HARCOV DAM – ENSURING SAFETY DURING FLOOD

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ABSTRAKT: The article deals with the technical solution and experience obtained from carrying out reconstruction works on the Harcov hydraulic structure (VD) in Liberec. The structure was built between 1902 and 1904 mainly in order to reduce flood flow rates and to provide partial protection for the area below the reservoir against the effects of floods. After more than a century in operation, it became necessary to proceed with a complete reconstruction of the hydraulic structure with regard to meeting valid requirements for the capacity of all structures for channelling water, mainly to handle a control flood surge. An additional challenge was the long-term increase in buoyancy in the foundation base of the dam and seepage appearing on the downstream face. The reconstruction of the VD Harcov dam and its structures is currently underway, during which a fundamental change in the parameters of this historic hydraulic structure takes place.
It consists mainly of increasing the capacity of the lower sluiceways and emergency crest spillway, including modifying a cascade below the spillway. Then, a fundamental measure to reduce buoyancy effects on the dam body is the sealing of the subsoil using a grout curtain.

1. DAM CONSTRUCTION HISTORY

The Harcov hydraulic structure is part of a system of dams for the regulation of outflow ratios in the Lužická Nisa river basin. The Harcov dam, constructed within the Liberec land registry, was built from 1902 to 1904, making it is the oldest one from a series of dams in Jizerské hory. The impetus for the construction of this dam stemmed from multiple catastrophic floods at the end of the 19th century, especially a flood on the 30th of July 1897, during which the highest flow rate recorded in the Harcov brook in Liberec was 20m3·s−1. A masoned, gravity, horizontally arched dam 19m in height is located on the Harcov brook. The design of the hydraulic structure was prepared by Dr. Ing. Otto Intze from Aachen, who was also the main supervisor of the construction. The construction was carried out by companies W. Streitzig of Liberec and H. Rell of Vienna. Construction commenced in November 1902 and concluded in December 1903. The final inspection was conducted on the 29th of April 1904. The dam was fully completed in June 1904. April 1904. The dam was fully completed in June 1904.

Fig. 1 The dam after completion - 1904

2. MOTIVE FOR RECONSTRUCTION

The goal of the reconstruction of the entire hydraulic structure is to safely channel PV10000 extreme floods (surge wave for a flood with an intermediate return period of 10,000 years) over the crest of the

dam, ensuring that the safety and stability. of the hydraulic structure would not be compromised. On top of not allowing the increase in the height of the limit safe level (MBG), there is also an increase in the uncontrolled retention space.

As part of the reconstruction, significant modifications of individual sections of the hydraulic structure are ongoing. More precisely, the dam crest, the upstream and downstream faces, the catchment area and underneath it, and on technical safety monitoring (TBD), technology, and monitoring devices.

The implementation of a set of designed measures will increase the capacity of all structures used for water fl ow control (lower sluiceways and safety spillway) while simultaneously securing the stability of the dam even in case of water fl owing over its crest (reducing buoyancy in the foundation base by sealing the subsoil with grout curtains constructed from a grouting corridor at the upstream heel of the dam, restoring the sealing function of the upstream face, contact grouting of dam masonry, reconstruction of the dam crest. The measures will significantly improve the conditions for channelling "common" floods with a shorter recurrence time. Concurrently, the safety of a category II hydraulic structure during floods will be secured. After realising the measures, a reallocation of reservoir space, with an increase in volume of the "emergency" retention space (increasing the limit safe level) and an option to more effectively utilise the controllable retention space

will occur. The purposefulness of the investment (overall reconstruction of the historic work) will further prove itself by an increase in operational reliability, extended service life of the structure, and reduction in costs for small repairs, all while preserving the historic character of the structure. Throughout preparation and realisation of the structure, information about the state of the masonry and subsoil of the dam will likewise be added on top of improving conditions for the monitoring of the structure within the TBD. The operator and also investor of the VD Harcov reconstruction is Povodí Labe, a state enterprise.

3. PREPARATION

Following catastrophic flooding in 2005, it was decided to review the approach to the safety of hydraulic structures in the Czech Republic. The outcome of this review is that VDs should be able to contain and channel a PV10000 flood. Therefore, for the Harcov hydraulic structure in Liberec, an expert assessment of hydraulic structure safety during floods was made in 2006 (J. Chroumal 2006). The base for the assessment was hydrographs of theoretical flood surges with a return period of 10,000 years (PV10000), deducted from single-day, or rather two-day precipitations established within a hydrological study from ČHMÚ (R. Tyl, M. Boháč 2005). It was clear from the results of transformed flood surges that the protective space of the reservoir cannot transform the theoretical flood surges, and in both cases, the dam crest will overflow by 70 and 20cm, respectively. The main reason for the negative result of the assessment was insufficient capacity of structures meant for water channelling, which are not sufficient even for transferring less extreme flows with values smaller than the theoretical PV100. Both the long-time problem of increased buoyancies in the area of the foundation base and planar moisture on the downstream face due to a seepage in the masonry needed to be addressed.

To secure the safety of this historic hydraulic structure during extreme floods, on top of the safety and long-term serviceability in normal operation conditions, it was necessary to proceed to designing measures that would increase the stability of the dam body and capacity of structures for flood flow channelling. The technical design, effect, and expected financial expenses were elaborated in more variants in the Studie opatření (Study of measures) from 2008 by the VD-TBD Company (D. Richtr, T. Klemša 2008). Results of executed stability calculations proved that the safety factor of movement and overturning of the dam body does not reach values required by a norm with water surface at the dam crest. On the basis of discovered realities, the study also recommended, besides other things, an increase in the capacity of the crest safety spillway by decreasing the level of the spillway area, and a reduction of buoyancies underneath the dam by way of subsoil sealing using a grout curtain. In 2010, the above-mentioned measure for the increase in safety spillway capacity was verified by a physical hydraulic model constructed by the Faculty of Civil Engineering, CTU in Prague (L. Satrapa, a kol. 2010).

Then, an engineering geology survey (IGP) was conducted in 2012 by the AZ CONSULT Company (K. Alföldi a kol. 2012). An assessment and selection of a suitable variant for the reconstruction of the hydraulic structure and rehabilitation of the dam subsoil was conducted in 2015 by the VD-TBD Company (D. Richtr, T. Klemša 2015), creating a Návrh opatření (Proposal of measures), taking into account the above-mentioned surveys. As part of the preparation of project documentation for a planning permission by VALBEK (VALBEK spol. s.r.o. 2017), a supplementary IGP was carried out by AZ-GEO (AZ GEO, s.r.o., R. Králík 2017) in 2017. Work design was made in 2020 by the company VALBEK (VALBEK spol. s.r.o. 2020), which was used as tender documentation for the selection of the contractor. Construction works commenced in 2022 after more than 15 years of preparation. The managing contractor is the company GARDENLINE in an association alongside the YUCON CZ Company. The estimated time of construction completion is April 2026.

4. COURSE OF RECONSTRUCTION

After draining the reservoir and removing a prefill protecting the upstream face it became obvious that historic documentation, which served as one of the main sources for the design of the technical solution of the reconstruction, does not entirely correlate to reality. While preparing the implementation documentation it was necessary, mainly in the case of some structures, to factor in their true state. Owing to its extraordinariness, the VD is a cultural technical sight, and all construction activities are carried out under supervision and with a permit from the NPÚ, whose fundamental requirement was not to disrupt the historical stone appearance of the dam. The reconstruction addressed the following: modification of the downstream section, excavating sediments, downstream and upstream faces, adjustments of the dam prefill, reprofiling of safety spillways, cascades, construction of operational technological structures (PTO), repair of right-bank wall and beach, altering the technology of lower sluiceways, realisation of operational electrical and control complexes, and the implementation of remote access and security. These different structures in terms of professions are not included in the article, and further on only building structures related to excavations, foundation engineering, and dam subsoil grouting are included.

5. REALISATION OF SELECTED BUILDING STRUCTURES

5.1 DAM MASONRY

After evaluating results of water pressure tests (VTZ) executed during the IGP (engineering geology survey, which also consisted of a construction technical survey of the dam masonry thanks to bore cores drilled from the dam crest), it was proven that the masonry of the dam has a very high permeability ($4.76 L\cdot min^{-1}\cdot m$ at a pressure of 0,3MPa) and it can be rated as of poor quality mainly due to expected high degradation of binder. As a measure for the improvement of primarily physical-mechanical characteristics of masonry, the design proposed grouting of the dam body masonry. After assessing grouting performed on a test fi eld, it was discovered that it is possible to alter the expected design values of grouting compound consumption per borehole meter from 157L to ca. 50L and to increase the maximum grouting pressures from 0.2MPa to 1.3MPa. Then, it was proven that the usage of previously designed chemical compounds is pointless, and grouting can be realised entirely using substantially cheaper materials based on clay-cement. To verify whether the dam masonry could be grouted, the realisation of test grouting boreholes was proposed. For their execution, a Technological regulation was made with the goal of assigning grouting works more precisely for the testing of masonry grouting ability. The grouting was carried out using clay-cement compounds in 3.0m grouting stages. Essential was the discovery that it is relatively easy to grout the masonry using clay-cement grouting compounds.

Grouting boreholes Ø 56 a 76mm were elongated contrary to the design (VALBEK spol. s.r.o. 2017) so that they would reach at least 0.5m below the dam foundation base. Maximum borehole length reached 20m. A double-core barrel was used to improve the core recovery quality. The technology of rotary-percussion drilling was not suitable in the given conditions. Then, in the lower section of some boreholes in the dam subsoil, entirely weathered granite was encountered, which also had to be grouted. The average consumption of clay-cement grouting compound came out at 52.26L/m. All boreholes utilised ascending grouting with a simple packer. According to the results of VTZ in control boreholes, the dam masonry permeability was three times smaller after grouting (1.6L·min−1·m at a pressure of 0.3MPa).

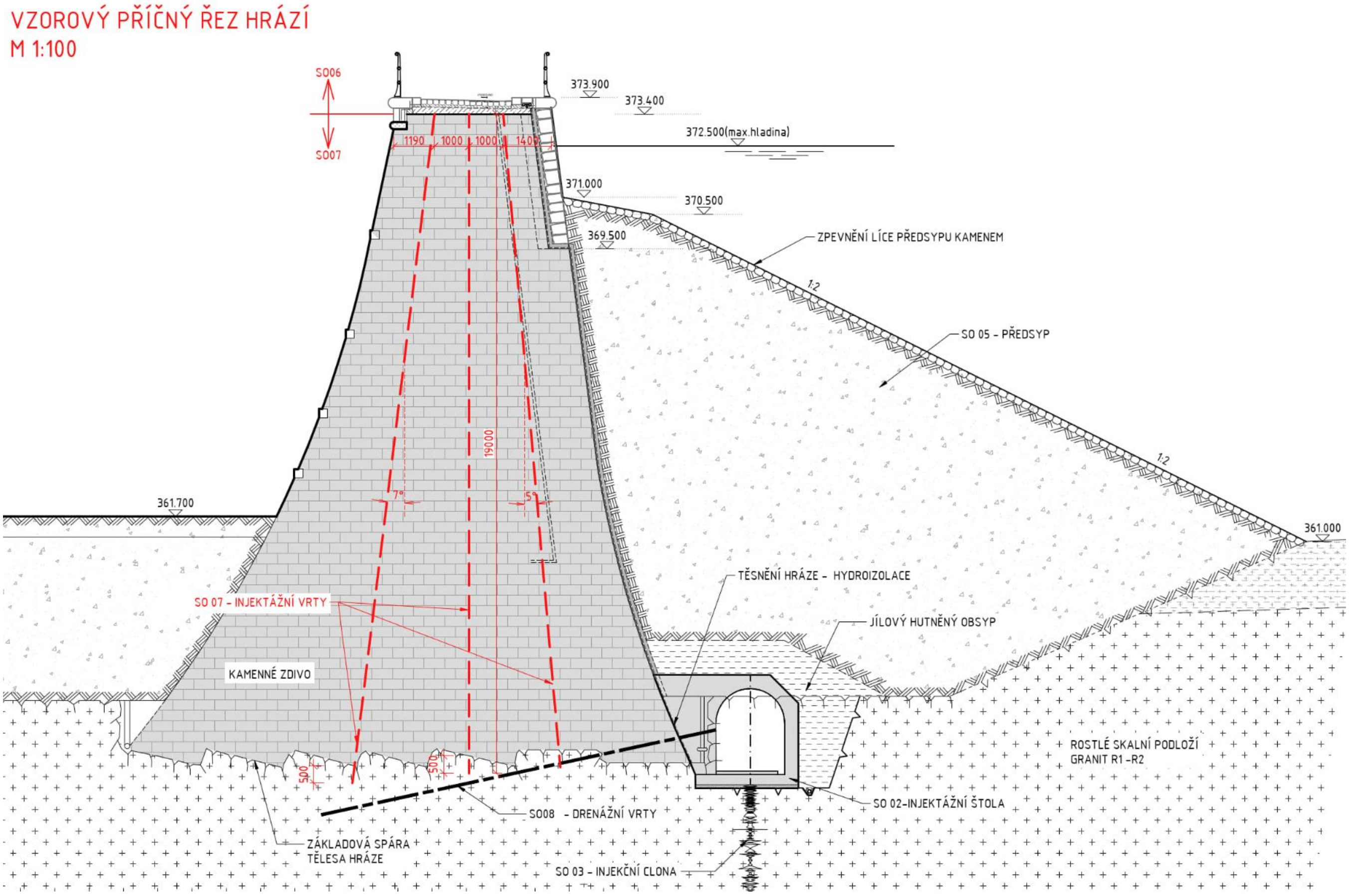

Fig. 2 Cross-section of the dam body grouting

5.2 GROUTING CORRIDOR

A newly built grouting corridor (ICH) with a total length of ca. 130m is led in front of the upstream face slightly below the foundation base of the dam body. An entrance shaft is equipped with a spiral stainless-steel staircase, and it is located on the right bank by the new operations building, enabling a descent below the terrain. Right after is the first section of the gallery with an assembly shaft, which facilitates the transport of mechanisation and construction materials for eventual future repairs. The ICH then continues to the left, where it is terminated on the boundary of the dam body and safety spillway. The ICH course follows the dam foundation base longitudinally and vertically. In descending legs,

the gallery reaches a longitudinal incline of 40%, and it is provided with steps. The clearance width of the corridor is 2.0m, and the clearance height is 2.4m. Definitive lining of the gallery was designed and executed as a cast-in-situ reinforced concrete structure from C30/37 hydraulic-construction concrete separated by sealed dilatation and construction joints into 24 blocks. Individual blocks have a default length of 6.0m. Construction and dilatation joints between blocks are insulated by an internal joint band 400mm in width and are equipped with safety grouting systems for potential grouting after the filling of the dam. The construction joint on the level of the base and wall is fitted with the KAB 125 insulating band. Minimum thickness of the reinforced concrete wall is 400mm.

5.3 EXCAVATED SECTION OF THE GALLERY

The majority of the length of the grouting corridor is realised in an open trench carried out using mechanisation and blasting works. Concreting of corridor lining was realised on infill levelling concrete. Only at the location where the ICH intersected with feeder galleries for manipulation towers of lower sluiceways, the excavation was carried out underneath the protection of a micropile “umbrella” from MAI

SDA Ø 51mm level grouted steel rods. The surface area of the excavation amounts to 10.5m2. The excavation was stabilised throughout construction with 200mm thick primary lining consisting of BTX lattice girders, SB 25/30-X0 SBII J2 sprayed concrete with two layers of KARI wire mesh. Excavation in the nearest vicinity of the towers, dam body, and lower sluiceway galleries was carried out using blasting works with minimal overburden size and 0.5m long rounds. The effectiveness of blasting works was decreased by healthy R1–R2 granite, which was encountered at the face, with high jointing and an intertwining layer of weathered granite at the dam foundation base. After partial completion of the open

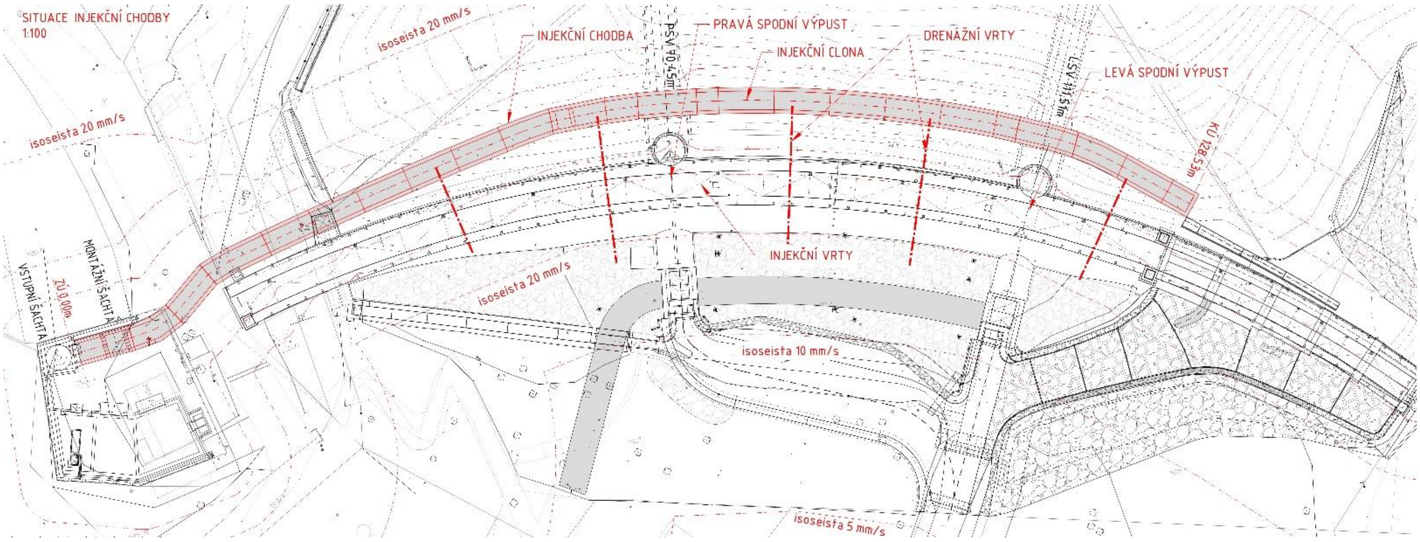

Fig. 3 Plan view of the grouting gallery along the dam heel

trench near the structures of the lower sluiceways, it was then discovered that the foundation of the lower sluiceways is not built down to the level of the dam body foundation base; therefore, it was necessary to proceed with the stabilisation of the area near the excavation. The material composition of this area was very diverse, and it was formed by low-quality concrete, individual stones, and weathered granite with strength ranging from R2 up to R5. The construction pit stabilisation in the area of lower sluiceway tower foundations was carried out with SB 25/30-X0 SBII J2 sprayed concrete, a KARI wire mesh, and grouted IBO Ø 32mm soil nails 4m in length. The nails and umbrellas were grouted using a micro cement suspension with plasticizers in a ratio of 1.0–2.0% of cement weight and a water-cement ratio of 0.8–1.0.

From the viewpoint of NATM, the entire section was classified into one technological class – VT3. During earthwork for the realisation of the gallery, mainly taking place in a rock environment, pressurised water springs were discovered in the foundation base that were not expected in the design. Near the lower left sluiceway, a laboratory analysis discovered carbon dioxide aggressivity of water, and a corresponding concrete class had to be used. Since the excavation took place under a fl owing stream at the bottom of a dam, it was necessary to collect inflows into catch pits and continually pump water from the construction pit over the course of excavations and gallery construction. This seepage and fissure water was released into the stream. Even though the extent of excavations was only a few meters, due to the variability of the rock environment and presence of stone structures of the reservoir, the excavations were a difficult task for all involved parties.

Grouting corridor equipment

During operation, the inside of the corridor will be lit and ventilated by fans with a stainless-steel DN300 forced ventilation duct located in the arch throughout the entire length of the gallery.

The exhaust will be led out through the wall of the entrance shaft. The gallery has a seepage water pumping catch pit in its lowest point by the heel of the right tower, where water from control drainage and buoyancy-measuring boreholes led into the gallery will be collected.

5.4 GROUT CURTAIN

The Grout curtain (IC) is supposed to be an impermeable element that will eliminate water flow underneath the foundation base of the dam and, therefore, alongside drainage elements, significantly reduce buoyancy forces acting upon the dam body. Old hydrogeological surveys completed historically before the construction found that water fl ow below the foundation base is 58 $m^3 \times hr^{-1}$ = 16,1 $l \times s^{-1}$.


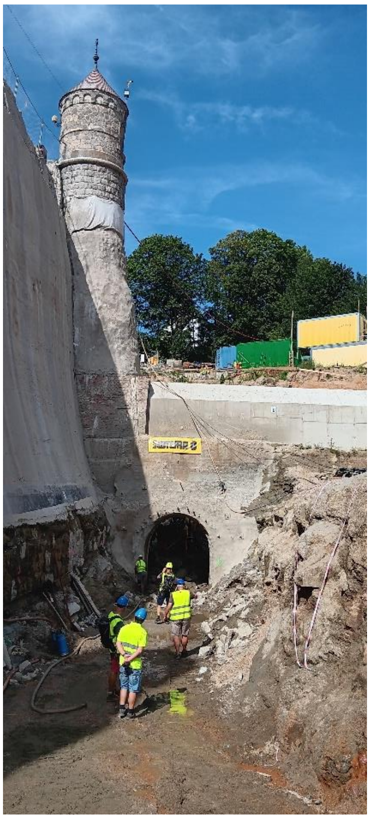


Fig. 4 a 5 View of both sections of excavations in the axis of the future gallery. Fig. 6 Gallery lining concreting

Na On the basis of test grouting, it was necessary to reconsider not only the expected technological process, which anticipated only ascending grouting, but it was also possible to change the designed grouting compounds, in turn significantly saving on material costs. Another change that occurred within Implementation documentation (RDS) was the route of the grout curtain on the left at the location of the first and second spans of the crest safety spillway. This change represents a substantial decrease in costs for securing the stability of a local road that is located in close proximity to the left abutment of the dam. Real parameters along the route of the grout curtain discovered on the basis of a carried out grouting test field led to important changes in the RDS. The grouting boreholes were executed through the infill concrete of the grouting gallery.

The following findings of explorations and their evaluations were vital for the subsoil:

- Granite in differing stages of weathering was discovered in all boreholes below the foundation base.
- It is true for the entire observed profile that even healthy, otherwise strong granite is broken up (most common fissures are almost vertical with an incline of 70°–90°), the disintegration is lumpy to blocky. The subsoil rock is substantially permeable, therefore water losses from the reservoir through the subsoil are sizeable.
- The rock subsoil has failures due to significant fissure systems. Open fissures reach at least 15 below the dam foundation base level.
- The subsoil in the area below the dam is less permeable than the subsoil below the reservoir. This fact is caused by an increase in buoyancy underneath the reservoir.

It was obvious from the results of exploratory works completed so far that in the right section of the dam, the most geotypes are represented, including a disintegrated (entirely weathered) course-grained granite. The grouting test fi eld was designed in these places.

The grouting test field consisted of nine grouting boreholes situated such that it would be possible to grout and then inspect an entire continuous section of the grout curtain in the subsoil. Grouting was performed by gradually concentrating the grouting curtain with three sequences of boreholes using grouting compounds based on cement.

The average clay-cement grouting compound consumption came out at 50.62L/m and microcement grouting compound at 65.29L/m. It was clear from the VTZ results that the dam subsoil in the section of the grouting field reached an average permeability of 6.86L·min−1·m at a test pressure of 0.3 MPa prior to the completion of test grouting. The subsoil permeability undeniably decreased more than eightfold. Maximum grouting pressures ranged from 0.5 to 1.3MPa in relation to the location of the stage. Fundamental was a finding that the dam subsoil can be relatively well grouted using clay-cement grouting compounds and cement compounds from really finely ground microcements.

The grout curtain itself was then carried out down to a depth of 24.0m in the middle section of the dam and 15.0m at the attachments to the VD shores. The number of grouting boreholes was 183pcs, 8 of them being control boreholes. The total length of boreholes was 2872m. The grout curtain was executed preferably using ascending grouting. Although descending grouting, or a combination of both, had to be utilised in places with significantly damaged rock. The grout curtain was designed as a single row with four sequences. The final spacings of grouting boreholes were adapted to the expected rock environment. For example, in areas with completely weathered and disintegrated granite and in tectonically faulty granite 0.75m, and in sections with more compact and healthy rock 1.0m. Core boring was utilised for grouting and control boreholes with a minimum diameter of 59mm (maximum 76mm). A double-core barrel was used for higher-quality of core recovery. The technology of rotary-percussion drilling was inappropriate and unacceptable in the given conditions.

At the time of writing (11/2025), connection (fortification) grouting of the space beneath the base of the final grouting corridor is completed. Grouting is carried out only after the grouting corridor has been constructed and the prefill has been poured.

Sufficient “loading” and consolidation of structures is required. The objective of connection grouting is to carefully grout the entire “delicate” area below the foundation base of the grouting corridor.

The discovered grouting parameters of the compound and results from the grouting test field were successfully applied to the entire grout curtain. The resulting quality of the dam subsoil grouting will be determined by the amount of seepage water and water pressure below the dam, which will be monitored by five drainage boreholes.

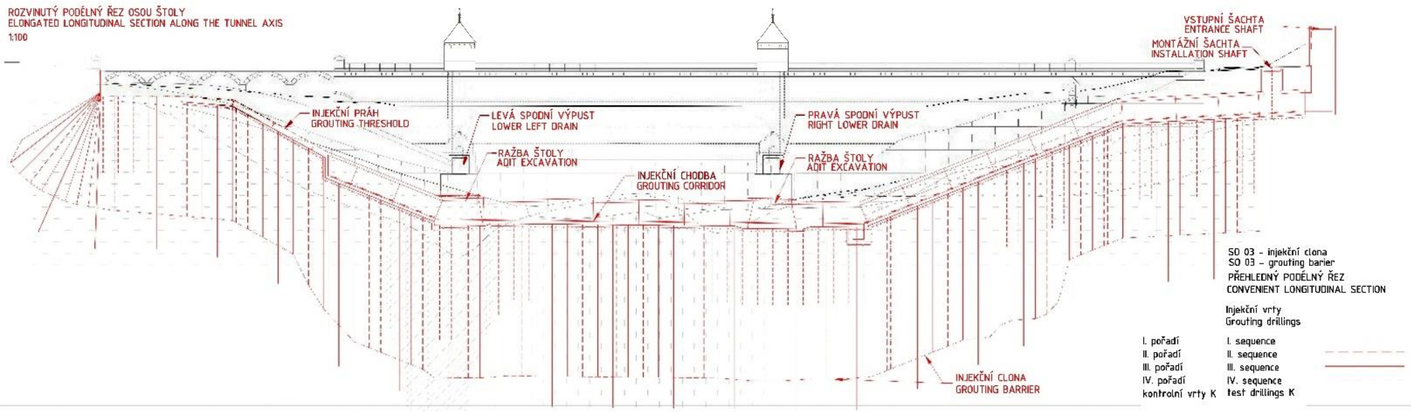

Fig. 7 Grout curtain of the dam subsoil

5.5 DRAINAGE BOREHOLES

It was clear from the stability analysis of the dam body carried out by the VD-TBD Company as a part of (D. Richtr, T. Klemša 2008) that the safety factor for movement during an extreme load state is unsatisfactory.

A distinct factor in the calculation is buoyancy in the foundation base of the dam body. For their monitoring and if necessary short-term lowering, five relief boreholes will be available that will be led from the ICH at a slightly downward angle underneath the foundation base of the dam body. The length of drainage boreholes is 8.5–14m. In regard to the presence of highly weathered Liberec granite attitudes that caused substantial instability of borehole walls already during executed drilling works, on top of the presence of pressurised water, it was even necessary to alter the design and equipment of the borehole, using a Ø114.3/94mm perforated steel pipe.

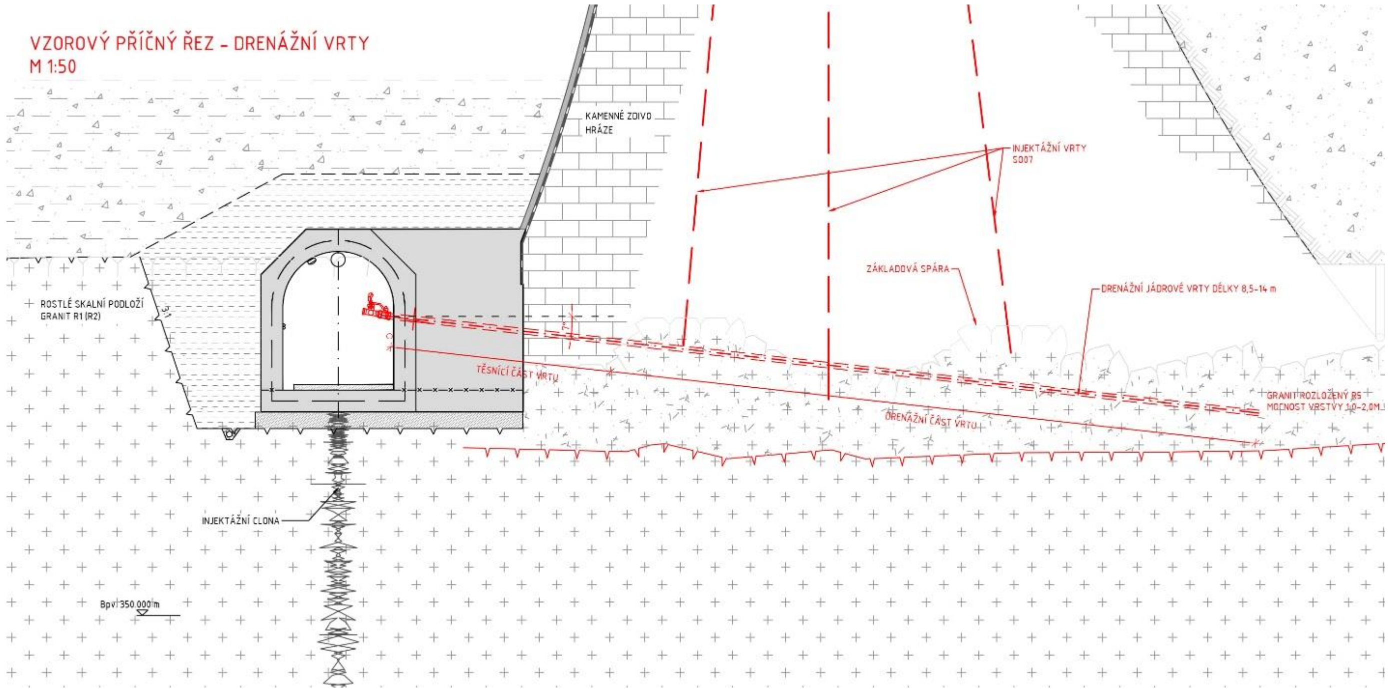

Fig. 8 Sample cross-section – drainage boreholes

5.6 LOWER SLUICEWAYS

The current masonry intake galleries were strengthened prior to the excavation due to their technical state, utilising a reinforced concrete sarcophagus and a reinforced concrete anchored floor. The chambers themselves and brick plugs of lower sluiceways were demolished, expanded, and deepened such that it would be possible to mount new DN 1000 stainless-steel runoff piping for the left and DN 1200 for the right sluiceway. Then, new indoor technological equipment of the lower sluiceway towers was installed.

6. MONITORING THROUGHOUT CONSTRUCTION

Specialised geodesy measuring, monitoring, and assessment of TBD results are, as with measuring during permanent operation, carried out by workers of an authorised company, VD-TBD. Contrary to monitoring during permanent operation of the hydraulic structure, the rate of some measurements is increased during construction. Vertical displacements are measured with very precise levelling (VPN). Horizontal displacements are measured using polar surveying of a spatial geodetic network in rows and groups from reference net stations.

6.1 DEFORMATIONS OF THE DAM BODY

Over the course of the last 24 years, horizontal displacements of the dam crest fluctuated during a year, ranging from 2mm (downstream) to 3mm (upstream). The total horizontal displacement of the dam, therefore, was not larger than 5mm with no observed growth trend. The size differential of determined horizontal displacements throughout construction in the first year was minimal. The emptying of the reservoir, lowering of water pressure, and buoyancy did not manifest in a meaningful way deformation-wise. These results were assigned to the arch effect of the masoned dam and loads from the protective prefill.

Entirely different horizontal deflection measuring results were then obtained during the excavation of a trench for the foundation of the gallery beneath the dam heel. The impact of construction works showed itself in the middle portion of the dam and its crest by a horizontal deformation upstream with a value of +20.0mm. Reversible deformation after the cooling of the dam in the winter months and while carrying out the prefill again was –5.0mm.

Fig. 9 Installing the bottom drain segment DN1200

Observed points at the ends (abutments) of the dam showed minimal movement in between stages in a range of ± 0.3mm. Total vertical movements have characteristics of verifiable deflections that are larger at the centre of the dam, with a maximum of –4.3mm. Total movements at the heel of the downstream face have characteristics of deflections, with a maximum of –2.9mm.

Overall, the measuring can be assessed after three years of construction works, such that the dam body, after draining and during reconstruction, reacted to construction intrusions, mainly the uncovering of the foundation base and prefill extraction, by an irreversible deformation (+15.0mm downstream). The dam also reacted, with a delay, to the impact of temperature in the winter and summer seasons. The impact of seasons was represented by cyclic horizontal deflections of ±5.0mm, depending on the heating of the dam body.

6.2 DEFLECTIONS OF UPSTREAM LOWER SLUICEWAY TOWERS

Movements perpendicular to the stream, i.e., in the direction of the dam axis, reach half the values of measured maxima in the direction of the stream, and they do not display the cyclic temperature loading.

Total tower deflections at the level of the dam crest were conclusive, with a maximum value of –4.5mm.

6.3 DEFORMATION OF INFLOW GALLERIES AT THE EXCAVATION

After transferring water into the lower right sluiceway at the beginning of the construction, a fissure between the stone masonry of the gallery and the brick lining of the lower sluiceway tower foundation was discovered during an inspection of the left inflow gallery. Due to a lack of knowledge of the structure foundation and planned excavation of the grouting gallery beneath the inlet structure, the 5.0mm wide fissure in the lower sluiceway ceiling was fitted with a deformeter base. In the horizontal direction, the fissure was pressed together by ca. 1.0mm (step by step since August 2023). This phenomenon can be attributed to the structure warming after summer, discovered during geodesy measurements throughout the year. Vertical deformations of the fissure were minimal at 0.65mm. Total deflections of inflow galleries after completing rehabilitation measurements and excavating the grouting gallery beneath add up to –6.6mm. Measuring was done with a precision of ± 0.05mm. Measuring frequency was 1 x monthly in succession to the date of geodesy measuring of the dam body and its surroundings.

Results of the VPN measurements confirmed stable behaviour of the dam over the entire evaluated period. The surpassing of the alertness warning state of ±7.5mm from the default measurement was observed only on some points by the left and right inlet corridors, precisely in the area of the inlets. The limit movement values of ±15.0mm from the default measurement were not reached anywhere.

6.4 BANKING OF LOWER UPSTREAM SLUICEWAY TOWERS

The platforms were placed inside the towers perpendicularly to the dam. Every tiltmeter platform consists of a couple of ball joints that are attached by consoles to a tower wall. Similar deformations occur at both towers. Measurement results are in agreement with executed geodesy measurements on the dam crest.

In the direction of the stream in the winter months, both platforms experienced an incline downstream (cooling of the dam). Maxima were reached during the year, with a value of 0.376mm/m (right tower, end of February 2024, maximum movement downstream – dam cooling after winter) and –0.155mm/m (right tower, halfway through September 2024, maximum movement upstream – arch of the dam heating). The largest tower incline throughout the year in the direction of the stream was 9.1mm on the left tower and 9.8mm on the right tower.

6.5 PRESSURE CONDITIONS

Pressure conditions in the dam subsoil are observed using buoyancy measuring, i.e., relief boreholes and observation probes located on the dam crest, in the corridor of the lower sluiceways, by the downstream heel of the dam, and in the area below the dam. No significant water level fluctuation manifested in the boreholes after emptying the reservoir.

6.6 SEEPAGE CONDITIONS

Seepage through the body of the dam and subsoil is measured using a volumetric method at the outfall of a vertical drainage system in the lower sluiceway corridors. It is expected that during grouting works from the dam crest, the clogging of the entirety of these drainage systems will occur.

7. CONCLUSION

The authors were delighted to participate in the design and realisation of an overall repair that this dam surely deserved after 120 years of safe operation. They view adding a grout curtain to the subsoil and constructing a grouting gallery as technically most significant. While realising the grout curtain, it was possible to add important findings to geological and hydrogeological data from the dam subsoil, which were unavailable while preparing the design. They will then be used for eventual curtain repairs in the future. The appearance of the dam, supervised by the NPÚ, will once again be how our predecessors built it at the beginning of the 20th century. The dam crest will continue to be made of stone with a cast iron railing and historic lanterns; however, on the inside of the dam body, it will be packed with monitoring technology and equipment of the 21st century.

The authors believe that the remaining finishing phases will follow up on the current course of reconstruction, and after three construction years, the hydraulic structure will facilitate not only safe operation for at least the next 120 years, but also a recreational function for citizens.

Fig. 10 Air side of the dam

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