Experience with Unlined Pressure Shafts in Norway

Expérience gagnée par l'utilisation de galeries en charge sans revêtement, en Norvège

Erfahrungen mit Druckstollen ohne Auskleidung in Norwegen

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SUMMARY

On the basis of experience from more than 25 unlined pressure shafts in Norway with heads of 100 to 450 metres, requirements for successful operation are outlined. Rules for placing the shaft are given particular attention. Cases of failure are discussed.

RÉSUMÉ

Sur la base de l'expérience tirée de l'utilisation de plus de 25 galeries en charge norvégiennes, sans revêtement, à hauteur de chute allant de 100 à 450 mètres, un aperçu est donné des exigences à poser pour assurer une exploitation réussie. Les règles à respecter pour l'implantation desdites galeries sont étudiées plus particulièrement. Des exemples de rupture de galeries en charge sont discutés.

ZUSAMMENFASSUNG

Auf Grundlage von Erfahrungen mit mehr als 25 Druckstollen ohne Auskleidung in Norwegen mit Fallhöhen von 100–450 m wird eine Übersicht über die Anforderungen gegeben, die man für eine erfolgreiche Durchführung der Arbeit stellen muss. Besondere Aufmerksamkeit wird den Richtlinien für die Anbringung von Stollen gewidmet. Beispiele von Stollenbrüchen werden diskutiert.

More than 25 unlined pressure shafts with a static water pressure between 100 and 450 metres have been constructed in valley sides in Norway. These pressure shafts, smaller shafts, and presssure tunnels with higher water pressure in metres than the overlying rock, have performed successfully during operation when the geological and topographic/geometrical conditions meet certain requirements.

It must be emphasized that our experience is restricted to works situated in generally massive, metamorphic, or plutonic rocks with permeability that is negligible. It is also restricted to rock masses with generally low permeability along joints and faults. The total water inflow in the shafts during the construction period has been less than 3 litres per second, concentrated to widely separated joints and clay-filled zones. When such zones, calcite veins, and other local areas of water inflow have been sealed by lining and grouting, the water loss from the shafts during operation has generally been negligible.

The ice erosion during the last glaciation has left the rock surface in Norway without any appreciable weathered zone. There is, however, a tendency to an increased frequency of jointing close to the surface, sometimes including sheet jointing parallel to the surface. Thus, the permeability of rock is usually relatively high to a depth of 5 to 40 metres. This is considered favourable as to the stability of a valley side when the spacing of the joints is not too small. Shafts are always placed under this zone of very high permeability.

Under the conditions mentioned, the following principles should be followed when designing unlined pressure shafts:

1. The shaft should be placed at sufficient depth to ensure a greater in situ stress per-

pendicular to the shaft before construction than the water pressure. This should be applicable at any point along the shaft.

2. The shaft should be placed in areas without major clay-filled zones or very impermeable layers of schists provided they have a) a strike nearly parallel to the valley side, b) a steep or medium steep dip towards the valley, c) a location between the shaft and the valley side. In cases where minor zones or layers with such orientation and location do occur, the following is particularly important: all points of possible leakage from the shaft must be very carefully sealed to prevent the build-up of a high cleft-water pressure behind the zone or layer. In general, an effort should also be made to place the shaft in such a way that major clay-filled faults are not crossed. In particular, attention should be paid to faults that crop out in the valley side, having a medium dip towards the valley. Such faults may cause major slides.
3. All zones in the shaft that may lead to water loss should locally be lined with concrete and grouted. Such points along the shaft include areas with porous rock, open joints, and all clay-filled zones and extensive calcite veins.

DISCUSSION

At the planning stage of an unlined pressure shaft, rock types in the area should be mapped carefully, crushing zones surveyed in detail, and systems and character of jointing studied. As mentioned, particular attention should be given to the possibility that water leakage from the shaft may build up a high cleft-water pressure behind any zone between the shaft and the valley side. Deformation of the rock mass outside the zone may lead to an increase in the in situ stress in the area around the shaft. In evaluating this possibility, it must be noted that not only the permeability but also the extraordinary distribution of stresses along clay-filled zones are of importance. It may often prove difficult to evaluate such conditions at the planning stage. When, however, it seems possible that these conditions do exist, the area should preferably be dropped as a potential site for an unlined pressure shaft.

It is necessary to follow the principle mentioned of sealing the shaft locally by means of concrete lining and grouting, and also to have a reasonable margin of safety as far as the overlying rock is concerned. It seems hazardous to place an unlined pressure shaft in typical porous rock types like certain lavas, sandstones and limestones, or in very blocky and seamy rock. In limestone and marble, karst phenomena may be encountered. Such caverns, as is well-known, are often very difficult to survey at the planning stage, and may lead to difficult or impossible conditions for unlined pressure shafts.

Concerning the topographic/geometrical conditions for placing a shaft, it has been mentioned that the in situ stress must be greater than the water pressure at any given point along the shaft to prevent any deformation out or to the side of the surrounding masses. If this requirement is not met, some selected fissures or joints along the shaft will open, and water from the shaft will have easy access outwards and will cause new deformation. A build-up of the cleft-water pressure along a selected plane in the valley side will be the result. Within a few days or weeks the shaft may 'crack'. The development in detail depends on the geometry of the joints and faults in the valley side. The stress distribution in the valley side indicates that the least favourably orientated joints and faults will govern the deformations that follow. It is quite common to have three systems of joints and in addition different 'wild' joints. Therefore, a continuous failure plane will almost always develop. It can be quite erratic and stepwise, and the water may thus find its way to the valley side several hundred metres away from the shaft. It seems difficult to find a system of joints and faults where the deformation process could certainly stop by itself. Also, it seems unreasonable to design with safety an unlined pressure shaft where such an ideal situation could lead to a reduction in the burden required.

The requirement that the in situ stress normal to the roof of the shaft at any point along the shaft must be greater than the water pressure can be expressed in a simple way as follows:

$$\gamma h \cdot \cos \alpha > H$$

where

γ = bulk density of the rock mass,
h = the vertical depth of the point studied,
α = dip of the shaft,
and H = the static water pressure in metres at the point studied.

This means that for a shaft with a dip of 45° in granitic rocks, $\frac{h}{H} > 0.6$ at any point higher up in the shaft than the lined steel and concrete tunnel that leads into the station. For a 60° shaft, $\frac{h}{H} > 0.8$.

The relationship is not valid for shafts that are steeper than 60°. Such a single straight shaft with a dip of more than 60° should always be placed inside the line representing minimum depth for a 45° shaft. It must be noted that we do not have experience with 100 metres or higher vertical unlined pressure shafts.

If the topography along the valley side is erratic with protruding 'noses' between gorges, the shaft must be placed well inside a line through the bottom of the deepest gorges in the vicinity. Furthermore, unlined shafts must be placed at somewhat deeper levels in valley sides which are shaped as 'protruding noses', caused by tributary valleys or irregular curves of the main valley. This is due to low horizontal stress parallel to the valley side. In regular valley sides in massive rocks in Norway the horizontal stress is sufficient if the shaft is placed as the rule-of-thumb indicates.

The numbers given above are based on a maximum head as achieved with a full intake magazine and without operation of the power station. A sudden stop in the operation of the power plant will lead to a short increase in the water pressure. It may be considerably higher than the static head that has been used above. However, the pressure increase is of such short duration that it seems improbable that it may have serious consequences for the stability of the shaft.

In the rule-of-thumb calculation presented, the contribution to the stress around the shafts from Poisson's effect is not included. Furthermore, the stress distribution that may exist in the valley side due to topographic conditions has not been considered. The reason for this is that these factors are hard to evaluate at the planning stage.

The Poisson's effect should most of all increase the safety at the bottom of the shafts. For a vertical or near vertical shaft this is of course an essential factor. However, the rule laid down above about the placing of such shafts does not necessitate calculations based on Poisson's number.

The anisotropic stress distribution that we find in steep valley sides in Norway when the vertical distance between the works and the mountain-plateau behind is greater than 500 metres, is often characterized by induced extension towards the valley side in crystalline and massive rocks. Experience seems to indicate that in steep valley sides up to 1500 metres, extension cracks can be registered down to 30 metres from the surface. In exceptional cases they may be followed even deeper. Under such conditions, the shaft should be placed as many metres deeper in relation to the rule-of-thumb values as the extension phenomena can be followed from the surface.

It should be quite possible to measure the stress conditions at the bottom of the shaft before it is excavated. A check of this kind, however, has so far not been carried out in Norway.

CASES OF FAILURE

In three of the more than 25 unlined pressure shafts with greater water pressure than 100 metres, the water pressure has led to cracking of the rock around the shaft. It is self-evident that the economic consequences have been considerable, but this will not be discussed here. The three failures will only be discussed in relation to the safety criteria that have been outlined above. It should be added that the other shafts have all satisfied these criteria.

An unlined pressure shaft was designed for the Herlandsfoss Kraftverk, built around 1919. The shaft consisted of 2 parts: a pressure shaft dipping approximately 40°, and a horizontal pressure tunnel below approximately 140 metres long (Fig. 1). From the pressure tunnel, a lined (steel and concrete) tunnel leads into the station. The rock types in the area are hornblende schist and biotite schist.

When water pressure in the pressure tunnel reached 120 metres (measured from the roof of the pressure tunnel), the rock cracked at the spring-line over a distance of approximately 50 metres upstream from the lined part. The line for $h = \frac{H}{\gamma \cos \alpha}$ for this head is drawn in Fig. 1. The cracked part of the pressure tunnel was lined with concrete, but when the water pressure in the pressure tunnel reached 129 metres, even the concrete lining cracked in the same area. The lining was reinforced concrete with a minimum thickness of 50 centimetres. Grouting behind the lining had been carried out. This lining should – in addition to the weight of the overlying rock – have been able to sustain the water pressure. However, it seems reasonable to assume that the rock masses over the tunnel had been lifted during the first cracking, and then deformed downwards again as a plate over the tunnel.

Lining was thereafter carried out near the foot of the shaft, and the water led into the station in a steel tube. No failures have occurred since, even under pressures up to 130 metres above the roof at shaft bottom. The line for $h = \frac{H}{\gamma \cos \alpha}$ at this head is also drawn in Fig. 1.

That no failures occurred in the tunnel at a distance from 60–110 metres from the original lined part during the first two increases in head, may be ascribed to the competence and direction of the hornblende schist. The hornblende schist has few joints cutting across the foliation, dips parallel to the shaft, and has a strike nearly perpendicular to the shaft plane. Due to this, the tunnel section in question may have a certain extra load due to stresses mobilized from the rock further up the valley side. A quantitative value for this increase in stresses around the tunnel, however, is hard to estimate.

Another failure of an unlined pressure shaft

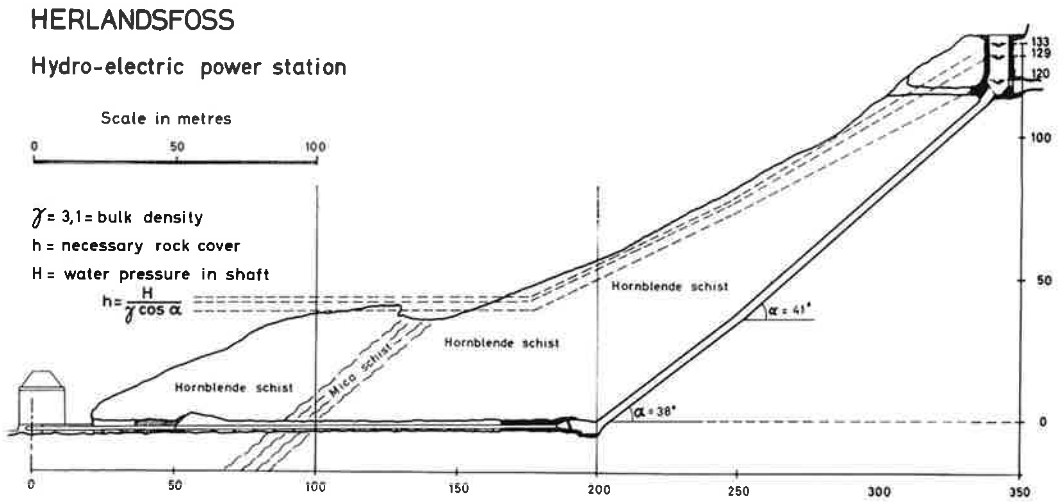

Fig. 1. Failure of unlined pressure tunnel – Herlandsfoss Kraftverk, 1919.

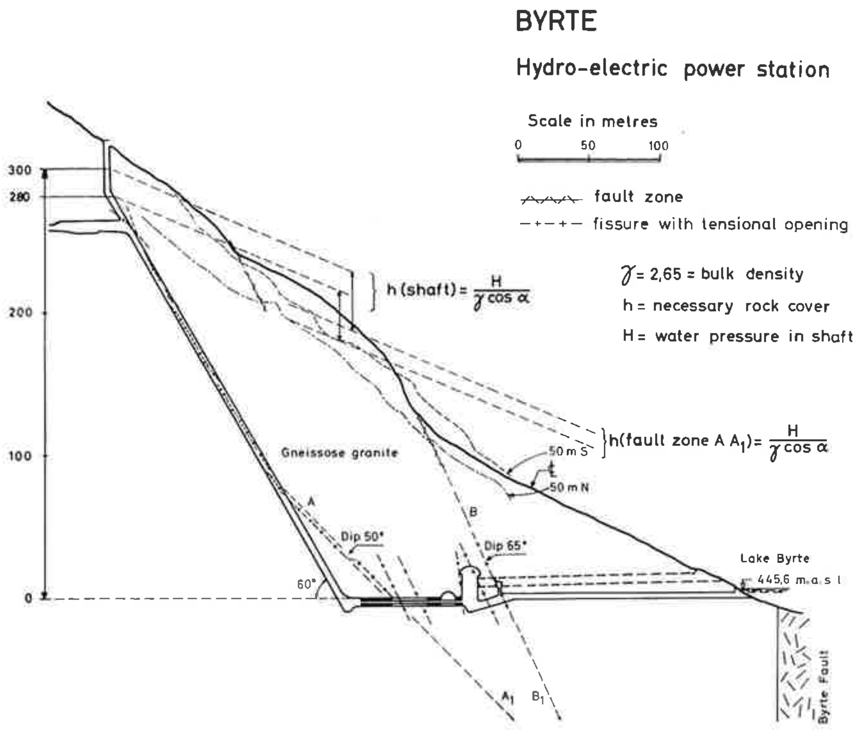

Fig. 2. Failure of unlined pressure shaft – Byrte (Tokke 5), 1968.

took place at the Tokke works in 1968. This case is still under investigation, but it seems possible tentatively to consider the major aspects of the failure on the basis of the rule-of-thumb that has been outlined.

The pressure shaft in question has a dip of 60°. The rock type is gneissose granite ($\gamma \approx 2.65$). Several systems of joints and minor faults occurred with strike approximately parallel to the valley side. The dip is usually between 55° and 70°. No failures were registered when the water pressure for two months was held below 280 metres, measured from the bottom of the unlined shaft. (This corresponds to 720 metres above sea level.) A little leakage, however, was registered. The line for $h = \frac{H}{\gamma \cos \alpha}$ is in this instance quite close to the surface, but for the lower part of the shaft a little higher (Fig. 2).

When the head was slowly increased, the leakage increased somewhat. However, when the following month the head was rapidly raised 20 metres, the rock in the shaft and in the power station area cracked according to the line given in Fig. 2. The line for h at this water pressure is also drawn. It is seen that it lies considerably over the terrain.

It must be mentioned that sealing of the fault zone AA_1 was carried out before the shaft was filled up, but with rather unsatisfactory results. It is therefore reasonable to assume that this zone had sufficient permeability to allow a build-up of a cleft-water pressure corresponding to the full water pressure in the shaft.

The opening of the small clay-filled fault zone AA_1 that followed the cracking of the shaft probably led to an increase in cleft-water pressure and opening of the zone BB_1. The opening of these two zones seems to indicate that the whole rock mass outside the zones has been somewhat moved towards a wide clay-filled fault (Byrte fault) outside the tail race tunnel.

It may be concluded that the directions of the faults and joints were particularly unfavourable in this case. However, some sufficiently unfavourably orientated faults or joints may certainly occur in all rock masses, even if the most careful investigations have been carried out at the planning stage.

The third case of failure happened at Skar Kraftverk, which was built in 1919. The maximum water head was 142 metres. However, since the unlined pressure shaft was placed unreasonably shallow, it seems unnecessary to discuss this case in detail.

CONCLUSION

In conclusion, it seems reasonable to suggest that unlined pressure shafts may well be used when certain basic requirements concerning the geological and topographic/geometrical conditions are met. These requirements are not complex, but they do necessitate careful and extensive engineering-geological investigations at the planning stage. They also necessitate careful inspection during the construction period to ensure adequate lining and grouting of local zones of potential leakage. The cases of failure seem to support the validity of the requirements presented.

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