Underwater maintenance works in the Gepatsch reservoir

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As a result of sedimentation problems in the Gepatsch reservoir, a special structure was created to enable inspection and maintenance work to be carried out in the headrace and bottom outlet tunnels upstream of their locking devices, without the need to drain down the reservoir. Steel guide frames with sliding stoplogs were installed in front of the bottom outlet intake structure and the lower headrace intake at a depth of about 110 m by divers, using a saturation system. Innovative design features of this project are described here.

The Kaunertal hydro plant, owned by TIWAG (Tiroler Wasserkraft AG) was constructed in the Kaunertal valley in the Tyrol, Austria, between 1961 and 1965. It was designed as a high-head storage plant, with the Gepatsch reservoir having a storage volume of about $138 \times 10^6$ m$^3$, see photo (a) and Fig. 1. With an installed capacity of 390 MW and an average annual production of 660 GWh, it was the most powerful hydropower plant scheme in Austria at the time [Lauffer, 1968].

There is a bottom outlet on the right slope of the valley, which has a maximum discharge of 75 m$^3$/s (see Fig. 2). The two bottom outlet valves are located in a cavern about 300 m from the intake structure.

The headrace intake structure, see Photo (b), is on the left slope of the valley with a design capacity of 54 m$^3$/s. The first water retaining structure, equipped with two butterfly valves, is approximately 540 m downstream of the intake structure.

1. Maintenance work

After 50 years of problem-free operation, it was necessary to carry out work on the intake structures of the headrace tunnel and the bottom outlet. New trashracks were to be provided at the headrace intake, which comprises an upper and lower intake structure, with a difference in elevation of about 35 m. In addition, it was decided to install a frame and sliding stoplog at the lower intake structure to monitor the status of the pressure tunnel up to the first valve chamber. This installation will enable future inspections to be carried out without having to drain down the reservoir completely.

The bottom outlet also has two intake structures. The lower structure was used for water diversion during the construction of the dam; the upper intake, about 27 m higher, is a three-field structure and is designed for bottom outlet operation. As it is closed by a sliding lock gate, it was not possible to check the lower intake for its functionality and operational safety. This intake needed to be sealed permanently. Again the objective...
was to install a guide frame with a sliding stoplog in front of the upper intake (similar to the one in front of each of the three fields of the lower headrace intake) so that future inspections will be able to be carried out in the adjacent bottom outlet tunnel up to the valve cavern without having to empty the reservoir.

It was decided to carry out this work in dry conditions, in the conventional way, so measures were taken to draw down the Gepatsch reservoir in the late autumn of 2015. During the drawdown, large volumes of fine material and silt were transported to the foremost area of the reservoir, see Photo (c). This was because of the limited transport capacity of the stream flowing through the reservoir (there was very little water as it was the start of winter) and because of the porewater pressure in the substantial volume of fine deposits in the centre of the reservoir. The dead storage capacity of the reservoir had virtually been filled with about 400,000 m³ of sediment. The dewatering and continuous sediment transport had to be stopped to guarantee the operational safety of the intake structures. Consequently, the only work that could be done was to install new racks at the upper headrace intake.

As a result, an alternative working method had to be adopted to carry out the remaining important tasks. This method was complex and challenging. However, it would make future major drawdowns unnecessary. All of the concreting and installation works were carried out by divers during the summer and autumn of 2017. Because of the considerable depth at which this work had to be done (110 m), the divers used a saturation system.

2. Saturation diving
Depending on diving conditions (depth and dive time), conventional dives require decompression. In principle, the longer the dive and the higher the pressure, and the longer the decompression time.

TIWAG opted for saturation diving for the following reasons:
- Environmental impact would be low. By not emptying the reservoir, sediments would remain in place and not be moved.
- There would be no impact on the aquatic ecosystem, because sediments would not be transported to the rivers downstream of the reservoir and the powerhouse.
- Saturation diving could be carried out regardless of the reservoir level. Diving would therefore not be restricted to a specific time of the year.
- Reservoir inflows would be of no significance, so the diversion system would not need to be closed.
- Classic diving accidents when resurfacing would be avoided, as the divers would spend several days (or even weeks) in a diving bell, pressurized to the same depth pressure.
- There could be uninterrupted operation of the reservoir and the hydropower plant, thus avoiding loss of revenue.
- Despite the technical complexity of the diving equipment, saturation divers were used to carry out a wide range of measures under water at Gepatsch. They could work around the clock because they did not need to spend time returning to the surface and decompressing. The diving equipment is shown in photo (d).

TIWAG had two main objectives. The first was to ensure long-term and sustainable operation of the powerplant. The second was to allow for future inspections of, and maintenance work to, permanently submerged plant components in dry conditions upstream of the gate cavern in the bottom outlet tunnel, and upstream of the butterfly valves of the headrace tunnel. The following measures were therefore carried out with saturation diving:
- **Lower headrace intake**: Installation of steel trashrack towers, and a frame with a stoplog
- **Bottom outlet**: Installation of an exterior concrete seal in front of the lower intake as well as support structures including stoplogs.

3. Choice of steel
The water in the Gepatsch reservoir has very low corrosive properties, which is very favourable for the use of either stainless steel or black steel.
The chloride content of 3.85 mg/l is well below the critical value of around 200 mg/l. Consequently, stainless steel (without the need for anti-corrosion coating) was considered suitable for use in the reservoir, even in terms of crevice corrosion.

If appropriate measures would be taken (for stainless steel, avoidance of direct contact with black steel, and for black steel, avoidance of damage to anti-corrosion agents), the use of either material would be possible at Gepatsch.

In these circumstances, it was decided to use stainless steel for the intake towers, and black steel for all the stoplogs, frames and supporting structures.

4. Construction measures at the lower headrace intake

4.1 Intake towers

Tower-like intake racks were chosen, to ensure that a sufficient rack area would be provided for powerplant operation even if it would not be possible to prevent the future transport of fine sediments. To minimize turbulence caused by water streaming through the system, the support structure and rack bars are shaped as round bars. A triangular, 9 m-high intake tower is installed in each of the three intake fields.

For hydraulic reasons, the intake bars are 35 mm-diameter round bars, fixed in vertical alignment. From a structural and dynamic perspective, it would also have sufficed to use 35 x 5 mm-diameter round pipes, but these could not be delivered on time. The bar spacing is 40 mm, making the centre-to-centre distance 75 mm.

The contact face on the reinforced concrete is inclined. As a result, horizontal displacement towards the walls, which have a trapezoidal layout, is achieved when the racks are lowered.

The bottom parts of the towers are then concreted to the top level of the cross beam of the existing vertical racks. Because of the inclined surface of the contact face on the reinforced concrete traverse, and the trapezoidal layout of the wall-like piers, and also because the bottom parts of the intake towers are concreted to the structure, it is not necessary to fix the intake towers with heavy duty anchor bolts. For constructional reasons, however, the panels are nevertheless fastened to the top of the traverse with assembly bolts.

The intake towers are equipped with a removable roof of rack bars. The heads of the bars are designed so that the height of the towers can easily be increased.

The area of a single screen array is 19.2 m². The rear screen array above the parapet has an area of 3.40 m², and the one below has an area of 2 m².

The following main activities were carried out by divers:

- Dismantling of the horizontal trashracks at lower headrace intake
- Removal of any siltation on the inside of the lower headrace intake and pumping it to an area some 300 m south of the lower headrace intake
- Installation of eyebolts in the corners between the parapet and wall-like piers (see Fig. 4).
- Attachment of cables to eyebolts to act as guide cables for the lowering process
- Lowering of the intake towers one after the other, followed by their installation
- Concreting of the lower part of the intake towers to the top of the existing cross-beam of existing vertical racks.

Photo (e) shows the stainless steel intake tower device shortly before it was lowered.

4.2 Loads and structural verification of the rack bars

There are considered to be two principal load scenarios:

- Operating load: Water flowing through the trashracks of the intake towers.
- Special load: Partial blocking of the trashracks by sediment.

To calculate this load, it is usual to assume a uniformly distributed load of 50 kN/m² and a reduced partial safety factor on the load side of 1.1 (instead of 1.35). This results in a design load of 50 kN/m² x 1.1 = 55 kN/m².

The rack bars are welded to the cross-pipes. Giving a centre-to-centre distance between the bars of 7.5 cm, and taking into account blocking of the trashrack by sediment, the design load is calculated as 55 kN/m² x 0.075 m = 4.125 kN/m.

This allows for the bending moment at the welded joints, on the basis of which the rack bars are designed.

4.3 Dynamic verification of the rack bars

4.3.1 Calculation of highest velocities in the area of the intake racks

To be on the safe side, it is assumed that the intake height is just 3 m. Consequently, the three intake towers each with two screen arrays with a width of 2.45 m provide a total intake area of:

\[ 3 \times 3 \times 2 \times 2.45 \text{ m} = 44.1 \text{ m}^2. \]
Assuming the water level is below the intake surface of el. 1694 at the upper headrace intake, the maximum water abstraction during full turbine operation will amount to 50 m³/s at the lower headrace intake.

This results in a (gross) flow velocity of 50 m³/s / 44.1 m² = 1.13 m/s. For the 35 mm-diameter rack bars with a net distance of 40 mm, this produces an obstruction rate of 75 mm/40 mm = 1.875.

Consequently, the (net) velocity between the round rack bars will be:

\[ 1.13 \text{ m/s} \times 1.875 = 2.13 \text{ m/s} \]

To be on the safe side, the velocity is assumed to be 2.5 m/s for the following verification.

Numerical fluid calculations of the flows and velocities in the area of intake racks have determined maximum velocities of 2 m/s. These velocities occur in very few areas where the flows are deflected, and are shown in red in Fig. 3.

Consequently, the following calculations based on a velocity of 2.5 m/s are very much on the conservative side.

**4.3.2 Flow excited frequency**

Using a Strouhal number for the circular cylinders of 0.2, and a velocity of 2.5 m/s, the following excitation frequency is calculated in accordance with the work of Schleiss, (1985):

\[ 0.2 \times 2.5 \text{ m/s} / 0.035 \text{ m} = 14.28 \text{ Hz} \]

Based on the formula of [Schleiss, 1985] the natural frequency of the rack bars amounts to 97.65 Hz.

**4.3.3 Rack adaptation**

To prevent a resonant condition with the bars vibrating with increasing amplitude, the flow-excited frequency must be sufficiently smaller than the natural frequency of the rack bars [Schleiss, 1985]. The flow-excited frequency should not exceed 60 to 65 per cent of the natural frequency.

The 35 mm-diameter bars easily meet this requirement: 14.3 < 0.60 × 97.65 = 58.6.

**4.3.4 Calculation of flow load and comparison with rock obstruction load**

The calculation was based on the following equation:

\[ F = q \times v = 981 \text{ kg/m}^3 \times 50 \text{ m}^3/\text{s} \times 0.035 \text{ m} / (2.45 \text{ m} \times 2 \times 3) \times 2.5 \text{ m/s} = 292 \text{ N per rack bar at a height of 3 m} \]

where:

- \( F \) = impulse force (N);
- \( q \) = flow rate (m³/s);
- \( v \) = velocity (m/s); and,
- \( \rho \) = density of water (kg/m³)

To compare this impulse force with the rack obstruction load, it must be converted into an area load:

\[ 0.292 \text{ kN} / (0.035 \text{ m} \times 3 \text{ m}) = 2.78 \text{ kN/m}^2 \]

The application of a partial safety factor of 1.35 to the load produces a compressive force of 2.78 × 1.35 = 3.75 kN/m² at the design level.

The design rack obstruction load of 50 kN/m² × 1.1 = 55 kN/m² must be assumed for the entire rack area, whereas the calculated flow load only exerts pressure on the bar. Consequently, the following rack bar loads are assumed for 35 mm-diameter bars with a centre-to-centre distance of 75 mm at design level:

- Rack obstruction: 55 kN/m² × 0.075 m = 4.125 kN/m
- Flow pressure: 3.75 kN/m² × 0.035 m = 0.131 kN/m

The force of flow pressure is therefore just 3.2 per cent of that of the force of rack obstruction pressure. Fatigue verification also shows that there is no risk of damage to or failure of the construction as a result of dynamic loads. Consequently, the dimensions of the rack bars must be determined on the basis of the structural load scenario 'rack obstruction'.

**4.4 Temporary closure of the emergency gate slot**

It was necessary to be able to close the emergency gate stoplog temporarily if the tunnel directly adjacent to the intake structure needs to be emptied for inspections or any remedial work.

In the course of the saturation diving assignment in 2017, a frame structure was installed in the emergency gate slots at the intake structure. This structure can hold the stoplog in both the closed and open positions.

![Fig. 4. The trashrack towers and stoplog at the lower headrace intake structure.](image-url)
To ensure that only a small crane with cable winches or chain hoists (which can be flown in by conventional helicopters) will be needed to lower the stoplog from a 'parked' position, the stoplog had to be designed so that air could be injected into the structure and the buoyant force could be used to facilitate the raising and lowering process. A small crane with a cable winch was needed to aid the process if required.

To this end, the stoplog had to have a chamber system and needed to be as light as possible so that the difference between its weight and buoyant forces would be as large as possible.

Fig. 4 shows the trashrack towers and stoplog at the lower headrace intake structure.

4.5 Construction

The stoplog is made of unalloyed steel and is 4 m wide, 0.48 m thick and approximately 5.8 m high. The two cover plates each have a thickness of 12 mm and are connected by horizontal reinforcement plates with a centre-to-centre distance of 200 mm and a thickness of 6 mm. In the event of one-sided water pressure when the tunnel is emptied, these reinforcement plates serve to transfer shear forces. In the event of double-sided water pressure, they have been designed to withstand a pressure difference between the interior and exterior of about 11 bar. This major difference in pressure will not occur in routine operation because the lower part of the stoplogs have been perforated. The further the stoplog is lowered and the pressure increases, the more water will be able to penetrate these perforations and compress the air within.

The sides of the stoplog and/or guide frames are designed so that they run in the existing U300 slots.

4.6 Plant Fabrication

First, all horizontal reinforcement plates are welded to the tunnel-facing cover plate. This cover plate can no longer be welded to the horizontal plates in a single piece (weld seams are not possible on the inside), so it would have been necessary to divide this plate into very many small plates and weld each, one by one, to the horizontal plates.

To dispense with this time-intensive process, the water-facing cover plate is equipped with slots at the height of the reinforcement plates. The slots are just long enough to ensure that the cover plate still holds together as a single piece. The slotted cover plate is then laid onto the reinforcement plates and welded in place from the outside.

The cover plate and reinforcement plates are only connected in the area of the slots. Consequently, this is the only place where shear forces are transferred between the two plates. Since the shear forces increase towards the support, the slots are longer in the outer area of the cover plate and shorter towards the centre.

It is not possible to apply a lasting anti-corrosion coating to the inside of the stoplog. It would be possible to apply an anti-corrosion coating before the second cover plate is welded in place. However, the welding process heats the stoplog to such an extent that the anti-corrosion coating would melt, peel off and be compromised.

Since the stoplogs can be raised, it is possible to inspect their interior through the existing air/water inlet openings. If the interior shows evidence of major corrosion, the stoplogs will need to be remanufactured. However, because of the very favourable properties of the water in Gepatsch reservoir (low oxygen content, hardly any chloride, high pH value and very little conductivity), little corrosion is expected.

4.7 Chamber system filled with air or water

The stoplogs are raised and lowered by a cable winch. To minimize the hoisting forces required, the stoplog can be filled with air. The buoyant force means that very low hoisting forces are required.

The horizontal reinforcement plates on the inside have circular openings with a diameter of 30 mm, so that the entire stoplog can be filled with air and/or water.

The stoplog has two chambers: one on the left and one on the right. In a worst-case scenario, if the stoplog should become jammed during raising or lowering, different buoyant forces can be applied to the right and left side of the stoplog.

The two chambers constitute an 'open system'. In other words, there are openings in the lower section through which air and/or water can escape. When the stoplog is lowered into the water from the operating platform, it is filled with air. As it is lowered, the water pressure increases with depth. Water penetrates the chambers through the openings and compresses the air inside (originally at 1 bar atmospheric pressure). The further the stoplog is lowered, the more water enters the system and the smaller the volume of compressed air will be. At a depth of 90 m, the stoplog will be filled to about 90 per cent with water and to just about 10 per cent with air, which has a pressure of 9 bar.

Thanks to this open chamber system, the pressure on the inside and outside of the stoplog will be equalized, preventing it from being crushed.

To facilitate the raising process, each chamber is equipped with a valve at the top of the stoplog. Through these valves, air can enter the two chambers independently of one another. At a depth of 90 m, air pressure of about 9 bar will be injected into the chambers to force the water through the lower openings. The buoyant force can be increased by injecting more air.

The hydrostatic pressure on one side is generated by a maximum water height of 35.5 m when the reservoir is at el. 1694.5 m, which is 0.5 m above the surface of the upper headrace intake that cannot be closed off.

Consequently, a partial safety factor of 1.25 for water pressure on the load side is a safe assumption to make. With an assumed safety factor of 1.25 the stoplog will be dimensioned against a water height of 44.4 m.

4.8 Frame construction in the existing slots

The two existing emergency gate slots consist of U300 steel profiles in a vertical alignment. The net distance between the two slots is 4 m.

When it comes to the insertion and lowering of the stoplog in the slots, it is necessary to make the width of the guide frame and that of the stoplog slightly smaller to compensate for any dimensional tolerances. In addition, there is little knowledge of the condition of the existing U300 profiles made of black steel.

For this reason, a special frame construction has been manufactured which is installed between the U300 profiles in the headrace intake and the stoplog. In other words, a new emergency gate slot has been created which is precisely tailored to the stoplog and is installed in the existing slots.
The frame construction is slightly extended at the top. In addition, removable supports are attached so that the guide frame will also act as a support structure for the stoplog in the raised position.

Fig. 5 shows the stoplog with details of the frame construction.

5. Construction measures at the bottom outlet

5.1 General situation

Fig. 2 shows the bottom outlet intake structure. A course rack immediately followed by a steel sliding lock gate are located in front of the lower intake. The upper intake cannot be closed off. A course rack is also located directly in front of the lower intake.

The silt deposits caused by sediment transport as a result of the reservoir drawdown in 2015 are at els. 1664 to 1664.5 m, that is, at least 0.5 m below the intake surface of the upper bottom outlet intake structure. Silt deposits were located at about el. 1659 m before this event.

Until the recent project, it was only possible to empty the bottom outlet tunnel upstream of the gate cavern to carry out visual inspections by lowering the reservoir level to below el. 1665 m via the headrace channel, because there is no shut-off device in the upper bottom outlet intake.

Because of the risk of further sediment being transported forwards by a drawdown to slightly below el. 1665 m, it was decided to avoid drawing down the reservoir in future.

In the event of water levels exceeding el. 1665, and so that water can be evacuated from the tunnel upstream of the gate cavern for inspections to be carried out, it was therefore necessary to be able to block off the upper outlet intake.

However, when the tunnel is emptied, the sliding lock gate at the lower bottom outlet intake can only withstand water pressures including (siltation) earth pressure up to el. 1665. In addition, the sliding lock gate itself could not be inspected. Consequently, measures also needed to be taken at the lower outlet intake, incase it would be necessary to empty the tunnel when water levels rise above el. 1665 m.

5.2 Closure of the lower bottom outlet intake with an exterior concrete seal

The following measures were taken:

- Removal of silt in front of the lower bottom outlet intake (at el. 1648) to achieve a water depth of about 110 m. This was done using an airlift method. A steel pipe with a diameter of 25 cm and bellmouth at the lower end was lowered vertically to the sediment. An air pipe was then attached to the steel pipe so that the end of the pipe could be inserted into the steel pipe bellmouth. A compressor was used to inject air into the air pipe, and as a result of buoyant force, a mixture of water and sediment could be transported upwards through the pipe. This mixture of water and sediment was then pumped via a pipeline located on the water surface by a booster pump to a site about 300 m away. The sediments were then transported to an elevation slightly above the reservoir base by a downpipe that extended to a depth of 75 m.
- An excavator shovel was used to transport rhizomes and blocks to the surface.
- Sediments were removed down to the original terrain and/or existing concrete slab (top level at el. 1648). It was necessary to avoid damage to the existing banks and riprap work.
- To safeguard the stability of the sediment banks, the slope steepness was kept at a maximum of 23° during sedimentation excavation. Consequently, the volume of excavated material amounted to about 12,500 m³.
- To gain direct access from above to the construction site of the concrete seal, the existing ice protection roof of the winch building and the winch frame of the movable rack were removed, using a wire saw.
- After the construction pit had been excavated, the movable rack was dismantled by divers, and then lifted out of the water by a crane.
- Residual sediments and sections of wood which could not be reached by the excavator shovel (or extracted by the airlift method) were manually removed by divers.
- After cleaning with high-pressure water jets, injection pipes were laid in the area of the sliding lock gate to enable grouting injection at a later date. All in all, three injection hose systems were laid for three-stage grouting. Directly after the concreting works had been completed, grouting was carried out using the first injection hose system (which was the first stage). The second and third stages are for post-grouting.
- The entire concrete seal was reinforced with steel (including connections to the slots).
- Subsequently, the excavated pit was filled with concrete to a height of approximately 4.5 m. This required a concrete volume of about 400 m³. A vertical water-filled steel pipe (DN125) was used to transport the concrete to a depth of about 110 m, so the concrete did not fall freely, but was conveyed in the water-filled pipe. The concrete was placed by the tremie method, with the bottom of the tube always being kept below the rising concrete level.
- The water temperature was measured throughout the entire depth of the reservoir on 4 July 2017. It was found that the temperature, even at the bottom of the reservoir, does not drop below 5°C, which is very advantageous for the placement of concrete. This is probably because the water flows from a diversion channel into the reservoir near the bottom outlet intake at el. 1665 m. The water temperature from the diver-
sion channel is also 5°C. For natural lakes which do not usually have any inflows at the bottom of the lake, the water temperature at the basin bottom is normally 4°C all year round, because water is most dense at this temperature and therefore remains at the bottom of the lake.

- The concrete seal is founded on a concrete slab in front of the bottom outlet intake on rock (see Fig. 6). Consequently, one-sided settlement of the concrete seal can be ruled out.

- It is impossible to completely rule out the contact of water between the concrete and sliding lock gate (from above, below and from the side) which could exert water pressure on the sliding lock gate. However, the concrete seal is not vulnerable to settlement because it is founded on concrete and rock. Consequently, there are very small gaps between the concrete and structure so the water afflux and/or through-flow will be minor when the bottom outlet tunnel is emptied after the stop logs have been lowered. Nevertheless, holes were drilled into the sliding lock gate to prevent the build-up of excessive water pressure. This is a further measure to enhance safety when the tunnel is emptied of water.

5.3 Concrete technology

Because of the length of the pump lines and the vertical pipes, the maximum grain size of the aggregate particles was limited to 16 mm, to minimize the risk of blockages or segregation. Based on a standard specification, the concrete mix design was required to exhibit adequate plasticity and good cohesiveness, and to flow well and compact adequately without compression measures.

The following mix design was chosen for the underwater concrete:

- 1019 kg/m³ sand 0/4
- 85 kg/m³ 4/8
- 595 kg/m³ 8/16
- 350 kg/m³ Cement Cem I 42.5 R SR0
- 180 kg/m³ Admixture Hydraulit-M (AHWZ – active hydraulic admixture)
- 210 kg/m³ Total water content
- 1.7 per cent Superplasticiser MasterGlenium Sky 707
- 0.3 per cent Stabiliser Delvo

To meet the requirements of fresh concrete in terms of transportation and placement, the composition of the concrete needed to exceed clearly the standardized minimum requirements made of concrete type C25/30/XC4/KA1I/UB1/GK16/TV180.

The binder content amounts to 530 kg/m³, with a water/cement ratio of 0.45 which produces excellent cohesiveness (with no tendency to segregation) and achieves the strength required even in unfavourable conditions. The design of the concrete mix took into account the long transportation route, long pump lines, significant water depth, bad visibility on the construction site, and the size of the component to be built.

As a result of the high binder content, the concrete is hardly at risk of segregation and is very suitable for pumping. Because of its low w/c ratio, the compressive strength required could be achieved even if the concrete is not fully self-compacting.

Thanks to an extension in processing time, unscheduled interruptions in concreting of up to two hours could be dealt with.

The concrete exhibited enhanced consistency (slump of 60-70 cm) and good self-leveling properties, spreading over a very large area. Consequently, it was not necessary to move the vertical pipe continuously.

Cement type CEM I 42.5 R SR0 C3Afree was used for two reasons. First, it takes into account the temperature of fresh concrete which is expected to be very low; second, because of the size of the concrete seal. Type C3Afree cements develop far less hydration heat than normal Portland cements.

The bottom outlet intake structure with the concrete block is shown in Fig. 6.

5.4 Compressive strength of underwater concrete

Because the divers were no longer present at the bottom outlet several days after concreting, it would have been very costly to take core samples from the underwater concrete to check its compressive strength. Therefore the divers filled six 10 litre buckets with concrete from the area of the concrete seal on site. These samples were kept in the area of the seal so that they were exposed to exactly the same external conditions as the concrete of the seal during hardening.

Two buckets were raised after three days, two after seven days and two after 23 days. At least three core samples were taken from each bucket, which were then subjected to testing to ascertain their compressive strength. As early as three days after concreting, a mean compressive strength of 23.2 N/mm² was determined; after seven days, 38.6 N/mm²; and after 23 days, 54.6 N/mm². Consequently, the compressive strength of 28 days stipulated by the standard had been considerably exceeded after just seven days.

5.5 Structure for temporary closure of the upper bottom outlet intake

The existing plant was not equipped with a facility to block off the upper bottom outlet intake, because it had been assumed that the reservoir level could be lowered to el. 1665 using the headrace channel, and that the bottom outlet tunnel could then be drained of water.

As a result of the situation described in Section 2, it needed to be possible to block off the upper bottom outlet intake at a reservoir level of about el. 1685, so as to be able to evacuate water from the bottom outlet tunnel upstream of the gate cavern. Therefore it was necessary to develop and install appropriate stoplogs.
The upper bottom outlet intake has three trashracks, each 5.1 m high. The central field has a width of 4 m and each of the two outer fields has a width of 1.5 m.

The central screen array is blocked off by a rear suspension structure in the existing vertical slots for the sliding lock gate, the two outer screen arrays are blocked by plates supported on three sides.

The stoplogs can also be ‘parked’ directly above the intake structure of the bottom outlet. To achieve this, holding supports are mounted on the frame structure of the stoplogs. Redirection systems are mounted at the top of these holding supports so that the stoplogs can be raised and lowered in their plane. This will greatly reduce the risk of the stoplogs jamming in the slots when they are being raised or lowered.

5.6 Assumed water load

The stoplogs of the upper bottom outlet intake are designed for a reservoir level just above the intake surface of the upper headrace intake, that means, at el. 1694.5. This means that a water column of 29.5 m acts against the bottom level of these stoplogs (1665 m). The stoplogs need to withstand this water pressure and remain tight.

5.7 Raising and lowering the 15° angled stoplogs

If the guide rails were not at an angle and there were no redirect points, the winch cable would have a vertical pull action when raising the stoplog. Since the stoplog is located in the rear suspension structure at an angle of 15° to the vertical, there is a risk of the stoplog jamming or becoming stuck in the groove when being raised.

The provision of 15° angled holding supports above the rear suspension structure, and having a redirection system attached to the top of the supports, means that the stoplog can be raised in a 15° angled plane by the cable winch. The winch cable runs around the redirection system.

After the stoplog has been raised by about 5 m, divers lock it in position on both sides using 60 mm-diameter sliding bolts. The holding supports can also be used to hold the ‘parked’ stoplog.

Consequently, the holding supports have two functions. First, they can be used to hold the raised stoplog. Second, thanks to their inclined position and the use of redirection systems, they ensure that the stoplogs can be raised and lowered in an angled plane without jamming in the groove.

The holding supports each weigh about 750 kg and are stiff box sections (300 x 180 mm) which are fastened to the rear suspension structure by a screw plate. The screw connection facilitates the dismantling of the holding supports for inspection purposes.

The rear groove prevents the stoplog from sliding through the opening in the unlikely event of the supports becoming misaligned.

Once the stoplog has been raised and locked in position, the winch cable can be transferred to the other side of the redirection system. With a small lift action, the locking bolts can be removed and the stoplog lowered to the water surface.

The holding supports also make it easy to insert the stoplog. The stoplog is lifted into the slot at the top level, and then the top of the stoplog is tilted towards the holding supports at an angle of 15°. To ensure that insertion into the groove opening is as smooth as possible, the holding plate on the stoplog has been equipped with two openings for cable hooks. The left opening is used for lowering the stoplog because it is left of the centre of gravity, causing the stoplog to hang at a slight angle, and facilitating insertion into the guide frame opening. The right cable hook opening is directly above the stoplog’s centre of gravity.

Relatively little horizontal force acts on the redirection system during the raising/lowering process because the stoplog can be filled with air. Consequently, the buoyant force and weight of steel (about 7 t) more or less offset each other. There is, however, friction. Assuming frictional forces are about 4 t, and given a 15° angled redirection system, the horizontal force acting on the redirection system is 1 t (or 5 kN per holding support). This force can easily be withstood by the holding supports and the rear suspension structure below.

The same holding supports are installed on the two small stoplogs at the bottom outlet intake structure. The supports are connected by cross struts because the width of the guide frame for the stoplog is precisely predetermined.

5.8 Stoplog and rear suspension structure in central field

Since the existing rack and cross beam in the upper bottom outlet intake have only been designed for a load of 30 kN/m², it is not possible to attach a stoplog with a water pressure of 29.5 m x 9.81 kN/m² = 289.40 kN/m² to the rack. Therefore it was necessary to develop a special support structure to transfer stoplog loads to the concrete structure.

Because of the continuous vertical slots for raising and lowering the existing sliding lock gate in front of the lower bottom outlet intake, it was possible to use a steel load-bearing rear suspension structure on each side of the rack. Consequently, the water loads acting on the stoplog can be transferred to the sliding lock gate slots.

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[Image: Suspension structure and holding supports for the stoplogs at the upper intake of the bottom outlet]
Behind the former cable winch building there are openings with widths of 58 to 65 cm. These can be used to insert the stoplogs so the stoplogs can be lowered from above, and there is no need to install them from the front, which would be time consuming and a labour-intensive process.

The rear suspension structure consists of a rear suspension wall with five suspension arms each side. To ensure that the stoplog does not touch the existing rack in a state of deflection when loaded (see Fig. 7), the rear suspension arms are 15 mm shorter than the actual distance between the rack and the sliding lock gate slot. In addition, provisions were made for the installation of filling plates between the arms and slot surface, in case there should be any dimensional deviations between as-built and as-planned. However, maintenance work on the headrace and bottom outlet intake structures had already shown that the former execution drawings corresponded very well with dimensions taken on site.

The two rear suspension walls of unalloyed steel, including the holding supports, each weigh about 3.5 t. Rotated by 90°, they were each lowered from above through the slot in the slab so that they could slide down the rack at an angle of 15° to the perpendicular. The rack was partly covered with a steel plate to allow the walls to slide downwards.

A special cantilevered arm was designed to ensure that the rear suspension wall's centre of gravity lay exactly below the suspension point and the rear suspension wall including holding supports is suspended at the right angle of installation (see Fig. 7).

The cantilevered arm has three holes. The central hole is exactly above the calculated centre of gravity. The two adjacent holes were provided in case small modifications needed to be made during manufacturing, which would have led to a slight change in the centre of gravity.

After the rear suspension wall had been lowered, it was rotated by 90° and the five suspension arms were inserted into the sliding lock gate slot. The rear suspension wall was still attached to the cable while it was being rotated. At the base, the rear suspension wall touched the intake sill in front of the rack. An elastomer bearing seal was used to compensate for any unevenness in the concrete surface at the base (and at the stoplog).

The rear suspension walls had to be fastened directly to the concrete wall, and this was done with bolts. In the area of the rack, one bolt per suspension arm was required; in the area of the slot, two M24 bolts per arm were used. This amounted to a total of 15 bolts per rear suspension wall.

5.9 Transfer of load from the stoplog to the sliding lock gate slot

The 35 cm-wide stoplog runs in bilateral guide rails with a centre-to-centre distance of 37 cm. The suspension wall has a thickness of 50 mm. The five rear suspension arms are fixed to the side plate.

When the tunnel is emptied, the stoplog presses against the guide rails and support arms in the area of the rack, and generates a clockwise moment about the right rear suspension structure. As a result, the side plate and the front part of the rear suspension wall are pressed against the concrete wall, generating a force which acts against the concrete wall.

Simultaneously, water pressure equivalent to a maximum water head of 29.5 m (the reservoir level at el. 1694.5) for each side of 29 m × 9.81 kN/m² × 1.95 m = 554 kN/m acts in the direction of the rack. The abutment force is transferred via the five rear suspension arms and transmitted to the slot.

The load of the stoplog acts at an eccentricity of about 6 cm from the centroid axis of the rear suspension structure and the support arms in the slot do not transfer this force directly at the slot corner but again some 6 cm further inside. This amounts to a total eccentricity of about 12 cm.

The vertical distance between the rear suspension arms is about 1.19 m. Consequently, every support arm must absorb 554 kN/m × 1.19 m = 660 kN of tensile force. This results in a clockwise couple of forces (moment) per rear suspension arm of 660 kN/m × 0.12 m = 79 kN/m about the right rear suspension structure.

To achieve a balance, it must be possible to generate a restoring anti-clockwise couple of forces via the rear suspension structure.

This anti-clockwise couple of forces is generated thanks to the effect of the stoplog whose side metal plates in front of the rack press against the 80 cm thick concrete wall and thanks to the stiff suspension arms with heavy duty anchors that anchor the arms to the concrete.

Calculated displacements caused by water pressure (27 m) on the stoplog rear suspension structure and holding support are shown in Fig. 8.

6. Diving and inspection works in early 2018

Since the powerhouse began operation, the bottom outlet and headrace tunnel upstream of their blocking devices had only been inspected and checked once. This was when the reservoir was flushed in 1977.

In April 2018, it was possible to inspect the bottom outlet and headrace tunnel upstream of the gate caverns, and to check the status of the structures.

In March, the ice cover on the reservoir was melted in the working area by an injection of air from the bottom outlet gate cavern into the bottom outlet tunnel. Then the operating platform was flown by helicopter (in sections)
to the surface of the reservoir, and assembled on site.

To ensure that existing structures were not exposed to excessive water pressure and to minimize the ingress of mountain water to the emptied tunnels, the reservoir was drawn down to el. 1678 (about 19 m above the headrace intake structure and 13 m above the bottom outlet intake structure).

The first step was to empty the bottom outlet tunnel after positioning the stoplogs. To this end, a small difference in pressure was achieved between the reservoir and tunnel, so that divers could identify any possible leakage in the stoplog structures through minor suction forces. They found that the sealing plane between the stoplogs and guide frames was virtually watertight. The only notable leakage was between the guide frames and the existing concrete. This was to be expected, because although surface sealants had been used, it was not possible to achieve completely tight joints between the guide frames and the existing concrete which was, in part, rough and uneven.

Divers sealed the leakage on the water side with cloth, slag and special adhesives, so that water ingress at the three stoplogs at the bottom outlet upper intake was reduced to about 30 l/s, and an inspection of the tunnel could be carried out without a problem. The inspection showed that there was no water seepage from the sliding lock gate. Consequently, it can be concluded that the concrete seal is completely tight, so it is not necessary to carry out post-grouting.

Fortunately, the entire existing structure and the plant components upstream of the gate cavern are in very good condition, so no remedial action is required.

The second step was to empty the headrace tunnel after positioning of the stoplogs. There was a small difference in pressure between the water in the reservoir and that in the tunnel, but few sealing measures were required. The leakage of about 2 l/s was considered negligible, and the headrace tunnel could be inspected without a problem. The tunnel and plant components were also in very good condition. This helps to support the safe operation of the hydropower plant.

The good condition of both tunnels, which are continuously subjected to water pressure but with constant conditions, shows that these are the best prerequisites for the optimal preservation of concrete and steel parts.

7. Conclusion

In late 2017, the project which has been described, involving innovative concreting and installation works, was successfully carried out in very turbid water at a depth of 110 m in the Gepatsch reservoir, part of the Kainertal project in the Tyrol, Austria. As a result, it is now possible to carry out inspections and maintenance work in the headrace and bottom outlet tunnels upstream of the gate caverns in the reservoir during a drawdown to about 20 m above the minimum operating level.

Previously, inspections and maintenance could only be carried out if the reservoir was completely emptied. Complete drawdowns cause the transport of substantial volumes of sediment and bedload from the back to the front of the reservoir, which can result in siltation of intake structures. The drawdown in December 2015 demonstrated how great the danger of siltation can be.

In April 2018, when the reservoir level was low, the new stoplogs were used for the first time and the bottom outlet and headrace tunnels upstream of the gate caverns were emptied of water. The investigation showed that the new stoplogs, their guide frames and the reinforced concrete seal fulfil their function perfectly and are easy to operate. In addition, it was confirmed that the two tunnels and the plant components are in very good condition.

The lower headrace intake was heightened by several metres through the installation of intake towers. The objective was to ensure increased operating safety by preventing the risk of increased siltation.

These concreting and installation measures have been successfully implemented to safeguard the future operation of the Kainertal hydropower plant.

References


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