Design features at Tjodan save time and money

By A. Palmstrøm and K. Schanche,
Senior Geo-engineer* and Senior Structural Engineer*

The Tjodan hydro scheme in Norway incorporates an unlined pressure shaft 1250 m in length; this design allowed a considerable cash saving and also reduced the construction time by approximately two months. The first part of this article deals with the design and construction measures taken for the 880 m-head unlined pressure shaft. The latter part deals with a new concept for sealing of the concrete section at the base of the pressure shaft which has been introduced at the project; this has proved to be an effective method for shafts subjected to high water pressures.

Unlined pressure shafts and tunnels have now been applied in more than 70 Norwegian powerplants. The method can offer reduced construction costs and shorten the construction time. The design criteria used, cost savings and the experience gained from construction of the 1250 m-long unlined shaft at Tjodan powerplant in Norway are described.

The Tjodan plant is situated in southeast Norway, some 50 km east of the town of Stavanger. The powerplant has an output of 110 MW from a single Pelton turbine with a head of 900 m and a production of 310 GWh/year. The drainage area is 54 km^2 and the precipitation is approximately 3000 mm/year.

The powerplant is a typical example of current Norwegian hydropower construction, consisting of 20 km of tunnels collecting water through five reservoirs and five secondary intakes from a wide area and utilizing it in a 900 m head Pelton turbine (Fig. 1). It comprises almost all the different, advanced hydropower solutions now regarded as traditional by Norwegian hydropower engineers.

The characteristic features of the project are:

*Ingeniør A.B. Berdal A/S, Partner of Norconsult A/S, Norway.

- two rockfill and six concrete dams;
- one pumping station with a head of 130 m;
- four lake taps at depths of 15-25 m;
- short planning and construction time with only 3½ years from start of detailed planning until completion;
- unlined pressure shaft with a static head of 880 m; and;
- TBM drilling of the pressure shaft and a 5 km-long tunnel in hard, granitic rock.

The term unlined means that no steel piping is installed in the shaft, with the result that the rock itself is under direct pressure from the water.

The possibilities and benefits of constructing unlined pressure tunnels and pressure shafts have attracted little attention in other countries. In Norway, however, there are more than 70 unlined, high-head pressure shafts and tunnels, and about ten more are under construction or are at the planning stage. The Tjodan scheme has one of the highest heads, with a pressure of 880 m on unlined rock; another shaft with 1000 m head is presently under construction. The method gives both a lower construction cost and a reduced capital cost during construction due to a shorter construction period for the whole scheme. In addition, a bonus is added for the plant coming on stream sooner, as will be shown later.

Location of the unlined pressure shafts was at first based on the simple theory that the weight of the rock above was greater than the pressure of the water in the shaft.

In 1972 a better simulation model was introduced, based on the finite element method (f.e.m.). This model makes use of the theory that the minimum principal stress in the rock should not be exceeded by the water pressure. The requisite rock cover is arrived at by transferring the scheme to topographical models adapted to local conditions. In determining the final location of the shaft, however, special attention has to be paid to any significant geological factors that may be present.

This method provides a better base of calculation than the

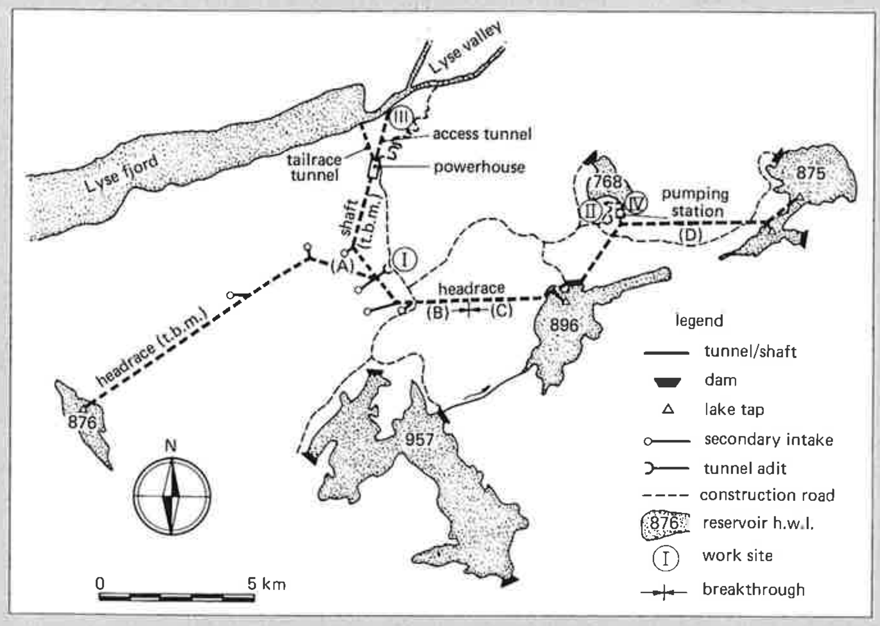


Fig. 1. General view of Tjodan hydropower plant.

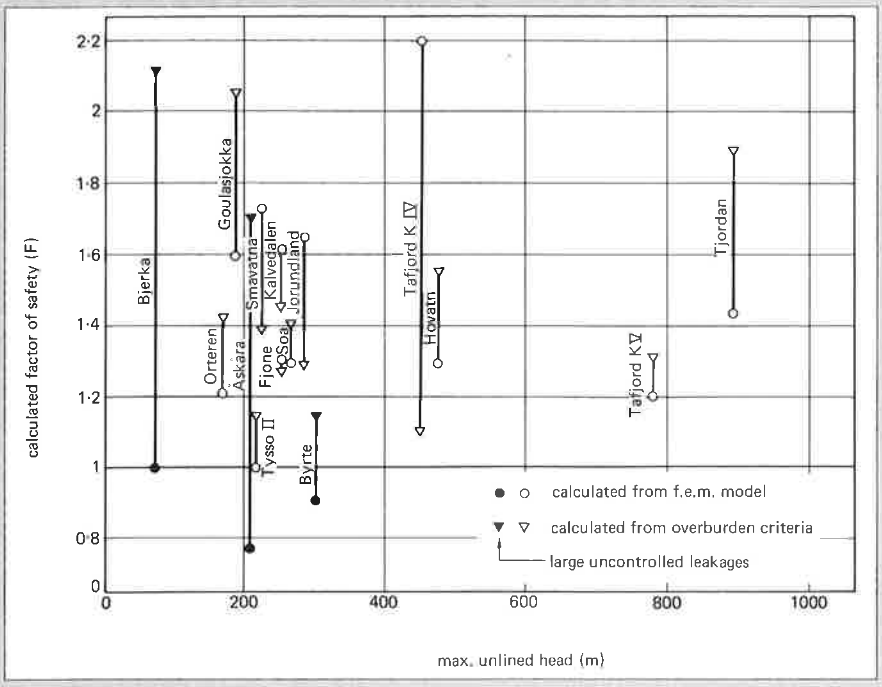


Fig. 2. Safety factors of some Norwegian pressure shafts/tunnels calculated on the basis of f.e.m. and overburden criteria.

overburden criterion formerly employed against hydraulic splitting. Fig. 2 shows that it indicates a safety factor of less than 1 in respect of certain plants which have suffered such leakage. Had this method been adopted for these plants it would probably have resulted in a different, and safer, location. For this reason the f.e.m. was chosen for the computations required to determine the site of the Tjodan pressure shaft.

Preliminary surveys

A water pressure of 880 m directly onto the rock at the base of the shaft was about 100 m more than that met with at the Tafjord hydropower plant, see Fig. 3. In dealing with such great pressures on unlined rock it is essential to operate with reliable assessments of both the geological factors and the overburden required, to be assured of a suitable location where large and undesirable leakage is avoided.

The geological mapping of the area showed that the gneisses had a low to moderate degree of jointing. No weakness of fracture zones of importance were found which would intersect the planned shaft. Laboratory tests revealed that the gneisses had an average compressive strength of 160 MPa and a Poisson's ratio of 0.2.

To assess the overburden required, a simple f.e.m. calculation was first performed with standard diagrams. For this purpose the terrain in a section along the length of the shaft was simplified (Fig. 4). With a selected safety factor of 1.2 the power station was preliminarily located 800 m inside the mountain.

To provide a more accurate picture of the probable magnitude of the minimum principal rock stress, a special f.e.m. analysis was performed. The computations carried out were adapted to the topographical and geological conditions met with at Tjodan. Five different models of the terrain were used for this purpose. These indicated that the pressure shaft

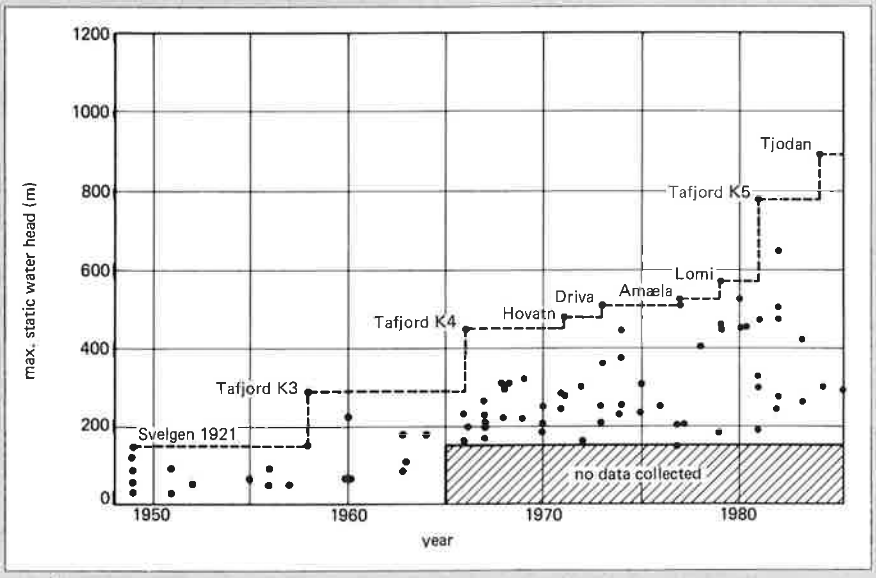


Fig. 3. Development of pressure shafts/ tunnels in Norway.

It was assumed that this feature affected the rock stress pattern. The tendency towards spalling phenomena inside the zone supported this assumption. Accordingly, it was decided to rely more on the hydraulic splitting tests to be performed in the penstock chamber inside the zone (see Fig. 5) before the final location of the pressure shaft was decided.

The results here showed that the minimum principal stress was about 12.5 MPa, 0.6-2 MPa more than had been estimated in the f.e.m. calculations. Fig. 6 shows the variation in the safety factor along the shaft, based on these results. With the same location as had been preliminarily chosen, this gave a minimum safety factor of 1.43 against hydraulic splitting for the shaft. This was accepted as satisfactory and the preliminary

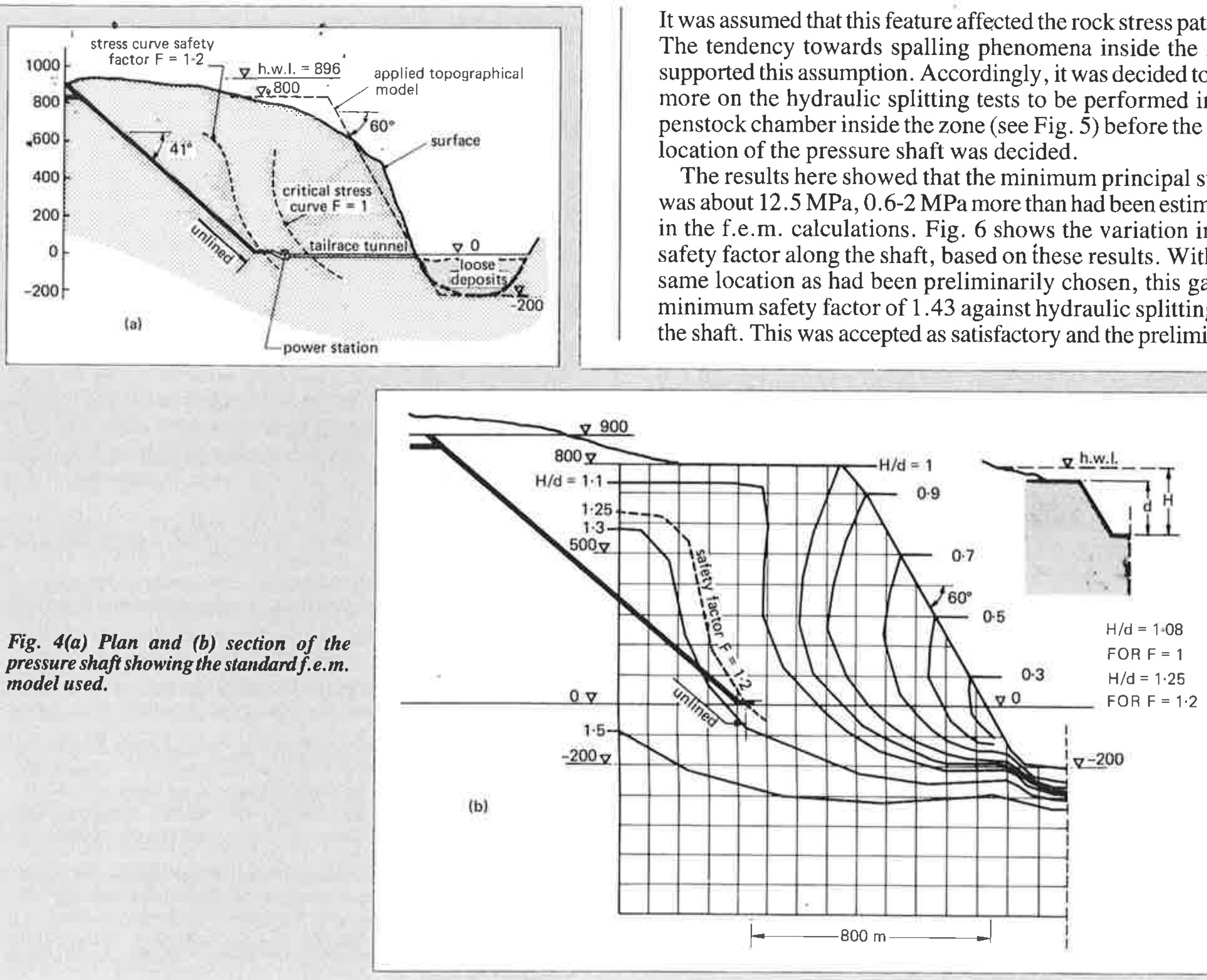


Fig. 4(a) Plan and (b) section of the pressure shaft showing the standard f.e.m. model used.

had a probable safety factor of 1.2-1.35, depending on the different assumptions of rock stress distribution. The preliminary location was therefore found acceptable for detailed design.

Measures carried out during construction

Uncertainties remained about the exact magnitude of the rock stresses. To find a more exact magnitude of the minimum principal stress it was decided to perform rock pressure measuring tests during tunnel excavation. These consisted of both three-dimensional stress measurements and hydraulic splitting tests in the powerhouse area. The measurements were first performed in the adit down to the tailrace tunnel 150 m from the station (Fig. 5). The reason for carrying out the first stress measurements here was to be able to change the planned location of the power station if there proved to be an appreciable difference between stresses actually measured and those computed.

The survey showed that the directions of the principal stresses were more or less what had been expected, with the main principal stress running just about parallel to the steepest portion of the mountainside. The magnitude of the stresses was, however, less than had been estimated, as the stresses appeared to be contingent only upon the weight of the overlying rock, whereas included in the f.e.m. analysis were factors pertaining to the horizontal stresses. The results of the hydraulic splitting tests, likewise, showed low stresses.

When the access tunnel was driven a little further it was found that there was a joint zone just inside the measuring point.

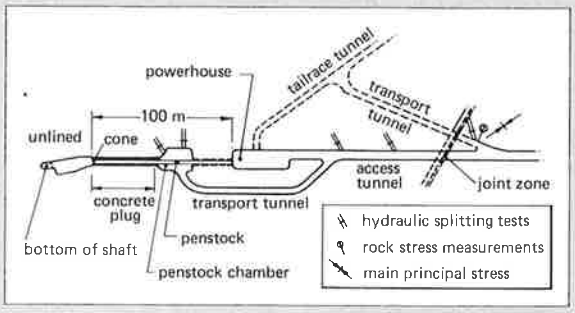


Fig. 5. Locations of rock stress measurements performed in powerhouse area.

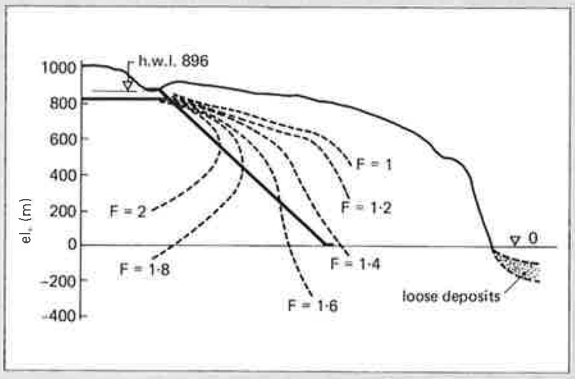


Fig. 6. Safety factors calculated from f.e.m. analysis and rock stress measurements.

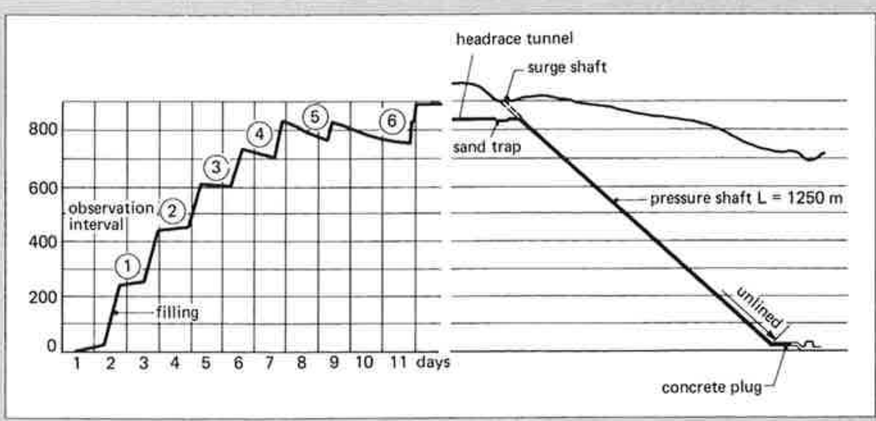


Fig. 7. Filling the pressure shaft in six steps.

location was maintained as final.

While the shaft was being TBM drilled, both water leakages and geological conditions were continuously monitored. On the basis of the data thus compiled, the rock support and sealing measures needed were planned and described. At five places extensive sealing and grouting work had to be carried out. This work was supervised by an experienced geo-engineer, who stayed at site during the whole drilling and sealing period.

Sequential filling of the shaft

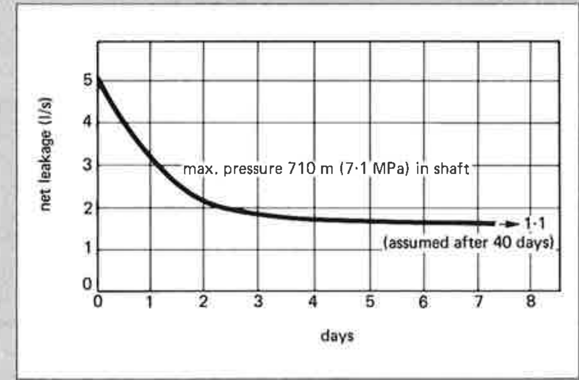
While a shaft is being excavated, joints and pores in the surrounding rocks are drained to the point that leakage, fed by the groundwater in the rock mass, is reduced to an even flow. For Tjodan this was recorded as 0.2 m^3/min into the entire shaft.

When a shaft is filled with water the joints, and pores in the drained zones around the shaft become filled too. This generates a considerable pressure, which inevitably introduces the danger of deformation. This hazard can be reduced substantially by slowly filling the shaft.

If the first filling of the shaft is done slowly and sequentially, extensive, unforeseen leakage can be observed much sooner. This means that the tunnel system can be emptied in time before flooding occurs and major damage is caused. These observations can be done during the intervals between filling steps. Earlier experience has shown that during an interval of 10 to 20 h a steady state of leakage is achieved.

At Tjodan it was decided to fill the shaft in seven steps (Fig. 7). During the intervals between filling periods, the level of water in the shaft was continuously and accurately monitored by an extra-sensitive manometer. By deducting the estimated natural groundwater leakage into the shaft and the measured leakage at the cone, it was possible to calculate net leakage into the rock.

Fig. 8. Net leakage out from shaft to surrounding rock massess with 710 m head in shaft.



Each filling took from 6 to 12 h, separated by step intervals of 18 to 58 h for monitoring. The longest intervals were made when the level of water in the shaft was highest. In the course of these monitorings it was found that on average it took far longer for a steady leakage to occur than had been experienced in other shafts earlier (Fig. 8). It was estimated that at the highest level it took as much as 1000 h (40 days) for a constant rate of leakage from the shaft to be achieved. This suggests that the rock masses are of very low permeability. The estimated

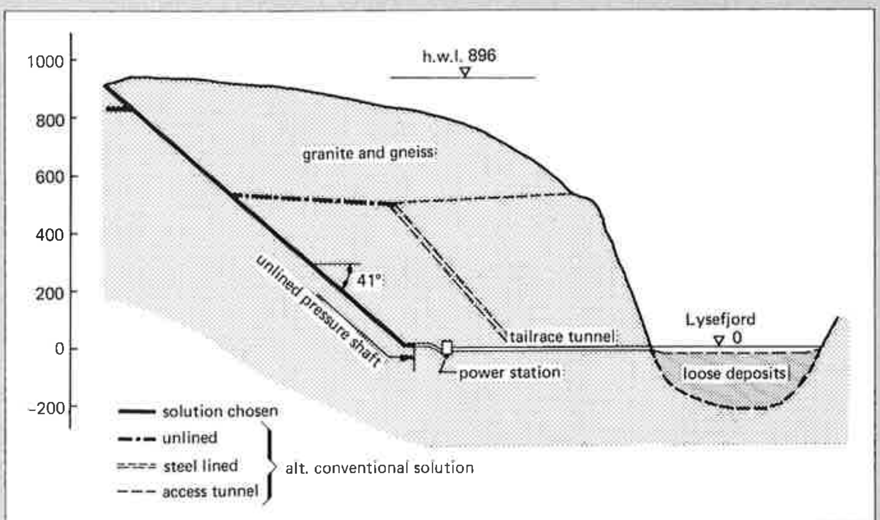


Fig. 9. Two different pressure shaft solutions were evaluated.

average permeability coefficient of the rock masses from the results was 7×10^{-9}m/s, which can be considered very low.

No leakage into the powerhouse, the transport tunnels or the access tunnels was recorded. The powerhouse is located 100-150 m from the base of the shaft.

Total leakage from the shaft (water loss) was found to be about 2 l/s, including some 0.7-0.8 l/s from the plug. (Leakage from the plug has subsequently been reduced to 0.1 l/s.) This low figure was considered to be because of the low permeability of the rock masses, coupled with careful and thorough injection in the shaft and plug area.

It can be mentioned that prior to the construction of the Tjodan plant, at least seven pressure shafts/tunnels in Norway had been similarly filled in stages. Leakage from these averaged 0.5-5 l/s per km of tunnel, depending on the pressure of the water. At Tjodan the leakage was measured to approximately 1 l/s per km.

Cost savings of the unlined shaft

The design of Tjodan powerplant started out with the lower half of the pressure shaft being steel lined. This could have been done from a construction adit as shown in Fig. 9. In the detailed design phase it became evident that the geo-engineering survey of the area revealed that the rock mass conditions were favourable for unlining the whole shaft. An estimate was made which showed that not only would an unlined shaft be US$3 million cheaper, but that it would probably be possible to shorten the construction period by at least two months. The overall saving resulting from bringing the plant on stream earlier was estimated at US$2.7 million. The costs of the extra geological survey work, computations, and assessments totalled US$22 000 giving an overall saving of US$5.7 million.

The cost savings compared with a fully steel-lined shaft are substantially higher.

Sealing of the concrete plug

A new sealing method has been devised specifically for concrete plugs at the base of the unlined pressure shafts which are subjected to high water pressures. This method offers a better sealing of the concrete joints, and of the interfaces between concrete/rock and concrete/steel by injecting an epoxy resin of low viscosity through fibre-strand hoses placed before concreting. Total leakage through a plug exposed to 900 m water head has been recorded to 0.1 l/s.

The method was developed and first used for the plugs at Tjodan powerplant. The shaft is not lined, hence the water pressure direct on the rocks and the concrete plug is about 900 m. Sealing work for a concrete plug against a static water pressure of 90 bar is a considerable challenge. Although similar projects with lower pressure have been sucessfully completed in the past, a situation with a pressure of this magnitude was altogether new.

Leakages generally occur both through cracks in the rock, at rock/concrete and concrete/steel interfaces, through joints in the concrete or through the concrete itself. The rocks in the area consist of Precambrian gneisses with a compressive strength of 150-200 MPa. They are moderately jointed and occasionally intersected by fault zones up to 10 m wide. To guard against leakage through the rocks, an area adjacent to the sealing zone in the upstream end of the concrete plug was injected with cement through 20 m-long holes, drilled radially to form a fan.

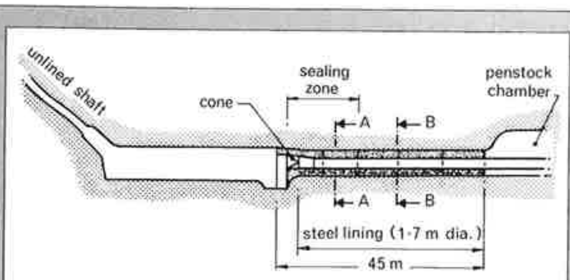


Fig. 10. Concrete plug, longitudinal section.

Development of the sealing method

Contact grouting between concrete/rock and concrete/steel is generally done through predrilled holes from the steel lining into the concrete and rock. However, for maintenance reasons this was not desirable at Tjodan, and alternative methods had to be considered. Two different requirements had to be complied with:

- no holes were to be drilled in the steel lining; and,
- precise positioning of the injection compound from the concrete cone upstream or from the penstock chamber downstream was required.

This led to the adoption of the Swiss Jekto system, which was found best suited to meet the requirements. It had, however, never previously been employed at such pressures. Earlier it was generally used in ordinary construction work, for water reservoirs, and in the shafts of concrete platforms for the oil industry, up to 150 m head.

The plan for the work was drawn up by the consulting engineer in cooperation with representatives of the building contractor, the supplier of the Jekto system and the epoxy resin supplier.

The system developed special advantages because of its flexibility, easily adapting to the irregular rock surface. The Jekto system consists basically of a fibre-strand hose for injection. It is made up of two hoses with an intermediate fleece to prevent clogging, and in order to prevent collapse under fresh concrete a helical spring is inserted into the inner hose.

The sealing works

The task of concreting the steel pipe in the plug at the base of the shaft commenced in August 1984. The length concreted was 45 m, divided into five 9 m-long vertically jointed sections with a horizontal joint underneath the steel lining, Fig. 10. A sealing zone was prepared in the two sections farthest upstream, and it was here that most of the injection hoses were located. At the rear of the sections epoxy was injected in the

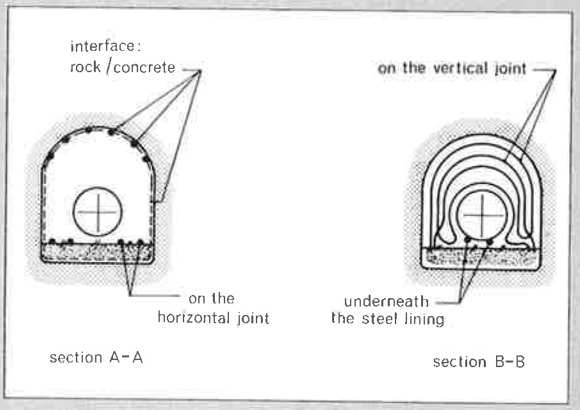


Fig. 11. Typical sections showing the injection hoses before concreting the upper part. (All connection tubes from the injection hoses in the two sections farthest upstream were led to the cone, the rest were led to the penstock chamber).

vertical joints to ensure that the sections functioned as a monolitic "plug" to withstand the horizontal thrust resulting from the pressure of water. In addition to installing Jekto fibre-strand hoses, it was necessary to fill all voids between concrete and rocks in the tunnel roof by cement-based compound injected through steel tubes. This was done from the vertical joints in every section, Fig. 11. All ends of the fibre-strand hoses were fitted with smaller high pressure polyamide tubes for connecting the injection pump leading to the upstream cone or to the penstock chamber. Concreting of the plug proceeded according to plan and took five weeks.

The epoxy used was of extra low viscosity and was custom-made for injection; it has a long service life, hardens without difficulty at low temperatures, and is specially designed for sealing and anticorrosion properties. Its viscosity at 20°C is about 130-150 cps and the gel time is about 4 h. At 10°C viscosity is up to 450 cps and the gel time is about 8 h.

During injection it was found that the penetration of the epoxy resin was very good. It crossed constructed barriers and in some places it forced its way through what was apparently flawless concrete. This presented a problem: how to limit the area to be injected?

The problem was solved in some places by pumping in polyurethane, which reacted rapidly with the epoxy resin to form a barrier. As the injection work drew to a close, the epoxy resin was allowed to harden in the outer section before more was pumped in. In this way the resin itself acted as a barrier.

Once the epoxy resin had hardened, test holes were drilled close to the concrete cone through the rock/concrete interface. These holes were then checked for water loss. The inflow of water was found to be minimal.

At full water pressure in the shaft an overall leakage at the downstream end of the concrete plug of about 0.9 l/s was recorded. Since then leakage has constantly decreased, and by the beginning of 1986 it was about 0.1 l/s. This result was considered to be very satisfactory.

Later experiences

Since the positive results at Tjodan, the method has been used at several sites in Norway. In February 1987 the unlined pressure shaft of Naddvik power station in the Nyset-Steggje hydropower scheme was filled with water. The gross head is 980 m and the sealing work of the concrete plug is similar to the method used at Tjodan. At full water pressure an overall leakage of about 1 l/s was recorded.

Although the results till now are good, there is still room for improvement. It is especially desirable to be able to restrict injection to sharply delineated areas to reduce the volume of grout used.

Recently, other designs of hose have been produced such as the German FUKO and the Belgium INFILTRA-STOP fibre-strand hoses.

At some of the sites where this method has been used, the results have not been as expected. The method itself is quite reliable but it requires solid concrete construction. In addition to a systematic working procedure and constant surveillance, experienced sealing personnel is a good guarantee of a successful project. □

Acknowledgements

The construction of the unlined pressure shaft at Tjodan was made possible by a close and fruitful cooperation with senior engineer Nils G. Ihlen, representing, the owner Lyse Kraft A/S.

Colleagues at Berdal, in particular Halvor Hagen, Erik Fleischer and Bjørn Buen have given worthful advice during the planning and follow-up of the shaft construction.

The experienced, practical work of engineering geologist Jan Idar Kollstrøm, during site follow-up, has been most important for the successful result.

Bibliography

1. "Norwegian unlined tunnels and shafts with more than 100 m static head", Report 54402, Norwegian Geotechnical Institute, Oslo, Norway; 1972.
2. Bjørlykke, S. and Selmer-Olsen R., "Necessary overburden for caverns with high internal water or air pressure," Report No. 6, Department of Geology, Technical University of Norway; 1972.
3. Selmer-Olsen R., "Experience with unlined pressure shafts in Norway," International Symposium on large underground openings, Oslo, Norway; 1969.
4. Buen, B. and Palmstrøm, A., "Design and supervision of unlined hydro power shafts and tunnels with head up to 590 meters," ISRM Symposium, Aachen, Balkema, Rotterdam, Netherlands; 1982.
5. Broch, E., "The development of unlined pressure shafts and tunnels in Norway," ISRM Symposium Aachen, Balkema, Rotterdam, Netherlands; 1982.
6. Broch, E., "Unlined pressure tunnels in areas of complex topography," *Water Power & Dam Construction;* November 1984.
7. Oddlang, P., "Full face drilling — experience from shaft drilling at Tjodan power plant," Fjellsprengningsteknikk — Bergmekanikk, Tapir, Trondheim, Norway; 1984.
8. Palmstrøm, A. and Kollstrøm, J. I., "Construction of the unlined pressure shaft at Tjodan power plant," Fjellsprengningsteknikk — Bergmekanikk, Tapir, Trondheim, Norway; 1985.