The Refurbishing of the Nibelungen Bridge Worms, Germany

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Summary

If the Nibelungen Bridge (Fig.1), opened on traffic in 1953, is forced into the corset of present-day technical regulations, then it loses its adequate load-bearing capacity. However, if one follows Finsterwalder's ingenious design idea of 1930 and enriches it with the ideal of fully prestressing, a careful strengthening will result with just a few external tendons laid carefully inside the two hollow boxes and an allowance for shear approach that will enable the structure to cope with today's live loads. Apart from the replacement of the tiebacks in one land pier, the further defects are due to the greatly changed, aggressive environmental conditions and the effect of de-icing salt that could not be foreseen in 1953.

Knowledge of construction history also helped to preserve this masterpiece of civil engineering without having to make any concessions with regards to its use.

Fig. 1: Nibelungen Bridge Worms, 1953

Keyword: refurbishing of early post-tensioned bridges, live load, external tendon, shear capacity, construction history

1. Introduction

With the construction of the second superstructure for the Nibelungen Bridge in Worms, the growing volume of traffic using the Rhine crossing at Worms is being taken into account. After the opening for traffic of the cantilevered hollow box-girder bridge, prestressed externally using a mixed construction method, a complete refurbishing of the early masterpiece of the chief engineer of Dyckerhoff & Widmann A.G., Ulrich Finsterwalder (1897-1988), is to follow in 2010.

2. State of the structure and historical assessment

The original bridge was opened to traffic in 1900. It consisted of three steel two-centred arches with the carriageway superimposed which were supported on pillars with caisson foundations. The shore bridges are solid, three-centred arches made of tamped concrete faced with natural stone.

The present Nibelungen Bridge in Worms forms part of federal highway 47 and links the municipalities of Worms and Bürstadt. The bridge has a total length of 745 m and is divided into three part structures of differing construction consisting of the some 109 m long shore bridge on the left side of the Rhine in Rhineland-Palatinate, the 351,8 m long centre part of the bridge crossing the Rhine and the shore bridge on the Hesse side of a total length of 295.5 m. The current traffic volume amounts to some 23,000 vehicles/24h with a moderate heavy vehicle share of 8.7%.

In the case of the shore bridges, the twelve three-centred arches made of non-reinforced concrete from 1903 are still in existence. Solid sandstone faces the front side of the 11.4 m wide arches and the areas directly in front of the hinges. The thickness of the arch lies between 0.67 m at the apex and c. 1.00 m at the quarter points of the arches. The arches are designed as compound curves with a radius of between 33 m in the case of the largest arch and 26 m in the case of the smallest one. The transverse prestressed carriageway plate made of concrete of B 450 quality incorporated in 1953 is supported over the whole surface in the crown area and above the masonry segments of the arches in the other areas. It is connected three times above the crown and impost hinges in each arch.

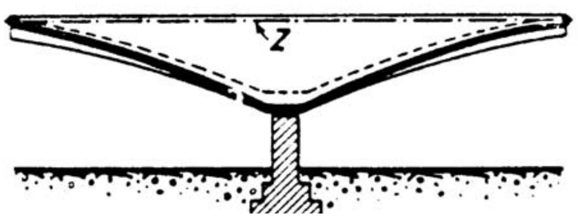

Fig. 2: Finsterwalder’s design idea from 1931 [06]

Traffic used to drive directly on the carriageway plates, as was quite usual for the first prestressed concrete bridges, until the only major repair of the bridge in 1974. Between 1972 and 1974, the bridge was completely sealed using a mastic seal, and in the 42 transverse joints single-seal-expansion joints, similar to the present-day German standard drawing Uebe 1, were provided. In the area of the shore bridges, the expansion joints had to be extensively touched up in 1981. Other preservation measures are in keeping with normal maintenance practice.

In his lecture at the general meeting of the German Concrete Association (DBV) in Berlin in May 1952, Finsterwalder [1] himself presented the fundamental design idea of cantilevering in prestressed concrete that dated back to the year 1930. On the occasion of the competition for the construction of a new Dreirosen Bridge in Basle, his company, Dyckerhoff & Widmann, had proposed two half arches connected rigidly above the pillar, the crowns of which were to be short-circuited via a tension member made of 60 mm thick wire ropes without connection. The prestressing was generated independently here by the dead weight (Fig. 2).

The further historical development of prestressed concrete and the outstanding position of the Nibelungen Bridge in the history of construction engineering is acknowledged in detail in [2] and [3]. The load-bearing structure and construction of the Nibelungen Bridge can be outlined as follows:

Around 1948, Finsterwalder [4] found the right material for his single bar steel tendons in the self-hardening St 90 steel, called St 60/90 shortly afterwards, from the Krupp Rheinhausen steel works. With nearly double the tensile yield strength, the steel was unsusceptible to brittle fracture and stress corrosion. Finsterwalder succeeded in rolling a screw thread onto the some 26 mm thick rods, thus making them suitable for anchoring and connections, which he joined ingeniously simply using plates, sleeves and screws. One single bar exerted some 0.2 MN (= 20 to) compressive stress into the concrete. Even more, Finsterwalder’s single bars had the advantage that there was nearly no slippage when the prestressing jacks were removed which was the prerequisite for cantilevering in short sections.

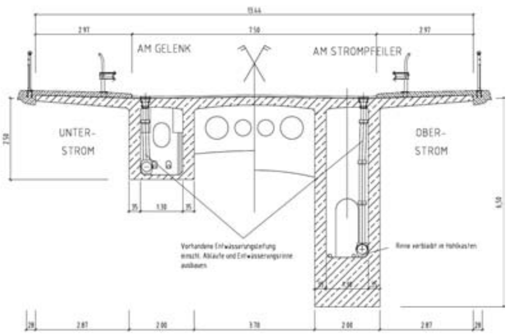

Fig. 3: Cross section of the centre bridge
Left: at hinge; Right: at pier

After the successful construction of the Balduin Bridge and a further pilot structure, the bridge over the Neckar at Neckarems [5], Ernst Wahl (1898-1982), the then head of the Road Construction Administration of Rhineland-Palatinate was able to tackle the replacement of the steel arched bridge in Worms, that had been destroyed in the war, by Finsterwalder’s cantilevered construction which had gained acceptance in the competition against the steel bridge structure as the most cost effective solution. Tied to the position of the caisson foundations of the destroyed steel bridge, the the Nibelungen Bridge crosses the Rhine by Worms in three large spans with the effective widths of 101.65 m, 114.20 m and 104.20 m. The cantilever girders protrude like monoliths from the reinforced concrete hollow pier, following Finsterwalder’s idea formulated in 1931 of a “cantilever whose vault thrust is cancelled out by tensioned, straight cables” [6]. Two hollow boxes connected through the carriageway plate with structural depths of between 6.5 m at the pier cut and 2.5 m in mid-span form the 13.5 m wide cantilever girder cross section (Fig. 3). A vertically prestressed joint, transmitting transverse forces, with stilted cast steel roller bearings avoid any reciprocal displacement of the cantilever girder ends.

Whereas the superstructure grows out of the river piers simultaneously on both sides in sections of 3.0 m towards the middle of the span, thus keeping the moments of difference to be absorbed for the subsoil small, the end piers had to be ballasted to control the overturning moment from the

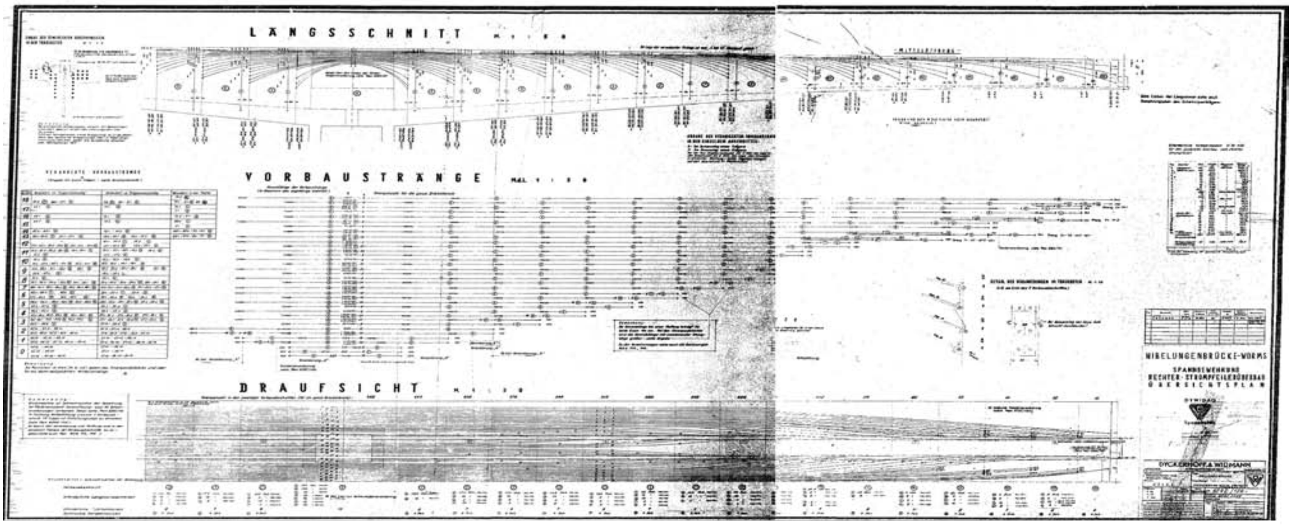

Fig. 4: Original plan of the "*prestressing iron*" Nibelungen Bridge, Worms (Source: Landesbetrieb für Mobilität)

growing superstructures or hung back via the prestressed structural components. This was helped by the existing foundations which had absorbed the horizontal thrust of the destroyed steel arch bridge. "The key to the construction" Finsterwalder observed [1].

Finsterwalder used his well-tried St 90 single bars for prestressing. He graded the prestressed reinforcement for the superstructures cantilevering out up to 57.1 m in accordance with the reduction of the bending moment towards mid-span (Fig. 4). The haunch helped him to keep the shearing stresses constant over the length of the structure. Under full load with the most unfavourable live load conditions, Finsterwalder allowed for a tension wedge that was later called limited prestressing. 486 bars with a diameter of 26 mm were necessary in order to be able to resist the fixed end moment of the full load amounting to 446 MNm (= 44 600 tm) at the centre piers (Fig. 5). Their initial eccentric pressure force of 108 MN (= 10 800 to) fell victim to the creeping and sliding losses of 20 MN. At the end of each section, on average 24 bars were stressed and anchored, [1]. One vertical reinforcement tendon per web directly alongside the cantilever joints absorbed the tensile splitting forces from the anchored tendons.

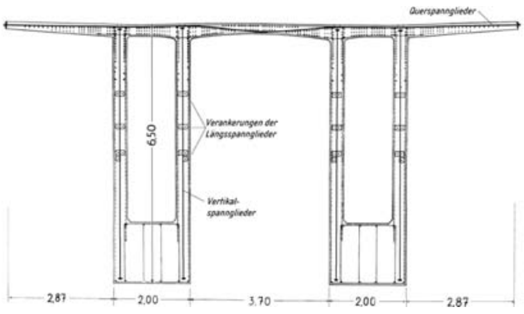

Fig. 5: cross section of Finsterwalder's longitudinal and transverse prestressing

The concrete grade of B 450 was a technical, but also a logistical challenge for the first post-war years. After 22 months of construction, the bridge was opened to traffic on 30 April 1953.

3. Steps to refurbishment

The examinations of the bridge carried out between 2000 and 2005 in accordance with DIN 1076 - Civil engineering works in connection with roads - inspection and test - on the shore and centre parts of the bridge showed damage that made a complete refurbishment necessary.

The planning for the refurbishment of the Nibelungen Bridge was subdivided into four sections:

3.1 Fundamental expertise on the state of knowledge and possibilities of application of the prestressed concrete method of construction at the time of execution

3.2 Technical building material examinations as well as examinations of the tendons and hinge prestressing of the centre part of the bridge

3.3 Checking of the bridge with the findings from and preparation of variants for strengthening the Nibelungen Bridge into bridge class 60/30

3.4 Preparation of the refurbishment design

3.5 Additional and follow-on special tests and examinations during building works to evaluate the measures planned in 3.4.

3.1 Fundamental expertise

In the opinion of the early prestressed concrete engineers, as far as possible no tensile stresses should be permitted in prestressed concrete components. This led to prestressed stirrups in order to avoid principal tensile stresses. Creeping and shrinking were known, however, engineers were still uncertain how to determine prestressing losses. Thus flat-rate reductions of 1200 -1500 kg/cm² were usual until the end of the fifties which went back to the Frankfurt Trials by the Neue Baugesellschaft Wayss & Freytag in 1936. As there was still no danger from salt used for thawing ice, friction and not protection from corrosion was the central topic for construction with subsequent bonding. Limiting the concrete tensile stresses in a transverse direction by an appropriate choice of system or transverse pretensioning led the engineers of that time to the conviction that the concrete would remain crack free and not require any sealing.

Following Freysinnet in this tradition, the Nibelungen Bridge is prestressed three dimensionally, however, as a result of his less high tensile strength prestressing bars, Finsterwalder did not achieve full prestressing for every load position. The design of the bridge was based on the 7th Draft of DIN 4227, but also included the discussion process still in progress. With the exception of the more precise proof of shear reinforcement in the case of exceeding the permissible main tensile stresses, the 7th Draft was essentially like the definitive first German prestressed concrete standard.

According to the 7th Draft for DIN 4227 in the January 1950 edition, the following fundamental proofs had to be furnished:

- Proof of compliance with the permissible stresses under use load in the concrete (tension and pressure) and in the prestressing reinforcement before and also after creeping and shrinking.
- Proof of the safety against cracking by covering the tension wedge as a result of increased loading by reinforcement when applying yield stresses.
- Proof of the safety against rupture under 1.75-times the load when applying a parabolic compressed zone of the concrete that has its vertex with 2/3 W_b at 1,8 ‰, without taking account of the tensile strength of the concrete (W_b = cube strength of concrete).
- Proof of the safety against shear failure and the principal tensile stresses. So long as the principal tensile stresses under ultimate load lay below a limiting value (B 450: σ_I =16 kp/cm² equivalent 1,6 MN/m²), no shear reinforcement was necessary, on the other hand, if they exceeded this value, all the principal tensile stresses in this section of the girder were to be covered by reinforcement.

The DIN 4227 - concrete construction elements with limited or fully prestressing - finally introduced in 1953 differed from its 7th Draft essentially in just two requirements:

- In the case of the proof of safety against rupture, the compressed zone was formed by a parabolic rectangle that had its vertex at 1,5 ‰ with 2/3 W_{28}. The compressed zone was then continued constantly up to 2 ‰ (W_b = cube strength after 28 days).
- The limiting value of the principal tensile stresses in the shear design was raised for B 450 to σ_I = 20 kp/cm². It was not complied with, in the girder section, in which 75% of the value was exceeded, the principal tensile stresses had to be absorbed by reinforcement.

A minimum reinforcement was only introduced with the Additional Provisions for DIN 4227 by the Federal authorities in 1966.

Finsterwalder interpreted the value of the shearing stress rather generously with σ_I = 18 kp/cm² compared with the limiting value of the 7th Draft of DIN 4227, but the definitive standard confirmed his value.

With the haunched cross-section design, the principal tensile stresses could be kept nearly constant over the length of the structure and were so limited in calculation that an inclusion of the stress and strain by reinforcement could be omitted. Nevertheless, Finsterwalder ordered a low shear mesh reinforcement and mild longitudinal reinforcement. On average 6.22 cm²/m is present per web which would not lead to an adequate shear capacity according to present standards. The longitudinal reinforcing bars of the carriageway plate verifying from 1.52 cm²/m (steel grade BSt I, yield point 220 MN/m²) up to 4.40 cm²/m (BSt III, yield point 420 MN/m²).

The edge stresses calculated did not lie within the tension zone under the most unfavourable load combination, so reinforcement to limit cracks was not required.

The carriageway plate is transversely post-tensioned with prestressing bars of steel grade St 60/90 every 40 cm with a diameter of 26 mm and additional reinforcing bars spreading from 1.25 cm²/m (BSt I) above up to 10.62 cm²/m (BSt III) below.

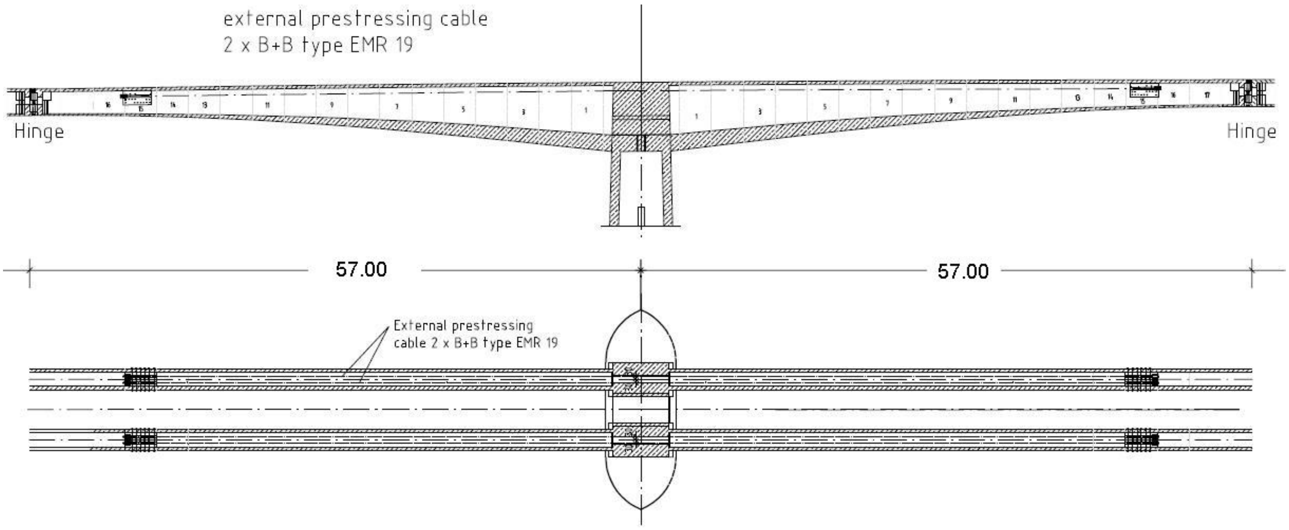

Fig. 6: Reinforcing the cantilever beams with external longitudinal tendons

3.2 Technical building material examinations

3.2.1 Material expertise

Shore bridge: The existing jointed reinforced concrete slabs are badly penetrated by humidity. Concentrations of chloride are to be found to a depth of 12 cm. The damage described extends further into the tamped concrete and the natural stone masonry. There is breakage, flaking and sintering both in the crown and in the impost area.

Centre bridge: The sealing system is defective. At the hinges and the eypansion joints, increased carbonation and chloride contamination are to be observed. The concrete cover complies with the standards of that time. The drilling cores taken showed a homogeneous, uniform and dense structure. The concrete grade corresponds on average to a B 450, which was later equivalent to a B 40 and to a C 30/37 today.

3.2.2 Examination of the tendons

Spot drilling checks were carried out with visual inspection for defects in the ducts and corrosion in the prestressing steel, ultrasonic echo examinations to locate breakages in the prestressing steel of anchorage fixtures as revealed in openings in the surface of the bridge carriageway, as well as additional visual inspections, in particular to locate bar breakages and damage to the protection against corrosion on the hinged prestressing bars.

The random drilling checks showed an average grouting error of some 20% for the longitudinal tendons. A quite normal value for the DYWIGAG prestressing method of that time. Only 2 tendons showed more severe corrosion. In the case of the transverse tendons, the grouting error falls to 9.1 %. The grouting error in the tiebacks at the Hessian land pier is too high with over 40 %, it being accompanied in addition by excessive and severe corrosion. The prestressing bars of the cantilever beams are not remarkable either for their degree of grouting or corrosion. On the other hand, the hinged prestressing bars are severely corroded, in part also broken.

3.3 Check calculation

Building up on the findings of, the check calculation shows that in order to achieve bridge load class 60/30 in accordance DIN 1072 German live load standard until 2002, the Nibelungen Bridge has to be reinforced for proof of safety against bending rupture. On the basis of the design theory of that time, as explained in more detail above, which allowed the concrete a contributory role in the transmission of shear, so long as 70% of the permissible principal tensile stresses were not exceeded, the Nibelungen Bridge has just a minimum shear reinforcement. This apparent deficit, from today's point of view, cannot be eliminated either by the later incorporation of a shear reinforcement, as was verified by the repair variants described here below. Rather a later shear reinforcement threaded into the webs would reduce the safety against rupture as a result of cutting through a large number of tendons. On the other hand, the level of the permissible principal tensile stresses can be achieved by external prestressing for bridge load class 60/30. One should not lose sight of Finsterwalder's ingenious act of erecting an arch with tieback and less a beam.

3.4 Refurbishing design

The necessary repair measures have been shaped by the results of the examinations (3.1 to 3.3) which have been supported and calibrated by further follow-up examinations (3.5).

3.5 Bridge inspection

The examination of the bridge by the State Agency for Mobility of the State of Rhineland-Palatinate checked the crack situation. In its report it did not determine any marked crack situation. Only in individual cases are incipient cracks to be seen on the outer sides between the carriageway plate and web. From this it is to be concluded that the Nibelungen Bridge follows the load-bearing structure model chosen by Finsterwalder.

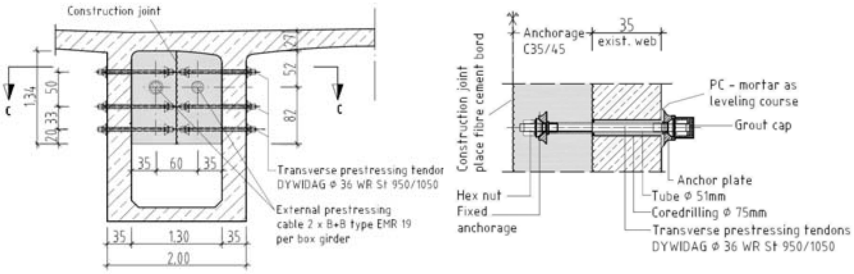

Fig. 07: Detail of the anchorage of the external additional prestressing tendons Left: Cross section of a hollow box at end of the cantilever Right: Detail of the anchorage block with transverse prestressing bar St 950/1050)

After the diversion of traffic onto the new bridge in 2009, the carriageway deck was freed of the dead loads in order to check the extent of the replacement of concrete and degree of corrosion of the tendons for the call for tenders and determine the quantities required for the call for tenders. The carriageway was subjected to a complete potential field measurement for this; transverse tendons were once again opened at random. In the course of this, the state and degree of damage of the bridge were confirmed. Only the crown hinges gave cause for concern. Underseepage of the waterproofing in conjunction with the pervious expansion joints had severely softened the concrete over about one metre length and let it become rapidly porous so that shipping on the Rhine had to be protected from falling lumps of concrete by means of scaffolding and nets.

3.6 Summary assessment of the state

The refurbishing of the Nibelungen Bridge is based on the finding that the bridge's structural stability is not endangered. The following inspection points describe the state of the centre part of the bridge:

- Favourable crack situation without shear cracks
- Good concrete quality, however without subsequent hardness
- Usual contamination with chloride and degrees of carbonating
- Flaking of concrete only locally in the area of the crown elements as well as humidity penetration on the wearing parts of the expansion joints
- Locally inadequate degree of grouting of the tendons due to the then too small ducts and the too narrow construction of the tendon sleeve coupling
- Tendons of the tieback to compensate for the cantilever moment on the land pier on the Hesse side inadequately grouted and corroded
- Severe corrosion and wear phenomena on the crown hinges coupling the individual cantilever - systems and their tie rods endanger traffic safety.
- Unsuitable drainage system (open gutter inside the hollow box along with free fall system).

The durability of the shore parts of the bridge is reduced to a greater extent through the defective expansion joints fitted later and the defective drainage system. The jointed carriageway plates are no longer a match for today's aggressive environmental influences.

4. Refurbishing measures

4.1 Centre part of the bridge

For the centre part of the bridge, four fundamental variants were developed and assessed on the basis of an economic viability check in accordance with Ri-Wi-Brü [7].

Variant 1C: Structural refurbishing without reinforcing the shear (€ 3.5 million)
Variant 2C: Structural refurbishing with reinforcement of the shear (€ 6.5 million)
Variant 3C: Refurbishing of the substructures and renewal of the superstructure (€ 10.4 million)
Variant 4C: Complete replacement of the superstructure and substructures. (€ 15.1 million)

The decision was taken in favour of Variant 1C that shows clear financial advantages and, as already explained above, avoids any structural damage to the longitudinal tendons. Its extent can be outlined as follows:

- Cleaning the dirty parts (pier and crown hinges soiled by bird droppings)
- Demolish the railings, crash barriers, lighting, curbs and carriageway surfacing
- Check of the defective building material quantities after demolition of the sealing and before awarding the contracts for further works
- Examination of the tendons including the regrouting
- Reinforcement of the superstructure by means of straight, external prestressing, lying on top by 2 B&B type EMR 19 or equivalent (Fig. 6 and 7)
- Substition of the existing stilt bearing to spherical bearing, including bracing the hinge (Fig. 08)
- Renovate the existing expansion joints in the crown hinge area, as well as in the transition to the separating piers in accordance with ZTV-ING 8.1 [8]
- Renovate the existing drainage system in accordance with ZTV-ING 8.5 [8]
- Repair of the concrete in the damaged areas in accordance with ZTV-ING 3.4 and 3.5 [8]
- Renovate sealing and surfacing (ZTV-ING 7.1 [8]), curbs, crash barriers and lighting.

4.2 Shore parts of the bridge

Three variants were prepared for the refurbishing of the carriageway plate of the shore parts of the bridge:

Variant 4S: Refurbishing of the existing carriageway plate in accordance with ZTV-ING 3.4 [8] and 3.5 and the 36 expansion joints (€ 0.63 million)
Variant 5S: Partial demolition of the carriageway plate in the joint areas and joint end by CFRP strips (€ 0.83 million)
Variant 6S: Replacement of the carriageway plate by a monolithic continuous carriageway plate made of light-weight concrete (€ 0.85 million)

Variant 6V convinced through a significant cut in the maintenance expenditure. It allows minimum costs to be expected for future expenditure for investment and maintenance. With the choice of light-weight concrete, the restraints as a result of the continuous carriageway plate are clearly diminished, as both the coefficient of thermal expansion and the dead weight of the carriageway plate are reduced and thus the deformation of the plate and the horizontal force are decreased as a result of the friction. Expansion joints are only envisaged at the plate ends. The carriageway plate is given sliding support at the ends by the arrangement of fibrated concrete slabs between the superstructure (carriageway plate) and reinforced concrete head beams on the masonry. The fastening to the substructures (arches and also masonry diaphragms) in the intermediate area is made by shear connectors.

The renovation of the curbs, seal, surfacing and vehicle restraint systems, the repair of concrete and masonry surfaces, as well as the fitting of a closed drainage system complete the measures for the shore parts of the bridge.

4.3 Land piers (Separating piers)

The land piers transmit the moments of the externally mounted cantilever girders (cf. par. 2) into the subsoil. In the land pier on the right bank of the Rhine, the cantilever moment is dissipated into a force couple via cross girders through connecting compression columns and tiebacks. The tiebacks consist of two edge tension members with 14 single bar tendons, each with a diameter of 26 mm, and a middle tension bar made up of 34 single bar tendons. They were found to be the tendons with the highest degree of damage (3.2.2). For reinforcement, the edge tension members are given 2 mono-strands (St 1570/1770, A_z = 140 mm) each, the middle tieback is given 3 mono-strands which are anchored by means of core drill holes in the foundation and the upper tieback transverse girders. Recesses in the middle of the tieback accommodate the tensioning wedge for prestressing the additional tendons. Further mono-strand tendons serve to strengthen the tieback transverse girder.

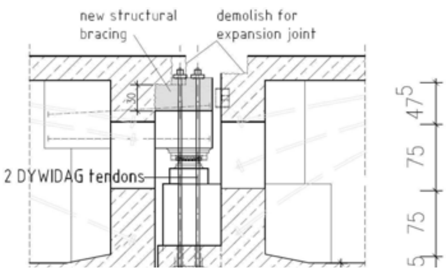

Fig. 8: Hinge in the crown with prestressing anchor and spherical bearing

Further measures for both land piers, such as e.g. repairing the natural stones and their joints, improvement in accessibility or renovating damaged manhole covers, are routine tasks. In conclusion, the land pier on the Hesse side will be given a new, external staircase tower.

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