# Strengthening of the Inn Bridge Obernberg-Egglfing

## *Balancing Act Between Preservation and New Construction*

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**Abstract** The Obernberg - Egglfing Bridge is an important border bridge between Austria (L510) and Germany (St 2117). It was constructed in 1963 as a post-tensioned concrete structure, designed as a double-T beam in the span and a box girder in the area of the supports. Characteristic of this period, the steel reinforcement was minimized in favour of a high level of prestressing, and 100% of the tendons are coupled at the construction joints. Due to cracks in the coupling joints, the condition of the structure was assessed through comprehensive structural and metrological analyses, adhering to levels 1 to 4 of the German guideline for recalculation of road bridges. By reducing the load on the structure to the load model BK30, in combination with coupling joint monitoring and alternative design approaches to describe shear load-bearing capacity, the interaction of bending in transverse direction and longitudinal shear, as well as torsional stiffness in level 2, the structural deficits were sufficiently limited to make an economical strengthening of the structure feasible. But also during the planning and installation of the various reinforcement elements, the existing structure presented unexpected challenges.

## 1 Description of the structure

The structure was built between 1963 and 1965 in eight construction phases on a falsework. The superstructure is a longitudinally prestressed double-T beam with a reinforced concrete slab. In the support areas, there is a haunched compression plate, resulting in a box girder at the piers. The spans are 35,00 – 37,00 – 40,00 – 43,50 – 52,00 – 2x 62,00 – 52,00 m with a total length of 383,50 m.

For the post-tensioning of the superstructure in the longitudinal direction, the post-tensioning system „BBRV“ St 150/170, with 44 individual strands of Ø6 mm, was used. The structure was designed for the target load level BK45 according to DIN 1072.

The condition of the structure was assessed according to the main inspection in accordance with DIN 1076, resulting in a condition rating of 3.5 (S=3.0). Two systematic deficiencies were identified: separation cracks in the compression plates < 0.35 mm and cracked coupling joints (CJ). Locally, a

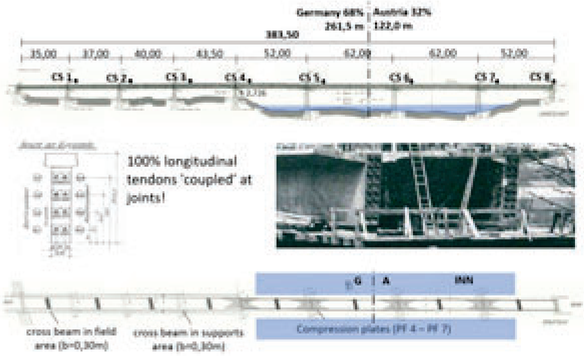


**Figure 1:** Longitudinal section and plan view

3 m diagonal crack in the support area < 0.5 mm was also observed.

## 2 Recalculation

The structure was initially evaluated for the target load level BK60 according to levels 1 and 2 of the German guideline for recalculation of road bridges (RG) [1], [2]. Significant deficiencies were identified in the longitudinal direction across the entire length of the beams, particularly concerning the shear capacity and the connection reinforcement of the flange, as well as the fatigue safety of the prestressing steel at the coupling joints. In the transverse direction, deficiencies were also noted in the shear capacity of the pressure plates, see Fig. 2.

Since significant damage to the prestressing steel at the coupling joints could not be ruled out, a coupling joint monitoring was initially conducted. This allowed the fatigue deficiencies to be narrowed to CJ 1.

In addition, the compressive strength of the existing concrete was determined through material tests.

A comprehensive strengthening to meet all requirements according to Level 2 would have involved a substantial intervention in the structure, combined with compensatory measures and a usage restriction of 20 years, see Fig. 3. Ultimately, this measure was dismissed as uneconomical due to the high costs.

Due to the high costs involved, instead of maintaining the structure, the authorities in charge of the construction subsequently opted for emergency strengthening until a new replacement could be built, with the aim of maximising the remaining service life ($\geq$ 20 years) with minimal strengthening intervention while maintaining two-way traffic and the BK30. In this context, scientific methods according to Level 4 of the Recalculation Guideline (RG) should be applied, particularly for reducing the deficits.

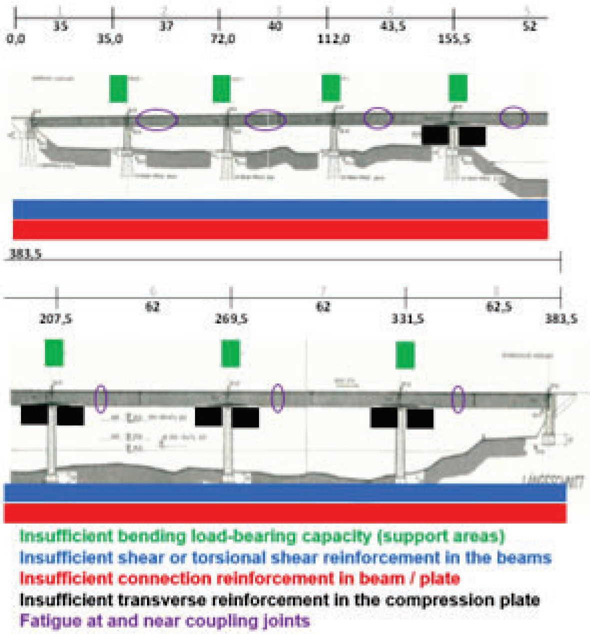


**Figure 2:** Extent of the calculated deficiencies (load model BK60) according to Level 2 of RG

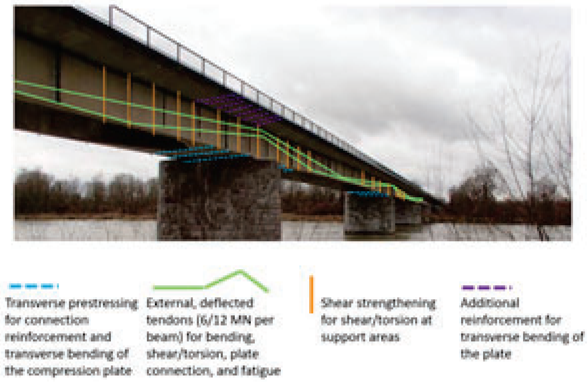


**Figure 3:** Overview of the strengthening concept for BK60

## 2.1 Approaches according to Level 4.

For the assessment of the shear capacity, the shear force model according to Herbrand [3] was used, which shows potential load reserves, particularly with low shear reinforcement but high prestressing levels. In Germany, it is also intended in the future to be considered within the framework of the 2nd amendment of the Recalculation Guideline (RG), which will appear in BEM-ING Part 2, already at Level 2.

Based on experimental observations [4], the design procedure is based on a truss model (in accordance with EC2) but includes an additive concrete load-bearing component, which can primarily be attributed to the shear capacity of the uncracked concrete compression zone, see Fig. 4.

With the verification the large deficits of shear capacity was significantly reduced since the design standard at the construction time of the bridge did not require a minimum shear force reinforce-

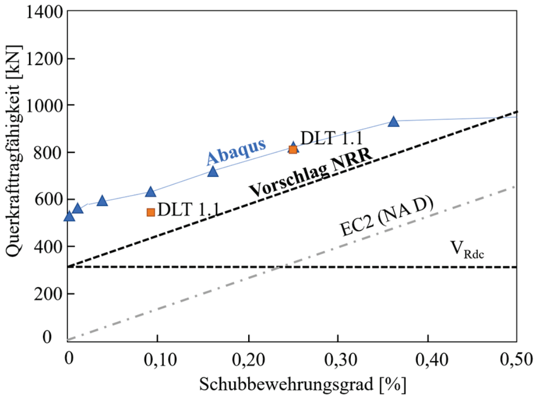


**Figure 4:** Shear capacity as a function of the shear reinforcement ratio. FE parameter study on test DLT 1.1, [3]

ment.

The use of open stirrup shapes resulted in significant deficiencies in torsional capacity. The deficiencies are primarily determined by the assumed torsional stiffness at Level 2. Without a more detailed verification, a reduction of the torsional stiffness in the ultimate limit state due to cracking to a value of 40% of the stiffness of the uncracked state may be assumed according to Level 2 of the Recalculation Guideline (RG).

In [5] various approaches to reducing torsional stiffness due to cracking for the recalculation of the Obernberg-Egglfing Bridge were discussed in the context of a literature study. Key influencing factors include cross-sectional geometry, the level of prestressing, the inclination of the compression struts, and the type of loading. Generally, the stiffness reduction in T-beams, without prestressing and/or subjected to pure torsional loading, is significantly more pronounced than in box girders, in prestressed beams, and/or in combined loading scenarios involving bending and torsion.

As a result, for the recalculation according to Level 4, a reduction of the torsional stiffness of the T-beam girder sections to 25% and the box sections to 100% of the linear elastic value according to Level 1 was recommended. This provided a sufficient redistribution potential of the torsional shear forces through transverse bending of the plate was demonstrated. This approach led to a significant reduction in the required torsional stirrups.

Another special design feature of the structure was the asymmetrical arrangement of the transverse reinforcement of the deck slab on the upper and lower sides, see Fig. 5.

According to DIN Technical Report 102, Section 4.3.2.5, the required reinforcement should be distributed equally between the bottom and the top (without more detailed verification). This approach is not effective for large areas and resulted in significant deficits according to Level 2. In Level 4, the procedure according to Bachmann [6], [7] was therefore used, taking into account the interaction between longitudinal shear and transverse bending. The centrically acting transverse tensile force according to the flange truss $Z = a_{sw,erf} * f_{yd}$ and the transverse bending moment $m_{yy}$, along with the associated normal force $n_{yy}$ are considered as acting together. The cross-section is designed accordingly. Thus, on the compression side, the longitudinal shear reinforcement can be

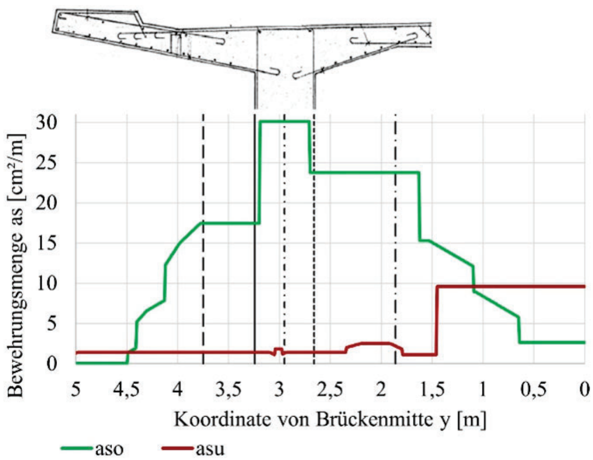


**Figure 5:** Distribution of the existing upper (green) and lower (red) flange connection reinforcement

reduced based on the effect of the bending compressive force.
The verification of the flange connection of the deck slab is thus only locally not fulfilled in areas that connect to the pier compression plates.
Overall, the deficiencies of the recalculation could be significantly reduced in magnitude and strongly limited locally through the approaches in Level 4, Fig 7.

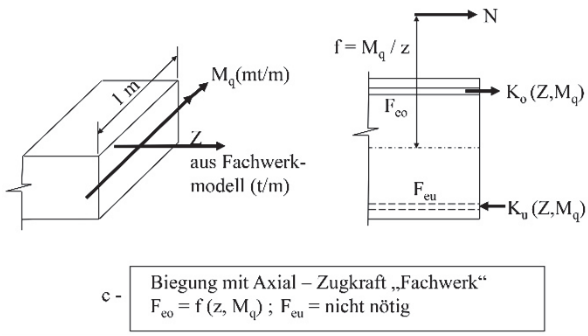


**Figure 6:** Design model for longitudinal shear with transverse bending, [7]

## 3 Strengthening

According to the results of the recalculation and discussions with the clients from Bavaria and Upper Austria, the structure was strengthened from July 2023 until September 2024. The strengthening measure allows for the use of the structure by normal traffic (load model BK 30/30 without the need for a permit for heavy traffic) during the period until the completion of a replacement construction with a planning horizon of $\geq$ 20 years.

For this purpose, the bridge superstructure was prestressed with numerous bar tendons. Tab. 1 provides an overview of the elements used for this purpose and their quantities. The specific features in the design and construction of the individual elements will be explained in the following chapters.

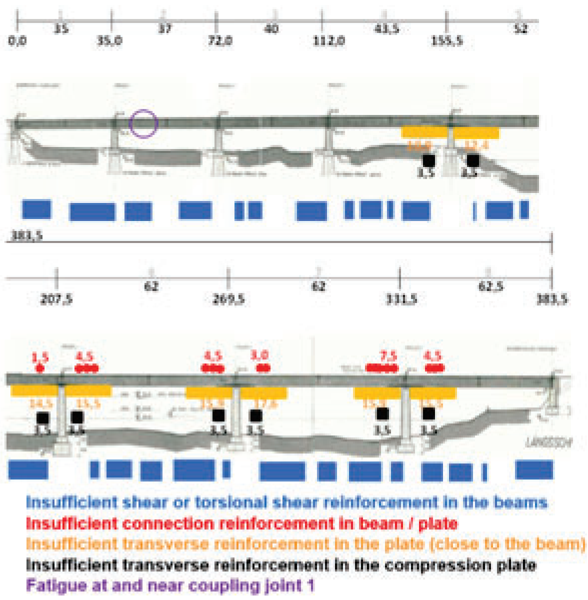


**Figure 7:** Deficiencies of the recalculation according to Stage 4

Table 1: Summary of the strengthening elements.

| Element | Strengthening | Quantity |
|---|---|---|
| Vertical tendon pairs with crossbeams | Shear force and torsion | 400 |
| Horizontal tendons through a bar | Torsion due to stirrup closure | 402 |
| Tendons between the bars (top/bottom) | Flange connection (top/bottom slab) | 120/56 |
| Anchor blocks / longitudinal / transverse tendons | Reinforcement of the coupling joint 1 | 8/4/16 |

### 3.1 Vertical tendons with crossbeams

The strengthening of the two main girders for the verification of shear with torsion is achieved through external vertical and internal horizontal bar tendons to create the stirrup closure.

The vertical tendons were arranged in pairs on the left and right sides of the beams at regular intervals between 1.0 and 2.0 meters. The necessary boreholes through the deck slab and, in the box girder areas, through the compression plates were drilled from above. On-site, the transverse reinforcement was located, and the boreholes were positioned between the bars.

The fixed anchoring on the top side is achieved using anchor plates with screw threads, which are installed in blasted recesses, known as anchor pockets, on the top side of the roadway slab. On the bottom side, the anchor plate designed as a tension anchor is positioned on the bottom edge of crossbeam structures made of U-profiles. This transfers the tension forces through a grout mortar connection into the bottom edge of the web. An overview of the system is shown in Fig. 8.

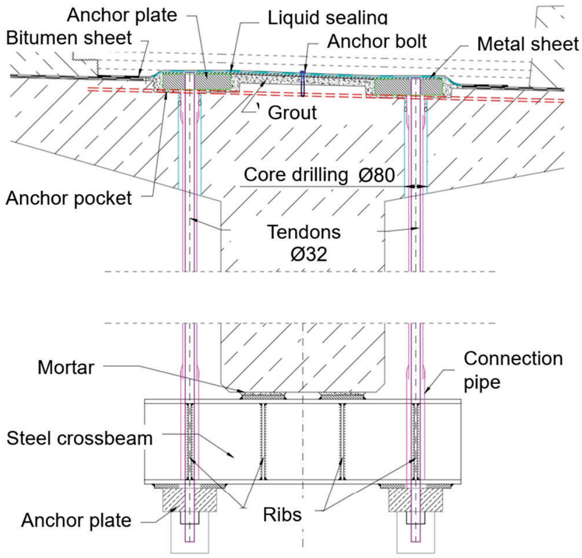


**Figure 8:** System overview of the vertical tendons for shear strengthening

The high loading of the upper transverse reinforcement required that any tensile forces, arising from the inclination of the compression strut between the anchor plate and the beam, be short-circuited by an additional plate between the two top anchor plates. This tension plate was precisely welded to the anchor plates with fillet welds after the installation of the anchor plates, and subsequently, the anchor plates and the plate were grouted with mortar. However, since the plate only actually acts as a tension band in ultimate limit state, strong compressive forces occurred during the installation of the up to 200°C hot mastic asphalt due to the thermal expansion of the plate being hindered by the simultaneous fixation on both sides. To prevent buckling of the plates, they were secured downwards using bolt anchors. The required number of bolts was determined experimentally. The installation of the mastic asphalt was simulated by heating the plates with burners, see Fig. 9.

**Figure 9:** Distorted connection plate without a central anchor bolt after experimental heating.

## 3.2 Horizontal tendons through one main beam

The shear reinforcement of the structure consists of wire mesh reinforcement and, in the direction of the support areas, additional vertical bars with hooks on the top and bottom. These 'stirrups' can at least partially be accounted for shear forces. However, the verification model for torsion is based on the core cross-section enclosed by stirrup reinforcement, so formally only the wire mesh reinforcement could be considered.

To improve this torsion deficit, boreholes were drilled as low as possible and pre-stressed with short horizontal tendons. The (partially angled) anchor plates were applied to the beams with a separating layer of grouting mortar.

In the entire project, the load introduction of the tendons onto the existing structure was carried out using anchor plates made of ultra-high-performance concrete (UHPC), referred to as hybrid anchors. These were manufactured according to the various required angles, with a slope of up to 13.5°, thus compensating for the angles between the existing structure and tendon axis (see Fig. 10). In addition to their lower weight compared to steel plates, hybrid anchor plates do not require corrosion protection.

**Figure 10:** Hybrid anchor plates with inclination corresponding to the alignment of the existing structure

### 3.3 Horizontal tendons between the beams

In the support areas with box section, high shear forces must be transferred between the flanges and the beams. In the recalculation, this verification of the flange connection for the cantilever could be provided, while in the box girder areas, deficiencies occur at both the connection of the deck slab and the compression slab.

To strengthen this horizontal shear load capacity, both girders were prestressed using external tendons that run between the beams. The alignment of the boreholes for the tendons follows the structure’s angle of 76.5°. The necessary holes through the beam were drilled starting alternatively

**Figure 11:** View into the box girder area field 8

from one or the other side to minimize damage to the already deficient stirrup reinforcement. Then a laser was used to precisely measure the location of the hole on the opposite beam.

### 3.4 Longitudinal prestressing at the coupling joint 1

The lack of fatigue strength at coupling joint 1 could be established with an additional centric prestressing of 1.0 MN. In order to guarantee this in the long term, prestressing was applied with two external Ø 32 mm bar tendons per beam.

The approximately 8.5-meter-long tendons were tensioned to the beams using steel construction elements and four cross tendons (similar to the torsional reinforcement afterwards grouted bore-holes). Due to the alignment of the tendons and the stirrup reinforcement in the existing structure, the tendons were installed at an angle of 4.2° to the horizontal.

**Figure 12:** Longitudinal prestressing at the coupling joint 1

## 4 Construction Sequence

Both structural and traffic necessities influenced the construction schedule for the reinforcement measure. Due to the high calculated deficiencies, traffic has been directed in a single lane in the centre of the roadway since 2022. The relatively low average daily traffic allowed for alternating traffic flow in both directions.

In principle, the concept of alternating traffic management within a single lane was continued throughout the entire construction period. Initially, the lane remained in the centre, while in the first construction phase, the short horizontal tendons were installed in the lower beam area to enhance the torsional load-bearing capacity. Subsequently, traffic could be shifted to one side of the roadway, allowing work on the other side to begin.

To minimize structural risks, it was essential to strictly adhere to the order during the installation of the tendons in the box girder areas. The drilling through the beams was performed before the drilling through the deck slab for the vertical tendons. This sequence allowed for the necessary adjustment of the drill holes on both beams to ensure that the stirrup reinforcement could be largely preserved.

When pre-stressing the tendons, the order had to be reversed: the horizontal tendons induce lateral bending in the beams, which can only be absorbed after the application of the vertical pre-stressing forces on both sides.

Throughout the entire installation of the strengthening elements, the structure was kept open to traffic. For a smooth execution of the installation of new expansion joint constructions and a continuous asphalt surface layer, the bridge was fully closed for two weeks in August 2024.

## 5 Conclusion

The Innbrücke Obernberg-Egglfing has been made suitable for the final phase of its use through the strengthening measure. The additional pre-tensioning of the cross-section in vertical and partially horizontal directions increases the load-bearing capacity, allowing the structure to be used without restrictions for load model BK30/30. By compressing the cross-section, the cracks in the compression plates or in the beam area were closed. Future damage to the concrete and prestressing steel due to fatigue-relevant traffic loads and temperature gradients has been ruled out with the help of this measure.

## 6 Participants

Client in Germany and Construction Management:
Staatliches Bauamt Passau

Client in Austria:
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Recalculation, assessment, and monitoring of the coupling joints and planning of the strengthening: matrics engineering GmbH, Munich Report on Torsional Stiffness:

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Prestressing:
BBV Systems GmbH

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