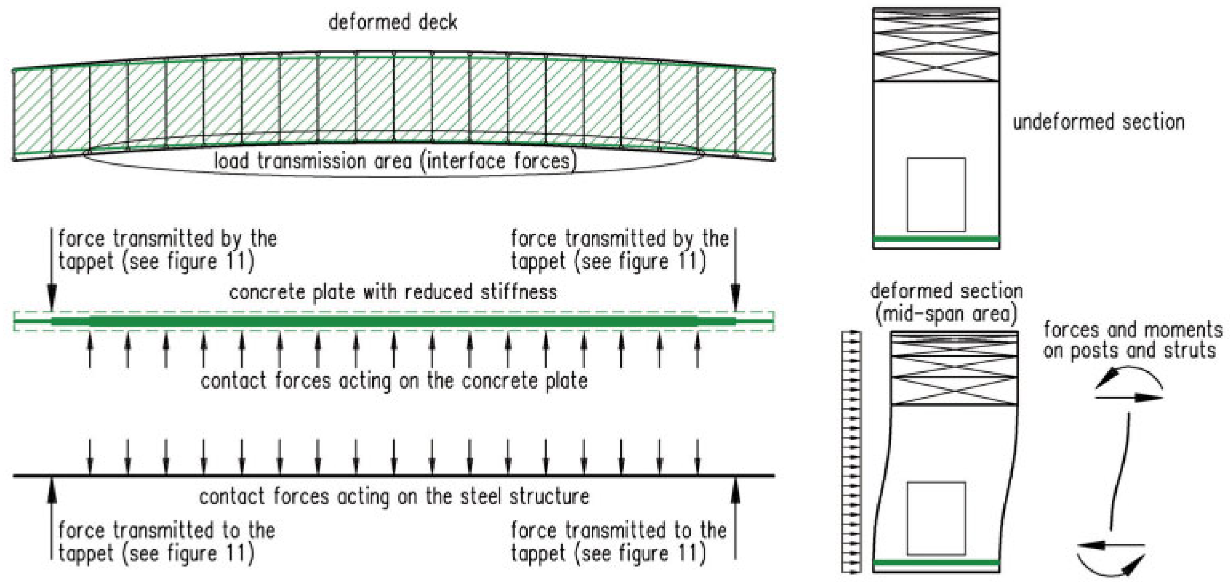

Fig. 13. Deformations due to wind actions

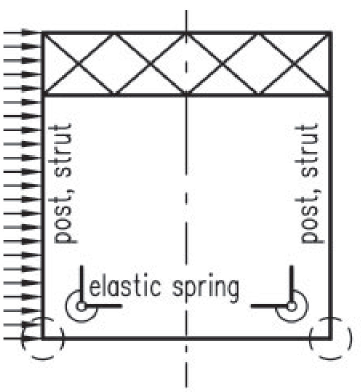

Fig. 14. Springs at the intersections between cross-beams and main truss bottom chords

EN 1992, EN 1993 ([10]–[17]), and Austrian guideline ONR 24008 [18] which lays down rules for existing bridges. The latter specifies partial safety factors γ_2, γ_2 and γ_2 and combination factors ψ differing from those given in the EN codes.

6.2 Load assumptions, load combinations and limit states

Vertical and horizontal traffic loads and wind actions were taken into account as well as self-weight and permanent loads. To form the appropriate load combinations, the load model UIC 71 and series of vehicles C2 (the latter according to [19] and featuring an average line load of $q = 64$ kN/m) were considered as an alternative.

The effect of wind was defined in two ways:

1. According to EN 1991-1-4 in combination with the Austrian National Document ÖNORM B 1991-1-4 [16]
2. According to measurements recorded in the period from 1998 to 2008

The former and the latter values differ considerably, with the non-directional gust speed calculated from the EN codes exceeding the highest recorded value by 66 %. Tab. 1 contains wind speeds, wind pressures and angles between the direction of the wind speed vector and the longitudinal axis of the bridge. The effective values of the wind speeds are:

$$v_{effective} = (v_{basic} + \Delta v)\cos\alpha \qquad (1)$$

where $\Delta v = 1.0$ m/s.

An expert report basically confirmed that the wind speed given in ÖNORM occurs with a probability of 0.02 (one event every 50 years). Therefore, and as the remaining service life is limited to the end of 2012, the following strategy was adopted for the structural analyses:

1. The maximum gust pressure occurring during hurricane "Kyrill" was considered as a transitory event. ULS and SLS checks were performed for combinations of all aforementioned actions, and with partial safety factors γ and combination factors ψ as given in [18], but with $\psi = 1.0$ for wind action.
2. The gust pressure according to [13] was considered as an extraordinary event, possible though unlikely to occur. Only ULS checks were performed, where no other actions except permanent loads and wind were taken into account with partial safety factors $\gamma_2 = \gamma_2 = 1.0$.

Several levels of wind action were analysed. In these, the full value "100 % wind" refers to [13] and corresponds to a gust speed of 148 km/h.

SLS combinations for the verification of the stress regime are not cov-

Table 1. Wind data

		Hurricane "Kyrill" (19 Jan 2007)		
Value	ÖNORM B 1991 1 4	Basic value (non-directional)	Angle α	Effective value (lateral to bridge axis)
Mean wind speed	38.6 m/s	24.0 m/s	40.7°	18.9 m/s
Gust speed	41.4 m/s	25.0 m/s		19.7 m/s
Mean wind pressure	0.93 kN/m^2	0.41 kN/m^2		0.26 kN/m^2
Gust pressure	1.07 kN/m^2	0.45 kN/m^2		0.28 kN/m^2

ered in [18] and therefore were defined by means of a specific interpretation of [10], [11] and [18]. As mentioned, no SLS checks were performed for the aforementioned "extraordinary" load combination with 100 % wind.

The original material had proved to be insensitive to notching and so because of the short period of remaining service life and the very low level of fatigue loadings, no fatigue safety assessment was considered necessary.

6.3 Allowance for plasticity

Although the structural analysis was basically performed according to the theory of elasticity, a plastic approach was used in the following cases:

As far as rigid connections are assumed between the cross-beams and the main girder bottom chords, considerable bending moments result at the joints due to restraint under vertical loads acting on the concrete plate as well as due to wind loads. For this reason, springs were incorporated at the joints with such stiffness magnitudes as would lower these moments to acceptable values (see Fig. 14).

For a further lowering of the aforementioned calculated restraint moments, an additional structural model was analysed, in which hinged connections were assumed for vertical loads acting on the cross-beams (see Fig. 15).

Limiting the forces occurring between the concrete plate to 120 kN (see section 5.1) allows for the principles of plasticity.

For the ULS global analysis, plastic deformations were considered both at the joints between the cross-beams of the deck and the diaphragms incorporated in the main girder bottom chords (Figs. 14 and 15) and in the contact areas between the concrete plate and the edge beams (Fig. 12). For the SLS global analysis, plastic deformations were allowed only in the contact areas between the deck concrete plate and the edge beams (Fig. 13).

6.4 Stress analysis

Basically, the stresses were calculated assuming elastic sectional resistances. For the ULS stress analysis, bending moments that result only from compatibility conditions, and therefore are not necessary to satisfy the conditions of equilibrium, can be neglected (see [18]) as long as the stress regimes in all other regions of the structure do not change significantly or remain on the safe side. (This requirement was confirmed by overall plastic analyses of the structure under some well-chosen, controlling, load combinations.)

When performing an SLS stress analysis, all stress resultants have to be considered, in particular to quantify the stress regimes of cross-sections damaged by corrosive attack.

7 Modelling

Owing to the size of the structure, built up from thousands of single elements, the large number of load combinations to be considered and the need to achieve comprehensive understanding of the structural behaviour, the analysis by way of a beam model was preferred to an analysis with an FE model with shell elements. For a consistent calculation of the internal forces and of the actual stress regime, all eccentricities that occur were incorporated into a three-dimensional beam model (see Fig. 16).

The laced post and strut members were modelled as single elements, allowing for shear deformations. Owing to the very poor condition of a number of elements and the even worse condition of some gusset plates, the lower bracing was not considered at all, an assumption that lies on the safe side. It can be assumed that riveted joints have a higher stiffness than welded ones, which are generally considered as rigid. Thus, the bending moments due to restrained deformation at the nodes are overestimated and the results are again on the safe side.

The modelling of the deck plate comprises an ideal beam with transverse bending and shear stiffness, whereas the remaining cross-sectional values are assumed to be equal to zero (see Fig. 17).

Basically, the internal forces were calculated according to the theory of elasticity, although a plastic approach was utilized in some cases as described in section 5 (ULS and SLS) and section 6.3 (ULS only).

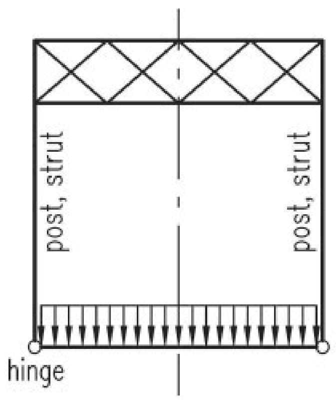

Fig. 15. Hinges at the intersections between cross-beams and main truss bottom chords

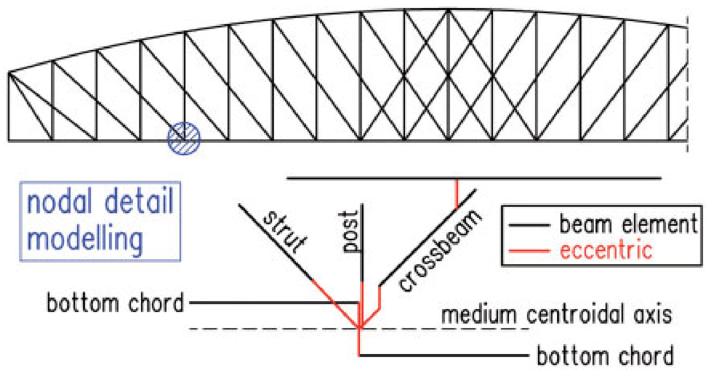

Fig. 16. Global modelling

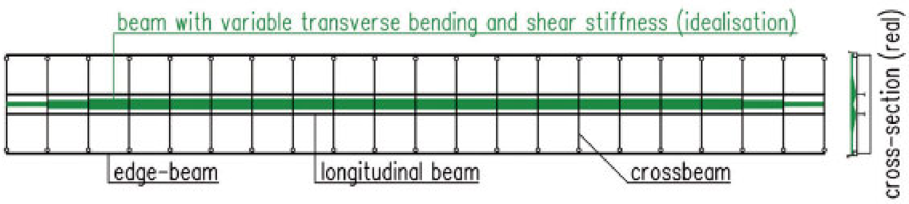

Fig. 17. Modelling of the concrete plate

8 Analysis results and verifications

Tab. 2 gives the load factors η > 1.0 occurring in the elements of the main girders (bottom chords, posts and struts) for regular (permanent und transitory) load combinations at ULS. The verification requires η ≤ 1.0.

Values in brackets occur locally at the ends of the bottom chords. The values increase disproportionally as the wind speed increases. If in the future the deck construction should be substituted by a new one, new conditions would arise.

For extraordinary load combinations comprising 100 % wind actions according to ÖNORM EN 1991-1-4, the maximum load factors are 0.59 for undamaged and 0.79 for damaged cross-sections.

9 Interpretation of the horizontal load transfer

For a comprehensive understanding of the transfer of the wind-induced forces, several models were analysed and the wind actions were assumed to act individually on single element groups of the structure and on the traffic band. Fig. 18 gives an example of the forces occurring in the system as given and wind loads acting on the structure and on a 4.0 m high traffic band over the full length of the bridge.

Tab. 3 lists the total amounts and individual portions of lateral loads in relation to the full values for a characteristic case.

It can be seen that the concrete slab – a stiff element, let us say – within the deck construction generally does attract loads to the correlated region comprising the concrete slab itself, the longitudinal beams and the edge beams, and the main girder bottom chords, thus relieving the upper regions (yellow). On the other hand, if there is no concrete slab stiffening the deck, not only the upper bracing is much more highly loaded (green) but, at the same time, the loads on the steel elements of the deck are lower than if there was a slab. This could be surprising at first sight since a – possibly substantial – enhancement of the stress regime in the deck elements might be expected, but this can be clarified by observing the three-dimensional integration of the deck within the structure as a whole: due to

Table 2. Load factors η for ULS

Traffic load	Wind: none	Wind: 75 km/h	Wind: 105 km/h	Wind: 148 km/h
UNDAMAGED SECTIONS				
(none)	0.34	0.36	0.37	0.41 (0.44)
LM71 / α = 0.83	0.73	0.74	0.76	0.79 (0.85)
LM71 / α = 1.00	0.80	0.82	0.83	0.87 (0.88)
LM71 / α = 1.21	0.90	0.92	0.93	0.97
C2	0.66	0.68	0.70	0.74
5022	0.45	0.47	0.49	0.53 (0.65)
5047	0.43	0.44	0.46	0.50 (0.59)
Road traffic	0.47	0.48	0.49	0.51
DAMAGED SECTIONS				
Traffic load	Wind: none	Wind: 75 km/h	Wind: 105 km/h	Wind: 148 km/h
(none)	0.47	0.48	0.50	0.54 (0.71)
LM71 / α = 0.83	1.01	1.02	1.03	1.07 (1.39)
LM71 / α = 1.00	1.12	1.13	1.14	1.18 (1.42)
LM71 / α = 1.21	1.26	1.27	1.28	1.31 (1.49)
C2	0.89	0.91	0.92	0.96 (1.16)
5022	0.62	0.63	0.65 (0.67)	0.69 (1.06)
5047	0.59	0.60	0.61	0.65 (0.96)
Road traffic	0.65	0.66	0.67	0.69 (0.83)

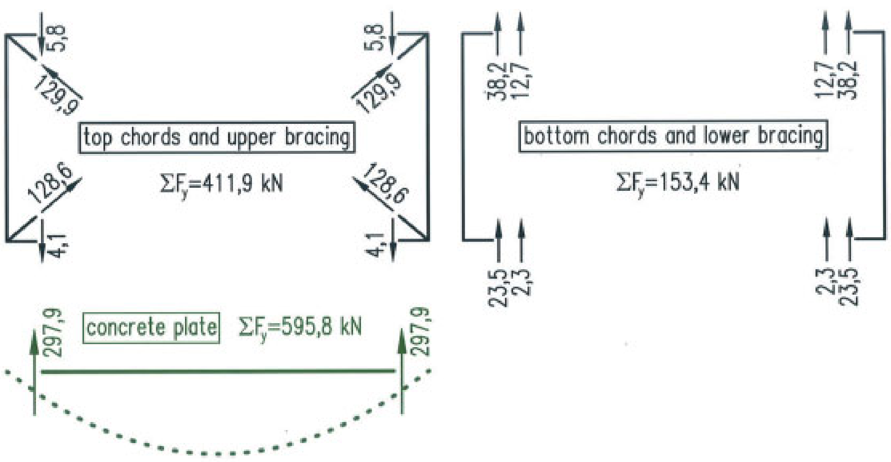

Fig. 18. Absorption of wind forces (deck construction comprising the concrete plate)

Table 3. Absorption of wind loads by the structure and by a 4.0 m high traffic band

Wind on …	Upper bracing	Lower regions	TOTAL
WITH CONCRETE PLATE			
top chords	69 kN	47 kN	116 kN
posts + struts	102 kN	141 kN	243 kN
deck region	62 kN	161 kN	223 kN
traffic band	179 kN	400 kN	579 kN
TOTAL	412 kN	749 kN	1161 kN
WITHOUT CONCRETE PLATE			
Wind on …	Upper bracing	Lower regions	TOTAL
top chords	102 kN	32 kN	134 kN
posts + struts	172 kN	80 kN	252 kN
deck region	125 kN	86 kN	211 kN
traffic band	343 kN	221 kN	564 kN
TOTAL	742 kN	419 kN	1161 kN

Table 4. Absorption of wind loads (comparison of four cases)

Concrete plate	Traffic band	Upper bracing	Lower regions	TOTAL
yes	no	39.2 %	60.8 %	100 %
yes	yes	35.5 %	64.5 %	100 %
no	no	66.4 %	33.6 %	100 %
no	yes	63.9 %	36.1 %	100 %

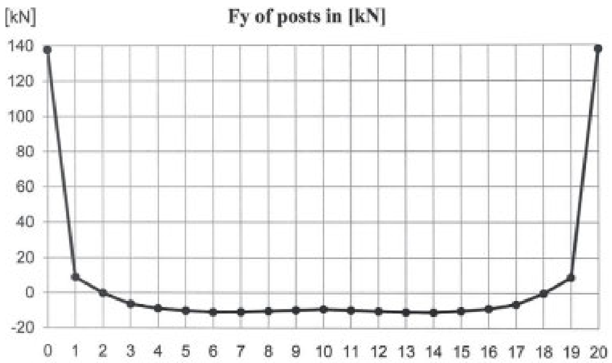

Fig. 19. Transverse forces in the main truss posts

a curving shape occurring in the steel elements of the deck construction induced by horizontal actions, the concrete plate carries, on the one hand, forces that are transmitted in the contact areas between the edge beams and the plate itself, and, on the other, it is supported by a great number of main girder posts and studs. These elements actually act as elastic bedding for the plate, absorbing a large portion of the load acting. Thus, a lower stiffness for the deck construction not only leads to a higher portion of the load being absorbed towards the top bracing but, additionally, relieves the deck elements. Fig. 19 shows the diagrams of the transverse forces occurring in the posts.

The results of further calculations are summarised in Tab. 4, which contains the percentages of the loads absorbed by the various elements of the structure. In this table, the colours as used in Tab. 3 apply.

The table clearly shows that the load absorption by the aforementioned elements (bracing, concrete plate, steel deck elements) – expressed as a percentage of the full load values introduced – does not differ much whether there is a traffic band or not. Again, this result can be understood by considering the global load transfer mode.

A comprehensive restoration of the lower bracing to allow for a defined absorption of wind forces would have a similar effect as adding the concrete plate to the unstiffened deck grid, relieving the upper bracing and the end frames and slightly enhancing the stress regime at deck level.

10 Conclusions

A comprehensive evaluation of the reliability of the 110-year-old railway bridge crossing the Danube in the Austrian town of Linz was performed recently, considering the poor condition of the structure because of corrosion damage and allowing for non-linear modelling where applicable. Sufficient reliability was able to be verified for a number of load combinations comprising wind loads up to the full values specified in design codes when considered as permanent, transient and extraordinary actions. Due to the critical condition evident in many areas and the current increase in the corrosion-induced damage, a limitation to the use in terms of both loading and time is given. This situation cannot even be improved to a satisfying degree by any kind of somehow reasonable repair measures, especially if performed in situ.

Considering the actual condition of the bridge, the expiration date for its use has been specified as the end of 2012, requiring a significant reduction in the rail traffic load model UIC 71 to C2 (q = 64 kN/m).

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