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Strengthening of the 120-Year-Old Substructure of a Railway Bridge

Renforcement de l'infrastructure d'un pont-rail de 120 ans

Verstärkung des 120 Jahre alten Unterbaus einer Bahnbrücke

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SUMMARY

The 120-year-old substructure of a railway bridge has been strengthened to allow for a considerably heavier new superstructure throughout a projected service life of 100 years. The joints of the original masonry piers and abutments have been reconditioned, the foundations of two piers have been strengthened by the soilcrete technique, and an abutment has been anchored in the bedrock in order to resist the high horizontal loads.

RÉSUMÉ

L'infrastructure d'un pont-rail de 120 ans a été renforcée pour supporter une nouvelle superstructure, nettement plus lourde, pendant la durée d'utilisation prévue de 100 ans. La maçonnerie en pierre naturelle des piles et des culées a été remise en état, et les fondations de deux piles ont été renforcées avec des pieux. Une culée a été ancrée dans la roche pour reprendre les importants efforts horizontaux.

ZUSAMMENFASSUNG

Der 120 Jahre alte Unterbau einer Bahnbrücke wurde verstärkt, um den beträchtlich schwereren neuen Überbau über eine geplante Nutzungsdauer von 100 Jahren tragen zu können. Die Pfeiler und Widerlager aus Natursteinmauerwerk wurden instandgesetzt, und der Baugrund unter zwei Pfeilern wurde mit Hilfe von Jetting-Pfählen verstärkt. Für die Aufnahme der grossen Horizontalkräfte musste ein Widerlager im Fels verankert werden.

1. INTRODUCTION

The steel superstructure of a 233 m long double track railway bridge over the river Aare at Brugg, Switzerland, is being replaced by a post-tensioned concrete box girder (Fig. 1). The substructure built in 1875 has been strengthened and the pedestrian suspension bridge is being reconstructed.

The new girder was constructed span by span on temporary concrete piers and scaffolding on the downstream side of the existing bridge. After having removed the five single span steel truss girders for either track of the existing bridge, the new continuous girder will (in two stages) be shifted laterally into its final position. Thus, except for one complete shut-down and two periods of single track use, the railway traffic is not affected by construction operations.

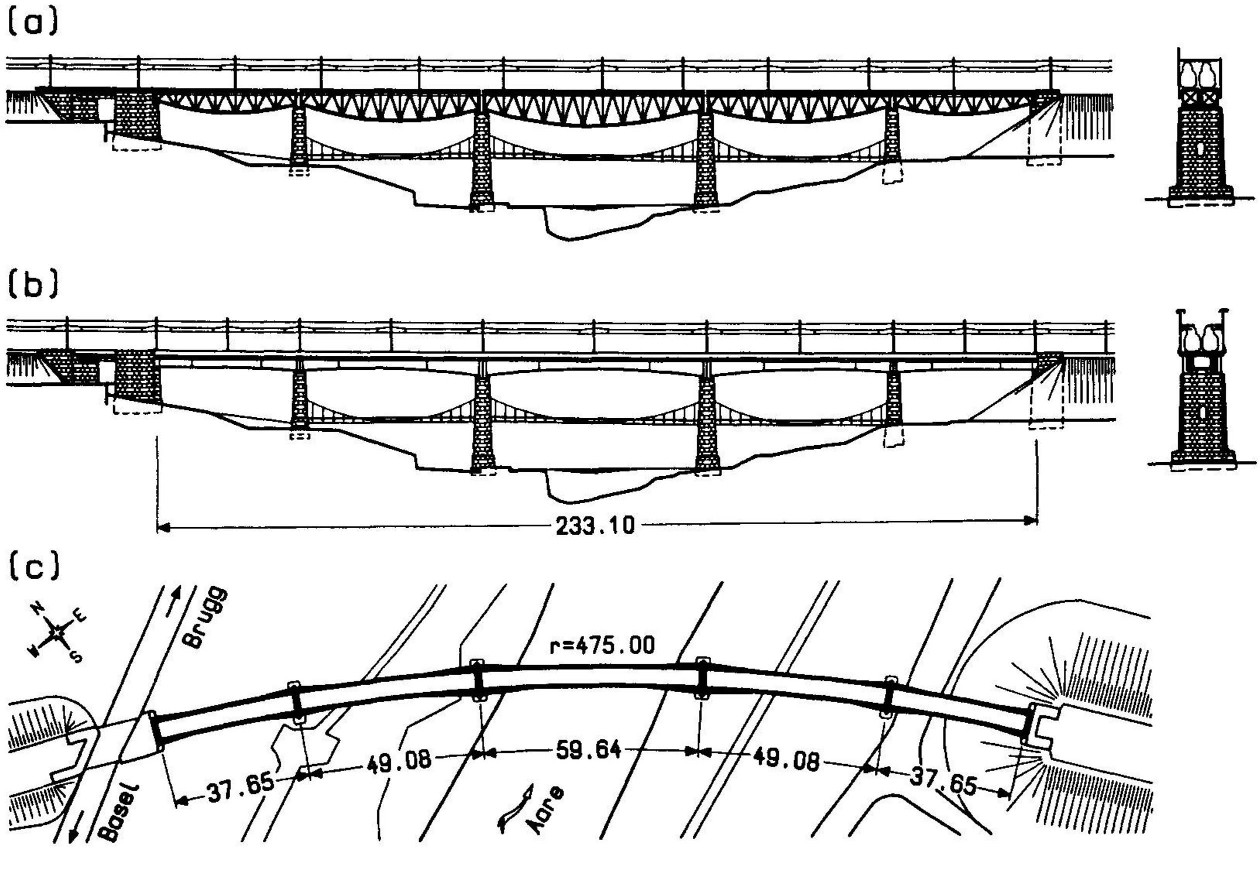

Fig. 1 Overview: (a) Steel superstructure on natural stone masonry substructure; (b) New post-tensioned concrete box girder on strengthened substructure; (c) Plan view of new bridge. Note: Dimensions in metres.

Compared to the open-decked steel structure the average dead load of the new concrete girder (including ballast) increased from 69 to 490 kNm^{-1}, i.e. by 610%. Design values for usual live loads increased from about 110 to 160 kNm^{-1}, i.e. by 45%. Strengthening measures for the substructure had to take into account these increased loads, the projected future service life of 100 years and the condition that all construction work had to be executed under full railway traffic.

This paper presents the techniques used to strengthen the different parts of the substructure, i.e. the foundation of the two side piers, the natural stone masonry of the piers and abutments, and the West abutment. Assessment of the existing structure, strengthening concepts and experiences gained during execution are discussed

2. FOUNDATION OF SIDE PIERS

While the two 25 m high central piers are founded on solid rock the somewhat shorter side piers are founded on layers of weathered molasse with a high clay content. These soft layers have a variable thickness of 1 to 8 m (Fig. 2).

Currently, average (extreme) soil pressures without traffic and with maximum traffic loads amount to 0.38 (0.45) and 0.55 (1.29) MPa, respectively. In the future, these values will increase to 0.62 (1.16) and 0.80 (1.50) MPa, respectively. In order to avoid unacceptable settlements and inclinations of the piers it was decided to replace the weathered molasse by a material with considerably higher modulus of elasticity and strength.

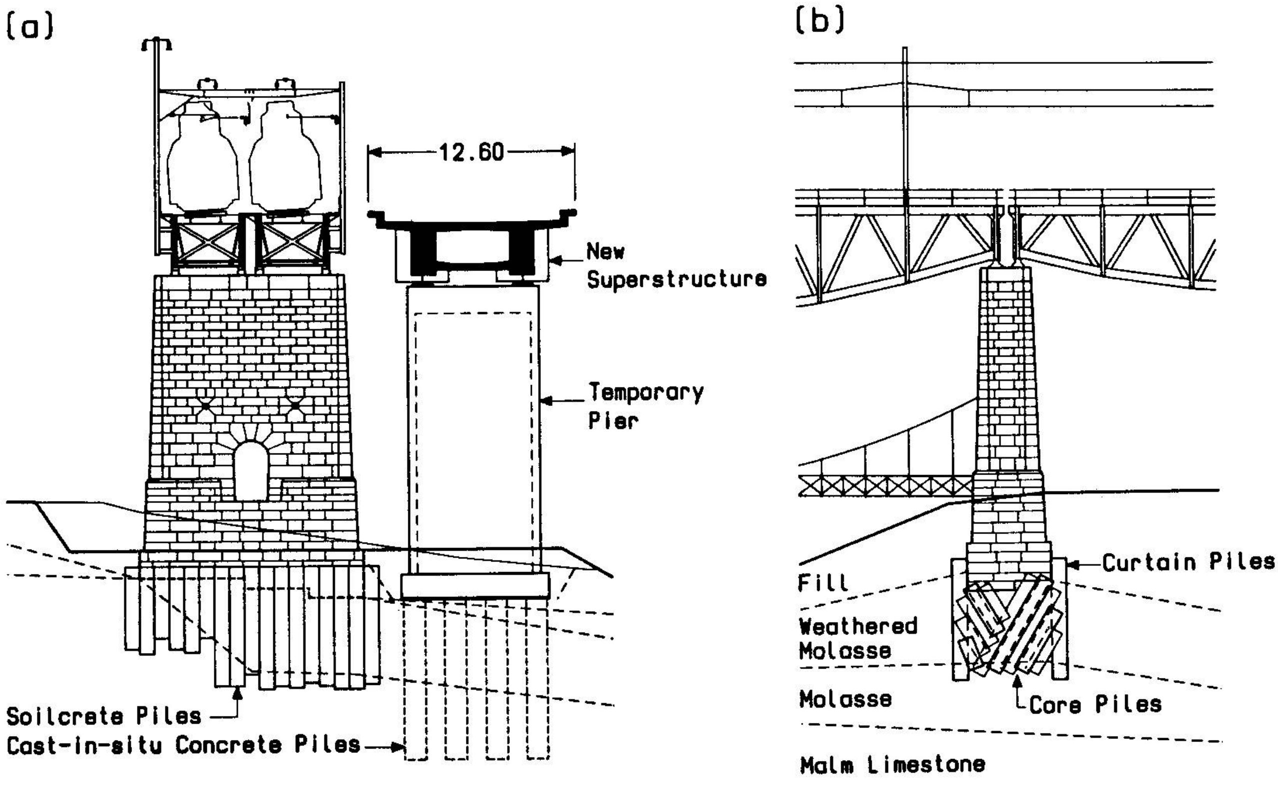

Fig. 2 Soilcrete pile foundation of side piers: (a) Cross-section; (b) Elevation.

Traditional underpinning was discarded because of high costs and risks for railway traffic during execution. The soilcrete technique was chosen after a feasibility test had shown that piles of sufficient quality could be made despite the soil's high clay content. It was decided to complete a curtain along the perimeter of the pier base prior to filling the core underneath the foundation. The curtain piles served to encase the material underneath the pier and allowed to gain further experience on the actual soil conditions. Construction of the core piles began only after the curtain piles had passed severe quality control test. Pier settlements and inclinations were continually monitored during the execution of the soilcrete piles.

The soilcrete piles were made by three-phase jetting. After having drilled down to the rock, the soil was cut from the bottom to the top by a high-pressure water jet followed by a cement injection. Retraction speed and jetting pressures for water and cement injection amounted to 1.1 mms^{-1}, 470 bar and 80 bar, respectively, and the injection material contained 690 litres of water and

870 kilogrammes of cement per cubic metre. The cement used allowed for early strength gain. The feasibility test had shown that soilcrete piles with a diameter of about 1 m could be expected, and thus, a spacing of 0.8 m between adjacent piles was prescribed. Pile production had to follow an alternating sequence to avoid concentration of fresh pile material. The main difficulty encountered during the execution was the adjustment of machine and injection parameters due to the variation of the clay content in the weathered molasse.

The quality of the soilcrete piles was checked by means of coring. While the average (minimum) values of the modulus of elasticity and of the compressive strength were found to be equal to 5 (2) GPa and 6 (2) MPa, respectively, the average density of the piles amounted to 16 to 20 kNm^{-3}. These values met the specifications and it was verified that the quality was sufficiently uniform throughout the soilcrete body.

3. NATURAL STONE MASONRY OF PIERS AND ABUTMENTS

In the 19th Century, most bridge piers and abutments were made of natural stone masonry according to the principle illustrated in Figs. 3(a) through (c). The circumferential masonry is responsible for carrying the loads. The inside core is an irregular and voided filling containing rubble and excess mortar from the construction of the exterior wall. Because of its low modulus of elasticity, the core does not carry any significant loads. The masonry blocks were placed on two strips of bed mortar having a 5 to 10 times lower modulus of elasticity than the stone material [Fig. 3(b)]. Joints between blocks were sealed with a weather resistant finishing mortar [Fig. 3(c)].

Examination of the condition of the piers and abutments revealed the usual damage due to weathering, i.e. deterioration of the outermost 100 to 200 mm of the joints, cracking of a few single blocks and overall wear of the surface. The exterior masonry wall has a thickness of about 750 mm and is made of limestone blocks with a compressive strength of 150 MPa. Due to the new superstructure, the average permanent stress in the exterior wall will increase from 0.35 to 1.6 MPa in the upper portion of the pier and from 1.4 to 2.4 MPa at its base.

The exterior walls have been prepared for the higher loads by reconditioning the existing mortar joint and by filling the voids in the mortar to allow for a smoother stress distribution in the joint. The head of the piers will be replaced by a massive 2.5 m high reinforced concrete block that transfers the forces from the bearings to the exterior walls. This concrete block also serves as a "roof" preventing water from entering into the pier.

Execution followed the steps illustrated in Figs. 3(d) through (f). First, the weathered surface was cleaned and the weak bed mortar, i.e. the outermost 100 to 200 mm, was removed by water jetting at a medium pressure. Second, the remaining mortar was supplemented by new bed mortar with similar mechanical properties. In the third step, remaining voids in the joints were grouted. Core grouting in Step 4 aimed at filling the voids immediately behind the masonry to achieve a back-anchorage of the wall in the core. In Step 5 the joints were sealed with a 20 to 30 mm thick layer of weather resistant finishing mortar. Where necessary, single masonry blocks were consolidated or replaced.

The requirements for the new bed mortar included a maximum value of the modulus of elasticity of 10 to 15 GPa and a limitation of the alkali-content of the cement to 0.6% to avoid crystallisation with subsequent damage in the stone material. Modern mortar and injection materials were used and adjusted to achieve the low modulus of elasticity required in the present application. In hindsight, the authors would have preferred to use more traditional materials, containing fewer additives and leaving less doubt regarding chemical compatibility with the natural stone masonry.

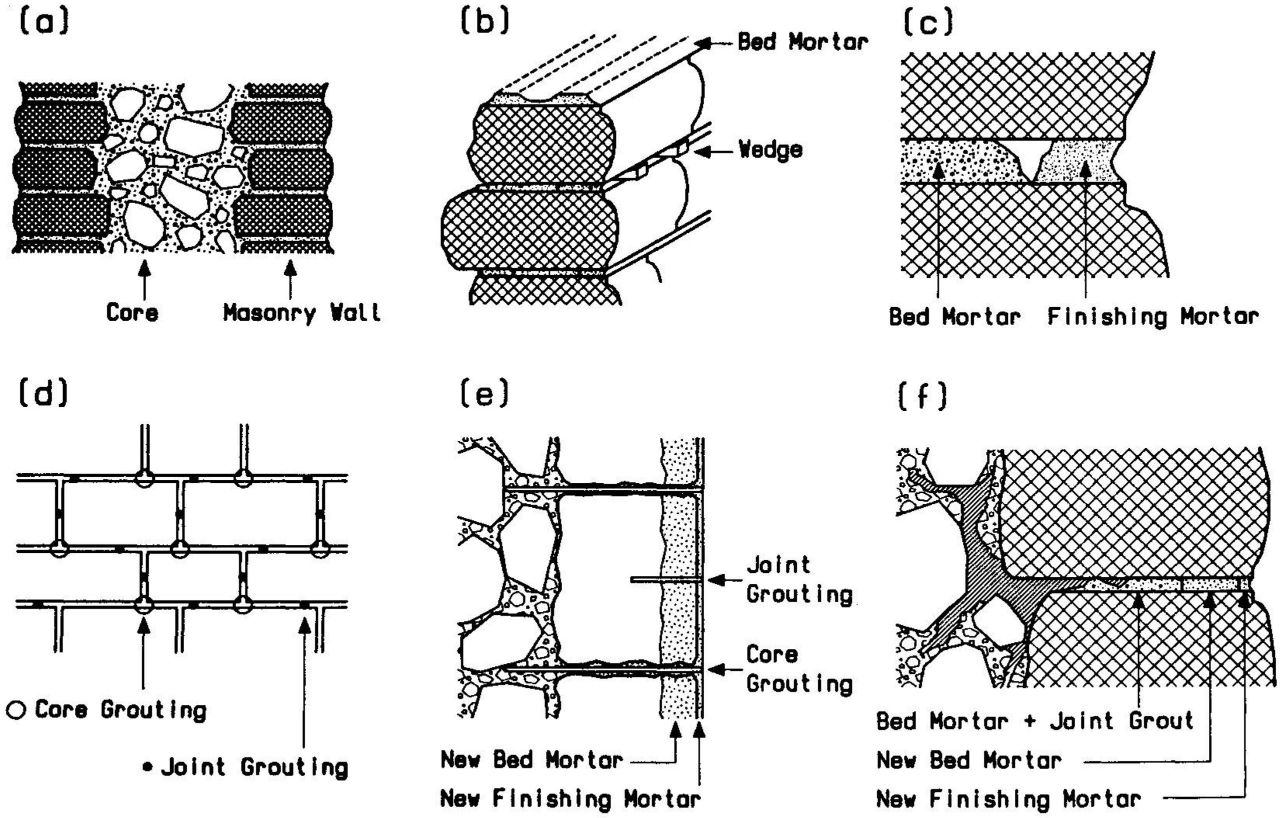

Fig. 3 Natural stone masonry: (a) Vertical section through pier; (b) Placing of masonry blocks; (c) Bed joint; (d) Elevation showing grouting locations; (e) Plan view; (f) Cross-section through bed joint.

Work was carried out in portions of about 40 m^2 of pier or abutment surface, alternating between front and side faces. The East abutment was used to gain experience and to refine the working procedures for the piers and the West abutment. The main difficulty was to find the right workability and technique to bring the new bed mortar into the joint. For quality control, cores were taken from the joints; remaining voids and mechanical properties of the new joint material were found to be satisfactory.

4. WEST ABUTMENT

Due to the change of the static system from a series of single span girders to a continuous five-span girder, the West abutment will be subjected to drastically increased horizontal loads. Breaking forces and friction forces induced in the bearings due to temperature changes are the primary actions in this regard. In passing, it is interesting to note that, compared to former provisions, the current design value for breaking forces of rail traffic has increased by 40%.

In spite of the congested space for the execution, [Fig. 4(a)] the West abutment was chosen as the fixed support of the bridge since it is of moderate height and founded on rock, while the pier-like East abutment is embedded in an earthfill dam.

After reconditioning the masonry, the abutment was strengthened by horizontally placed steel reinforcing bars to obtain a solid abutment block [Fig. 4(b)]. In addition, vertical steel reinforcement is placed to interlock the abutment block and the concrete block housing the bridge bearings. The required resistance to the horizontal design load of 11 MN, acting 13 m above the solid rock, is provided by the weight of 38 MN of the abutment block as well as by a number of inspectable and replaceable ground anchors with a total anchor force in service of 16 MN [Figs. 4(c) and (d)].

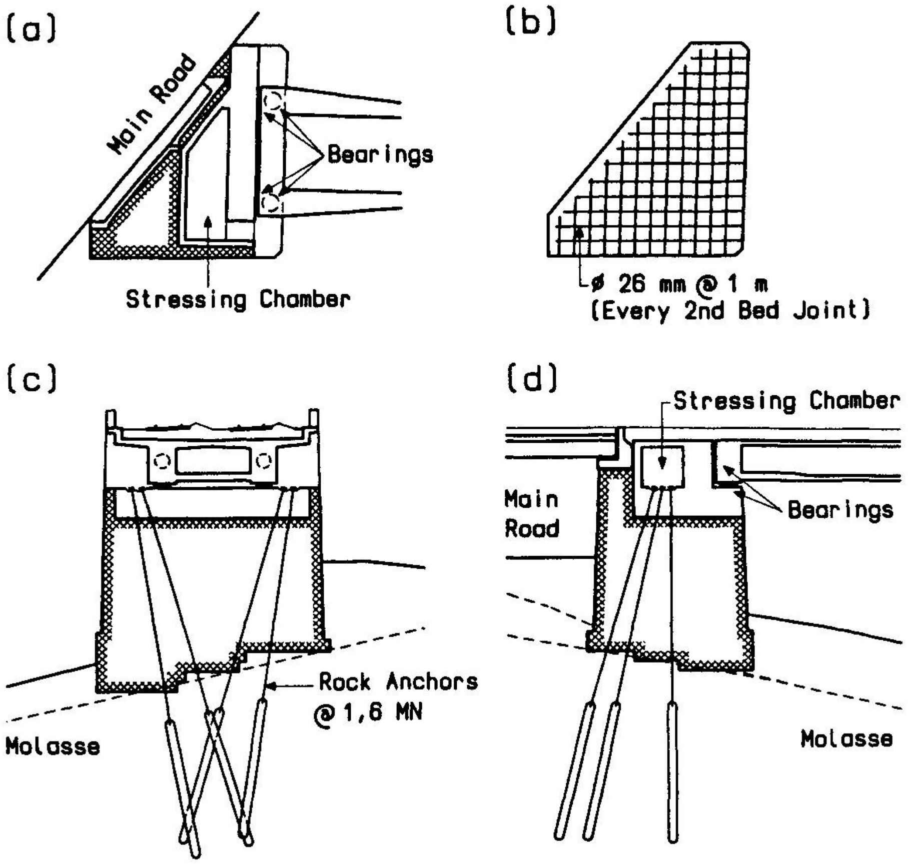

Fig. 4 West abutment: (a) Plan view; (b) Orthogonal reinforcement in bed-joints; (c) Cross-section; (d) Elevation.

5. CONCLUSION

The chosen project management has contributed much to the success of this project. The contractual arrangements between the client, the proof engineer, the consulting engineer (who was responsible for design and on-site management), and the contractors have allowed for the necessary close and efficient collaboration. In an area where no textbook solutions are available, a consensus based on engineering judgement is necessary among all partners in order to arrive at technically sound and cost effective solutions.