UPGRADING THE STABILITY OF THE EDER MASONRY DAM WITH PRESTRESSED VERTICAL ANCHORS

AMÉLIORATION DE LA STABILITÉ DE LA BARRAGE EDER PAR DES ANCRAGES PRÉCONTRAINTS PERMANENTS

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1 INTRODUCTION

The Eder dam was built between 1908 and 1914 as a curved gravity dam to provide Water for the Mittelland canal (fig. 1). The dam is founded partly on slate and partly on graywacke of the lower carboniferous. Graywacke and trass cement were used for the masonry. The dam, which is 36 m wide at the base and 6 m wide at the crest, is relatively thin. With its capacity of 202.4 Million m³ the Eder reservoir is one of the biggest reservoirs in Germany.

Height:	47 m
Base length:	270 m
Base width:	36 m
Crown length:	400 m
Crown width:	6 m

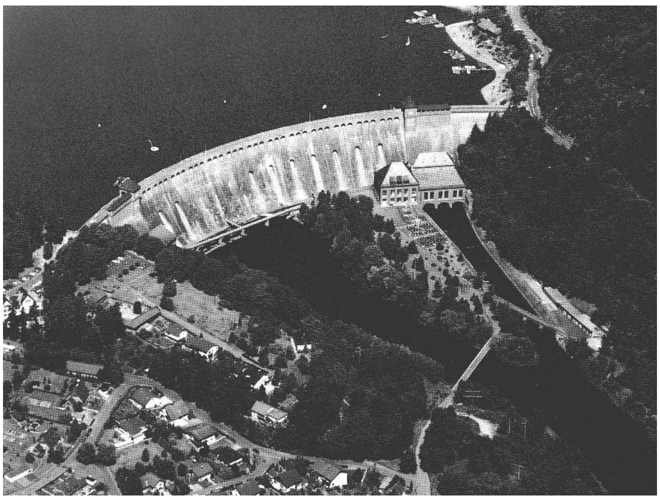

Fig. 1: The Eder masonry dam

As for many gravity dams built in Germany around the turn of the century, the absence of pore water pressure in the dam foundation was assumed in the original design. This assumption, however, cannot be upheld based on today's knowledge (fig. 2). Achieving stability was therefore only possible through either restricting the storage level or using other measures to improve stability. Because the original water level in the reservoir was to be maintained, extensive measures became necessary. The works started in October 1991 and were finished on May 6, 1994.

- foundation pore water pressures not taken into account in original design in 1908
- spillway now has to allow for
  HQ 1000 = 1100 m³/s instead of
  HQ 100 = 590 m³/s previously

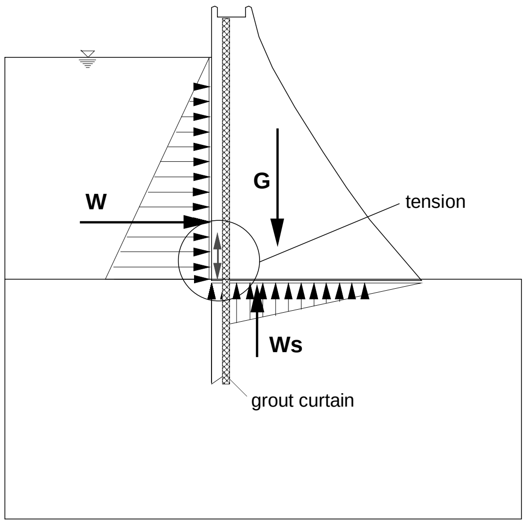

Fig. 2: Reasons for remedial measures

2. RESTORATION CONCEPT

After careful studies of several design alternatives, a solution involving 104 permanent rock anchors was given preference, whereby the dam would be anchored into the bedrock (fig. 3). To distribute the anchor forces, a load distribution beam was needed on the dam crest. The chosen type of anchor was the Stump/VSL prestressed anchor which carries a design load of 4575 kN. The design load on each anchor in the area between the superstructures was calculated as 4500 kN, with an average spacing of 2.25 m. The length of the load transfer section of the anchors in the rock was chosen as 10 m.

To construct the load distribution beam at the top of the dam, the existing crest had to be dismantled. At the same time the spillways on the dam were brought into line with the relevant standard (DIN 19702). This standard requires the dam to be able to cope with the 1000 year flood event, calculated in this case to be 1100 m³/sec. The existing spillway had been designed for the 100 year event of 590 m³/sec.

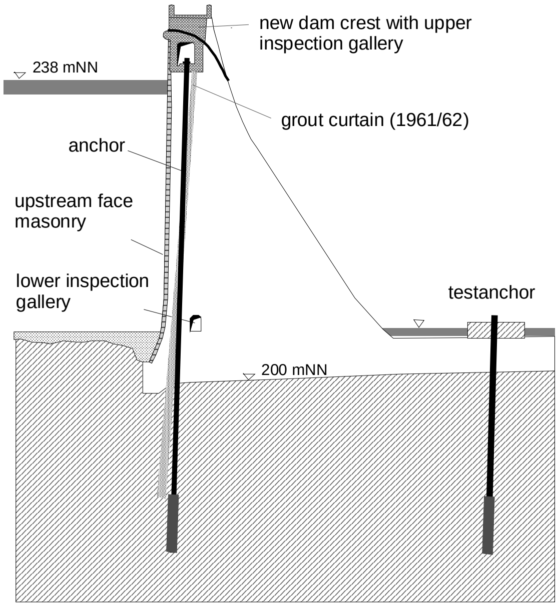

Fig. 3: Cross-section of the dam

3 INSTRUMENTATION

Instrumentation installed in this dam during construction in 1908 included 12 water pressure gauges and 12 temperature gauges. The horizontal movements of two points on the crown were monitored geodetically. Further instrumentation was added in later years, especially in 1984, when the following devices were installed (fig. 4):

3 7-fold and 5-fold, resp., inclined extensometers, located below the reservoir with their heads installed on the downstream side of the dam and in the cross adits, resp.

2 plumb lines, located between the crown and the inspection gallery

2 inverted pendulums, located between the inspection gallery and the bedrock

6 sliding micrometer tubes in 3 cross-sections.

3 reservoir water temperature gauges

36 dam temperature gauges

1 air temperature gauge

40 pore water pressure gauges located in the masonry dam and the bedrock.

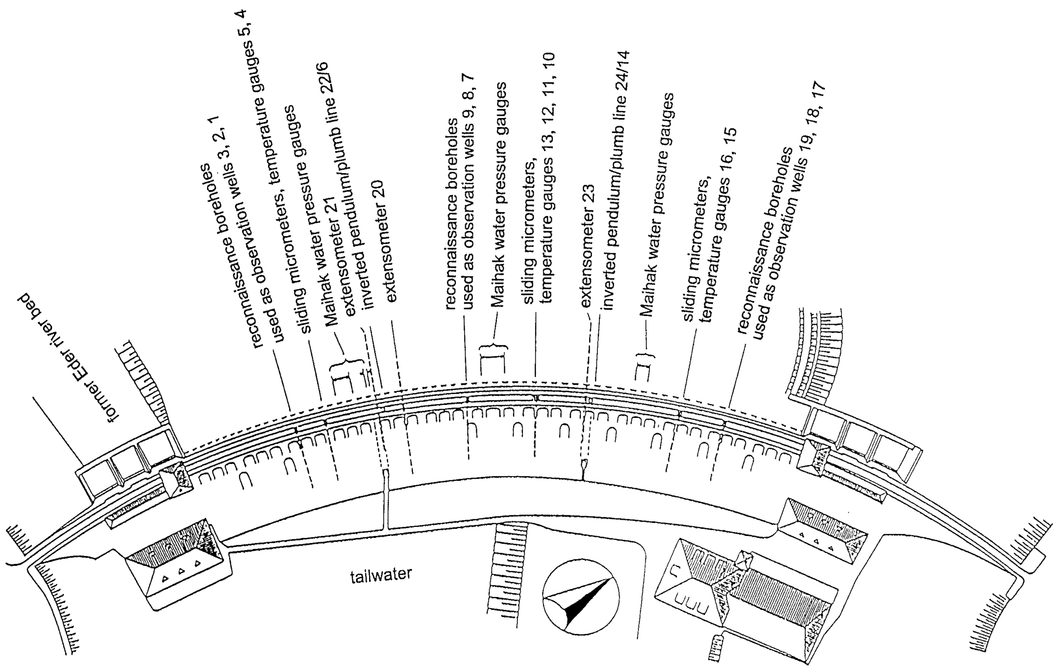

Fig. 4: Dam plan with measuring devices in cross-sections

4 STABILITY ANALYSES

To enlarge on the calculations done by the Federal Institute for Water Engineering (Bundesanstalt für Wasserbau, BAW) using beam theory [2], Prof. Dr.-Ing. W. Wittke Geotechnical Engineering Consultants (WBI) carried out finite element (FE) analyses. These served as an aid to the interpretation of the measured water pressures and displacements in the dam in order to thus help determine the parameters for dam and bedrock. In addition to this, FE-analyses were also carried out as a verification of stability analyses performed by the client [5]. These analyses will be explained in the following sections.

4.1 Computation Method

The computation of the stresses and strains in dam and bedrock were carried out with the program system FEST03 described in detail in [8]. For the dam and the surrounding rock an elastic-viscoplastic stress-strain relationship was assumed. Anisotropy in the elastic and viscoplastic range can be taken into account. For the intact rock as well as for the discontinuities of the rock, the Mohr-Coulomb failure criterion is applied, limited by a vertical line in the tension region of the τ–σ- graph ("tension cut-off"). The latter assumption is also relevant for the horizontal joints in the dam. The influence of the joints is taken into account by reducing the shear and tensile strengths parallel resp. orthogonal to the discontinuity in the finite elements in the respective areas.

The most commonly used type of element is the three-dimensional isoparametric element, the number of nodal points of which can be varied between 8 and 21. Rock is usually simulated using an element with eight nodes, while curved structures, such as arched dams, are modelled using more nodes. For the modelling of arched dams the application of twenty-node elements has proved necessary.

The piezometric levels resulting from seepage flow through dam and bedrock were calculated with the program system HYD03. The basis of these calculations is Darcy's Law. Anisotropic permeabilities resulting from the orientation of the discontinuities can be taken into account. Variations in permeability within the computation section as well as the topography and the geometry of the construction project can be modelled as well.

The calculations result in the piezometric head distribution within dam and bedrock. Furthermore uplift and seepage forces for use in stress-strain-analyses (see above) are derived from these computations.

4.2 Two-dimensional FE-Analysis for varying water levels

4.2.1 Area of Calculation, Element Mesh and Calculation Steps

As a verification of the existing stability analyses for the dam in the area between the gate superstructures [3], two-dimensional FE-analyses were carried out. Since no three-dimensional effects are present due to the considerable length of the dam and the large radius of curvature, the calculations could be carried out in one plane.

The 1m thick slab used for the calculations includes a 300 m long and 159 m high section of the bedrock as well as the 47 m high dam (fig. 5). The FE-mesh described here has 2757 isoparametric elements. As mentioned earlier it was used for the hydraulic as well as the mechanical calculations.

The analysis for the load cases dead weight, anchoring and filling of the reservoir was carried out in three steps (fig. 6). In the first step the stresses and strains resulting from the dead weight of the bedrock were calculated, taking uplift into account. The uplift forces necessary to account for the ground water conditions were determined in the hydraulic calculation. The second step simulated the construction of the dam and the installation of the pre-stressed anchors. The third step then considered the uplift and seepage forces resulting from seepage through dam and bedrock for varying storage levels.

In the section of the element mesh shown in fig. 5 the fine meshed modelling of the dam on the up and down stream sides can be seen. The purpose of this is to calculate as accurately as possible the stresses in these areas with large stress gradients. The inspection gallery and the elements used to simulate the low permeability grout curtain are also shown. The effect of the prestressed anchors was simulated by means of two equal and opposite forces with the same line of action, one of which acting on the rock, the other in the upper inspection gallery (fig. 5). This inspection gallery was not included in the FE-mesh as the aim was to determine the overall stability of the dam and not the local stress distribution in the vicinity of the dam crest.

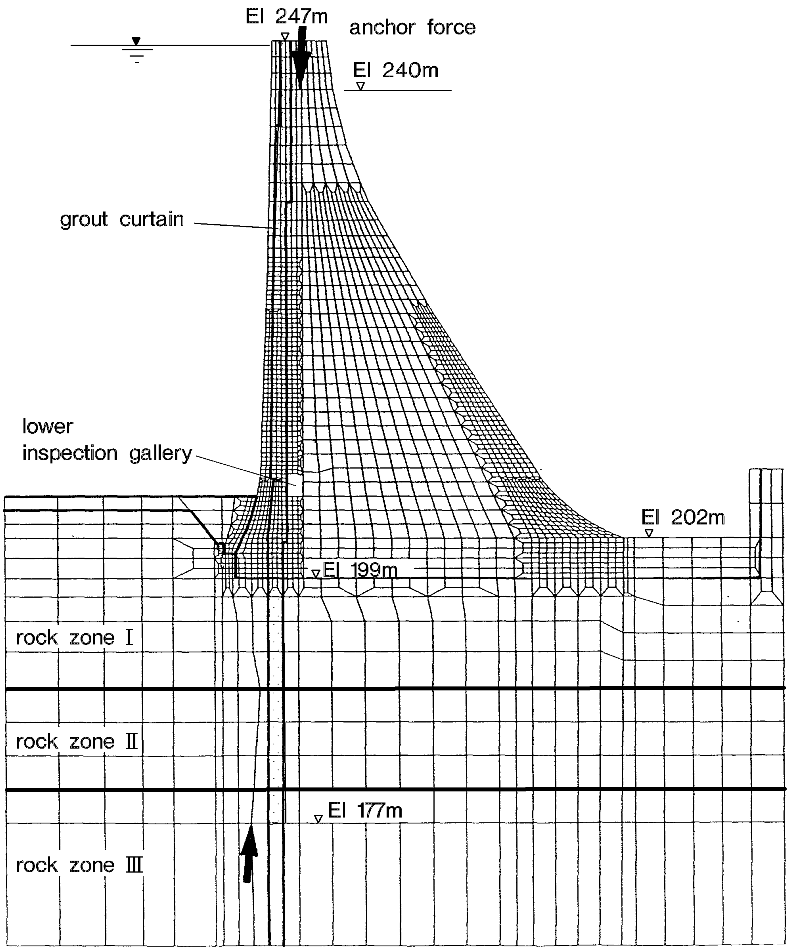

Fig. 5: Detail of the 2D finite element mesh

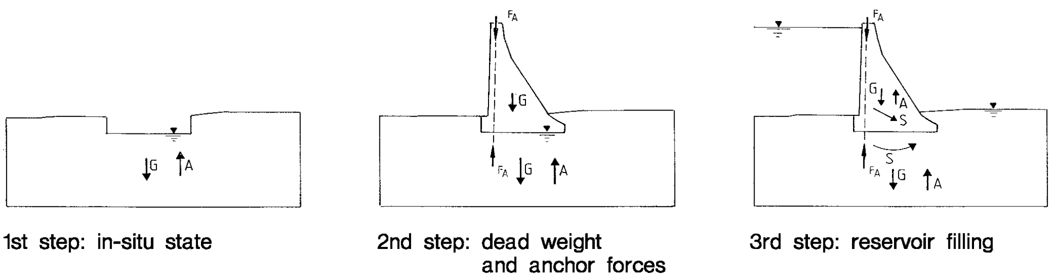

Fig. 6: Computational steps

4.2.2 Parameters

In the area of the Eder masonry dam the rock in the valley and on the valley slopes consists of carboniferous slate and quartzite. The rock strata on the right-hand side of the valley consist predominantly of slate, while quartzite prevails on the left-hand side. The bedding dips at angles between 65 and 75 degrees and strikes approximately from the upstream to the downstream side. Apart from the beddig parallel discontinuities, two further joint sets exist in the rock.

The permeability of the ground can be considered approximately isotropic in relation to seepage flow under and around the dam [2]. According to the results from core drillings and Lugeon tests carried out during the rehabilitation the bedrock can be divided into three zones (I-III) with respect to its permeability (figs. 5 and 7).

	permeability	deformation		dead weight
	k_f [m/s]	E [MN/m³]	ν	γ [kN/m³]
rock zone I	1×10^{-5}	5000	0,25	27,2
rock zone II	1×10^{-6}	5000	0,25	27,2
rock zone III	1×10^{-7}	5000	0,25	27,2
masonry	1×10^{-6}	7500	0,25	23
grout curtain	5×10^{-7}	7500 / 5000	0,25	23 / 27,5

*) dam-parallel discontinuities: $\sigma_t = 0$, c=100 MN/m², φ=30°
**) horizontal joints: $\sigma_t = 0$, c=100 MN/m², φ=30°

Fig. 7: Parameters for seepage flow and stability analyses

The dam is also considered to be isotropic and homogenous with respect to its permeability. The permeability assumed for the grouted area is within the range of permeabilities usually achieved with standard cement grouting. The grout curtain connects into zone III of the rock foundation. With respect to their elastic behaviour, the quartzite layers can be considered isotropic, whereas for the slate this is only an approximate assumption. The value of Young's modulus for the rock in fig. 7 results from back analysis and an interpretation of measured displacements for different water levels (fig. 8). Isotropic behaviour was also assumed for the dam in the elastic range.

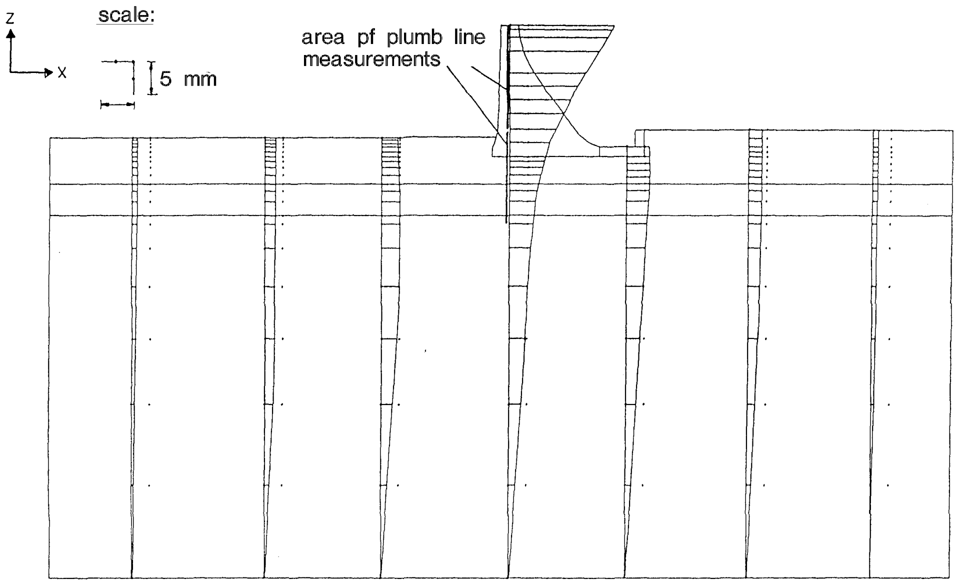

Fig. 8: Horizontal displacements due to raising the storage level from 217 to 241 m above MSL

For the rock discontinuities parallel to the dam, i.e. at right angles to the plane of fig. 5, and for the horizontal joints in the masonry dam it was assumed that the tensile strength perpendicular to these planes is zero (fig. 7). Using these assumptions, the weakening effects of the joints in the rock as well as the horizontal planes of weakness in the dam were taken into account.

4.2.3 Results

In the first case examined the reservoir is empty and the only loading is the dead weight of dam and rock and the anchor forces. Fig. 9 shows the vertical stresses in horizontal sections. Assuming that the tensile strength at right angles to the sections is zero, cracks occur on the downstream side of the dam due to the anchor forces. These cracks reach approximately 3,5 m into the masonry dam and are unsignificant with respect to dam stability. In the same way, the compression stresses of 1.6 MN/m^2 on the upstream side of the dam are easily sustained by the masonry.

For the filling of the reservoir to the maximum storage level, it has to be absolutely ensured that the masonry dam is stable and the sealing remains functional. The equi-potential lines resulting from the seepage flow calculation are shown in fig. 10. The draining effect of an inspection gallery located at the foundation level on the upstream side and the effectiveness of the grout curtain were taken into account. The analysis lead to the result that the water pressure on the foundation on the downstream side of the grout curtain amounts 40% of the hydrostatic pressure (fig. 11). The seepage forces from this analysis together with the loading due to dead weight and prestressed anchors produced the stress distribution shown in fig. 12.

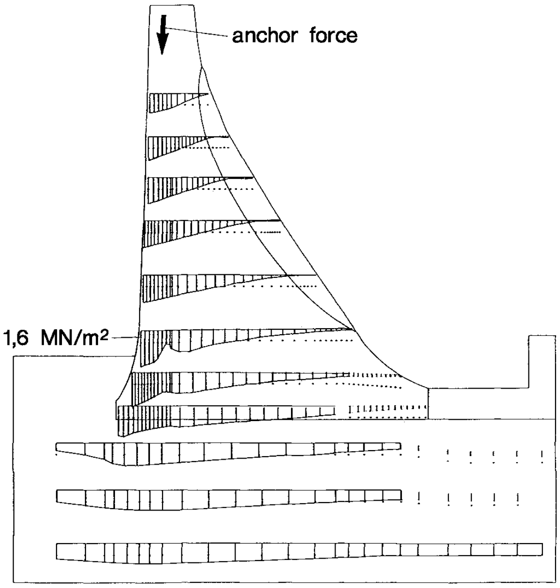

Fig. 9: Vertical stresses due to dead weight and anchor forces

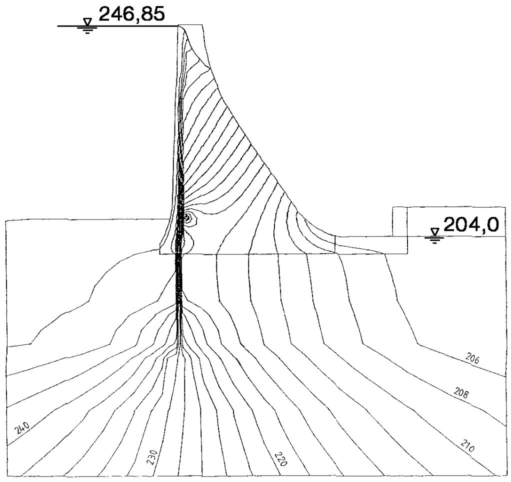

Fig. 10: Potential distribution: storage level at el. 246.85m (maximum storage level)

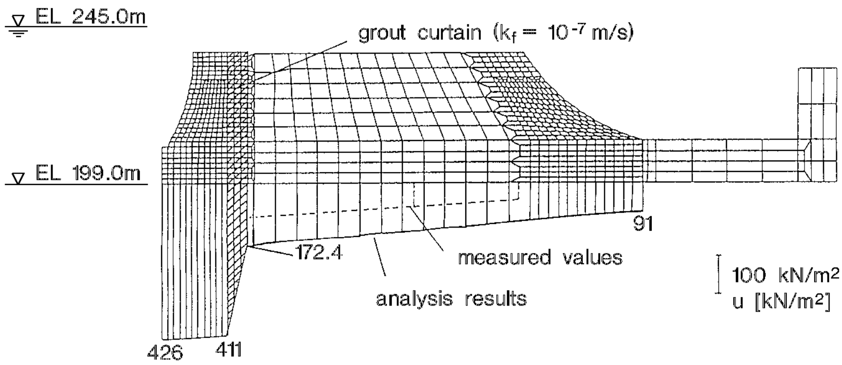

Fig. 11: Seepage analysis: Distribution of water pressure in the dam foundation area

As mentioned earlier, the masonry cannot sustain vertical tension. Horizontal cracks therefore develop at the upstream side of the masonry dam base. However the area where these horizontal cracks occur is relatively small. Since the cracks do not reach the grout curtain, its sealing function is not affected, and restrictions on the use of the dam do not have to be applied. The maximum compressive stress of 1.6 MN/m^2 can easily be sustained by the masonry.

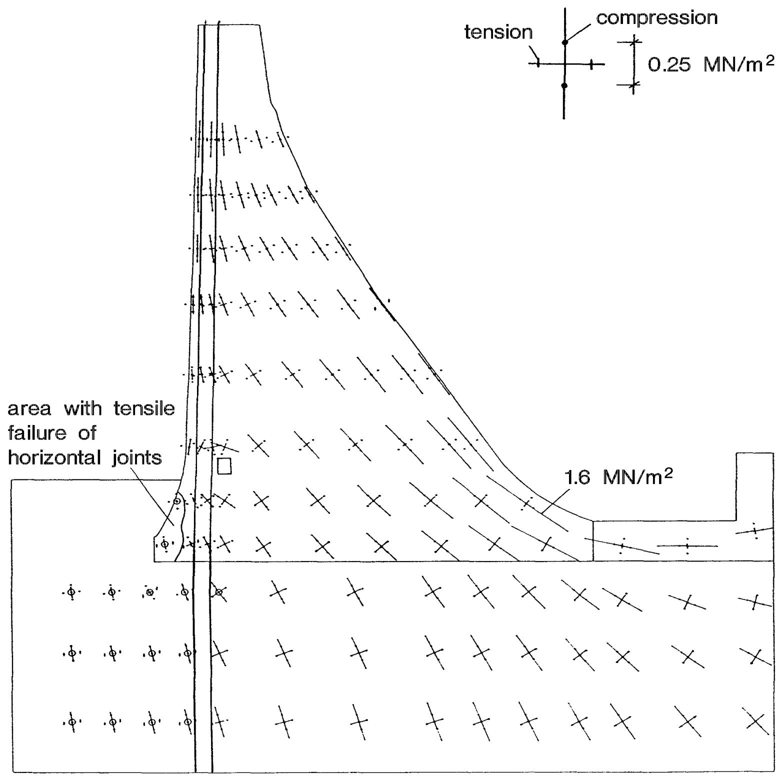

Fig. 12: Principal normal stresses: dead weight, anchor forces, storage level at el. 246,85 m

4.3 Investigation of the Load Bearing Behaviour in the area of the Valley Slopes

On the dam crest above the toes of the two slopes there are two 37 m long gate superstructures. In the vicinity of these superstructures and the adjacent slopes no prestressed anchors are located. It therefore had to be determined, wether the anchors in the central part of the dam are sufficient to also reduce the tensile stresses in the slope areas. To investigate this, three-dimensional FE-analyses were carried out using the mesh shown in fig. 13. It consists of 3673 isoparametric elements. The length of the analysis region is 280 m, the width 232 m and the height 147 m. The masonry dam is modelled in the area of the slopes, the superstructures and over a length of 50 m in the central part. This section of 50 m length was chosen long enough to prevent effects from the slope from having an influence on the central edge of this part. The small curvature of the dam in plan was neglected in creating the FE-mesh.

The three-dimensional calculation was also carried out in three steps (fig. 6). For the calculation of the seepage flow under and around the dam, the maximum storage level of 246,85 m (HQ_{1000}) was assumed because this loading case had turned out to be the worst case in the two-dimensional analyses. The assumptions about the effectiveness of the grout curtain correspond to those from the two-dimensional analysis. The draining effect of the horizontal gallery was neglected in the calculations.

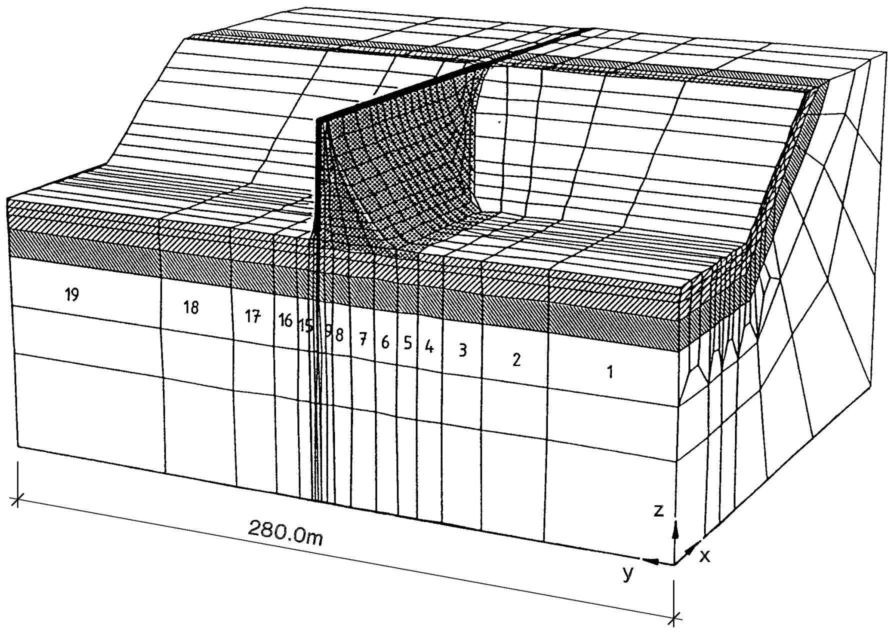

Fig. 13: 3D finite element mesh

The three-dimensional load bearing behaviour is easiest to recognize in the results of the second analysis including only the dead weight of the dam and the anchor forces. Fig. 14 shows the size and orientation of the principal stresses within slice 11 (fig. 13), located close to the upstream side of the dam. Comparably large principal stresses occur in the lower slope area resulting from the dissipation of the loading from the dead weight of the dam and the anchor forces. Due to these increased compressive stresses, there are no tensile stresses reaching the grout curtain even at maximum storage level.

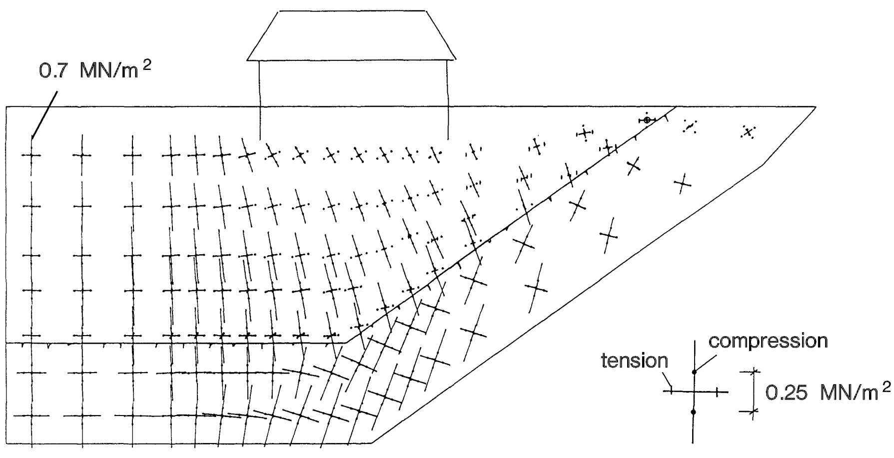

Fig. 14: Principal normal stresses: dead weight and anchor forces, longitudinal section on the upstream side of the wall

5 METHOD OF CONSTRUCTION

To construct the load bearing beam; a section of the original dam crest was separated from the body of the dam by gap blasting. The parts that were to be removed were prepared for demolition by loosening blasting. The downstream facade remained in its original form. In order to retain the old sand stone arcs above the spillways. In the next step, the load bearing beam and the upper inspection gallery were constructed. After completion of the overflow crest the piers for the road were concreted. The sand stone parapet was, as far as possible, reconstructed with the original stones. Finally the front on the upstream side of the crest was faced with quarry-stones.

6 MANUFACTURING AND PLACEMENT OF ANCHORS

6.1 Preliminary field tests

From December 1991 to February 1992 extensive preliminary field tests were carried out in order to determine the suitability of the anchors and to investigate the transfer of prestressing forces into the bedrock.

In the stilling basin of the masonry dam three anchors were placed on the right side of the valley in an alternating sequence of graywacke and claystone, while three further anchors were placed into graywacke strata on the left side of the valley.

The anchors were subjected to a suitability test, including pull-out-tests with a maximum loading of 8000 kN. The outcome confirmed the suitability of the anchors for the envisaged construction measures.

6.2 Drilling and Grouting Works

The preparatory drillings for the anchors to an alternating depth of 68 and 73 m were carried out from the newly constructed crest as wire line core drillings (ø 146 mm) using two double tube core barrel drilling machines (fig. 15).

As the thickness of the masonry dam amounts to only about 2,5 m between the existing face liner wall and the lower inspection gallery, the client demanded the borehole deviations less than 1 % of the borehole length at the level of the lower inspection gallery. This means that after 40 drilling meters the actual drilling center line was to deviate no more than 40 cm from the envisaged center line. Typical for core drilling boreholes deviations of 2 - 3 % are expected. The contractor achieved an average borehole deviation of 0,36 % at the level of the lower inspection gallery and of 0,45 % at the deepest point of the borehole.

After drilling, the boreholes were cement grouted. In total, 100 t of cement were injected into the masonry dam and the rock during 2000 h of grouting works. In the next step, the injected boreholes were re-drilled with a roller bit and subsequently expanded from 146 mm to 273 mm by means of a down-the-hole hammer. The hammer head was equipped with a pilot spike in order to ensure the adherence to the original drilling center line. After the expansion, the treatment of the boreholes was examined by means of Lugeon tests. An absorption capacity of less than one liter per minute under a pressure of 100 kPa for all of the bonding length of 10 m was used as treatment quality. Only 8 out of 104 borings in which this was not achieved had to be injected again.

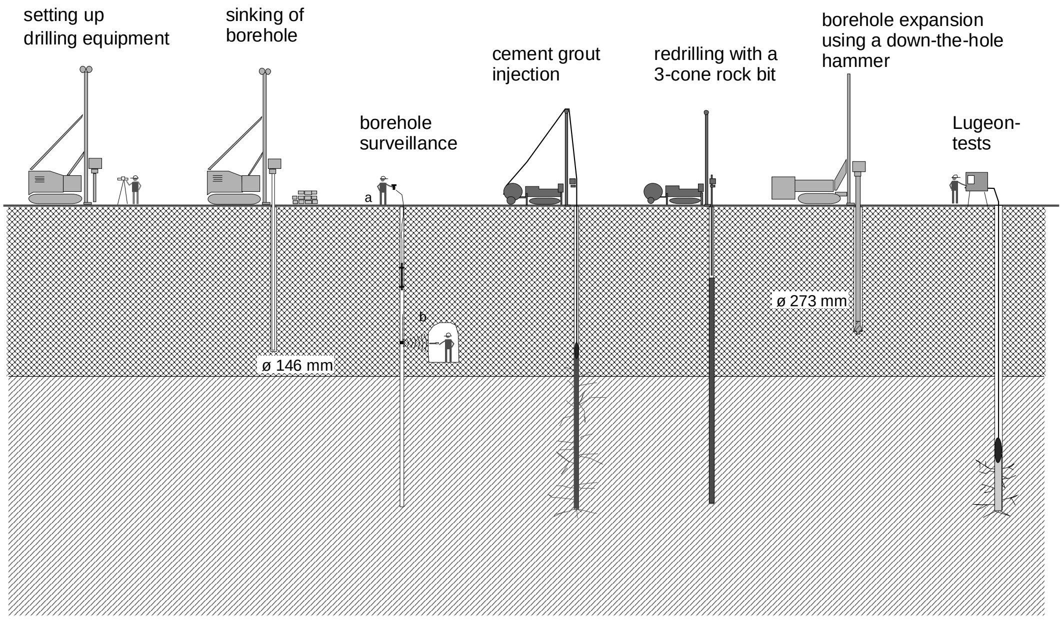

Fig. 15: Borehole procedure as a preparation for the prestressed anchors

6.3 Anchor Production

The most important elements of the reconditioning works are 104 permanent rock anchors as described before. With this anchor type the anchor forces are transmitted from the anchor head via a load bearing beam made of reinforced concrete into the masonry. At the anchor foot, the forces are transmitted by the grouted bonding section of the anchor into the bedrock. The load transfer from the anchor head to the anchor foot is effected by 34 wire strands, ST (steel quality) 1570/1770 with a 150 mm^2 nominal cross-sectional area each (fig. 16).

In the free moving anchor section each individual strand is enclosed in a mantle consisting of anti-corrosive grease and a plastic coating (PE). The steel strands are inserted in a 11.4 mm strong, smooth tube (ø 200 mm) in the free moving anchor section and in a ribbed PE-tube in the bonding section. The transition between the smooth PE-tube and the ribbed PE-tube is supported on the inside by a steel socket and sealed by a shrunken-on plastic tube from the outside.

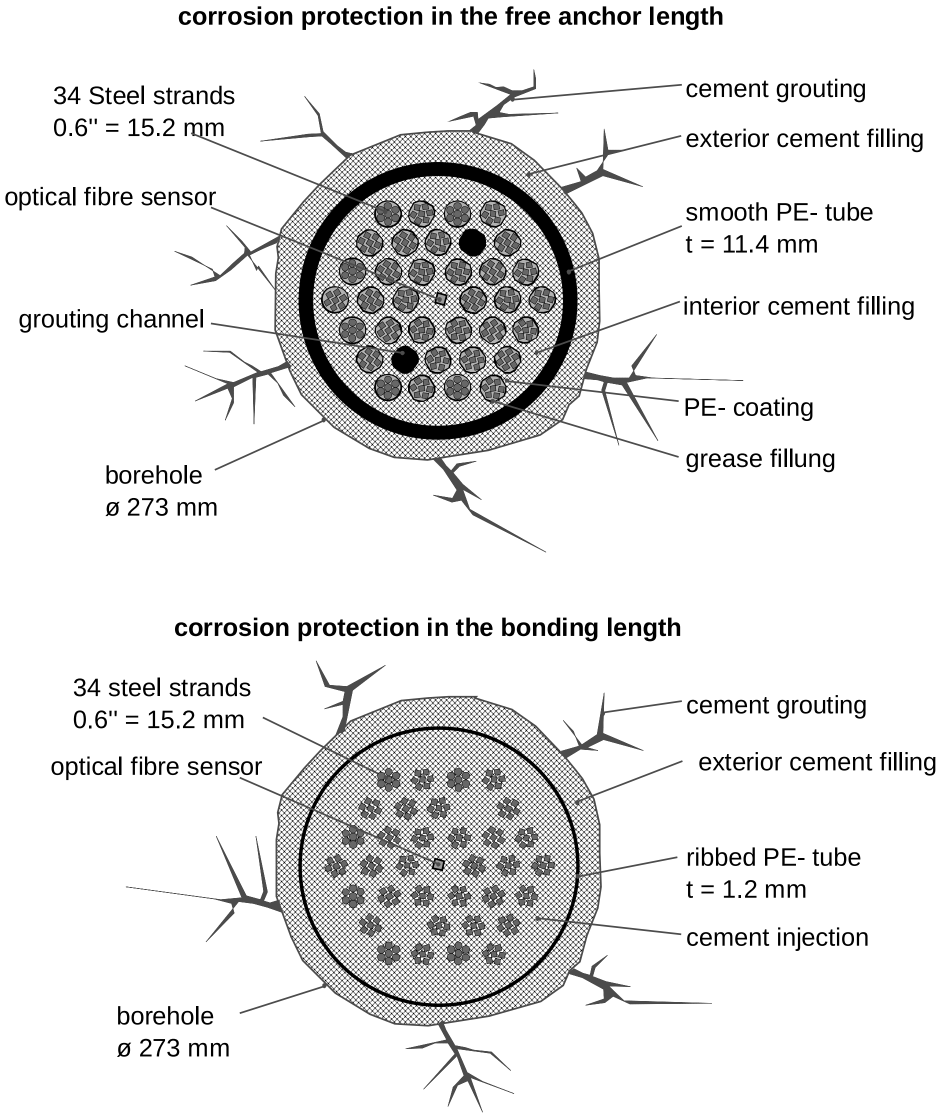

Fig. 16: Anchor cross-sections

As a length of 70 and 75 m and a weight of about 4 t prevented the anchors from being transported by lorry or train, they were assembled on site (fig. 17).

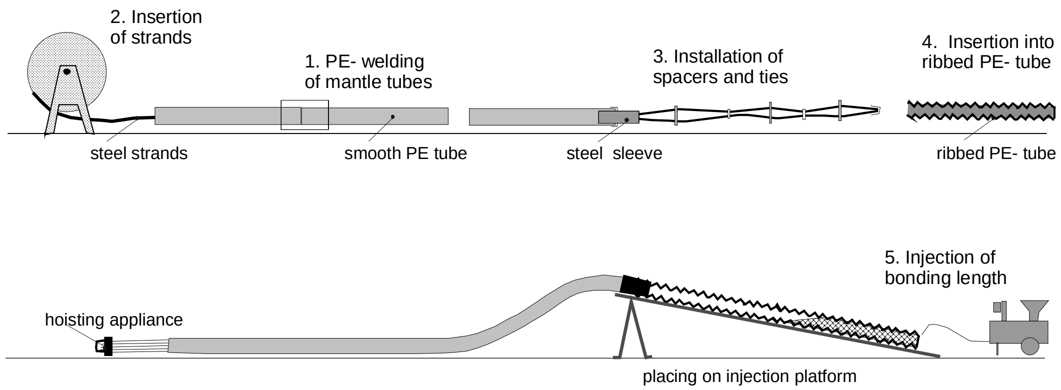

Fig. 17: Anchor manufacturing scheme

6.4 Transporting, Installation and Tensioning of the Anchors

From the production stage to the installation of the anchors the bonding section was protected against mechanical damages by an additional foil coating. The anchors were moved to the drilling site on the crest by means of rollers and subsequently placed in the borehole by means of a mobile crane and an installation frame that had been developed specifically for this purpose (fig. 18). The tensioning of the anchors was carried out from the upper inspection gallery by means of a hydraulic jack.

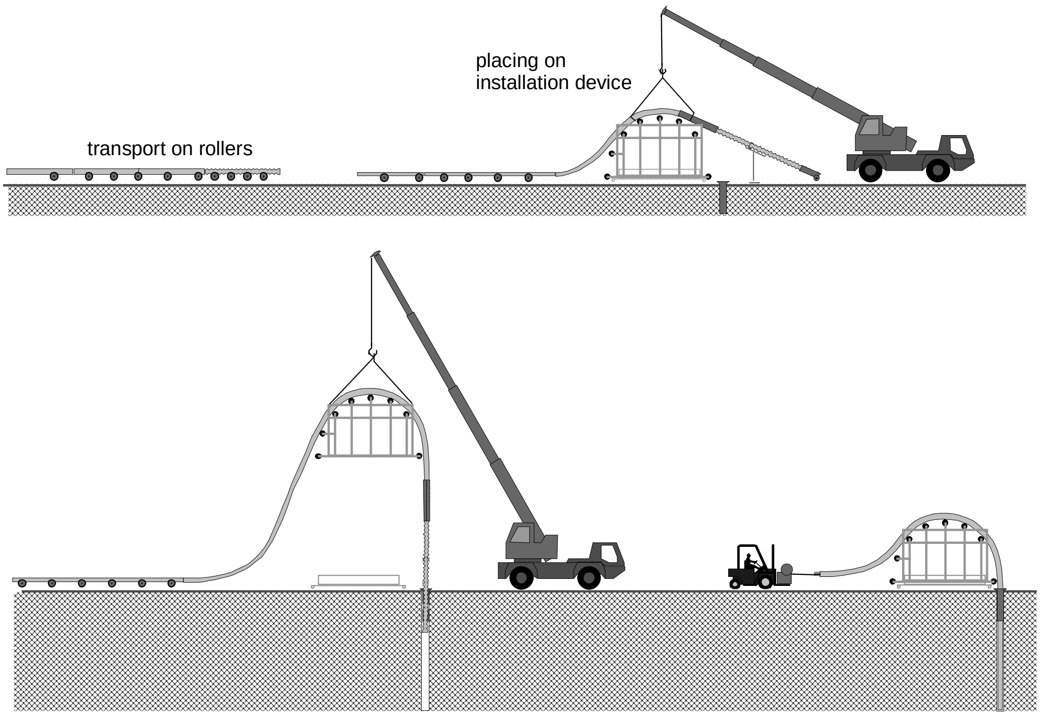

Fig. 18: Anchor transport and installation

7. CONCLUDING REMARKS

The anchoring of the dam with prestressed anchors in the bedrock results in the masonry dam stability being in line with current requirements for a future design life of 80-100 years. Two- and three-dimensional stress-strain and seepage flow FE-calculations confirmed that applied anchor forces of 2000 kN/m reduce tensile stresses in the dam to tolerable values. The reconstruction project proved that high-tension anchors can be produced under site conditions. This presupposes an expert and diligent execution of all steps as well as constant supervision up to the very end of construction.

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Summary

The 47 m high Eder masonry dam was built of graywacke quarry-stones and lime trass mortar in 1908-1914. Some years ago the dam stability was reinvestigated which the consequence that the storage level had to be lowered. Prestressed permanent anchors, reaching from the top of the dam down to approximately 30 metres into the bedrock, were installed in order to achieve dam stability according to todays standards. The anchors, placed with 2,25 m spacing, were prestressed with an anchorforce of 4500 kN each. The paper describes the stability analyses and the installation of the prestressed anchors.

Resumé

Le barrage Eder (47 m de hauteur) est édifié comme mur en pierres brutes de carriére de grauwacke de 1908 á 1914. Une investigation du barrage a montré qu´on ne peut plus prouver la stabilité pour les charges supposées actuelles et au niveau actuel des consignes de sécurité. Afin d´élever la stabilité, on a installé des ancrages permanentes, qui descendent du couronnement jusqu´ à 30 m dans la fondation rocheuse. Les ancrages ont un éspacement de 2,25 m et un effort de précontrainte de 4500 kN. Dans l´article, des calculs statiques à la méthode des éléments finis sont présentés et le procédé de l´installation des ancrages est décrit.