NORWEGIAN HYDROPOWER TUNNELLING II

NORWEGIAN TUNNELLING SOCIETY

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INTRODUCTION

The publication "Norwegian Hydropower Tunnelling II" is part of the English language series published by the Norwegian Tunnelling Society, NFF. The aim is to share with our international colleagues information on rock technology, - this time with focus on tunnelling and underground works related to hydropower. As indicated in the title, this publication is the second publication in our series devoted to hydropower tunnelling. The first publication was published in 1985 as No. 3 in this series, contained 18 papers and has during the years been spread all over the world. A list of content is included in this publication as an appendix.

Some brief information about the hydropower industry in Norway: Annual production is in the order of 140 - 150 TWh and covers 95-96% of the electricity consumed by the 5 million inhabitants, - one of the highest (if not the highest) consumption per person in the world. During the last 50-60 years practically all hydropower stations have been located underground, - in total more than 200, which is in the order of 25% of all underground power stations in the world. A total of 4000 km of tunnels have been excavated for the hydropower projects, - the majority by the drill and blast method, but also approximately 200 km by the TBM method. So-called small hydropower projects, i.e. less than 10 MW, are now built at a rate of almost 50 per year.

Special Norwegian solutions like the unlined high pressure tunnels and shafts, the air cushion surge chamber, the lake tap, the sand trap and concrete plugs are described and discussed in several papers in this publication. A very special solution is to use ice to plug a tunnel for inspection and repair works. Raise drilling and directional drilling for small and mini hydropower projects are also described. Norwegian engineers have been involved in hydropower projects in a number of countries and have learned important lessons in an international market. Some of this is included in the papers presented in this publication.

On behalf of NFF we express our sincere thanks to the authors and contributors of this publication. Without their efforts the distribution of Norwegian tunnelling experience would not have been possible.

Oslo, April 2013

Norwegian Tunnelling Society NFF - International Committee

Einar Broch **Elisabeth Grasbakken** **Werner Stefanussen**

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CONTENTS

01. UNDERGROUND HYDROPOWER PROJECTS - LESSONS LEARNED IN HOME COUNTRY AND FROM PROJECTS ABROAD

BROCH, Einar

1 INTRODUCTION

It is natural to start this paper by giving a brief description of the development of the hydropower industry in Norway, and in particular concentrate on the underground aspects. This is presented in Chapter 2. One special lesson learned from the Norwegian hydropower projects is that it is possible to replace the standard ventilated surge chamber by an unlined rock cavern operating as an air cushion. This described in Chapter 3. Having been involved in different ways on hydropower projects in many countries around the world, the author also includes some lessons learned from some selected projects. In Chapter 4 some samples of problems caused by special types of rock masses and stress conditions in water conveying tunnels are discussed.

2 THE DEVELOPMENT OF UNDERGROUND HYDROPOWER PROJECTS IN NORWAY

Topographical and geological conditions in Norway are favourable for the development of hydroelectric energy. The rocks are of Precambrian and Paleozoic age, and although there is a wide variety of rocks, highly metamorphic rocks predominate. From an engineering point of view they may in general be classified as typical hard rocks.

More than 99% of a total annual production of 140 TWh of electric energy in Norway is generated from hydropower. Figure 1 shows the installed production capacity of Norwegian hydroelectric power stations. It is interesting to note that, since 1950, underground pow-

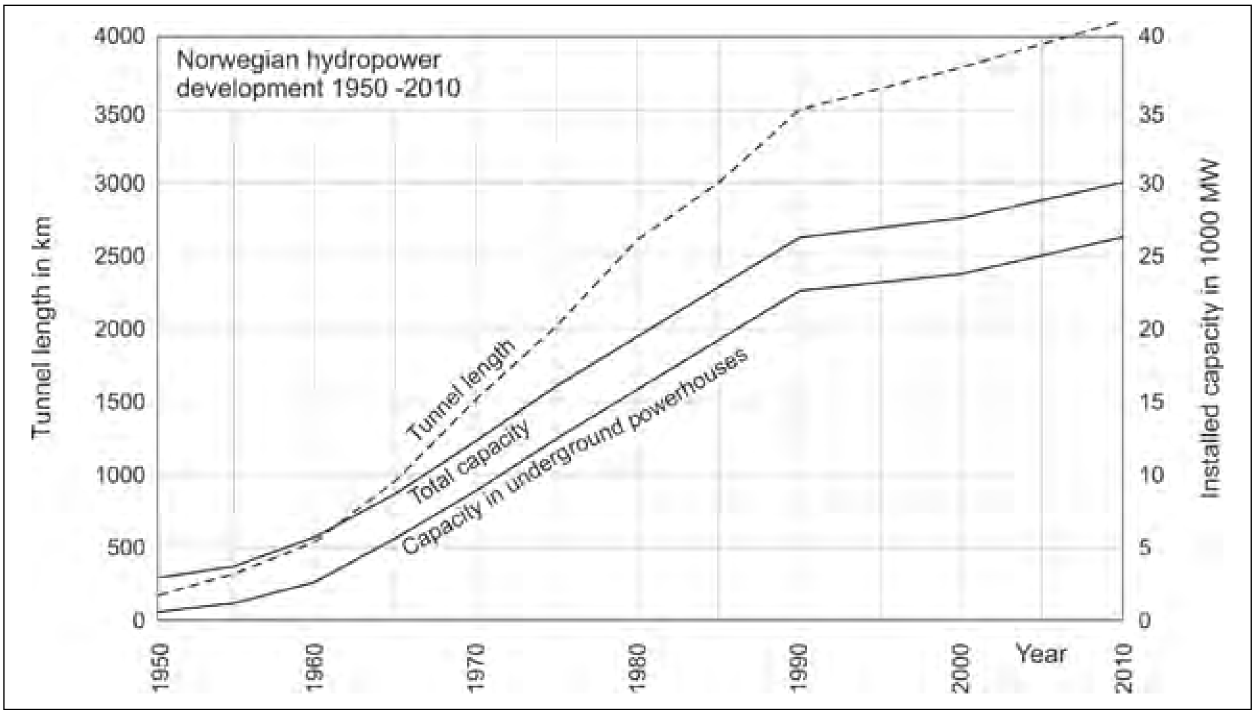

Figure 1. The development of Norwegian hydroelectric power production capacity and the accumulated length of tunnels excavated for the period 1950 -1990.

erhouses are predominant, (Broch, 1982). In fact, of the world's 600-700 underground powerhouses, one third, i.e. 200, are located in Norway. Another proof that the Norwegian electricity industry is an "underground industry" is that it today has 4000 km of tunnels. As the dotted line in Figure 1 shows, during the period 1960 - 90 an average of 100 km of tunnels was excavated every year.

Through the design, construction, and operation of all these tunnels and underground powerhouses, valuable experience was gained. This experience has been of great importance for the general development of tunnelling technology, and not least for the use of the underground. The many underground powerhouses excavated in rock masses of varying quality are to a large extent the forerunners for the varied use of rock caverns which we find all around the world today, (Edvardsson & Broch, 2002). Example of an underground powerhouse from the early 1950s is shown in Figure 2. In this case a concrete building has been constructed inside a rock cavern. The powerhouse has in fact false windows to give people a feeling of being above ground rather than underground.

Later people became more confident in working and staying underground, and powerhouses were constructed with exposed rock walls, often illuminated to show the beauty of rock such as demonstrated by two powerhouses commissioned around 1970 and shown in Figure 3.

Some special techniques and design concepts have over the years been developed by the hydropower industry. One such Norwegian speciality is the unlined, high-pressure tunnels and shafts, (Broch, 1982B, 2000). Another is the so-called air cushion surge chamber which replaces the conventional vented surge chamber, (Goodall et al., 1988)

Most of the Norwegian hydropower tunnels have only 2 - 4% concrete or shotcrete lining. Only in a few cases has it been necessary to increase this, and in these few cases only a maximum of 40 - 60% of the tunnels have been lined. The low percentage of lining is due not only to favourable tunnelling conditions. It is first and foremost the consequence of a support philosophy which accepts some falling rocks during the operation period of a water tunnel. A reasonable number of rock fragments spread out along the headrace or tailrace tunnel floor will not disturb the operation of the hydro power station as long as a rock trap is located at the downstream end of the headrace tunnel. Serious collapses or local blockages of the tunnel must, of course, be prevented by the use of heavy support or concrete lining where needed. Normal

Figure 2. The Aura underground hydropower station, commissioned in 1952

Figure 3. Åna-Sira (over) and Tafjord K-5 (under) underground powerhouses

water velocity in the tunnels is approximately 1 m/sec. During and after the Second World War, the underground solution was given preference out of considerations to wartime security. But with the rapid advances in rock excavation methods and equipment after the war, and consequent reduction in costs, underground location very soon came to be the most economic solution, see Figure 4. This gives the planner a freedom of layout quite independent of the surface topography. Except for small and mini-hydropower stations, underground location of the powerhouse is now chosen in Norway whenever sufficient rock cover is available.

When the hydropower industry for safety reasons went underground in the early 1950's, they brought the steel pipes with them. Thus, for a decade or so most pressure shafts were steel-lined. In 1958 at the Tafjord K3 hydropower station a completely unlined shaft with a maximum water head of 286 m was successfully put into operation. This gave the industry confidence in this time and money saving solution. As Figure 4 shows, new unlined shafts were constructed in the early 1960's and since 1965 unlined pressure shafts and tunnels have been the conventional Norwegian solution. Today more than 100 unlined high-pressure shafts or tunnels with water heads above 150 m are success¬fully operating in Norway, the highest head now being more than 1000 m. Figure 5 clearly demonstrates that increasing water heads reflect an increasing confidence in unlined pressure shafts.

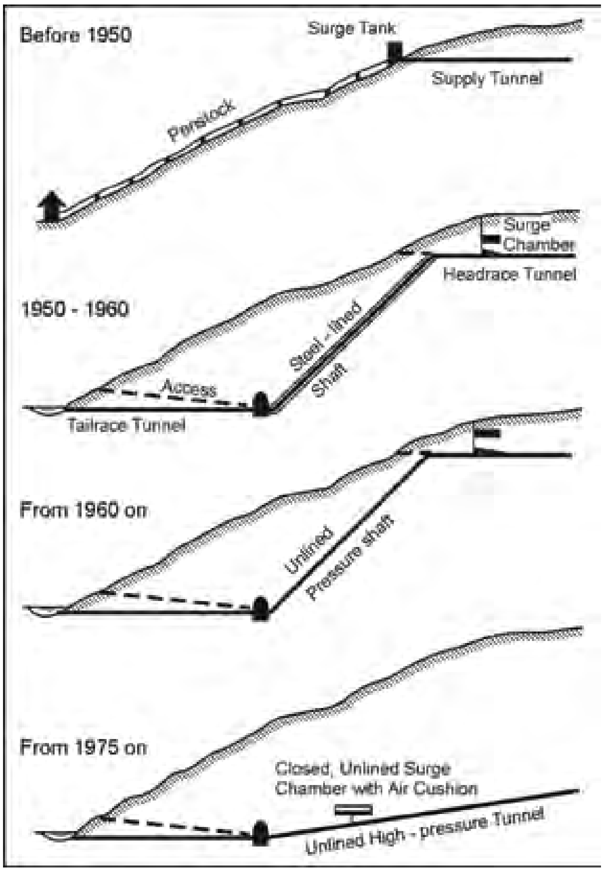

Figure 4. The development of the general layout of hydroelectric plants in Norway

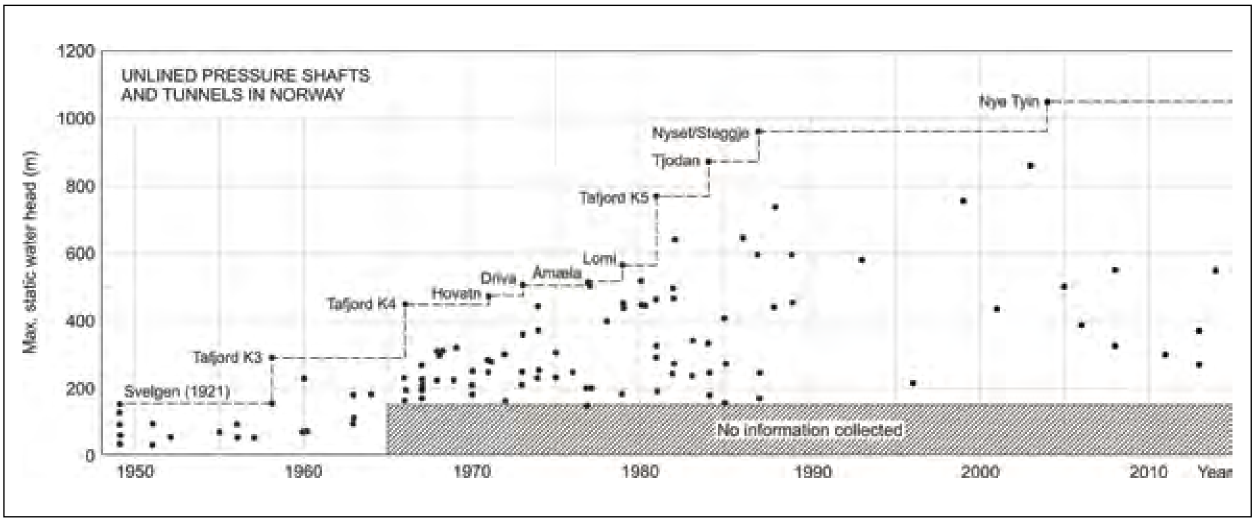

Figure 5. The development of unlined pressure shafts and tunnels in Norway.

3 AIR CUSHIONS

The confidence in the tightness of unlined rock masses increased in 1973 when the first closed, unlined surge chamber with an air cushion was successfully put into service at the Driva hydroelectric power plant, (Rathe, 1975), (Goodall et al., 1988). The bottom sketch in Figure 4 shows how this new design philosophy influenced the general layout of a hydropower project. The steeply inclined pressure shaft, normally at 45o, is replaced by a slightly inclined tunnel, 1:10 - 1:15 grade. Instead of the conventional vented surge chamber near the top of the pressure shaft, a closed chamber is excavated somewhere along the high-pressure tunnel, preferably not too far from the powerhouse. After the tunnel system is filled with water, compressed air is pumped into the surge chamber. ¬This compressed air acts as a cushion to reduce the water hammer effect on the hydrau¬lic machinery and the waterways, and also ensures the stability of the hydraulic system. Ten air cushions are now in operation in Norway, and compressed air with pressure up to 83 bars, equalling a water head of 830 m, have been successfully stored in unlined rock caverns. These air cushions may also be regarded as full scale test chambers for storage of gas in unlined rock caverns.

The first containment principle for storing of air in unlined rock caverns is that any internal storage pressure must be sustained by the minimum in-situ rock stress ($\sigma 3$) to avoid hydraulic splitting.

Secondly, the ground water pressure and the gradient of the water seepage towards the caverns provide the containment. The rock material itself has in most cases an insignificant permeability. Hydrodynamic control by the groundwater is the main principle of containment for storage of air in unlined rock caverns. In some cases, the hydrostatic head from the natural ground water may be sufficient. In other cases, one ‘assists’ the natural ground water by infiltrating water into the rock mass around and above the caverns, by ‘water curtains’. These are established by drill holes from the surface or designated infiltration galleries. Normally, the requirement to the hydrostatic head will be the dimensioning factor for the cavern elevation. The Norwegian Explosives and Fire Safety Authority require a safety margin of a minimum 20m water head above the water head corresponding to the storage pressure.

Thirdly, if the rock mass is more permeable than desirable, grouting is performed to ensure safe operation (permeability control). This reduces the overall inflow of water into the storage volume, reduces pumping costs, and ensures a high gradient close to the cavern contour, increasing safety against air leaking out of the cavern. As a rule, the grouting needs to be performed as pre-excavation grouting of the rock mass. Post-excavation grouting should only be allowed as a supplement after pre-grouting; it is not a substitute for pre-grouting.

Table 1: Overview of main data for compressed air storage, including air cushion surge chambers

Project	Commis-sioned	Main rock type	Excavated volume, m^3	Cross section, m^2	Storage pressure, MPa	Head/ cover*)	Experience
Driva	1973	Banded gneiss	6,600	111	4.2	0.5	No leakage
Jukla	1974	Granitic gneiss	6,200	129	2.4	0.7	No leakage
Oksla	1980	Granitic gneiss	18,100	235	4.4	1.0	<5Nm^3/h
Sima	1980	Granitic gneiss	10,500	173	4.8	1.1	<2Nm^3/h
Osa	1981	Gneissic granite	12,000	176	1.9	1.3	Extensive grouting
Kvilldal	1981	Migmatitic gneiss	120,000	260-370	4.1	0.8	Water infiltr. necessary
Tafjord	1981	Banded gneiss	2,000	130	7.8	1.8	Water infiltr. necessary
Brattset	1982	Phyllite	9,000	89	2.5	1.6	11Nm^3/h
Ulset	1985	Mica gneiss	4,800	92	2.8	1.1	No leakage
Torpa	1989	Meta siltstone	14,000	95	4.4	2.0	Water infiltr. necessary

*) Ratio between maximum air cushion pressure expressed as head of water and minimum rock cover

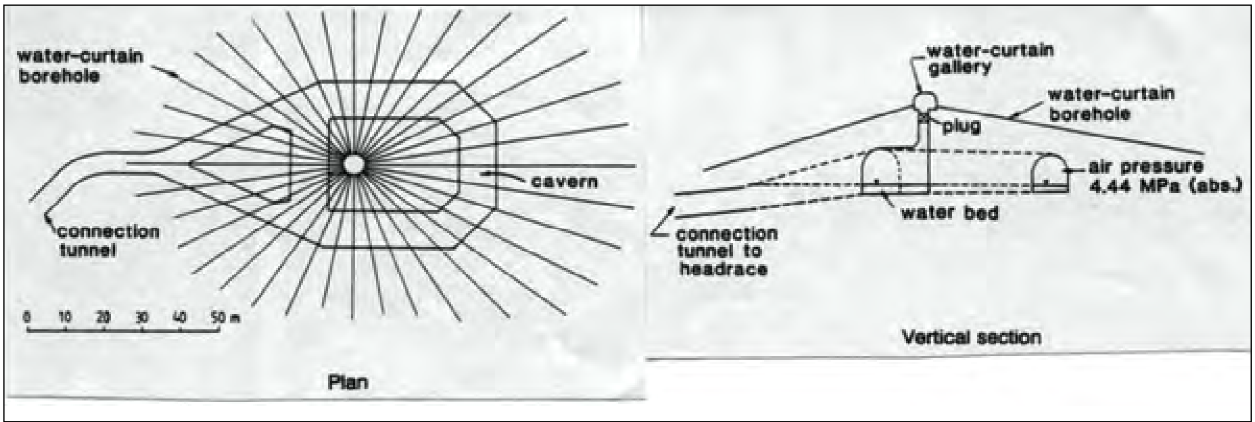

Figure 6. Air cushion surge chamber with water infiltration curtain at Torpa (Kjølberg, 1989)

Table 1 shows the main data for ten air cushion surge chambers built in Norway, (Kjørholt et al, 1992). Remarkably, the caverns range in size from 2,000m^3 to 120,000m^3 and have operating pressures of 1.9MPa to 7.8MPa, serving the need for surge dampening for power-plants with installations from 35MW to 1240MW. Note the increasing trend to greater ratios between water head and rock cover over the years, indicating the increasing confidence.

As shown in Figure 4 typically air cushion surge chambers are located adjacent to the headrace tunnel within a limited distance from the turbines. However, in some cases distances exceeding 1000m have been acceptable. This provides a large flexibility in the location of the chamber in the best available rock mass. The chambers are in most cases designed as a single cavern, but in two cases (Kvilldal and Torpa) they have been given a doughnut shape. All chambers are unlined with a minimum support of rock bolts and sprayed concrete, as minor rock falls during operation are accepted.

The air loss from the chambers may be due to both air dissolution into the water bed below the air cushion (annual losses for typical caverns are 3-10% of the air volume), and leakage through the rock mass. Three chambers have no leakage at all through the rock mass and six chambers have acceptable losses (within reasonable compressor capacity). Three chambers (Osa, Kvilldal and Tafjord) showed natural leakage rates that were too high for economical operation. This necessitated remedial measures. For Osa, extensive post-grouting reduced the leakage to an acceptable level. For Kvilldal, where the leakage probably was resulting from a near by weakness zone, a water infiltration curtain was established that totally eliminated the leakage. For Tafjord, it appears that hydraulic splitting took place during the first filling due to unusually low minor principal stress conditions considering the rock cover. Repair attempts by sealing of the split joint failed. The plant was operated for some years in 'tandem' with another plant without its own surge chamber. In 1990-1991, a water curtain was installed, and the air leakage disappeared when the curtain was put in operation with pressure 0.3MPa above the air cushion pressure.

For the tenth chamber, at Torpa, a water curtain was included in the original design, and installed from a designated gallery above the chamber as shown in Figure 6. During construction the rock mass around the doughnut shaped chamber was pre-grouted. Extensive rock stress measurements were performed with a variation of results; some indicated the minimum rock stress to be as low as the storage pressure at 4.4MPa (Kjølberg, 1989). Without the infiltration running, the leakage rate was 400Nm^3/h; with the curtain in operation at 0.2MPa above the air pressure, there is no measurable leakage.

The experience from designing and operating unlined air cushions confirms the following (Blindheim et al., 2004):

- Thorough geotechnical investigations to obtain relevant information about the hydro-dynamical and rock mechanical conditions are required.
- The rock cover must provide sufficient rock stress to avoid hydraulic splitting of the rock masses by the air pressure.
- The groundwater level should be maintained during construction with the use of water infiltration curtains, unless location is possible in very favourable rock mass and a high groundwater level.
- Water infiltration is an effective means for maintaining the ground water level, and thus the confining effect of the hydrostatic head (hydrodynamic control). Used in combination with pre-grouting, excessive water consumption can be avoided. Water infiltration curtains have successfully been installed in areas of groundwater drawdown.

- Systematic pre-grouting is necessary if strict requirements to tightness need to be satisfied (permeability control). High pressure pre-grouting of the rock mass with micro- or ultrafine cements minimising the remaining water inflow, ensures tightness by controlling the gradient close to the contour, and provides operational safety and economy. Grouting of concrete plugs needs special attention.

4 LESSONS LEARNED FROM WATER TUNNELS OUTSIDE NORWAY

Tunnels designed and constructed for carrying water are special in the way that during the construction period they are full of air, often dry air with high velocity because of the ventilation system, while in operation mode they will be filled with flowing water. Dry rocks are normally stronger than wet rocks, and some rocks may even contain minerals that start swelling and expanding when exposed to water. Also gouge material in faults and weakness zones intersected by tunnels often contains swelling minerals. In some tunnels and shafts for hydropower projects the stresses in the periphery of the tunnel may vary with changing water head in the tunnel. Thus there are several geological/topographical factors that need special attention for tunnels designed to convey water. In the following subchapters some selected cases from the author`s involvement in projects outside Norway will be discussed.

4.1 Tunnelling in "crazing" basaltic rocks

The 45 km long headrace tunnel for the Muela hydropower station in Lesotho, also referred to as the Transfer tunnel, goes through basalts for its entire length. This basalt is of Jurassic age and overlies the Clarens sandstone. It dominates the highlands of Lesotho. In the tunnel area the rock is in general hard and strong with a uniaxial compressive strength of between 85 and 190 MPa. The entire length of the Transfer Tunnel was very successfully excavated with 5 m diameter TBMs. Record breaking advancements rates were obtained.

Initially, some 91 % of this tunnel was expected to fall into a rock support class requiring no more than spot bolting. It was also impressive to see the quality of the finished TBM tunnel shortly after it had been bored. Rock falls were only observed in a few areas of very high rock overburden where the rock was clearly overstressed. These areas were supported with rock bolts and wire mesh. The tunnel was in general very dry, in fact over long stretches it was dust dry.

However, as time went by some cracking and "sloughing" of the rocks was observed in the few wet places along the tunnel. This was also typically observed along the invert where water from the boring process flows constantly. A phenomenon known as "crazing" was observed. This is a form of rock deterioration or weathering which occurs in highly amygdaloidal basalts. Studies revealed that this is caused by the reaction of two mineral types occurring in the highly amygdaloidal basalt. When in contact with water, smectite minerals in the characteristic amygdales or matrix of the basalt swell causing the rock face to desintegrate, see Figure 7.

Figure 7. "Crazing" due to weathering in amygdaloidal basalts in the Transfer tunnel in Lesotho.

In addition, active zeolites, in particular laumontite, caused fine fracturing in weak, highly amygdaloidal basalt. Both these conditions caused deterioration, ranging from very minor weathering of soft minerals to the sloughing of large slabs or weakened rock. Degradation was, however, not wholly confined to highly amygdaloidal basalts, although it was in this type of rock that almost all the areas of the more severe weathering occurred.

Having identified the nature of the problem, many solutions were considered. One immediate suggestion was the application of a protective skin of shotcrete. It was surprising to learn that the relative cost of this obvious solution was higher than the conventional in-situ concrete lining. There were also some concerns about the long term durability of the shotcrete in this high pressure water tunnel.

A comprehensive system for the evaluation of the quality of the rocks along the tunnel was made. The prime factor was a weatherability classification which was developed locally. The intention was to identify the places where concrete lining was needed. The major problem turned out to be that at any place along the alignment, the cross section of the tunnel was intersected by at least two horizontal basalt flows. Even though one or two of the flows were of good quality, very often one basalt flow of poor quality intersected the tunnel, and thus concrete lining was needed for this.

It is also an economical fact that the concrete lining procedure cannot be stopped without costs. In fact a 300 m long section of the tunnel was the minimum length of good rock which was needed to stop lining. Thus the task of the tunnel geologist was not any longer to identify the poor rock that needed support, but to find 300 m or longer sections where they could guarantee the long term stability of the rock. The final conclusion was that the whole 45 km long Transfer Tunnel needed lining.

4.2 Tunnelling in friable sandstone

4.2.1 CASE 1 – Guavio hydropower project
On November 7, 1983 the excavation from the downstream adit of the 5.25 km long, 65 m^2 tailrace tunnel for the Guavio Hydropower Project (8 x 200 MW) in Colombia, had reached Station K4+ 567 as shown in Figure 8 , Broch, (1996).

The lower part of the tailrace tunnel goes through the so-called Une-formation, which belongs to the Cretaceous era, i.e. approximately 100 million years old, - see Figure 8. The formation is dominated by aeolian sandstones, some of them with a rather high porosity and low diagenesis, which means that they are poorly cemented and have a uniaxial compressive strength as low as in the order of 10-20 MPa. During the drilling of a probe hole from the centre of the tunnel face, water under high pressure was struck at a depth of 25 m. The leakage increased rapidly to 7 l/sec (420 l/min), and fine sand started coming out of the borehole. During the following day two slides occurred. From these two areas up to 40 l/sec and 70 l/sec of water was pouring out for a couple of hours. After one day a total 350 m^3 of sand had been flushed into the tunnel, and the tunnel face had moved 4 m from Station 4+567 to Station K4+563. It was then decided to block the tunnel face with concrete.

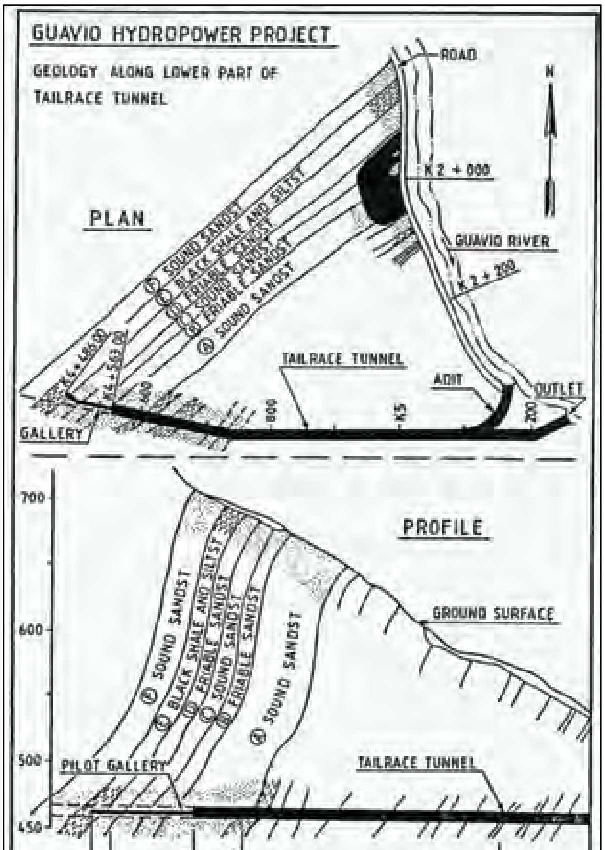

Figure 8. Geology along the lower part of the Guavio tailrace tunnel.

During the following months several attempts were made to reduce the ground water level above the tunnel as this is normally the most effective way of solving stability problems in friable sandstones. The result was a number of small inflows of sand. By early February a total of 5000 m^3 of sand had flowed into the tunnel, and it was obvious from all attempts that it was impossible to reduce the pore water pressure below 20 bars, i.e. 200 m water head. (The annual rainfall in this part of the Colombian jungle is as high as 4 - 5000 mm). It was therefore decided that the rock mass ahead of the tunnel face should be stabilised with a grouting procedure, and that a 3.5 m diameter pilot tunnel should be driven through the unstable zone. This was very complicated and time consuming work. In spite of all precautions several slides or inflows of sand occurred, so when the whole 77 m long pilot tunnel after 15 months was finished, a total of 15000 m^3 of sand had flowed into the tunnel.

Typical for these water and sand inflows was that they started as small water leakages followed by the inflow of sand which eroded the drill hole and thus increased the capacity of the hole, which again allowed more water and sand to flow into the tunnel. It was also commonly observed that the inflows had a pulsating character. After strong inflows which could last for some hours, the inflow decreased for some time and then increased again. The most serious water inflow was as high as 400 l/sec. To cope with this, the tunnel face had to be blocked with a bulk head, and the rock mass was re-grouted before new excavation could start.

The excavation diameter for the final tunnel was 8.5 m. Several possible solutions for the excavation of this tunnel were evaluated, among them freezing. A method based on grouting and drainage through radial drill holes was, however, chosen. This is shown in Figure 9. A 6 m thick ring of grouted rock outside the final tunnel was established.

The maximum distance between the grout holes was only 1.5 m in the middle of the ring. The grouted ring was then drained through holes which where 1.0 m shorter than the grout holes and had a spacing of 3.0 m. Grouting was done in three stages starting with cement/bentonite, followed by a thick silicate - mix and then a thinner mix as the final stage. All drilling was done through blow-out preventers and all drain holes were equipped with filter tubes. A large number of piezometers were installed to monitor and control the pore water pressure

In addition to the 3000 m^3 of grout used for the pilot tunnel 2250 m^3 of cement/bentonite and 6250 m^3 of silicate were used for the radial grouting. This gives 11500 m^3 grout for a 77 m long tunnel, or 150 m^3 per m tunnel. All grouting was completed by May 1986.

Final excavation of the main tunnel was done by the use of a roadheader. The upper half of the tunnel was excavated first and preliminary secured. The excavation was done in 1 m steps, followed by the installation of heavy steel ribs at 1.0 m spacing. Reinforced shotcrete was applied between the steel ribs. Final support includes a circular concrete lining.

This 77 m long section of the tailrace tunnel for the Guavio Hydropower Project was completed three and a half years after the first serious inflow of water and sand. Fortunately it did not delay the completion of the project as the difficulties were met at an early stage in the construction, and it was possible to speed up the excavation of the tunnel from the upstream side.

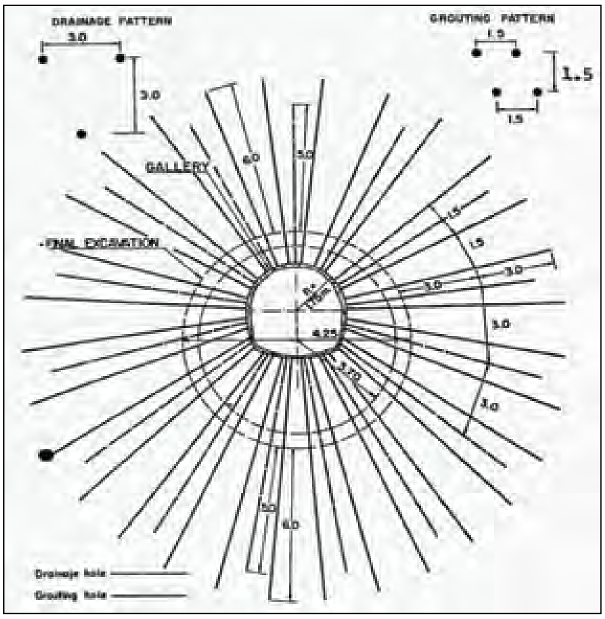

Figure 9. Pattern for the radial grouting and drainage for enlargement of the pilot tunnel of the Guavio tailrace tunnel.

4.2.2 CASE 2 – Delivery tunnel, Lesotho Highlands Water Project

The Muela hydropower station, which is part of the Lesotho Highlands Water Project, as well as Muela Dam and the Delivery Tunnel South are all in the so-called Clarens sandstone, which is a very uniform sandstone, partly of aeolian origin. It is quite similar to the Une sandstone in Colombia, but somewhat older (Jurassic) and stronger. The rock is however also friable, but is not subjected to high pore water pressures like in Guavio, and the stability in the powerhouse is very good.

The Delivery Tunnel South was excavated by a 5 m diameter TBM. Only minor stability problems were encountered during the tunnel boring process. After several weeks overstressing phenomena were, however, observed in the sidewalls of the tunnel. These phenomena were locally called "dog-earing" and are shown in the picture in Figure 10.

The "dog ears" developed slowly, but consistently. A full concrete lining was finally needed to stop further spalling. Measurements of the uniaxial compressive strength (UCS) and the vertical stress, showed that overstressing always occurred where the ratio was lower than 2.5. There were indications that time dependent overstressing might occur even for ratios up to 4.0. These stress induced spalling phenomena are rather different from the violent rock bursting that is observed in the Norwegian hard, crystalline rocks.

5 CONCLUDING REMARKS

As shown in the described examples with weak and unstable rock masses, headrace and tailrace tunnels for hydropower projects in such conditions need to be properly supported by concrete or shotcrete linings. There are, however, lots of cases around the world where hydropower tunnels are excavated in rock masses that are only slightly affected by the water. In such cases considerable cost savings can be made by reducing the amount of lining to an absolute minimum. The cost of lining a meter of tunnel is often in the order of two to three times the cost of excavating the tunnel. And to put it frankly: The water does not care if there are some minor rock blocks along the tunnel floor, - and a rock trap at the end of the headrace tunnel. This has been demonstrated through decades of successful operation of several hundred unlined hydropower tunnels in Norway.

With a good understanding of the rock stresses in the planned area for the underground powerhouse, Norwegian experience has also shown that it is possible to convey the water to the powerhouse through unlined highpressure shafts and tunnels with water heads more than 1000m. Steel pipes or steel linings are only used for the last 25 – 75m dependent on the water head. It is basically a question of putting the powerhouse and thus the shaft deep enough into the hill side so that the rock stresses along the shaft at any point is greater than the internal water pressure. Avoiding installation of a steel lining means not only considerable direct cost savings, but also saving of construction time for a part of the project that often is on the critical path.

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Figure 10. "Dog-earing" in sandstone due to high vertical stresses in the Delivery Tunnel in Lesotho

Lower Røssåga Powerstation. A waterhead of 246 metres. 6 Francis turbines, each 50 MW. The project started the operation in 1955. Photo: Statkraft.

02. PLANNING OF PRESSURIZED HEADRACE TUNNEL IN ALBANIA

AASEN, Oddbjørn
ØDEGAARD
PALMSTRÖM

ABSTRACT
A major hydropower project is under planning in the eastern parts of Albania. The 10.7 km pressurized headrace tunnel is designed based on the Norwegian "unlined waterway" principle, only including the required rock support.

1 BACKGROUND AND PROJECT DESCRIPTION
A major hydropower project is under planning in the eastern parts of Albania, consisting of three hydropower plants along the Devoll river with an installed capacity of approximately 272 MW, which will generate 800 GWh once the plants are finished and operating, corresponding to an increase of electricity production in Albania by 20%.

The upper hydropower plant, HPP Moglicë, is designed according to Norwegian design principles with unlined pressure tunnel, and utilizes a head of 300 m along an approximately 22 km long stretch of Devoll River, as schematically shown in Figure 1.

The intake is situated in the Moglicë reservoir created by the approximately 150 m high Moglicë Dam, outlined in Figure 2.

Figure 2: Outline of Moglicë dam

A headrace tunnel of length 10.7 km conveys the water to the powerhouse located underground on the north bank of Devoll River. The tailrace tunnel is approximately 900 m long leading to the Devoll River. HPP Moglicë powerhouse is equipped with two Francis units with a total combined capacity of 171 MW and an average annual energy production of 445 GWh.

The project owner is a joint venture between Norwegian Statkraft, and Austrian EVN, established in Albania under company name Devoll Hydropower Project (DHP). Main consultants for the concept study, feasibility study and tender design have been Norconsult AS supported by Multiconsult AS on all underground design works.

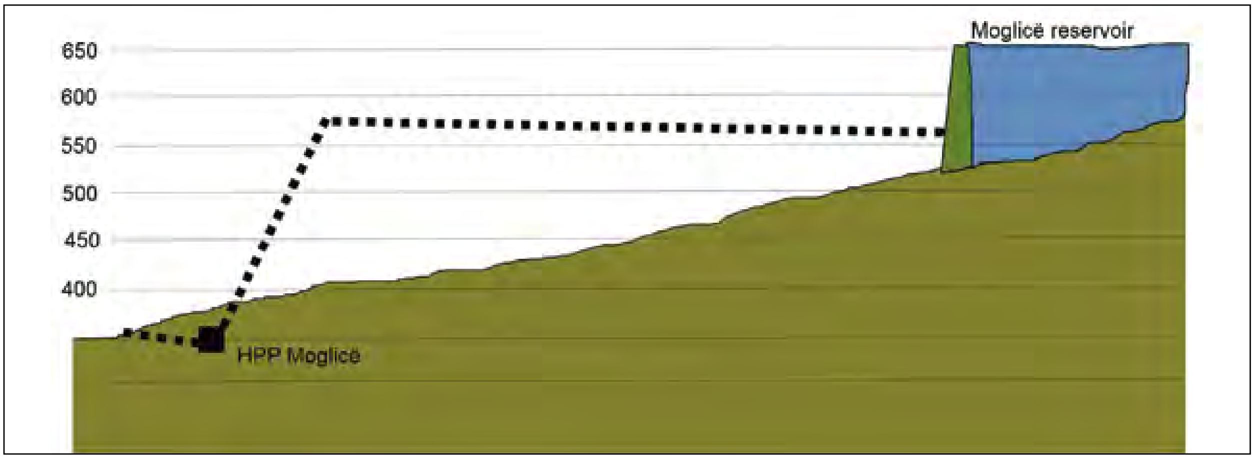

Figure 1: Schematic overview of HPP3

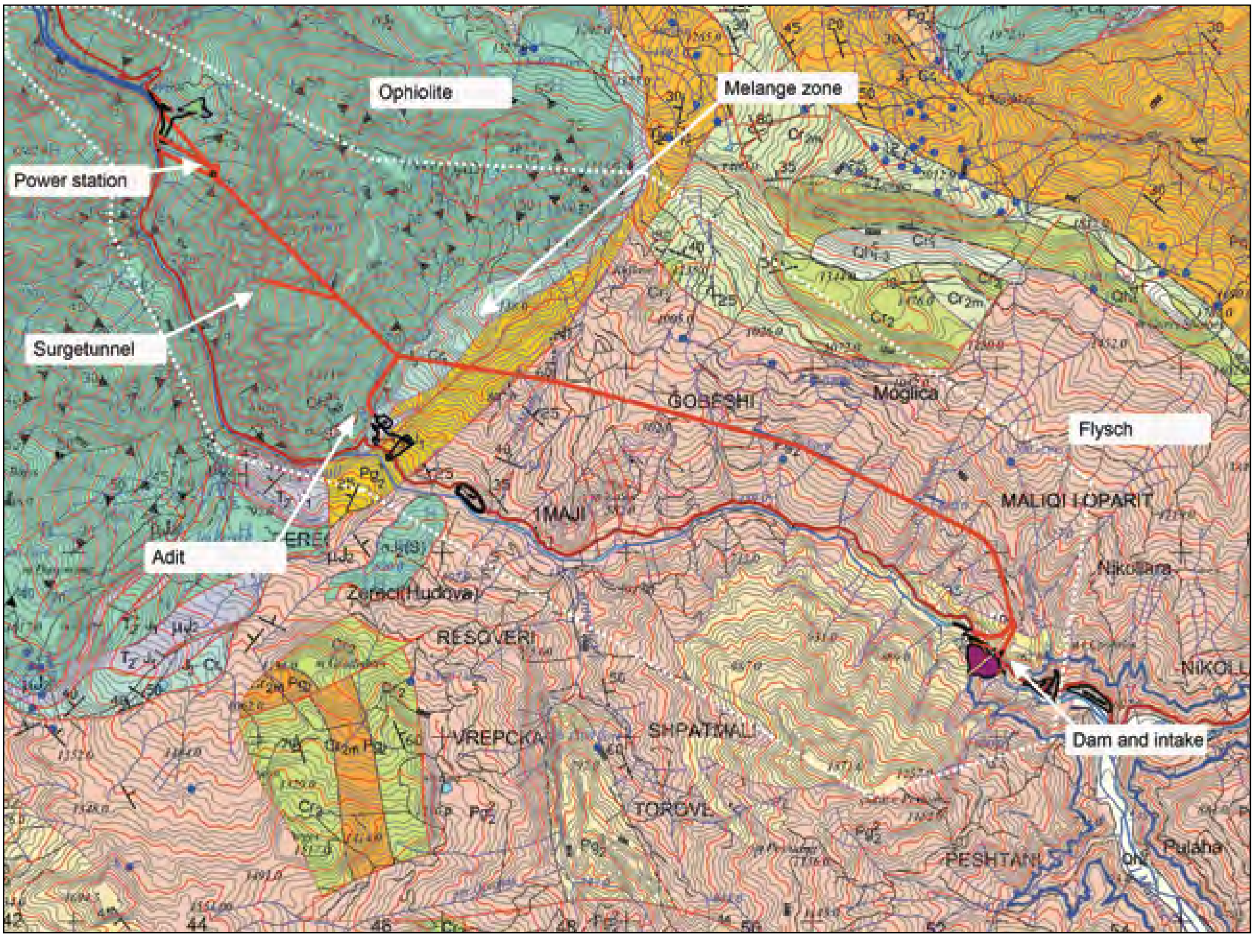

Figure 3: Excerpt of the geological map of Albania with the tunnel system shown in red

2 GROUND CONDITIONS AND INVESTIGATIONS

2.1 General geology

In any hydropower project the ground conditions are a great project uncertainty, and great efforts should be made to provide a good understanding of the geological conditions affecting the project. The geological units encountered in the project area vary considerably both in their origin and in their mechanical properties. There are two main lithological units, the ophiolitic rocks, mainly variants of Peridotite, shown in green color in Figure 3, and various sedimentary rocks, shown in pink and yellow color in Figure 3. The border between these two units is of tectonic nature and of very poor quality.

2.2 Rock mass conditions

2.2.1 Introduction

On the mechanically sound end of the rock mass scale are the Ophiolitic rocks found in the downstream part in this project. The main concern regarding rock mass quality within the ophiolite has been localized serpentinization of the peridotite and fault zones.

The flysch typically consist of alternating layers of claystone, siltstone, sandstone and rare conglomerates, with the stronger sandstones typically creating a rigid "skeleton" within the weaker silt- and claystones. The mechanical characteristic of the rock mass is thus governed not only by the characteristics of each individual layer, but also by the proportion of the different rock types.

While the ophiolite found in the project area is fairly homogenous and sound, this is not the case with the Flysch which can be extremely heterogeneous, and locally of very poor rock mass quality. A geological cross section showing the various rock types from core drilling is shown in Figure 4.

3 GROUND INVESTIGATIONS

Ground investigations for the underground part of HPP Moglicë consisted mainly of rotary core drilling and refraction seismic measurements. From the core drilling

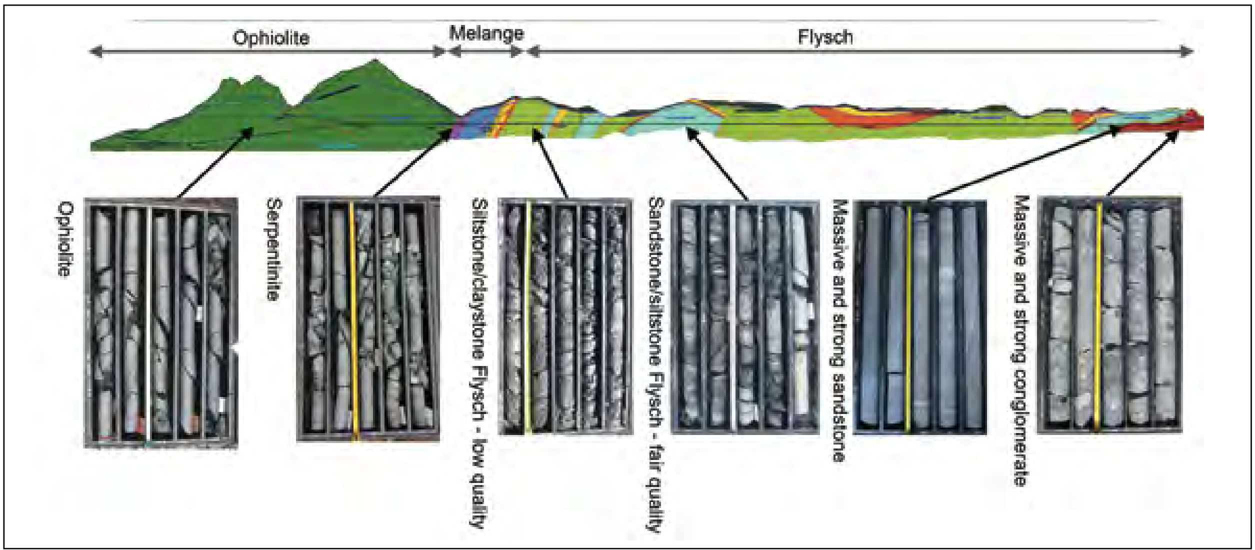

Figure 4: Overview of geology along the tunnlel alignment

representative samples of rock along the headrace tunnel alignment was gathered for geological mapping and laboratory testing.

Field testing of rock mass permeability, groundwater level and long term monitoring of ground water level were performed in all investigation boreholes. Additionally, stress measurements were performed in selected boreholes near the inclined pressure tunnel.

A verification of the interpreted rock mass quality, ground water and in-situ stress conditions will be done by a detailed ground investigation program performed concurrent with excavation of tunnels and caverns belonging to Moglicë power station area, including the pressure tunnel.

3.1 Rotary core drilling
A total of six core drilling locations were performed for the headrace tunnel, giving a total of 860 m core material. Due to the difficult accessible drilling locations, the rotary core drilling was performed by belted drilling rigs, as shown in Figure 5.

Besides providing essential information about the subsurface rock mass and hydrogeological conditions, the core drilling was aimed at providing representative core samples from all rock types that could be encountered during the tunneling works. All holes along the headrace tunnel alignment were equipped with stand-pipe piezometers enabling surveying of ground water levels.

3.2 Laboratory sampling and testing
Although any experienced engineering geologist could obtain extensive knowledge about rock strength and petrography aided only by a hammer and a magnifying glass, exact knowledge of the mechanical properties of intact rock require laboratory testing of representative samples of intact rock. Besides the standard index tests, such as density, porosity and thin sections, the following tests were considered necessary:

- Uniaxial compressive strength, UCS
- E-modulus, E
- Point load, Is50
- Brazilian tensile strength, BTS
- Sound Velocity, vp
- Petrographic analysis/thin section
- Drilling rate index/Cutter life index (DRI/CLI)
- Cerchar scratch test
- Slake durability

Figure 5: Core drilling in the Flysch

3.3 Rock stress measurements
To evaluate the rock stress levels and orientation several hydraulic fracturing (HF) tests were performed.

The HF testing had as objective to present indications on the minimum rock stress (σ3) in the area as close as possible to critical design components of the project. The purpose of the investigation is to provide information on the state of stress in the rock-mass at depth to:

- Confirm that σ3 is higher than the maximum planned head with a safety factor, avoiding any unwanted hydraulic fracturing and loss of water
- Enable an optimum orientation of the long axis of the power- and transformer caverns to the σ_1

4 DESIGN BASIS AND EXPERIENCE

4.1 General
Already at an early stage during the planning it was evident that both topographical and geological conditions were suitable for an unlined "Norwegian" design of the inclined part of the headrace tunnel. Unlined in this context means a water tunnel without steel lining or hydraulic concrete lining, with rock support only consisting of rock bolts alone or in combination with sprayed concrete applied only on parts of the tunnel surface, thicker reinforced sprayed concrete or shorter concrete sections where required.

4.2 Design features
The requirements to an unlined pressure tunnel are quite straight forward – to remain stable for the life of the project under the various loading conditions without undue water loss, nor severe maintenance requirements. For a successful design of the unlined pressure tunnel, the following desirable geological characteristics should be present:

1. Sufficient confinement; the entire tunnel must be set deeply enough within the rock mass to ensure that adequate in-situ compressive stress is available to prevent hydraulic jacking.
2. Suitable rock mass; the rock material must be long-term durable and preferably have good and fair tunnelling qualities without soluble or weak fillings.
3. Sufficient long-term tunnel stability; i.e. no slide, cave-in must take place during operation of the power plant.
4. Other important conditions, such as:
   - Low rockmass permeability, and
   - Sufficiently high groundwater level.

Once the initial stress field was estimated from a crude overburden assessment, analytical solutions were used to evaluate the critical tunnel water pressure where hydraulic jacking may take place. The location of the tunnel was verified by checking the in-situ stress conditions from hydraulic fracturing tests in deep drill holes. Later, during the construction phase, further hydraulic fracturing tests performed in drill holes in the vicinity

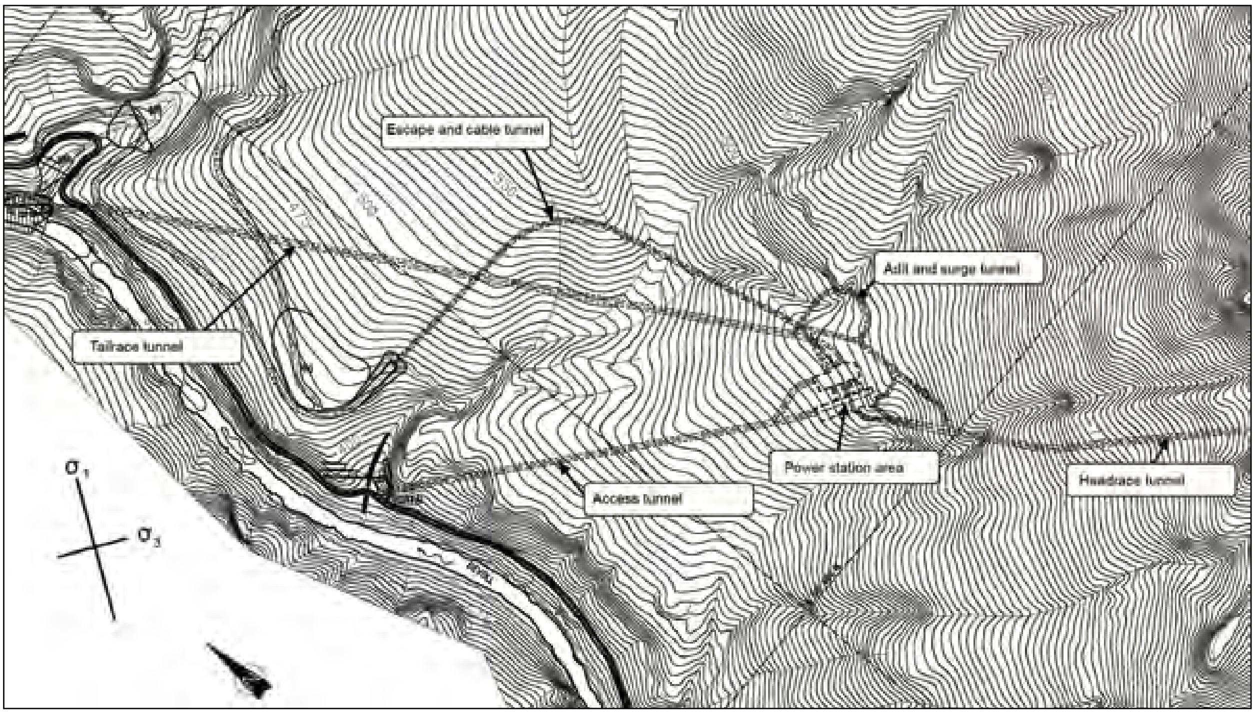

Figure 6: Power station layout and stress field directions

of the penstock area shall verify that sufficient stress conditions are met at the final tunnel location.

The choice of the factor of safety is influenced by the complexity and degree of knowledge of the geology, the accuracy with which the in-situ stresses and the maximum tunnel water pressure are known. As geology and the water rock stresses is known to a satisfactory degree, a value of F = 1.2 to 1.3 is used for the maximum dynamic pressure, and F = 1.3 to 1.5 for the maximum static pressure.

4.3 Measures to reduce risks of undesired failures in the unlined pressure tunnel

Special attention during the design of the unlined pressure tunnel was made to the:

- location of the tunnel with respect to the topography; ensure that the headrace tunnel at no point has less than 100 m overburden, and in flysch not more than 250 m overburden
- investigation of geological conditions and understanding of the geology, as presented in Chapter 3, and
- magnitude of the in-situ least principle stress.

The key measures to reduce the risk of failure are:

- The use of a design where it is possible to decide the final location after stresses can be measured in the tunnel during construction
- Geological mapping and decision on final support means by qualified personnel after each blast
- In-situ testing and monitoring during construction
- Observations of water seepage into the tunnel together with remedial grouting
- The installation of rock support will be followed-up to ensure appropriate support quality.

5 TENDER DESIGN

5.1 Layout

The tunnel layout presented below is the result of adapted design to findings from the ground investigations, together with the project specific minimum requirements.

5.2 Engineering Geological follow up during construction

It should be recognized that the Tender Design quantities do not represent the real and final distribution of rock support and grouting quantities to be used during construction, neither the required measures for investigation, instrumentation or tests.

During construction the observational method is used for establishing the final rock support and grouting design. In general the method is described as observation of the behavior of the newly excavated face, decide on the necessary support (including water treatment) and keep the tunnel under surveillance for a period of time to verify the functioning of the installed support. If the support is found insufficient, additional support has to be placed until stability is assured. This cause the real support to be distributed quite independent of the support classes listed in Engineering Geological Report.

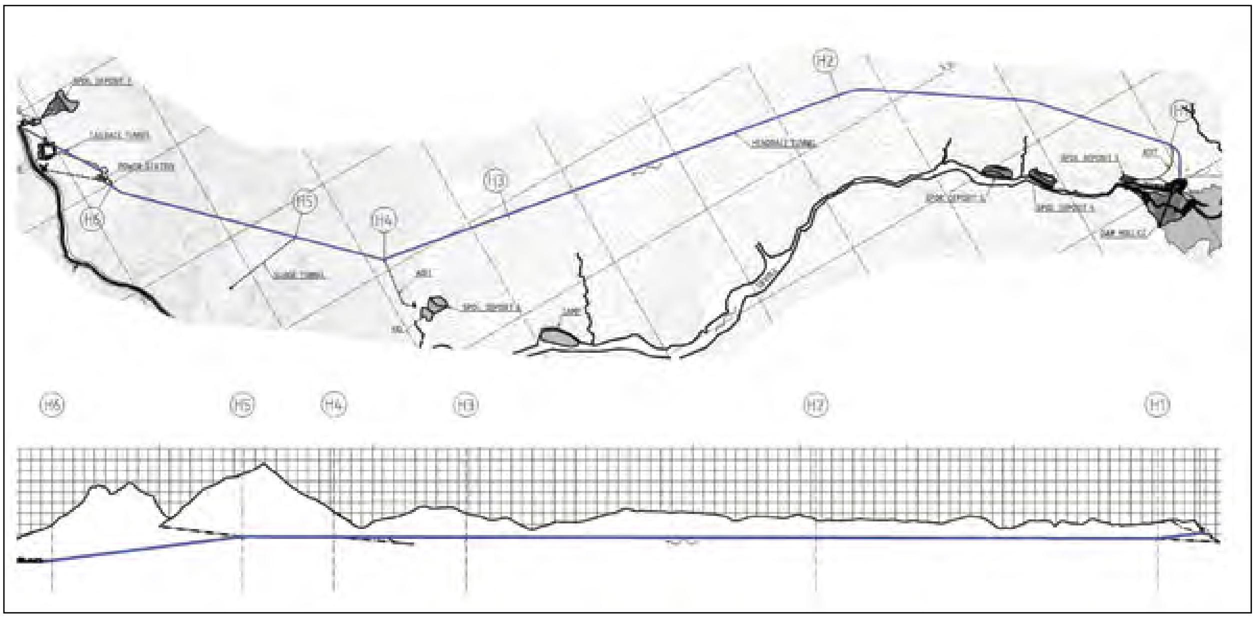

Figure 7: Layout of hydropower scheme - waterway highlighted blue

Headrace tunnel section	Internal water pressure	Terrain over-burden	General rock formation	Tunnel Design Concept/ Excavation method
Intake to H1	0-90	0-150	Flysch	Unlined/sprayed concrete (D&B)
H1-H3	90	100-250	Flysch	Segmental lining (TBM). Drained and without gaskets
H3-H4	80	100-300	Melange	Concrete lining (D&B), drained
H4-H6	80-340	350-800	Ophiolite	Unlined/sprayed concrete (D&B), drained
Penstock tunnels	340	350	Ophiolite	Steel lined (D&B)

Table 1: Selected Tunnel Design Concept for each tunnel section

The geology at each of the tunnel face will be mapped by qualified and experienced Engineering Geologists, and the rock mass quality classified immediately following the blasting of a new round. The appropriate rock support resources will then be selected based on the findings.

The permanent support will be designed incorporating the initial rock support and released for execution at suitable intervals.

6 TUNNELLING CONTRACT FORMAT

The aim of rock support is both to provide safe working conditions during construction and sufficient long-term stability of the underground opening. Safe working conditions, which are the responsibility of Contractor, are to be taken care of by the initial rock support. For long-term stability, the extent of additional rock support will be decided by Employer.

The contract format carefully selected for this project, is the FIDIC red book, and based on Employers design and mainly unit prices for the tunnelling works.

Basically, the Contractor has responsibility for his unit prices and unit capacities, whereas the Employer has the risk and responsibility for the total quantities defined and installed during construction. All rock support and grouting works elements have a separate payment item in the BoQ`s.

Basic principle is that each cost element (item) shall be measured and paid according to installed and approved quantities, not according to “rock mass quality” or “rock support class” as may be specified in some projects.

7 RECOMMENDATIONS ON FILLING AND DEWATERING OF THE UNLINED PRESSURE TUNNEL

The initial water filling of the pressure tunnel should be carefully controlled to limit differences in pressures between the groundwater and the tunnel water. During construction, the tunnel has been open for several years, and drainage of the rock massive has taken place. Slowly filling the tunnel allows pressure equalization to occur, and thereby limits deformation of the rock and supportive structures. The rate should depend on the rock mass conditions and the types and extent of rock support installed. An infilling rate of 5 to 20m head/hour has been found adequate where good rock mass conditions.

Dewatering of pressure tunnels should be done even more carefully, preferably at a rate between < 5 to 10m head/hour, utilizing slower rates for high-head plants. Ground water changes should be noted as the dewatering takes place. A detailed inspection of the tunnel should be done immediately after the dewatering is complete. Records of inflow, local failures of the rock or rock support, cracking or other distress should be recorded.

8 CONTROL OF THE TUNNEL DURING POWER PRODUCTION

An unlined water tunnel cannot be considered completed until the tunnel has been dewatered and its performance verified, included installation of any clean-up works and necessary additional support works. For this, it is generally recommended that such tunnel should be dewatered within approximately one year of operation, so that such works could be undertaken during the contractor's mandatory defects remediation responsibility. Head loss should be constantly recorded during power production and even head losses in the range of a few centimetres should be carefully analysed, as this may indicate local rock falls of several m^3.

9 CONCLUSION

An «unlined» pressure tunnel design concept is adapted to the local geological and topographical conditions at the HPP Moglicë. The tender design is based on extensive geological mapping and ground investigations whereas the final tunnel and rock support design shall be defined based on observational methods during construction phase. Construction is planned to start in 2015 and the client DHP has decided to continue to final design according to the principles described in this article due to the great economical savings compared to a more "standard" lining design.

5.1 ACKNOWLEDGEMENT

We would like to thank Devoll Hydropower Sh. A. for allowing us to present this interesting project.

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03. DESIGN OF UNLINED HEADRACE TUNNEL WITH 846 M HEAD AT LOWER KIHANSI, TANZANIA. FILLING EXPERIENCE

HALVORSEN, A.
ROTI, J. A.

1 PROJECT DEVELOPMENT

Lower Kihansi Hydropower Project in Tanzania is located in the Rufiji basin, some 550km southwest of the capital Dar es Salaam. It is owned by the Tanzania Electric Supply Company (TANESCO).

The project was conceived in the Rufiji Basin Hydropower Study of 1984, which suggested that the Kihansi river was the most favorable for future hy-dro power development in the basin. A feasibility study published in 1990 demonstrated the technical feasibil-ity and economic viability of the project. Based on the study, the World Bank and TANESCO subsequently agreed to incorporate the implementa-tion of the Lower Kihansi project into a sector loan package the Power VI Project Program.

In December 1991, NORPLAN was selected to carry out a feasibility review of the Lower Kihansi scheme, and if this proved positive, to continue with the final design. Tendering and supervision of con-struction fol-lowed as an extension to NORPLAN's initial contract.
Feasibility review and field investigations were con-ducted in 1992, followed by the detailed design and preparation of Tender Documents. Considering the ground conditions, NORPLAN found that a deeply sited tunnel system would be advantageous, compared to the shallow tunnel system previously proposed. The design suggested by NORPLAN in-cluded an unlined headrace tunnel with maximum head of close to 850 m and under-ground power house.

The actual construction phase started in July 1994, with the mobilisation of the Chinese contrac-tor SIETCO, who won the bid for preparatory works. In July 1995 the Italian contractor Impregilo SPA, who won the bid for main civil works, mobi-lized for the underground works. Construction was successfully completed in February 2000, within budget and time schedule.

At present, the power plant at Kihansi contrib-utes with in order of 40% of the total electricity pro-duction in Tanzania.

2 DESCRIPTION OF PROJECT

Lower Kihansi Hydropower Project includes a 25m high concrete gravity dam which impounds a small reservoir with a total storage of 1.6M m^3. The intake connects to the headrace tunnel via a circular unlined vertical headrace shaft ($25m^2$), some 500m deep. The unlined headrace tunnel slopes at a gradi-ent of 1:7. At the downstream end are the stonetrap and the transition section to the steel penstocks. The tunnel is 2200m long and has a cross section of $30m^2$, except in the last down-stream 600m where its cross section is $37.5m^2$ to allow for lining, if neces-sary, in the zone of the highest pres-sure. No surge shaft or chamber had to be provided due to the relatively short tunnel length and the low water velocities, combined with the use of Pelton turbines.

The tailrace tunnel has a length of 2100m and a cross section of $34m^2$. It slopes gently downstream at an incli-nation of 1:900 and connects to an 800m open cut canal that evacuates the water into the ex-isting Kilombero river system.

The power house cavern is excavated deep into the mountain massif and is 12.6m wide, 98m long and 32m high, with space for possible future instal-lation of 2 additional turbines. The power house is connected to the outside by a 1900m access tunnel ($40m^2$ in cross section) and also by a separate cable tunnel which was chosen as an extra security meas-ure for the 220kV cables passing from the under-ground power house to the outdoor switchyard.

An overview of the project is shown in Figure 1, show-ing the waterway as plan and as longitudinal section. A comprehensive description of the project is given by Saidi, F. X., Lindemark, J. and Wilhelm, V. C. (2000).

3 DESIGN OF THE HEADRACE SYSTEM DEPENDENT ON SUFFICIENT ROCK STRESSES

A head of 846m could be achieved with the loca-tion of the tunnel in the escarpment of the Udzungwa mountain range. This escarpment, be-longing to the eastern branch

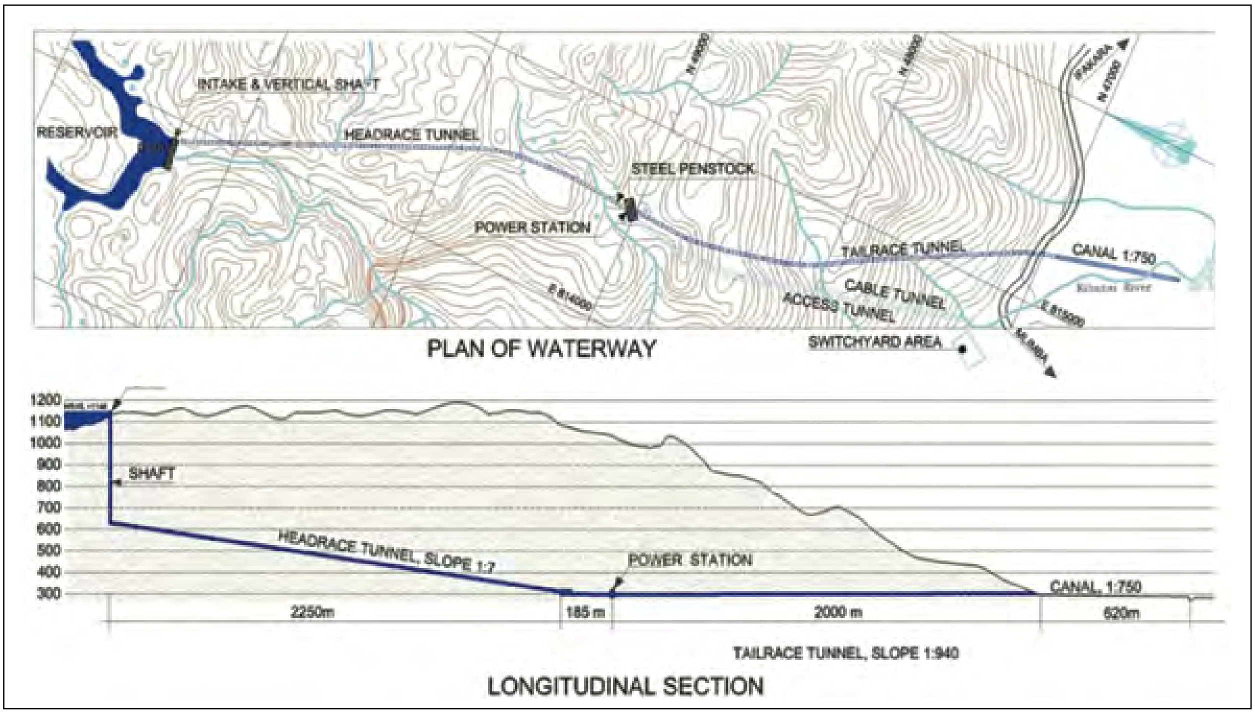

Figure 1: Project overview

of the east African rift system, is formed by large scale block faulting. The rocks are mainly competent gneisses of the Pan Af-rican Mozambique Belt, subjected to high grade metamorphism. The degree of faulting and jointing is in general moderate to low. Most pronounced is a joint/fault system oriented perpendicular to the tun-nel system with partly high permeability.

An unlined, low-gradient water tunnel with as high head as 850 m head had not been constructed before, anywhere in the world. In 2 other hydropow-er plants, Tjodan and Nyset-Steggje in Norway higher heads were obtained in 45° inclined shafts, 875m and 964m respectively. The design at Kihansi is more vulnerable in regard to leakages, as the high head section in a tunnel with inclination 1:7 is longer than in a 45° shaft, shown schematically in Figure 2.

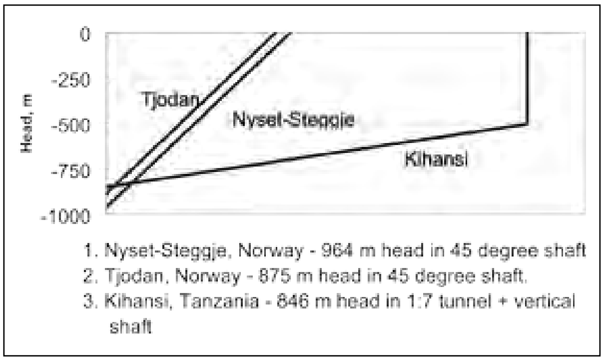

Figure 2: Highest water heads in unlined tunnels / shafts.

With a concept as chosen at Kihansi, detailed knowl-edge about the ground conditions and adapta-tion of the design to these are a condition for a suc-cessful construction. An important question for the design-ers was if rock stresses were sufficient. With too low stresses, the confinement of the tunnel could be insufficient, resulting in hydraulic failure. The primary aim of the field investigations was to ensure that there were sufficient internal stresses in the rock mass for adopting an unlined design for the headrace tunnel, giving large cost savings.

Stress measurements by use of hydraulic fractur-ing methodology were first conducted in deep, core drilled holes from the surface. After some costly and time consuming attempts, where test equipment was lost in deep drill holes, a different approach was chosen. Hydro-fracturing tests would be done from short holes drilled from within the tunnel during ex-cavation. If the results from these tests were unsatis-factory, the tunnel layout would have to be modi-fied. That meant that a tentative design had to be presented in Tender documents, based on assumed rock stresses. The contract conditions were written to allow the power house to be sited deeper into the rock massif if necessary, since this would result in larger rock cover and probably improve the rock stress conditions for the critical part of the headrace tunnel. Use of unit price as well as unit time system was the important basis for contractual regulations in case of relocation of the power house.

4 ROCK STRESS MEASUREMENTS

A detailed strategy plan for rock stress measure-ments and decision making was concluded on in due time. If sufficient rock stresses could be confirmed by testing at 3 different stations, located from Chainage 800 to Chainage 1000 in the Access tun-nel, the original, tenta-tive location of power house, at around Chainage 1070, could be maintained.

Hydro-fracturing testing as well as triaxial testing began when 800m of the access tunnel had been ex-cavated. Initial testing gave insufficient minimum principal rock stresses and relocation of the power house seemed unavoidable. The encountered stress pattern was char-acterized by sub horizontal minimal principal stresses oriented north - south, close to parallel to the tunnel axis and perpendicular to the main joint orientation. This pat-tern is assumed to re-flect the original stress situation with the low mini-mum principal stresses explained by extensional tec-tonism.

When the testing finally was completed at Chainage. 2093, totally 19 holes with lengths be-tween 20 m and 140 m had been drilled for hydrau-lic fracturing / jack-ing testing in the tunnel. The drilling was done partly from the tunnel face, ahead of the tunnel, partly from niches behind the face. In addition testing was done in 2 deep holes core drilled from terrain above the tunnel. Totally 97 hy-dro-fracturing tests, 27 hydro-jacking tests and 22 triaxial tests by use of overcoring method were con-ducted. The testing was done by the SINTEF Rock and Mineral Engineering, Norway, assisted by ex-perts from SOLEXPERTS, Switzerland. During the final stage of the testing, Dr. Tore Dahlø of SINTEF died as a consequence of a tragic accident in the tunnel.

5 INTERPRETATION OF ROCK STRESS MEASUREMENTS

The measurements were conducted to determine whether the level of minimum stress at the transition between unlined headrace tunnel and steel-lined pen-stock attained the level of 10 MPa, as required to leave the tunnel unlined. Continued stress measure-ments by hydro-fracturing / hydro-jacking and by overcoring methods indicated an improved stress situation deeper into the rock massif.

The minimum stress estimates from the instanta-neous shut-in pressures (ISIPs) of the hydro-fracturing tests generally showed large scatter and were often less than the estimated pre-disturbance pore pressure. It was suspected that the stress tests were affected by stress alteration around the tunnel due to drainage causing a pore pressure draw-down. To test this, hydro-fracturing and hydro-jacking tests were conducted in two long, horizontal holes drilled ahead of the excavation face into relatively un-drained rock, and a third vertical hole drilled behind the face. All were near the critical loca-

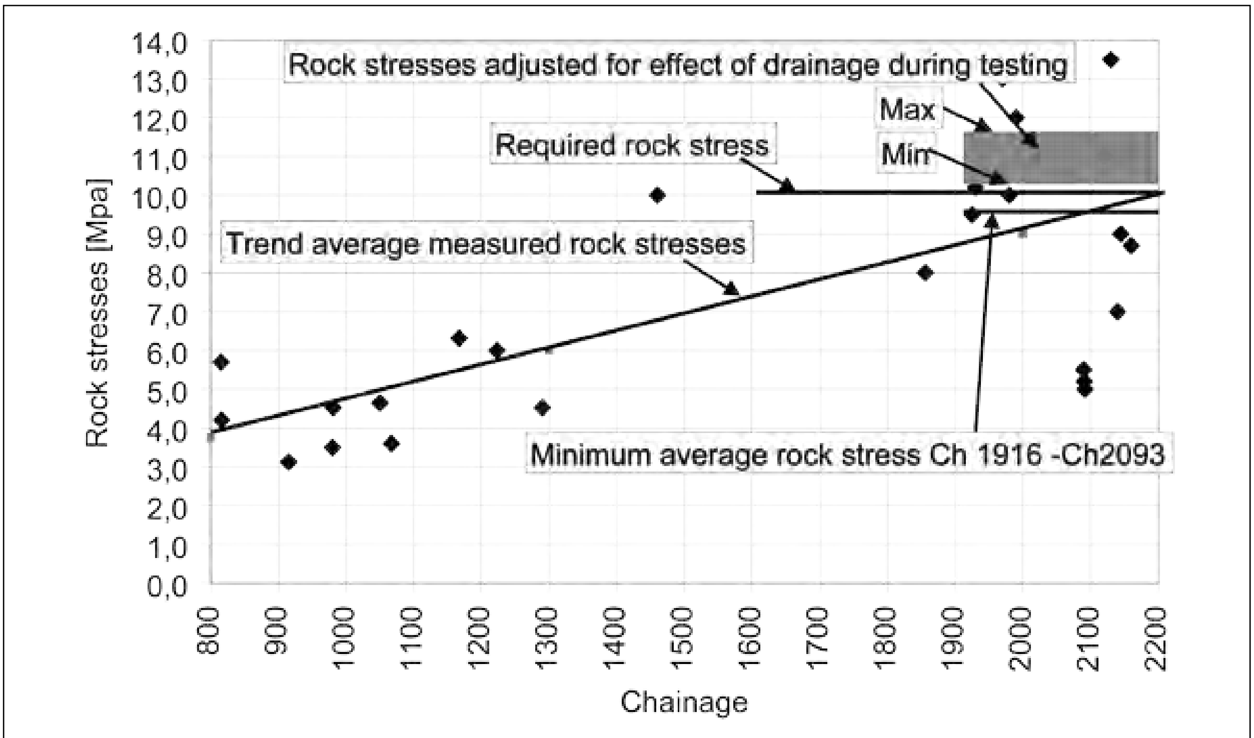

Figure 3: Measured rock stresses, with adjustment for drainage effect

tion where the pressure tunnel enters the powerhouse. The re-sults showed that the minimum stress estimates in the ahead-of-face holes were much higher than those from the behind-face hole, consistent with the hy-poth-esis. A model was developed to explain the coupling between the pore-pressure and stress fields in terms of fracture compliance and poro-elasticity, and was used to correct the stress estimates for the effects of pore pressure draw down around the bore holes during test-ing. The corrected minimum stress estimates exceeded the 10 MPa limit for an unlined tunnel. Figure 3 shows a compilation of stress meas-urements at the various test stations in the access tunnel, also indicating how the rock stresses were adjusted for effect drainage (pore pressure draw down).

The results of this study are relevant to any situa-tion where stress tests are to be conducted in deep tunnels or excavations. A detailed description of the rock stress measurements and the influence of pore pressure draw down is given by Dahlø, T. et al. (in press).

With this conclusion, the location of the transition between unlined and steel-lined tunnel could be de-cided on. The power house complex was moved 730m into the mountain beyond the initial location, and with more than 700 of overburden.

In Table 1 the required and measured minimum principal rock stress for initial as well as as-built de-sign is shown.

		Initial design	As-built design
Overburden at penstock, m		600	750
Water pressure / overburden ratio		0.65	0.89
Min. pricipal stress, σ3, MPa	Required	10	10
	Measured	6	≥10
Min. principal stress / water pressure ratio	Required	1.2	1.2
	Measured	0.7	≥1.2

Table 1. Required and measured minimum principal rock stresses, σ_3, in initial and as-built design

6 GEOLOGICAL CONDITIONS IN HEADRACE TUNNEL

A summary of the geological conditions of the headrace tunnel is shown in Table 2. Potential leak-age is mainly connected to E-W oriented, sub-vertical joints.

Degree of jointing is reflected in the RQD value, see Table 2 as well as Figure 4. The RQD values are in gen-eral low. During excavation the degree of wa-ter inflow was low in the lower part of the tunnel, where the RQD values are high.

Chainage	Lithology / tectonisation	Average RQD
0 - 825	Mainly massive granitic and dioritic gneiss / low to moderate degree of jointing and faulting.	95
825 - 920	Mainly dolerite and biotite rich gneisses / schistose (thrust faulting?), moderate degree of jointing.	80
920 – 1250	Massive granitic / dioritic gneisses with dolerite and biotite rich interlay-ers / low degree of jointing, but some schistose zones.	93
1250 – 1520	Granitic gneisses interlayered with meta-diorites and micaceous gneisses / faulted sections and moderate to high degree of jointing..	72
1520 – 2200	Granitic gneisses interlayered with meta-diorites and micaceous gneisses / low to moderate degree of jointing and faulting	95

Table 2. Summary of geological conditions in headrace tunnel

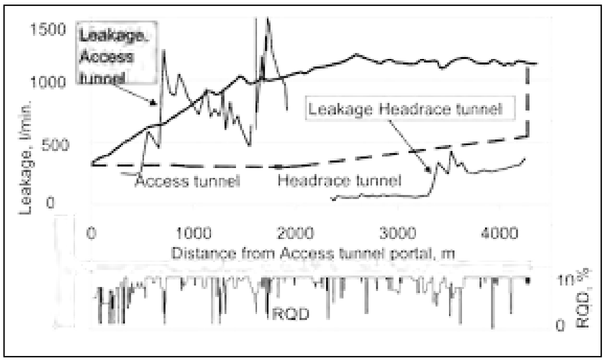

Figure 4. Leakage from access tunnel and headrace tunnel dur-ing excavation, measured weekly at portals as tunnel face pro-ceeded

7 TRANSITION BETWEEN THE HEADRACE TUNNEL AND THE DRY TUNNEL SYSTEM

A 120m long horizontal steel penstock liner from the power station to the headrace tunnel was de-signed to give an acceptable pore pressure gradient. The sealing between the water filled, high-pressure tunnel and the dry tunnels was taken care of by con-crete plugs in the by-pass tunnel and the penstock tunnel, lengths 60 m and 70 m respectively. Figure 5 shows the layout in the tran-sition area, including concrete plugs and grout curtains.

Grouting works were designed and performed ahead of the excavation in the areas of the concrete plugs. In addition, comprehensive grouting of the rock around both plugs was done in 24 m long holes before cast-ing, with use of cement and a grouting pressure of up to 90 bars, and after casting contact grouting between concrete and rock. After water fill-ing post grouting works were done to reduce the leakage encountered at the plug in the Bypass plug area.

8 HYDRO-GEOLOGICAL CONDITIONS

The tunnel is located in a North-South oriented ridge, formed by erosion of the two rivers on both sides of the ridge: Kihansi to the west and Udagaji to the east. A longitudinal profile of the ridge along the tunnel is presented in Figure 1. Figure 5 shows the calculated ground water pressure lines in a cross-section of the ridge, located just upstream of the plugs, prior to tunnel excavation and after com-pleted filling of the tunnel. Estimated flows to the Kihansi and Udagaji Rivers for one set of estimated permea-bility conditions of the rock. The model show an es-timated rise of water level above the tunnel in the order of 50 m.

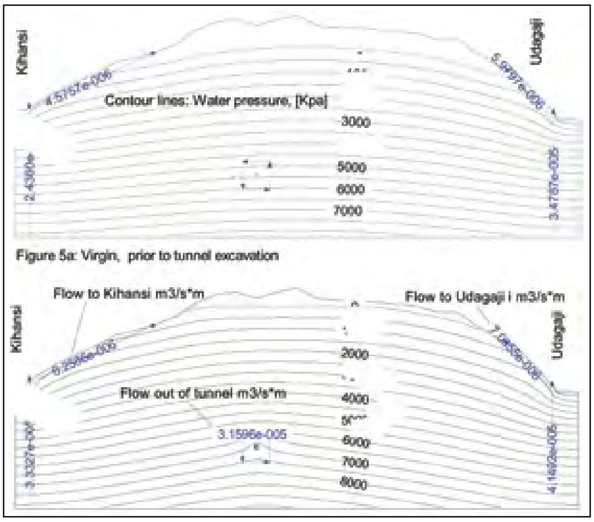

Figure 5: Water pressure lines in a cross-section upstream of the plugs prior tunnel excavation and after completed filling.

The hydro-geological conditions of the ridge are decided by:

- A system of sub-vertical E-W-oriented joints across the ridge approximately perpendicular to the tunnel axis. These joints, assumed to be ten-sional, are per-pendicular to the minimum princi-ple stress direction.

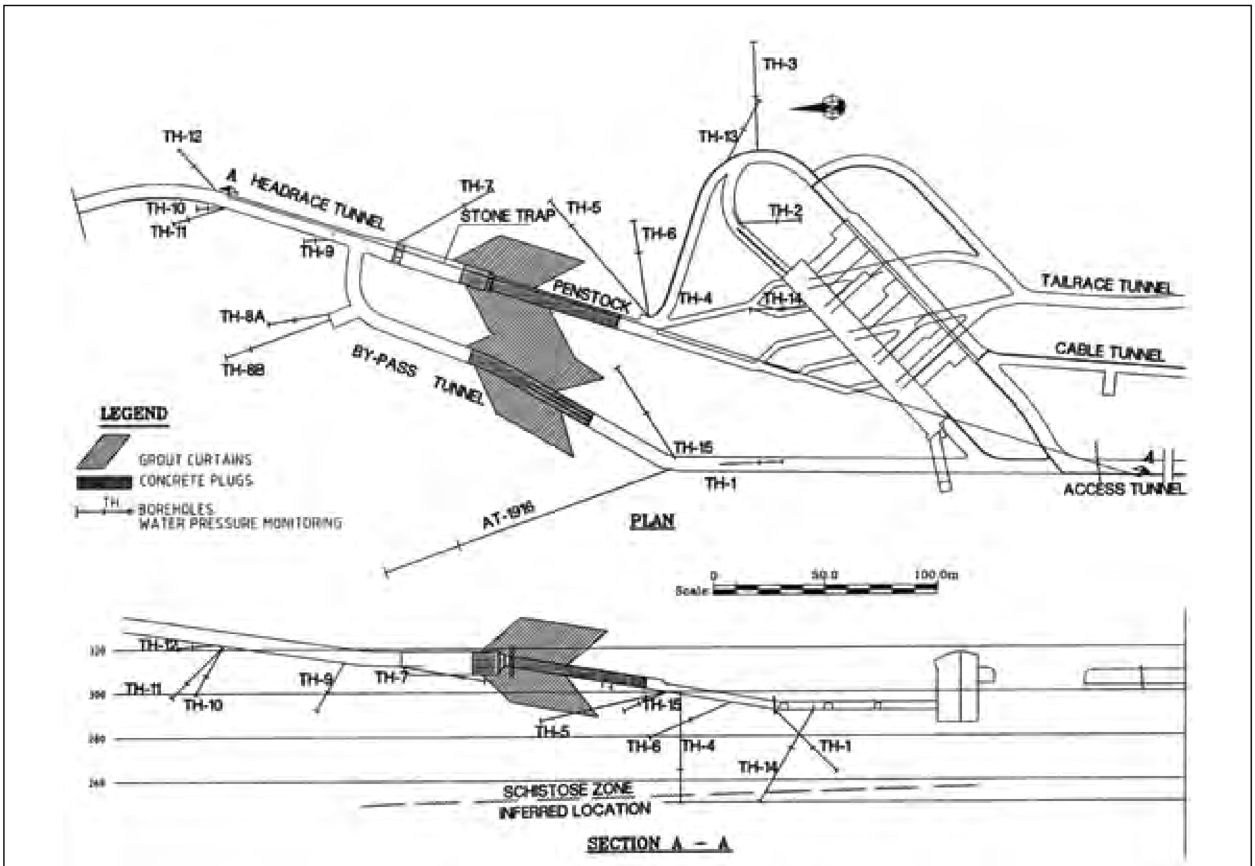

Figure 6: Layout of tunnel and monitoring system

They are water bearing, partly open / partly filled with weathering materials. Degree of weathering varies and generally these materials are residual soils of a uniform grading in the clayey silt to fine sand fractions. The weathered material is assumed susceptible to in-ternal erosion. Width of the joints varies general-ly from 0 to 10 mm. In places, however, the thickness of the severely weathered joint may measure up to 100 mm. Parts of the joints, where the weathered materials have been washed away, have a relatively high permeability compared to the rock mass otherwise.

- Distance between the sub-vertical E-W joints is generally in the order of 10-20 m. Between the joints, the rock is normally solid without pro-nounced joints or weaknesses.
- Permeability of the rock in N-S direction at tun-nel level is very low and negligible compared to the permeability of the sub-vertical E-W joints.
- The severely weathered zone, of up to 40-50 m thickness, at ground surface above the tunnel has a higher permeability, causing a distribution of ground water from the huge ground water reser-voir at the Plateau west of the project area along the ridge. Therefore, some of the tributaries to Kihansi and Udagaji carry water also in the dry season.

A pronounced system of E-W oriented joints crosses the tunnel system just upstream of the rela-tively compact section of the rock, in which the pen-stock and bypass plugs have been located. This joint system crosses Kihansi River at and downstream of the main Kihansi Falls and can be followed over to the Udagaji River. Tributaries to both rivers have water all year trough.

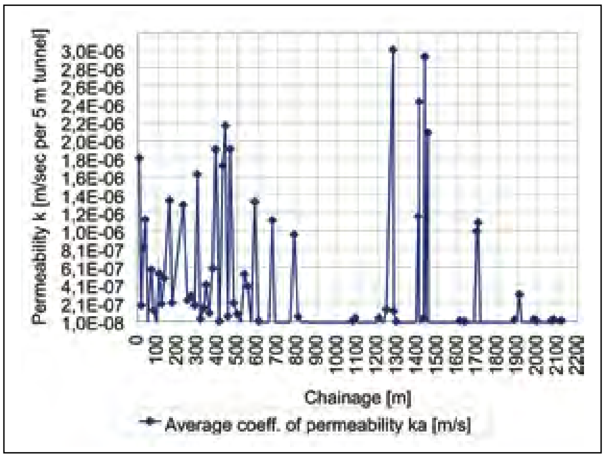

Figure 7: Permeability of rock perpendicular to the tunnel axis, and grout take for grouting at face. Coefficient of permeability k is calculated based on water pressure tests /water inflow in soundings and ahead of face

The hydro-geological conditions are characterized by a highly anisotrop permeability: high in the joints perpendicular to the tunnel and very low permeability parallel to tunnel axis. This pattern could be observed during excavation of the tunnel: Water pressures up to 6 Mpa could be measured in sounding holes crossing E-W joints only 3 m ahead of face. All water entering the tunnel during excavation came from the E-W joint system. Figure 7 shows estimated permeability of the rock perpendicular to the tunnel axis, calculated on the bases of water pressure tests and or measurements of water passing through soundings systematically drilled ahead of face.

9 LEAKAGE PREVENTION IN HEADRACE TUNNEL

An estimate of expected water leakage out of the headrace tunnel was made using the permeabilities, estimated on the basis of the soundings, taking the future net head along the tunnel into account. The estimated leakage at each joint system crossed by the soundings as well an estimated net head curve are shown in Figure 8. Economic analyses were made to obtain parame-ters for reducing leakage from the headrace tunnel. The net present value of production losses at the Ki-hansi project

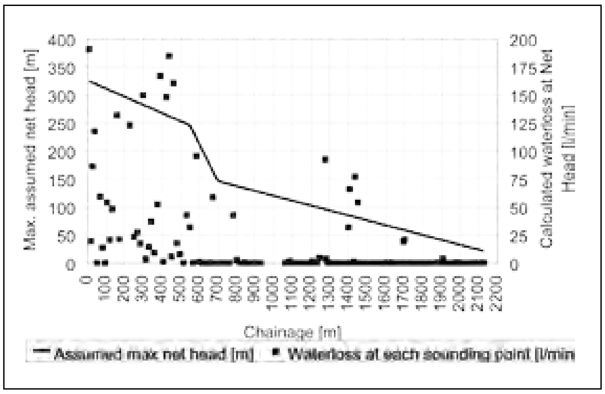

Figure 8: Estimate of water leakage from headrace tunnel, cal-culated on the basis of the permeability determined at the soundings ahead of face.

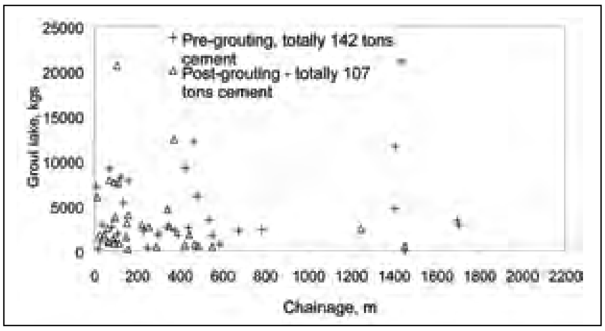

Figure 9: Grout takes in headrace tunnel

was estimated to USD 24.000 /l/s. This high value of the potential water loss entailed that great investments in reduction of permeability were warranted. Pre-grouting should always be performed at face when possible, to reduce permeability in zones in the most cost efficient manner. It was stated that pre-grouting was feasible and efficient when rock permeability, measured as Lugeon value, ex-ceeded 0,5-1 L.

Pre-grouting by use of cement, both ordinary Portland cement and micro-cement, was conducted in sections where unacceptable permeability was de-tected by water pressure testing. The criteria for grouting was set to the permeability k> 2 E-7 m/s. Usually, sounding was performed with three holes of 18 m length. The conductivity of the rock was de-termined by water pressure testing of each hole.

If deemed necessary, post grouting was also done from the excavated tunnel in leaking zones. Figure 9 shows the distribution of grout takes along the tun-nel.

Various types of sectional linings and local struc-tural measures in addition to grouting were consid-ered and designed for highly permeable sections. Type and length of such measures were selected based on a cost – benefit estimate.

In the lower part of the headrace tunnel, at Chainage 66 – 90 and 111 - 144, sections of concrete linings were applied to reduce potential leakage through some pronounced joint systems.

10 INSTRUMENTATION FOR MONITORING DURING WATER FILLING

Water filling of the headrace system was consid-ered an especially critical operation and the response in the rock mass around the tunnel had to be ana-lysed in detail. A monitoring system included pore pressure measurement in strategically placed bore holes, both upstream and downstream of the con-crete plugs. NGI (Geotechnical Institute of Norway) was engaged for the technical design and installation of the instrumentation. Pore pressure measurement was done in totally 17 holes, with lengths from 25 to 140 m. Location of holes is shown in Figure 6. High pressure steel tubes connected each piezometer hole to pressure transducers. The steel tubes were placed in concrete protection in the pressure tunnel and lead through the Bypass concrete plug to the downstream end of the plug. Measures were taken to prevent leakage along the tubes through the plug. All trans-ducers were placed in the dry zones downstream of the Bypass and the Penstock Plugs. No transducers for monitoring piezometer pressures or water pres-sures were located in the water pressurised tunnel system. DigiQuartz absolute pressure transducers with an accuracy of 0.005% were used. Signals from the transducers were transmitted to 2 automatic data loggers, with connection to computers in a control room in the power house.

Precise recording of the water level in the tunnel or in the shaft was necessary for analysis of loss of water during filling. For this purpose, a DigiQuartz transducer was placed in the lower end of the tunnel and connected to one of the data loggers. In case of failure in this system, and to be able to read the wa-ter level in the shaft even more precisely, a high pre-cision level transmitter was brought to the site for use in the shaft.

In addition to the monitoring of water pressures in the various piezometers and in the headrace tun-nel, a system to monitor strain in a sectional con-crete lining and possible deformation of joints in rock was installed. The system consisted of exten-someters placed at the rock / concrete boundary across E - W joints. Furthermore, two extensometers for monitoring possible radial strain in the concrete lining were installed. The vibrating string exten-someters were connected to a AC/DC transformer and to the datalogger downstream of the Bypass Plug by cables, which were lead through the con-crete plug in the same way as the steel tubes for pressure monitoring.

A simple, manual system for monitoring inflow of water into the dry part of the tunnel system downstream of the plugs was established. Furthermore, water flow in tributaries and streams to Kihansi and Udagaji rivers were monitored manually daily dur-ing and after filling. Water levels in bore holes in rock above the tunnel and close to the dam site were monitored manually on a daily basis.

11 FILLING OF HEADRACE TUNNEL

Filling of the headrace tunnel and the shaft was planned with at least 3 weeks duration to avoid high local hydraulic gradients in the joints close to the tunnel, and to avoid excessive changes in rock stresses around the tunnel. The plan included various stops at certain filling levels in the tunnel and in the shaft in order to monitor net inflow from rock into the tunnel or outflow from the tunnel into the rock at different water pressures. The filling was done using pumps as the construction program at the dam struc-ture did not allow for rising the reservoir level above the threshold of the intake structure in due time.

Filling of the headrace tunnel and intake shaft up to final reservoir level took totally more than one month.

During filling, the pore pressure build-up in the rock mass around the tunnel as well as the water pressure in the tunnel was closely followed by au-tomatic logging of some 20 piezometers installed in drill holes. This monitoring, with automatic readings every 10 minutes, allowed a continuous recording of the filling rate and gave valuable information on stress development around the tunnel. Detailed re-sults of a selection of monitored piezometers during the filling are shown in Figure 10. The monitoring continued for several months after the filling. Re-sults for a period of 4 month after start filling are shown in Figure 11. Locations of the piezometers are shown in Figure 6. The pressures recorded in the diagrams in Figures 10 and 11 are adjusted to the same reference level, i.e. the level of the transducers located in the gallery at the downstream end of the Bypass plug. Consequently, the difference between the recorded water pressure in the tunnel and the pressure monitored at a piezometer, reflects the hy-draulic gradient between the respective piezometer and the tunnel.

Based on results from the rock stress measure-ments, water pressure testing ahead of face, as well as a hydro-geological analysis of ridge, the virgin ground water pressure in rock at the downstream end of the headrace tunnel was estimated to 5,5-6,0 Mpa. Before filling, the piezometers showed variable in-fluence of the tunnel on ground water pressures. Pie-zometer AT1916, located some 120 m away from the pressurized tunnel in a joint system crossing the tunnel system just upstream of the Bypass plug, showed a pressure of approximately 3,0 Mpa before filling. Before tunnel excavation crossed that joint system, a pressure of 5,3 Mpa was recorded in AT1916. Piezometer TH7, located in the rock at the upstream end of the Penstock plug some 25 m away from the stone trap, showed a pressure of 4 Mpa pri-or to filling.

During the first stages of the filling, as recorded in Figure 10, the piezometers in rock showed only slight pressure increases until the water pressure in the tunnel had exceeded the piezometer pressures by approximate-ly 1,5 Mpa, i.e. a water head differ-ence in the order of 150 m. This applies to all pie-zometers located in rock beside the pressurized tun-nel. The distances from the tunnel were between 10 to 120 m. The average hydrau-lic gradients were cor-respondingly between 1 to 15.

All piezometers in rock around the pressurized tunnel, the “wet zone”, were located in joints cross-ing the tun-nel. Most of these joints yielded water during drilling of soundings ahead of face during tunnel excavation, or showed high permeability dur-ing water pressure test-

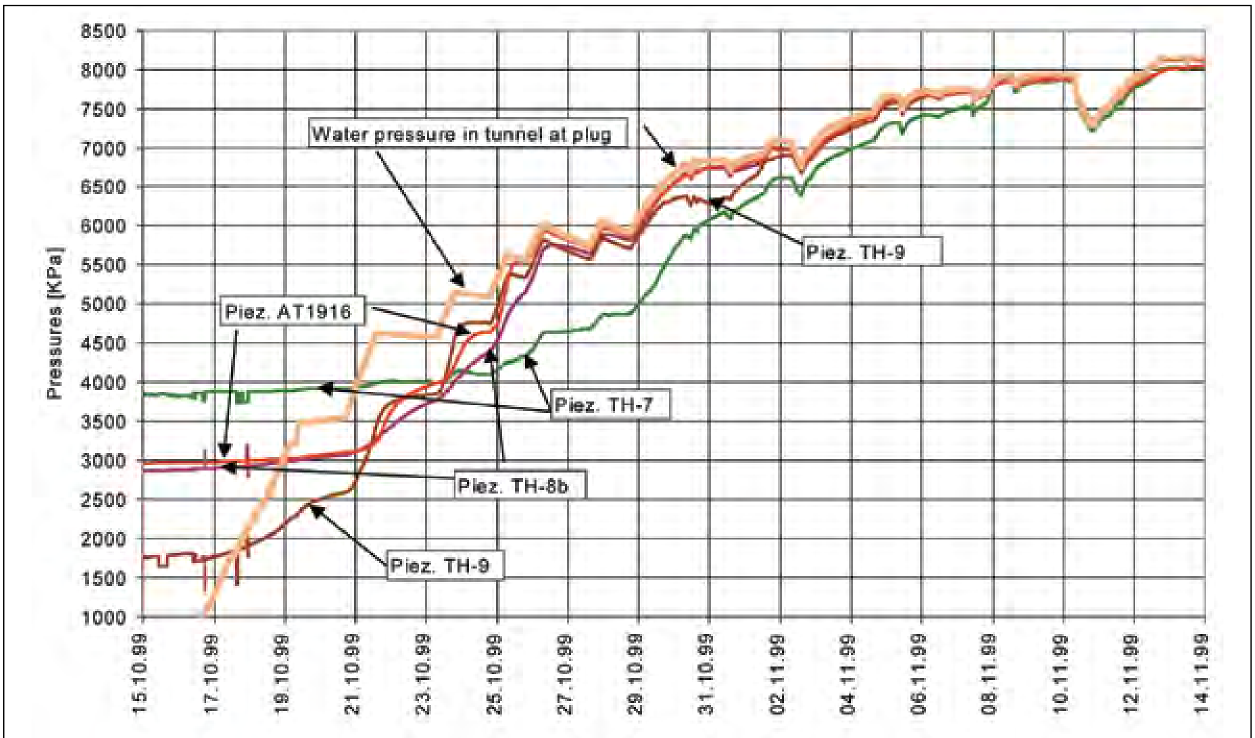

Figure 10: Recorded water pressure in tunnel and in selected piezometers during filling of tunnel/shaft.

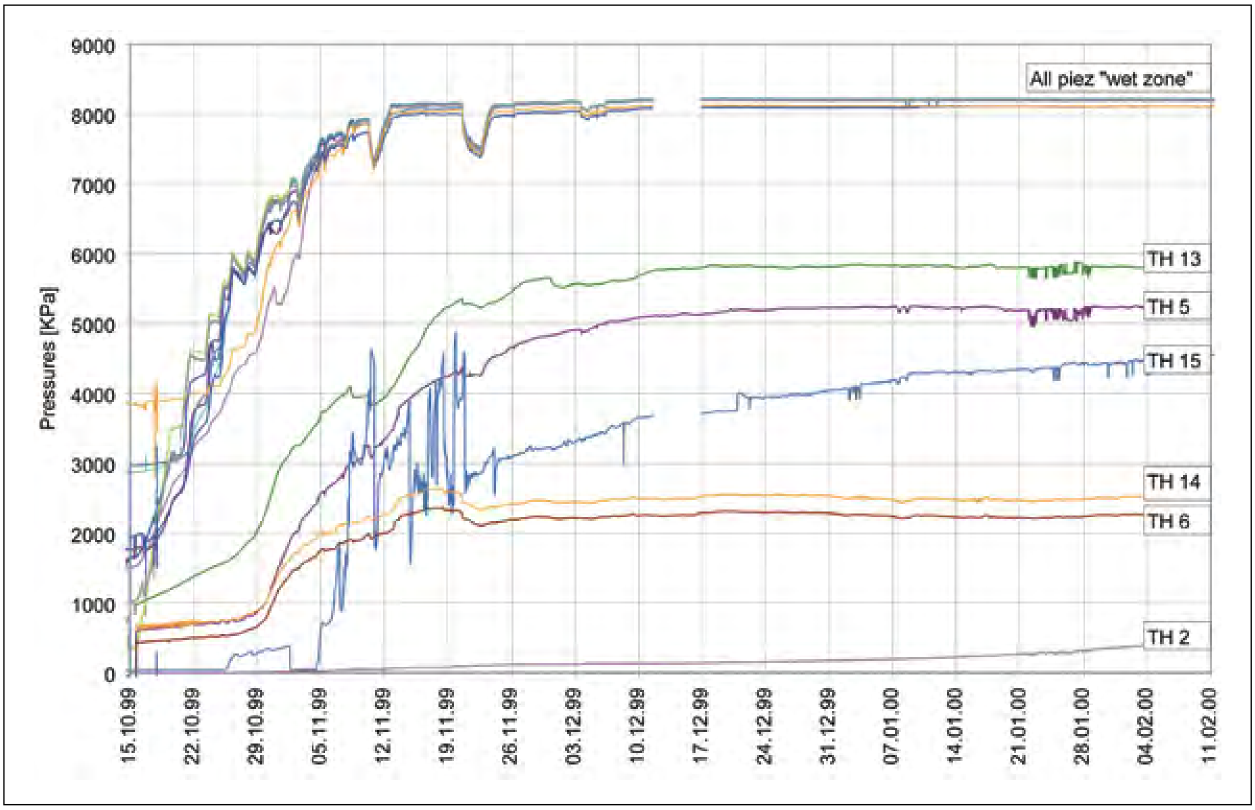

Figure 11: Recordings of piezometers and water pressure until 4 month after start filling.

ing. The joints had a clear communication to the tunnel, and were consequently grouted at face and, in most cases, also post-grouted.

After grouting, all joints were dry in the tunnel. The results of grouting were tested by test-holes and water pressure testing generally at pressures of 9 Mpa. The joints generally contained some decom-posed material, assumed susceptible to erosion.

At filling levels in the tunnel, corresponding to pres-sures higher than approximately 1,5 Mpa above the piezometer pressure before filling started, the pressures in the "wet zone" piezometers increased at the same rate as the water pressure until the pressure reached the mag-nitude of virgin ground water of 5.5-6Mpa. Thereafter, the piezometer pressures in the "wet zone" equalized more or less with the water pressure inside the tunnel. The pressure differences remained generally between 0 and 0,3 Mpa. A de-tailed study of the pressure develop-ment, showed that the increases in most cases did not take place smoothly, but often stepwise. Also pressure decreas-es with time were observed. An explanation for this development may be found in the erosion sus-cepti-ble material in the joints: As the pressure and the gradients between the tunnel and the joints outside the grouted zones increased, high local hydraulic gradients across the residual soil increased, leading to erosion of this material. This lead again to sudden equalization of pressures between the tunnel and the piezometer in the corresponding joint. Such sudden increases of pressures, up to 2 Mpa, took place with-in 10 minutes at several occasions. The eroded mate-rial was re-located within the joint system, causing a new build up of pressure differences.

The recordings of piezometer TH 9 in Figure 10 shows clearly such pressure development at pres-sures 6.3 to 6.8 MPa. At the latter pressure, pie-zometer TH 9 again communicated directly with the water in the tunnel.

Pore water pressures recorded in some of the pi-ezome-ters located in the "dry zone", downstream of the plugs, are shown in Figure 11. A distinct brake in the pressure increase development took place at water pressure 6 MPa in the tunnel, corresponding to the virgin ground water pressure in the rock for all piezometers located in the joints. Piezometer TH 15, is located in rock of very low permeability without any distinct jointing, between the downstream ends of the concrete plugs. This pie-zometer showed prac-tically no pressure increase until the water pressure in the tunnel reached approximately

7,5 Mpa. Then it increased within short time to the same pressure level as the other piezometers located in joints with-in the “dry zone”, i.e. 4 Mpa. Thereafter the piezom-eter showed an unstable behavior with sudden fluc-tuations between 4,5 and 1,8 Mpa. About one month after start filling, the fluctuations came to and end and the further pressure increase followed a development similar to the other “dry zone” piezometers. It has finally stabilized at a pressure of 5 Mpa, i.e. in the same order as the others. The permeability in the rock were piezometer TH15 is located, is still very low, in spite of the observed pressure behavior.

The behaviour of the pressure in piezometer TH15 may be explained partly as the consequence of erosion and subsequently collapse and blocking of the residual soil in joints at high local gradients. Be-cause of low permeability, pressure changes within short time spans may be distinct. Similar occurrenc-es could also be seen in other piezometers, support-ing the idea about erosion. Natural stress adjust-ments in the rock when the ground water pressure exceeded by far the virgin water pressure in the rock, could also be an explanation for the sudden pressure changes.

One year after filling of the headrace, pressures in all piezometers have stabilised at reasonable levels.

The monitoring of strains in the sectional con-crete lining show that no strain occurred neither ra-dial or longitudinal in the lining.

During the filling operation, water inflow in the “dry zone” of the tunnel system was followed close-ly. Several stations for manual monitoring were es-tab-lished. Stations in the eastern area (Penstock plug and eastern tunnels) showed only small inflow, total-ly around 2 l/s. during the first month after filling, decreasing to less than 1 l/s. 2 months later.

In the western area, downstream of the bypass plug, the inflow was significantly higher. Total in-flow, including some inflow from the access tunnel, shown in Figure 12, was less than 10 l/s. when the water head in the tunnel was below 750m. With higher head, the inflow increased to 50-60 l/s. A ma-jor part of this came in the gallery of the bypass plug, or just downstream of the plug. The inflow within the plug area was apparently due to commu-nication between the sub-vertical joint system within the plug and casting joints. Subsequently, a grouting program in the bypass plug gallery was implement-ed. After this grouting, the inflow in the western area has stabilised at 27 l/s. A second significant water inflow from joints occurred some 60m

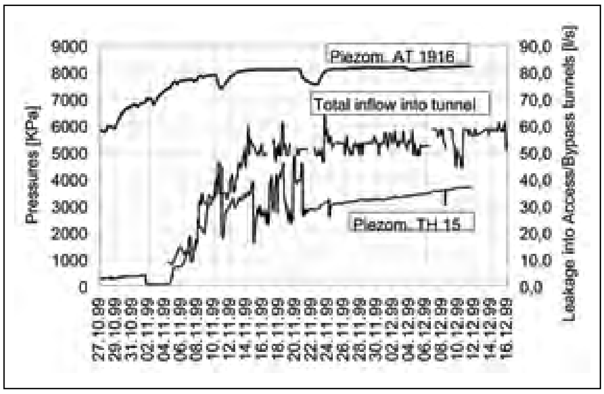

Figure 12: Water inflow into the western “dry zone” tunnel system. Pressure in piezometer AT1916, also representative for the water pressure in the tunnel, is shown for reference.

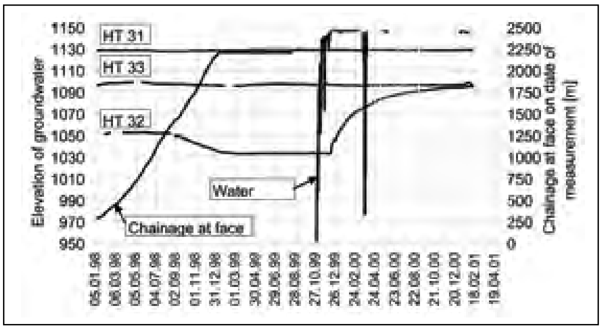

Figure 13: Ground water levels recorded in bore holes above the tunnel from before excavation started to after filling.

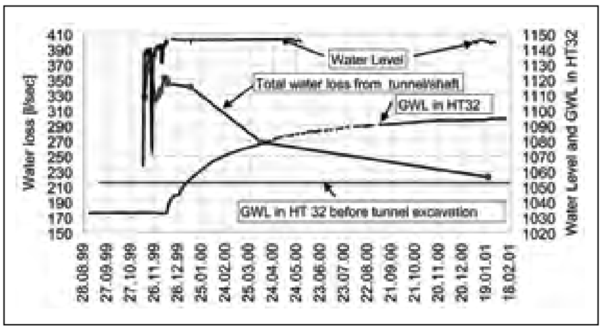

Figure 14: Total outflow from Shaft and Headrace tunnel just after and up to 4 months after filling was completed. Reservoir water level and water level in HT32 are shown for reference.

downstream of the plug. The leakage way for this water might be along joints crossing the tunnel upstream of the plugs, through the permeable weathered zone below terrain, and down again along the leaking joints. These joints also gives good drainage around the power house. Therefore, no attempt was made to grout these joints.
Before tunnel excavation started, ground water sounding was initiated in 3 bore holes in the ridge above

the tunnel. Changes in the water levels are shown in Figure 13. Any influence from tunnel ex-cavation and filling was only observed in one of the bore holes, HT32. Bottom of this hole is located ap-proximately 480m above the headrace tunnel at Chainage 310. The water level in HT32 started to sink about 3 weeks after the tunnel excavation met the permeable zones causing the abrupt increase in inflow in the tunnel, about 900m north of HT32, see Figure 4. The water level started to rise again some 1.5 months after start of filling of tunnel. 15 months after completed filling the water level is about 40m above the virgin water level. The water level has not yet completely stabilised (February 2001).

After completed filling of the headrace tunnel and shaft, the water loss has at some occasions been measured, reading the rate of drop in water level with a high-precision level transmitter when closing the intake gate. As seen from the "Outflow" curve in Figure 14, the total water loss decreased from 390 l/s. just after completed filling to 215 l/s. 15 months later. The loss of water is considered to consist of 3 components:

1 Water inflow in the "dry zone" of the tunnel sys-tem, amounting to approx. 30 l/s.

2 Water outflow to terrain, observed as increase in water flow in creeks. The complete flow increase is difficult to monitor. At altogether 8 stations, the increase is estimated to 100-150 l/s.

3 Filling up of ground water reservoir. So far the rise of water table in HT32 is more than 40m. This component of the water loss from the tunnel is temporary, until the new water table has stabi-lized.

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Nore 1 started operation 1928. It is recognized as the first hydropower project developed by Government. Installed are 8 units (Pelton) with the total capacity of 206 MW. NVE (Norwegian Water Resources and Energy Directorate) established 1921 is later reorganized as a public agency while Statkraft (established 1992) is owner and operator of Governmental owned power stations. The picture illustrates the steel penstock installation. Photo: Statkraft.

04. TUNNELS AND SHAFTS IN SMALL HYDROPOWER PROJECTS

STEFANUSSEN, Werner
GJERMUNDSEN, Tor

INTRODUCTION

The expression Small Hydro Power Plants (SHPP) normally is used for hydropower plants with an output of less than 10 MW. Some countries have defined higher values in their definitions.

Traditionally, the waterway for small hydropowers consists of an intake structure, canal or horizontal pipe and a steel pressure penstock to the power station above ground. The power station normally is above ground.

Due to the technological development, the environmental aspect and the lifetime costs related to the safety aspect and the maintenance cost, tunnels and shafts have been common in large and medium hydropower for decades. During the last 10 to 15 years this design and layout has been used also in Small Hydropower Projects in Norway.

DESIGN LAYOUT

Most of the Norwegian SHPP's are typically run-off the river projects with relatively high head (larger than 250 meters). The discharge is normally in the range of 1 to 10 m^3/sec, which requires quite small diameters for the tunnels, shafts and penstocks.

The traditional penstock above ground, or buried in a ditch, will normally be the cheapest and the less time consuming solution. However, the governmental requirements and the topographical conditions may require alternative solutions. In Norway the governmental environmental requirements normally does not allow for a penstock above ground. In some projects, the topographical conditions does not allow construction of a buried penstock due to steeply inclined slopes with exposed rock, or risk of landslides. Then, the alternative solution with tunnel and shaft may be relevant. These governmental and topographical conditions are also relevant in other countries, and the "Norwegian solution" may be an alternative. During the last 10 years Sweco Norge AS has designed tens of small hydropower projects with tunnel and shaft solution. In most of these projects the power house has been constructed above ground, but it might also be possible to design and construct an underground power house located in a rock cavern.

ALTERNATIVES

In the following, different and most common solutions for underground waterways (tunnel and shaft) are described. A combination with surface solutions can also be possible or preferable. Typical alternative design of the waterway is illustrated in fig. 2.

Figure 1. Installation of penstock in dich

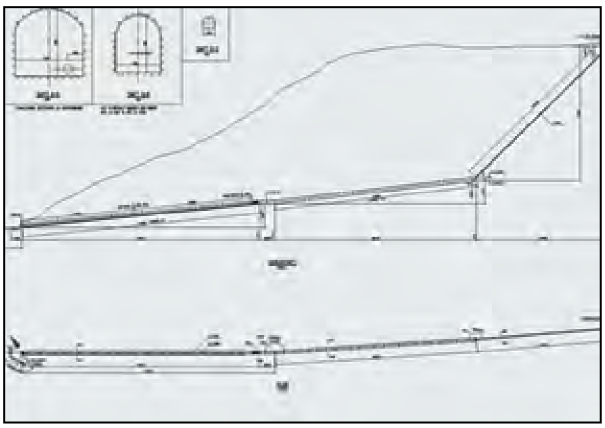


Figure 2. Alternative design with inclined shaft and unlined pressure headrace tunnel

Normally the tunnel length is within 1 km, with a cross section of minimum size, like 12 to 14 m^2. The length of the shafts are normally 300 to 500 meters, and with diameters from 1 to 1,5 meters. The limitations in length are due to the economically aspects of a small hydropower, or technical reasons for the shaft drilling. When performing an economical evaluation of the project, the cost of the underground works will certainly be a limitation. The restricted length of the shaft is because of the present technology for the light weight equipment to be used in these projects without road access to the intake location. The weight restrictions of helicopters are setting the limitations of the equipment to be used for shaft drilling.

Figure 3. Small size 2-boom tunnel machines

CONSTRUCTION METHODS

Tunnels

As indicated above, the required tunnel diameter is normally small, in the order of 8 to 10 m^2. However, to obtain high tunnel performance, the smallest tunnel profile is normally in the range of 12 to 14 m^2. With this cross section, small high performance two-boom jumbos can be used for drilling the blasting rounds. The loading and mucking equipment must be adapted to each other and to the tunnel profile. Our experience is positive using front loaders with extra low height. They are efficient in loading, with a large volume scoop and have high maximum speed. These can be used for mucking and transport of the blasted rock mass for until 400 meters. If the tunnel is longer, construction of a turning niche is necessary every 250 meters. By using the correct equipment, the tunnels can be constructed with an upwards inclination of 1:5 (20%).

Figure 4. Low height mucking and transport equipment

Shafts

The shafts are normally constructed by use of pilot hole and reaming (raiseboring), and are normally inclined (to about 45o). They can however be vertical in other situations. Due to the advantage of using light weight machinery to possibly use helicopter transport to the shaft location, the equipment has limited capacity by length. Normally, in our experience, the shaft length can be at a maximum of 600 meters. The diameters can be in the range of 0,7 meters to 4 meters. However, due to the water quantity, normally the shafts are constructed with a diameter of 1 to 2 meters. The shaft is connected to the tunnel at the end of the tunnel.

Lately, new developments in shaft drilling equipment have been developed in Norway. This gives the possibility to perform the drilling from the lower end, and upwards to the intake position. The length with this technology can be up to 1000 meters and even longer. Deviation controlled shaft drilling is also possible.

Helicopter is used for transportation of equipment if no access road is possible.

INVESTIGATIONS AND DESIGN CRITERIA

The geological and topographical investigations includes study of the geology by field survey and laboratory investigations of rock samples. Special focus is paid to the entrance area of the tunnel, and the intake area of the shaft. Relevant investigations are review of geological and topographical maps, experience from other projects in the area, field survey, investigation pits, core drilling, geophysical investigations.

To have exact topographical maps, it is recommended to perform aerial survey by scanning, and processing detailed topographical maps.

For the pressure tunnels, the rock cover must comply with the water head pressure to avoid hydraulic splitting. Norwegian splitting criteria is used, based on empirical formulas, or by performing hydraulic splitting tests.
In areas with severe geological conditions, it might be relevant to perform core drilling investigation. The core drilled hole may also be used to perform permeability tests of the rock mass.

ROCK SUPPORT

The Norwegian Tunnelling Method is based on unlined water pressure tunnels. The typical rock support methods in tunnels are rock bolts and fiber reinforced shotcrete. Norwegian reinforced shotcrete arches are also used to a certain extent. Full concrete lining is only used in special situations with severe fault zones with swelling clay materials.

The concrete plug (conus) is constructed at a location of the tunnel where the criteria to avoid hydrofracturing requirements are fulfilled. From this point a steel-, a cast-iron- or a GRP penstock is used to connect to the power station. The length of this penstock depends on the topographical conditions, and can be from 50 meters to several hundreds of meters.

The rock support in the tunnels where the penstock is used, the rock support normally includes systematic pattern rock bolting and shotcrete with fiber.

Rock support in inclined drilled shafts is not used. If severe geological conditions are encountered, grouting is performed as down-stage grouting.

COST AND CONSTRUCTION TIME

Construction cost and construction time is essential for all hydropower projects, and especially related to small hydropowers. Construction of the intake dam is normally a small investment, but may depend on the topographical and geological conditions. Small concrete dams are normally constructed with a dam height of 5 to 6 meters. The intake arrangement in projects in Norway needs special arrangement due to the cold climate with snow and ice.

The solution with buried pressure pipe is normally the most economical solution. However, use of high performance tunnelling and shaft drilling equipment has shown to be competitive, due the possibly shorter (straight on) underground solution.

Lifetime, future maintenance cost and safety aspects of the waterway should also be taken into consideration.

From Sweco Norge experience with design of several small hydropower projects, the cost of the waterway by using underground design (tunnel and shaft) will normally be 20 to 50% higher than the buried penstock solution. In the case when only the underground solution is feasible, the cost is not an issue.

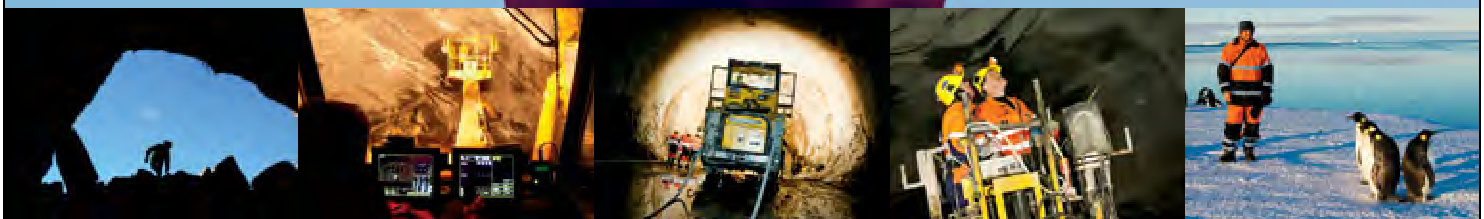
Based on ther typical Norwegian tunnel excavation, an advance rate of 40 meters per week, and about 2 months for the construction of the drilled shaft, the time schedule can in some projects be more favourable by using the underground solution, compared to the traditional solution.

CONCLUSION

Underground solutions are becoming more and more common in SHPP's in Norway. The reasons can vary, but keywords are environmental aspects, topographical conditions, safety and lifetime cost. With a future development of equipment, underground solutions will be even more competitive in the future, and will definitely be adapted into the international market.

Figure 5. Raise drilling. Ready for reaming the shaft


Our secret is
Norwegian
Tunnel Technology



05. LAKE TAP DESIGN FOR SISIMIUT HYDROPOWER PLANT, WEST GREENLAND

MATHIESEN, Thomas K.

SUMMARY
The traditional Norwegian method of underwater tunnel piercing called "lake tap" has been developed through more than a 100 years of Norwegian hydro power development. In Norway more than 600 lake taps have been successfully performed since the 1890's. The well proven technique has been applied mostly in the hydropower sector but over the last 20 years the technology has also found its use for shore approaches in the offshore oil and gas industry.

Besides blasting and excavation techniques the method relies on thorough evaluation of engineering geological aspects, and geometrical considerations are of significant importance in order to optimize the tunnel alignment making a lake tap possible at minimized risk. Further, analysis of hydrodynamic shockwaves is vital for the final adjustments to the geometrical layout. The operation requires thorough planning of monitoring systems for control of water levels and air pressures, and for measuring and documenting the hydrodynamic impact from the blast and surge of water after breakthrough.

The Icelandic contractor Ístak have constructed a 15 MW hydropower plant in "2nd fjord", 30 km north of the town of Sisimiut on the West Greenland coast. The lake tap to the reservoir was designed and supervised by Norconsult AS. Construction commenced in June 2007, the lake tap was successfully performed September 2009, and in November 2009 the electricity was switched on the distribution net of Sisimiut.

METHODS FOR SUBMERGED TUNNEL PIERCINGS (LAKE TAPS)
The idea of performing 'lake taps' originates from traditional Norwegian hydropower development where the method seeks to utilise the potential of lowering the normal water level of natural reservoirs, thereby increasing the usable reservoir volume. The method involves excavating a tunnel under a lake, leaving a short rock plug to the lake. A final blast round, prepared from the tunnel, will be piercing the lake bed from below.

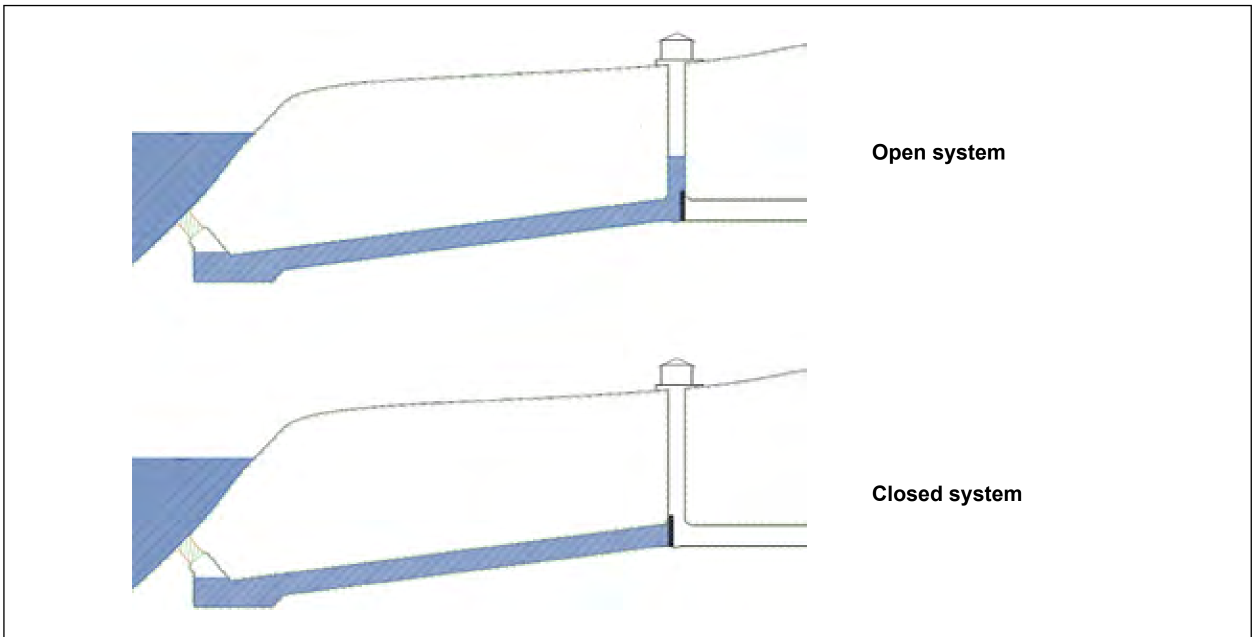


Figure 2 Principle layout of open (top) and closed (bottom) lake tap systems.

The lake tap method can generally be divided into to two groups; closed and open systems. Figure 2 illustrates the two systems in a typical hydropower layout. In the open lake tap system the lake piercing is linked to the atmosphere, for example through a gate shaft, allowing a surge to even out the pressure after blasting. In the closed system the lake piercing is isolated from the atmosphere by a gate or valve. Both methods can be performed with full, partial or no water filling. The purpose of water filling is to limit the amount of sediment transport as the water flushes in and to limit surge of water through the tunnel/shaft.

Water filling and compression of air pocket

If the intake tunnel inside the final plug is left dry, a rush water from the reservoir will fill up the tunnel once the final plug is blasted. The high velocity of the water flowing from the reservoir will transport rock debris from the blast, which may damage any nearby gate structures. However, if there are no structures that may be damaged by water pressure or sediment transport and the surge of water is acceptable, this method may be a simple and inexpensive option. This method is often referred to as "open dry system". A closed system may also in some cases be performed without water filling, as the compression of the trapped air between the piercing and the gate will reduce the hydrodynamic shock and slow down the velocity of the water sufficiently so that the rock debris does not damage the gate. The latter system, however, requires a significant distance between the piercing and the gate.

Filling of water in the tunnel prior to the blast may be an efficient method for slowing down or eliminate the water flow into the tunnel, thereby limiting the sediment transport. In this case the tunnel geometry must include a spoil trap for a concentrated settlement of rock debris. The explosives for the final blast must not be in contact with the water inside the tunnel as this may produce harmful shock-waves from the detonation that may damage the gate structure. An air pocket towards the final rock plug ensures this. The air pocket also ensures that the explosives, detonators, and connections are kept as dry as possible in order to minimize the risk of faulty ignition.

The water filled solution is generally considered to be a safer and more controlled way of performing a lake tap in cases where there are structures that may be damaged by pressure or sediment transport or where the surge of water is not acceptable. The method is, however, more complicated and requires special attention to geometry and a practical design of systems for controlling and monitoring water level and pressure in the air pocket.

If there is a difference in the pressure of the water inside the tunnel and in the reservoir, a pressure build-up will occur after the rock plug is blasted resulting in a hydrodynamic wave propagating through the tunnel system. In order to allow lean and cost-optimal dimensions of the gate/valve, it is desirable to keep the maximum pressure build-up as low as possible. This may be achieved by analysing the lake tap system in order to optimise the volume and pressure in the air pocket. Such analyses may be analytical, empirical, mathematical and/or numerical. The analyses must consider the geometry and dimensions of the tunnel system in relation to the volume and pressure of the air pocket and amount of gas developed by the explosives during the blast. Analyses depend on basic hydrodynamic theory; however, the complexity of the total lake tap system leaves significant uncertainty in many important factors of the analyses.

Blast design, charging, and detonation system

Once the final blast of a lake tap is initiated and completely or partially detonated, it is considered very difficult and/or dangerous to repair or improve a faulty or incomplete breakthrough to the reservoir. Unsuccessful lake taps do occur from time to time for various reasons, but it is possible to solve such problems. However, the solutions are generally time consuming and usually involves a significant cost compared to the original design. It is therefore prudent to implement a well thought through design of the lake tap system and pay special attention to close follow-up and monitoring of all elements in order to reduce the risk of failure.

It is important to achieve complete detonation, sufficient break and good fragmentation of the rock debris, leaving sufficient hydraulic opening area for the intended operation of the tunnel. Compared to normal tunnel blasting the following principles usually govern 'lake tap' design:

- High amount of explosives in a tight drill pattern; usually specific charge in the order of 2-3 times that of normal tunnel blasting
- Higher number of detonators and 2 or more separate ignition systems (to minimise risk of incomplete ignition)
- Shorter total ignition time (to ensure total ignition before blast fragmentation occurs)

DESIGN AND EXECUTION OF THE LAKE TAP AT SISIMIUT

Background

The town of Sisimiut lies on the west coast of Greenland, and is with its 6 000 inhabitants Greenland's second largest urban area. In 2007 the construction of a 15 MW hydroelectric power plant commenced, with the

Figure 1 Location and layout of the project of project.

purpose of replacing expensive and less environmentally friendly diesel generators. The project area lies in Kangerluarsuk Ungalleq, also known as "2nd Fjord", about 30 km northeast of Sisimiut. The project comprises intake at lake Taserssuaq (elevation ~78 m.o.s.l.), 5 km unlined headrace tunnel, underground power station, outlet in the fjord (see Figure 1), and also a 27 km long transmission line and a transformer yard in Sisimiut.

The project owner is Nukissiorfit (Greenland Electricity Company), and an EPC contract was awarded to the Icelandic contractor Ístak with its main sub-consultant Verkís. Norconsult was sub-contracted by Ístak for the special design and follow-up of the lake tap. Also previously Norconsult has cooperated with Ístak on a similar lake tap project in Greenland at Qorlortorsuaq, completed in 2006, and is also working with them at the on-going project in Ilulissat.

The main responsibility of Norconsult at Sisimiut focused on the final 30 m of tunnelling towards the reservoir and the final lake tap including; geometric design, system for probe drilling, blast design, system for water and air filling and instrumentation/monitoring.

Layout and geological conditions at the intake

The reservoir intake lies at a depth about 15 m below the highest water level. Just inside the intake the tunnel area is enlarged to accommodate rock debris from the final blast, leaving sufficient cross-section for the water to pass on to the headrace tunnel. The intake gate structure is a sliding gate with upstream sealing located at a distance of 120 m from the intake. The gate is operated through a 23 m high gate shaft to a chamber accessed from the surface. The principle layout of the gate and lake tap is shown on Figure 3.

The intake lies in a steep rock slope with gneiss of good quality with two main joint systems; parallel and normal to the surface. Some surface weathering was observed down to 3 m from the surface. Divers had verified hard rock surface with generally little sediments except some accumulated sand and slide debris on small shelves in the slope. Due to the joint systems and observed lack of sediments some water leakage in the tunnel was expected.

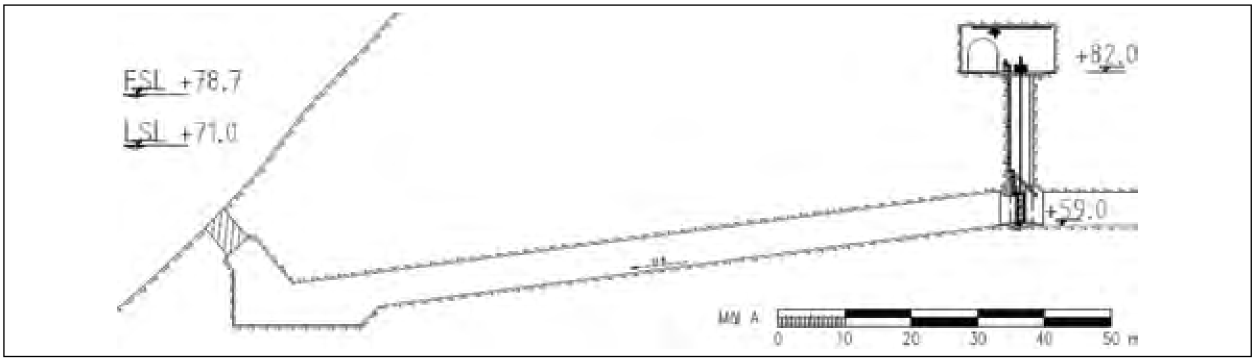


Figure 3 Geometric layout of intake at Sisimiut.

Tunnelling towards the intake
Excavation of the last part of the tunnel towards the intake is regarded as sub-sea tunnelling and gradually also tunnelling with very low overburden. General principles to be followed were:

- Systematic probe drilling in order to be well prepared for any adverse rock mass conditions or significant water leakage
- Probing all the way through to the reservoir at critical locations in order to verify the exact location of the tunnel in relation to the lakebed
- Careful blasting in shorter rounds as the face approaches the final rock plug
- System of probe drilling through the final face to gather data for the final blast design

Data from the probe drilling was used to generate a 3-dimesional model of the rock surface shown in Figure 4. This model was used to determine the appropriate length of each individual blast-hole for the final blast. Some grouting of the final rock plug was necessary. The result of the grouting was good leaving an almost perfectly dry face, also after all holes for the final blast had been drilled.

Besides normal rock support in the tunnel, pre-tensioned spiling bolts with 1 m spacing around the designed opening towards the reservoir were installed. The bolts were 3 m long CT tensioned and grouted.

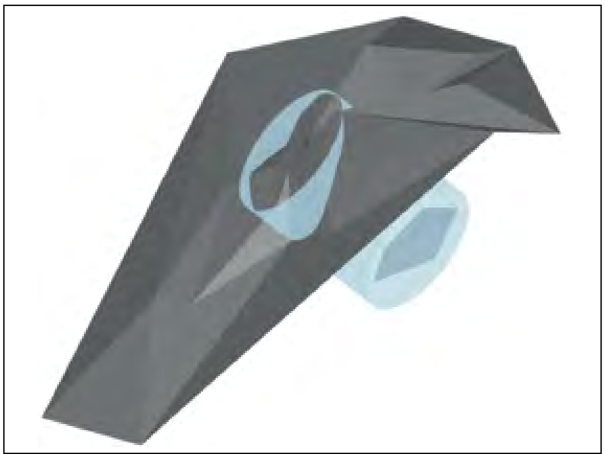


Figure 4 3D surface of the piercing area.

Blast design, charging, and detonation system
The final rock plug was circular with a diameter of 4 m and an average length of 4.3 m, resulting in a volume of 54 m^3. The blast design comprised 86 charged 51 mm blast-holes and 9 burn-holes of 102 mm diameter. The lengths of the holes varied from 2.7 m to 4.5 m, leaving approximately 0.4 - 0.6 m of rock towards the reservoir. A total of 347 kg of explosives was used, resulting in a specific charge of 6.4 kg/m^3. The explosives were 30 mm paper cartridges of “Dynomite” and NONEL MS detonation system from Orica.

All blast holes were controlled and measured and a final blast design was prepared based on the actual conditions and lengths of the drillholes. Charges for all 86 holes where prepared in plastic pipes of correct lengths corresponding to each individual blast hole. The charge in each blast-hole was equipped with 2 detonators connected in both parallel and series, resulting in 2 separate ignition systems with complete redundancy. 2 separate ignition lines were drawn from the rock plug to the gate structure and through pre-installed pipes through the concrete structure at the gate.

Filling of water and air, and design of monitoring system
The optimal level of water filling and pressure in the air pocket were designed based on analytical and semi-numerical calculations. The final optimisation of the filling level and air pressure was performed at site when the final blast design was complete and the exact amount of explosives was known. The analyses indicated an optimal pressurisation of the air pocket at about 9.1 mWc and a total volume of the air pocket of 575 m^3. Figure 5 shows the predicted pressure build-up at the gate with an expected maximum peak of about 30.7 mWc.

The water level inside the intake tunnel was monitored by level sensors at the elevation of the intended waterline in the air pocket. Pressure sensors were installed in the air pocket, at the invert below the rock plug, at the invert inside the gate, and on the air tube outside the gate. The sensors monitor pressure in the water and air-pocket during the filling process, and also functions as a back-up system for monitoring the water level. Further, the sensors record and document the pressure build-up and maximum load on the gate system during the blast. The different elements of the monitoring system were connected to 2 separate lines of data cables. For safety reasons, the data cables for monitoring are kept at the opposite side of the firing lines, ensuring complete separation of the cables.

Water filling was performed by pumping water from the reservoir, through a 6” filling valve at the intake gate structure. A 2” plastic pipe allowed compressed air to be injected into the air pocket, ensuring that the intended optimal pre-compression pressure could be obtained.

Results from blasting
Measurements of the pressure build up at the gate during the final blast and the surge is presented in Figure 5. Observations can be summarised as follows:
- The total maximum peak pressure build-up on the gate was 31.1 mWc

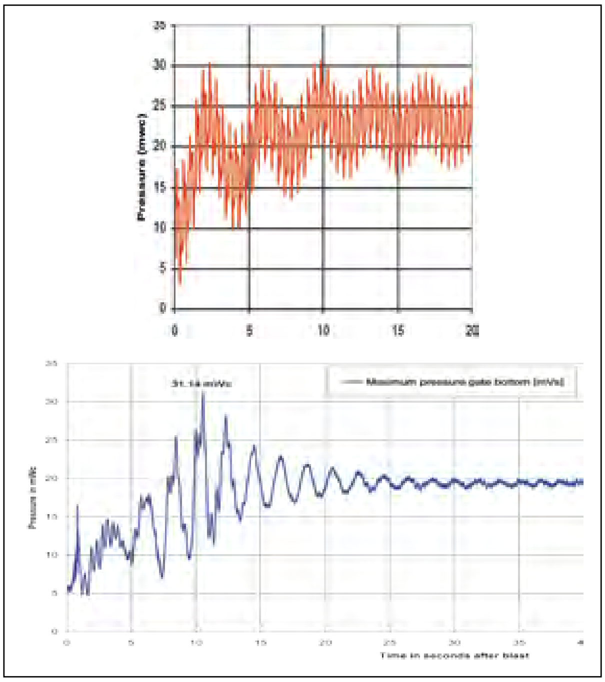


Figure 5 Simulated (left) and measured (right) pressure at the gate during after the final blast.

Figure 6 Picture showing the reservoir approximately 8 seconds after detonation.

- After the blast the recorded data show an irregular pattern, indicating turbulent conditions
- Besides the irregularity of data mentioned above, the recorded oscillating pressure corresponds well with the predicted behaviour based on the analyses of the lake tap
- The recorded time from blast until maximum amplitude and the magnitude of the amplitude indicate that the resulting cross-section of the intake opening met with design

Visual observations of the blast indicate that the break was successful and the resulting opening according to the design. Later inspections by divers verified a perfect circular opening with no significant overbreak. Figure 6 shows the lake tap blast at the surface of the reservoir approximately 8 seconds after detonation.

CONCLUDING REMARKS

In addition to blasting techniques, lake tap design involves engineering geology, hydraulic engineering and instrumentation. It is important that experienced lake tap expertise is involved in the project at an early stage when it is still possible to adjust the geometric layout of the tunnel systems in order to accommodate particular requirements for an optimal lake tap design.

In the preliminary stages of the design of a lake tap the key operation for specialists include optimising the location and geometry of the piercing and establishing procedures for rock support, probe drilling, careful blasting and grouting. Further, it is important that the layout and execution of all works accommodate all important elements and allows for a unified and practical design. It is particularly important to follow up at site the special construction and installation of all elements necessary for safe and successful execution of the works.

For the project at Sisimiut, Norconsult commenced the detail design of the lake tap approximately 1.5 years before the scheduled time for the piercing. 3 months before the piercing the design was updated based on as-built information and inspections at the site, including probe drillings through to the reservoir. Final adjustments to key elements of the design were performed during the execution and follow-up of the works during the last 2 weeks before the blast.

The monitoring program chosen for the lake tap at Sisimiut functioned well and the recorded data provides valuable documentation of the pressure conditions the gate structure had been exposed to. Compared to the predicted behaviour of the system with regards to pressure build-up, the measured results correspond well. The measured maximum peak pressure build-up was well within the limits of the capacity of the gate structure. Inspection of the gate structure after the blast showed no indication of damage.

It should be noted that the model for hydrodynamic analyses of the pressure build-up usually cannot accurately predict the highly turbulent conditions during the first 5-10 seconds after the blast. At this time, there is a great deal of unevenly distributed gas bubbles usually resulting in significant dampening of the expected early 2-3 pressure peaks. The model is mathematically accurate with respect to the input conditions, but it is not possible to accurately model deviations that may be caused by unexpected effects, such as reflections and bubbles, which may result in both reduction and increase of the peak pressure. Exact mathematical and or numerical models are very useful tools in the design of lake taps but should always be used with care and supplemented by experienced engineering judgement.

Alta Power Station. The project with 2 units and a total installation of 150 MW started production 1987. The project comprises an underground power house complex, a 145 metres high double-curved concrete dam in the Alta river in Finmark. The region is dominated by ethnic minorities and their reindeer herds. The development raised (1979-81) one of the first hot tempered campaigns against hydropower development in the country. By later decision, remaining parts of the Alta river system is adopted as environmental protected area. Photo: Statkraft.

06. SEDICON SLUICERS FOR SEDIMENT REMOVAL DURING OPERATION USED AT KHIMTI, NEPAL

JACOBSEN, Tom

INTRODUCTION

"Extreme sediment loads in rivers, mainly during monsoons, are among the major problems related to man's development of water recourses. Sediment transport will remain a natural phenomenon. Reliable and efficient systems for sediment control and the removal of sediments from withdrawn water will therefore always be one of several preconditions for the successful use of water recourses". These are the words of late Professor Dr. D.K. Lysne (Lysne et al, 1995). At Khimti HPP in Nepal, the consequences of these words have been fully appreciated.

This article describes the last sediment barrier at Khimti HPP in Nepal, which is a small rock trap at the end of the headrace tunnel. Located downstream of not less than 5 sediment barriers, and with a volume less than 1 % of the desander at the intake, it still removes substantial amount of harmful sediments.

1 KHIMTI HYDROPOWER PLANT

Figure 1: Khimti intake and desilting basin. The trap is incorporated at the end of the 7 km long headrace tunnel, just upstream of the pressure shaft.

60 MW Khimti HPP, 80 km east of Kathmandu in Nepal, is a run-of-the-river plant that exploits 680 m of head on Khimti Khola. It is owned and operated by Himal Power (controlled by SN Power), and has since year 2000 been a major contributor to the Nepalese electricity grid, and generates 350 GWh annually, nearly 15 % of Nepal's electricity. As most Nepal's rivers, Khimti Khola can carry large sediment loads. Khimti has perhaps the most comprehensive sediment removal facilities in any run-of-river power plant:

- An innovative intake in the steep Khimti Kola consisting of a low weir without gates.
- Stone trap with flushing gate immediately is after the intake.
- Gravel trap upstream the entrance of the desilting basins, with flushing gates.
- SediCon Sluicers in the retardation zone of the desilting basins
- Large desanders with a total volume of approximately 13 000 m^3 volume and designed to trap 99% of all sediments larger than 0,15 mm.
- A small rock trap in the downstream end of the headrace tunnel equipped with SediCon Sluicers.

2 SEDIMENT TRANSPORT IN THE HEADRACE TUNNEL

2.1 Suspended transport of fine sediment

The desilting basin at headworks is designed to trap all, or nearly all sediment particles that are larger than 0,15 mm. A substantial percentage of finer material will also be trapped. The trapped sediment will be removed from the desilting basin with the S4 system. However, no desilting basin can trap all suspended sediments. Some sediments will pass the desilting basin into the headrace tunnel, but this will mainly be particles smaller than 0,15 mm.

During the transport through the headrace tunnel, some of the suspended sediments will tend to be transported as bed load. Because of the bed load transport, the tunnel sand trap may be able to trap sediment particles less than 0,15 mm.

2.2 Maximum particle size mobilised from the tunnel floor

The tunnel floor at Khimti was left with an unpaved invert.

The sand trap is primarily intended to trap gravel and stones that are released from the base and the walls of the tunnel, which can be transported in the tunnel. The size of the gravel that can be transported in the tunnel can be determined by the formula: (Lysne, 1986)

$$d_{70} = \frac{\gamma_w}{\gamma_s - \gamma_w} \cdot \frac{V^2}{130 \cdot A^{1/6}}$$

γw	specific weight of water
γs	specific weight of sand
V	velocity
A	cross section area

$\gamma s = 2650$ kg/m^3, V = 1,0 m/s and A = 11 m^2 gives d70 = 3,3 mm.

However, the maximum particle size is considerably larger than d70. A study of new Zealand rivers indicate that maximum size of bed material is 2,6 – 6,2 times d60. It is therefore reasonable to expect particles up to 1,5 – 2 cm being transported as bed load and deposited in the sand trap. As explained later, even bigger particles up to 50 mm, have been found in the sand trap.

2.3 Amount of coarse material

A limited amount is available from the tunnel floor. Over the first years of operation the finer material will continuously move towards the lower end of the tunnel where the sand trap is located. If the water velocity during operation is less than 1,5 m/s, the coarser material will form a stable layer. For this velocity range the volume of transported sand amounts to 5 - 8 cm thickness over the whole tunnel bottom area. Of this 60 - 70 % will be trapped, while the finer fractions will pass the sand trap. (Lysne, 1986) Provided the headrace tunnel at Khimti is 7 km long, the amount of sediment from the tunnel floor may have been in the order of 1000 - 1600 m^3.

3 DESCRIPTION OF THE INSTALLATION

3.1 The tunnel sand trap

The tunnel sand trap was designed to trap sand and gravel which is released from the tunnel, in particular during the first months of operation. Due to the geology, the tunnel sand trap is of limited size, only 3 by 3 by 13 meters. The sand trap is covered with concrete slabs that prevent turbulence and redistribution of sediments from the sand trap during normal operation of the power plant.

The tunnel sand trap was equipped with SediCon Sluicers, for the purpose of removing sediments. The SediCon Sluicer technology permits removal of the trapped sediment during operation. This was the first time a sand trap has been designed with this technology.

3.2 The SediCon Sluicer technology

The SediCon Sluicer is a unique technology which invention dates back to 1993. The SediCon Sluicer consists of pipes with a continuous, longitudinal slot or row of slots along its lower surface. They are fixed close to the original bottom surface and connected to an outlet pipe, whose outlet is at a lower level than the water pressure. In this way suction is created by the use of gravity as shown in Figure 2.

1. Sediment is allowed to deposit on top of the slotted pipe until the thickness of the sediment deposit is sufficient for flushing.
2. The valve on the outlet pipe is opened, and flushing of sediment starts. Water is drawn through the slots and picks up sediment close to where the slotted pipe emerges from the sediment deposits (the “suction point”). As the sediments are removed, the suction point moves downstream until all sediment that cover the slotted pipe has been removed.

The SediCon Sluicer technology has several advantages compared to other sediment removal technologies:

1. The SediCon Sluicer has no movable parts, except the outlet valve. It is the unique hydraulic design that balances suction of sediment in such a way that clogging of the pipeline always is avoided. It has therefore an unmatched reliability and simplicity of operation.
2. Because there are no pumps or impellors, the SediCon Sluicer can remove particles up the same size as the pipeline itself. Typically particles up to 100 – 200 mm are removed.
3. The hydraulic design creates a sediment concentration which is always close to the theoretical maximum capacity of the outlet pipe, resulting in very low water consumption.
4. Removal of sediments does not cause suspension and the water consumption is only a fraction of the power plant discharge. Sediments can therefore be removed at any time without interfering with the power production.

3.3 Installation in Khimti tunnel sand trap

The sand trap is of moderate size, as its volume is only 133 m^3. There are two parallel SediCon Sluicers in the sand trap. Each of them is connected to a 260 m long outlet pipe. The outlet pipes discharges into a stilling basin outside the adit.

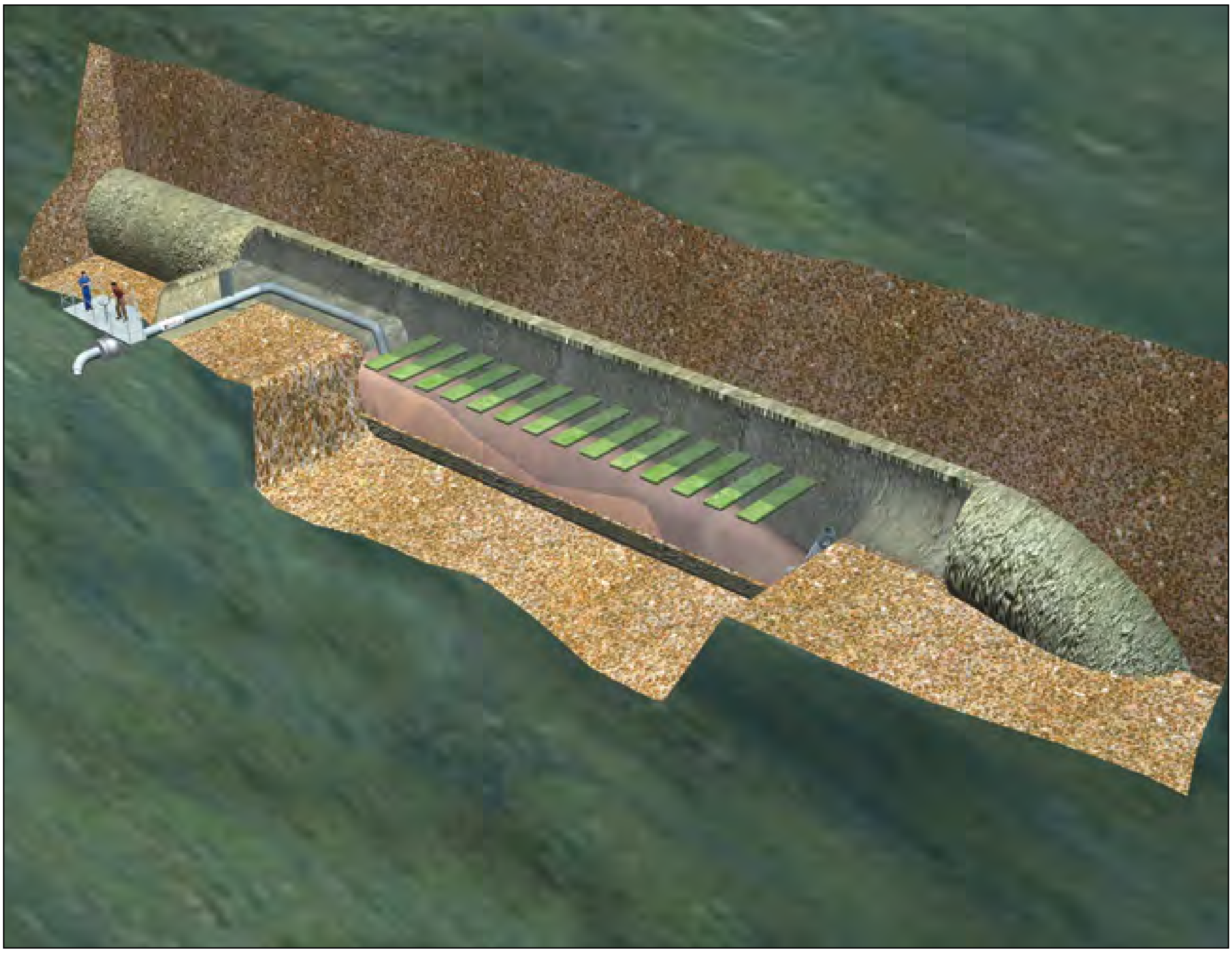


Figure 2: Principle sketch of the sand trap with SPSSs.

Figure 3: Tunnel desander above the concrete slabs. The upper ends of the SediCon Sluices are visible.

Figure 4: Tunnel desander and SediCon Sluices below the concrete slabs.

3.4 Automatic operation
The SediCon Sluicers was initially designed for manual operation, but long walking distance to the adit where the sediments are discharged (a vertical difference of 600 meters) and civil unrest in the area made the client to operate the SediCon Sluicers automatically, with fixed (basically weekly) time intervals.

4 EXPERIENCE FROM OPERATION

4.1 The first monsoon
The SediCon Sluicers were operated from the beginning and substantial amount of sediments were removed on a weekly basis. After the first monsoon season year 2000 the tunnel and the tunnel sand trap was inspected in December 2000. Everything seemed in order – Statkraft who was in charge of operation support, recorded that *"No sand was removed (during inspection) because there was no sand there"*.

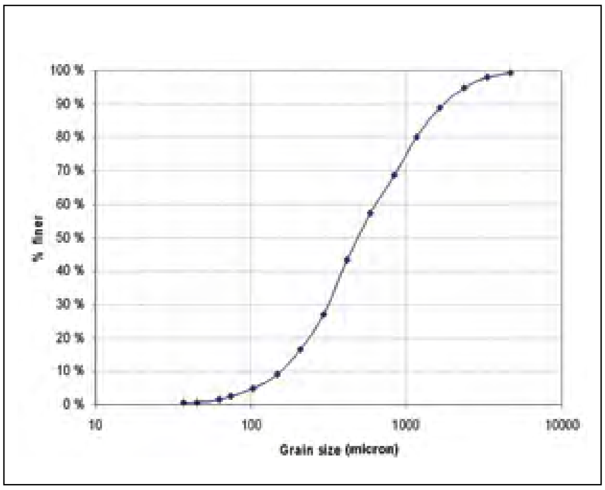


Figure 5: Grain size distribution of sand and gravel removed from the tunnel sand trap in 2000.

4.2 The years 2001 - 2003
The following experiences were recorded by Statkraft Engineering:

> *"The Flushing system at Adit 4 is working excellent… In the monsoon season we are flushing 2-4 times a month, in the dry season once per month or less...The system is working best if flushing is not performed to often…Flushing is performed with such frequency that brown, sandy water is discharged for 15 – 20 minutes. Total flushing time is one hour… We have no exact measures on how much sand that is discharged, but it is substantial amounts, but little in dry season. Dried samples consist of everything from sand particles to stones up 50 mm of size"*

According to our calculations, about 15 to 20 m^3 sediments were removed in each flushing and roughly 500 m^3 every year. Given that the removed sediment were 0,5 mm sand and that 7%, (35 ton per year) were gravel > 2 mm there should be no doubt that turbines were saved from substantial damage.

4.3 2000 – 2008: Eight years without inspection
The tunnel was operated continuously for eight years, from 2000 to 2008 without dewatering of the tunnel and without access to the SediCon Sluicers. During the 2nd inspection in 2008 it was verified that no sediments where left above the SediCon sluice pipes.

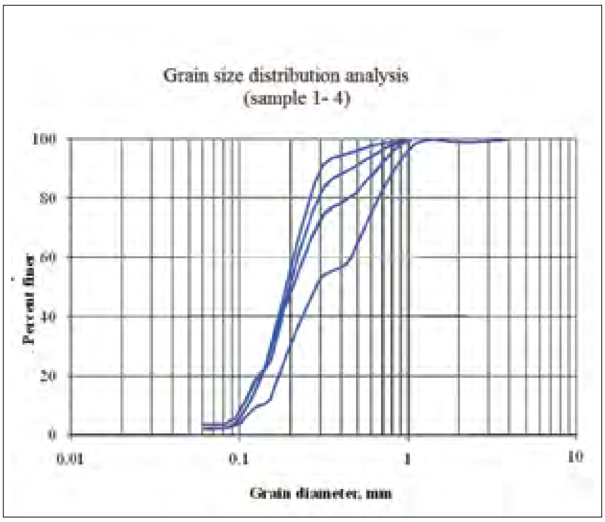


Figure 5: Inspection of the tunnel sand trap after 8 years of operation.

Figure 5: Inspection of the tunnel sand trap after 8 years of operation.from the tunnel sand trap in 2000.

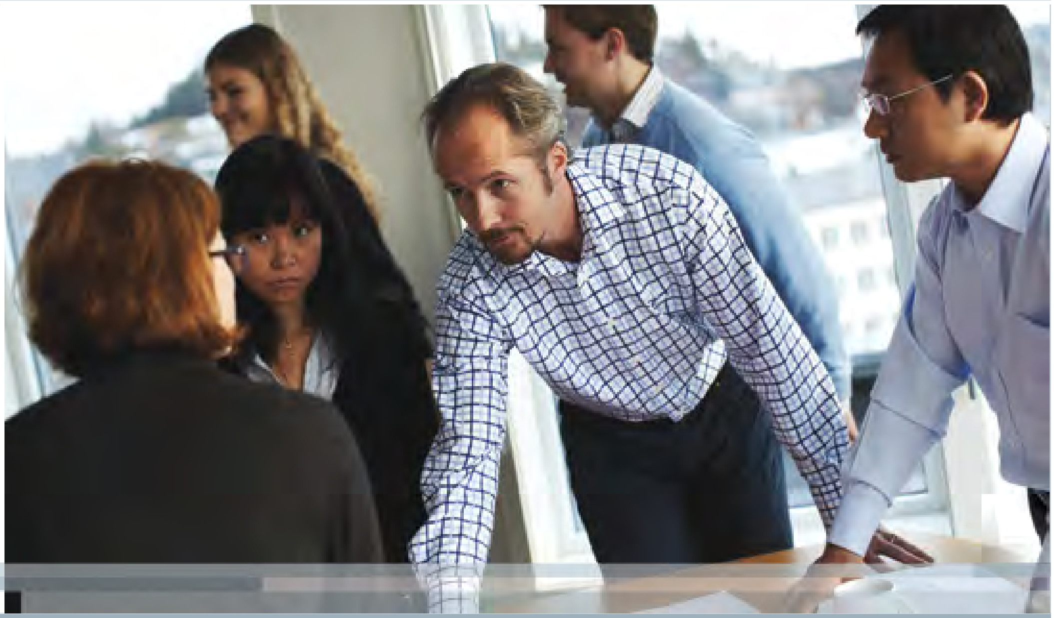
4.4 Use of the sediments
The sand fractions that are removed are rarely found naturally. They are suitable for concrete and other construction purposes, and are therefore a valuable benefit for the local population

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Efficiency of tunnel sand traps
It can be concluded that the tunnel sand trap efficiently traps sediments that is mobilised from the tunnel floor. This is coarse sediment – even at Khimti where water velocity in the tunnel is low, particles up to 50 mm have been found. The need for tunnel desanders downstream of unlined tunnels is therefore evident. It is also clear that a tunnel sand trap has very high trap efficiency compared to its volume. This is because the tunnel itself "traps" sediment and causes sediment to be transported as bed load. Even fine sediment from headworks tends to be transported as bed load and is trapped by a tunnel desander

5.2 Sediment removal with SediCon Sluicers
The SediCon Sluicers at Khimti have fulfilled all expectations; they are efficient and extremely reliable. Since year 2000, several thousand tons of sand and gravel has been removed with a minimum of effort and water consumption, saving turbines from substantial damage.

5.3 Future projects
It is recommended to design headrace tunnels with tunnel sand traps with SediCon Sluicers. With a moderate investment and minimum work and water consumption, the tunnel sand becomes a last and efficient sediment barrier which excludes tunnel floor material, sediment from the intake and rock and gravel from potential slides in the tunnel.



07. NORWEGIAN HIGH PRESSURE CONCRETE PLUGS

BERGH-CHRISTENSEN, Jan
BROCH, Einar
RAVLO, Aslak

ABSTRACT:
Concrete plugs in the conduits are important elements in hydropower projects. A study on design, construction and operation of concrete plugs in underground hydro-power projects was carried out during the period 1987-1991. Data from 150 plugs were collected and analysed, among these more than 30 exposed to static pressure heads between 400 and 1000 metres. Observed were: While the concrete length varies from 2 to 5 % of the static water pressure, the steel lining may be as short as 0.4 % of the water pressure head. The development of Norwegian hydropower howev-er, did not stop in 1991. Many new projects have been implemented; during the recent decade, - not less than 25 projects. Eight of these projects have concrete plugs withstanding water heads of 300 metres or more. Two of them are described in more detail in this paper.

1 INTRODUCTION

Concrete plugs have always been vital elements in the development of underground hydropower pro-jects. Plugs using the same technique are also elements in several other sectors of underground con-structions. For the oil and gas industry concrete plugs are used for shore approach of offshore pipe-lines or plugs for and between numerous storage caverns.

The basic requirements to concrete plugs are:

- The plug must be strong enough to withstand the actual pressure
- The leakage through the plug and the adjacent rock mass must be less than specified.

Project	Area	Owner	Waterhead (m)	Commissioned
New Tyin	Årdal	Norsk Hydro	1050	1944/2004
New Bjölvo	Ålvik	Statkraft	872	1918/ 2003
NewFlørli	Lyse	Lyse	755	1918/ 1999
Svartisen	Nordland	Statkraft	580	1993
Sønnå H.	Sauda	Elkem	550	2008
Eiriksdal	Sogn	Statkraft	547	(2014)
Innvik	Stryn	Stryn Energi	504	2005
Åbjøra	Oppland	Skagerak	433	2001
Framruste	Øvre Otta	Opplandskraft	325	2008
Jössang	Lyse	Lyse	300	2011

Table 1- Selected high head projects, new or completely reconstructed during recent years

Specific requirements deal with safety factors, durability, temperatures, plugs between caverns and more. For shore approach plugs the maximum head so far is around 200 metres. For high head hydro-power the New Tyin project is a new number one with a water head of 1050 metres out of which 1030 metres through unlined headrace shaft.

Two of these projects, New Bjølvo and New Tyin, will be described in some detail in this paper.

2 THE 1987-1991 STUDY

The 1987-91 study included information of about 150 concrete plugs. The data base included some 30 plugs with water heads above 400 meters constructed during the period 1970 - 1990. The projects Nyset-Steggje, Mel, Jostedalen and Torpa were selected for a detailed analysis and description of the plug construction, grouting operations and first water filling of the unlined waterway.

During the 1980's, several high pressure plugs were constructed for static water head close to 1,000 meters. The study was concentrated to, the at that time newest plugs which were designed and con-structed in line with improved quality standards and grouting technology. These plugs were more ex-pensive, but also more efficient in terms of reduced leakage as compared to older cement-grouted plugs.

3 PLUG TYPES

The two main types of concrete plugs used in hydroelectric power plants are shown in Figure 1. The penstock plug is located at the upstream end of the steel penstock, at the transi-tion to the unlined pressure tunnel. Access to the unlined tunnel system is usually provided by an access gate plug located in the access tunnel adjacent to the pressure tunnel.

SITE	WATER HEAD1 m	CROSS SECTION m^2	LENGTH CONCRETE m	LENGTH STEEL m	WATER LEAKAGE l/min
NYSET-STEGGJE	964	25	55	Penstock	< 60
TJODAN 5)	880	17	45	Penstock	2
TAFJORD K5	790	18	88	Penstock	50[3)]
SKARJE	765	252	20	5.5	< 15[3)]
MEL	740	22	27	27	1[3)]
SILDVIK	640	26	35	12	< 240
JOSTEDALEN	622	35	20	5	6[3)]
LOMI	565	20	15	9.5	190
LANG-SIMA	520	30	50	Penstock	120
SØRFJORD	505	20	20	12	10[3)]
KVILLDAL	465	31	30	4	4)
TORPA	455	32	20	6	< 1[3)]
EIKELANDSOSEN	455	20	20	5	8
STEINSLAND	454	20	20	10.2	[4)]
KOLSVIK	449	23	20	10	30
SKIBOTN	445	18	12	7.6	96[3)]
LEIRDØLA	441	26	30	Penstock	< 54
SAURDAL	410	49	40	1.5	5[3)]
ORMSETFOSS	373	22	22	7	< 3
DIVIDALEN	295	10	13	4.5	< 120

Table 2 - Key figures for some major concrete plugs.

1) Max. static head

2) Varies from 20 to 30 m^2
3) Remedial grouting at first water filling or later
4) Within accepted limits
5) Tjodan commissioned 1984 was the first to implement modern grouting technique in the plug area also using tubes etc for the contact grouting.

4 DESIGN

There are two fundamental requirements for the design of a concrete plug. Primarily, it must have the structural capacity to carry the static load from the water or gas pressure. Secondly, specific require-ments must be satisfied in terms of leakage. Both in the design and the construction, there are normal-ly few problems related to the load capacity. The length and layout of the concrete structure, however, often seem to be a subject for discussion. In the early years little attention was be paid to the leakage problems, although one conclusion from the 1987-91 study was that efforts to achieve the optimum tightness is important, both in terms of functioning of the plug and in terms of costs.

The plug design may vary with respect to the length of both the concrete structure and the steel lining. Figure 2 illustrates the design of two different access plugs constructed in 1989. For access plugs, the steel lining is normally shorter than the concrete lining, and may be located in the upstream, interme-diate or downstream part of the plug. The access gate may be located anywhere along the steel lined section. The shape of the plug may be simple or it may vary along the length axis in agreement with the established stress distribution.

Plug length

It is commonly accepted that the plug length should be related to the actual water head or gas pressure. As demonstrated, the length of both the concrete structure and the steel lining (for access plugs) may vary within wide limits, even for the same water head. The steel lining is usu-ally shorter than the concrete lining, the extreme being the Saurdal access plug with a steel lining of only 1.5 meter at a static head of 410 meter. Sometimes the steel lining of the access plug may even be of the same length as the concrete structure (Mel plug).

Figure 3 shows that the length of the concrete structure for a high pressure concrete ranges from about 2 to 5% of the maximum static water head (in meter) thus underlining that other plug structural aspects are vital. For tunnel cross sections ranging from 8 to 50 m^2, this represents a maximum shear stress of about 0.4 MPa at the plug circumferential area, assuming a uniform shear distribution in the rock to concrete interface. This used to be the maximum accepted shear stress for uniaxial situations according to former standards for concrete structures (for uniaxial concrete strength 25 MPa, i.e. C25). Today emphasis are put on aspects related to rock surface in the plug area and grouting procedures

The maximum linear hydraulic gradient along the plug axis (ratio of water head to concrete length) that may be calculated for a shear stress of 0.4 MPa will be ranging

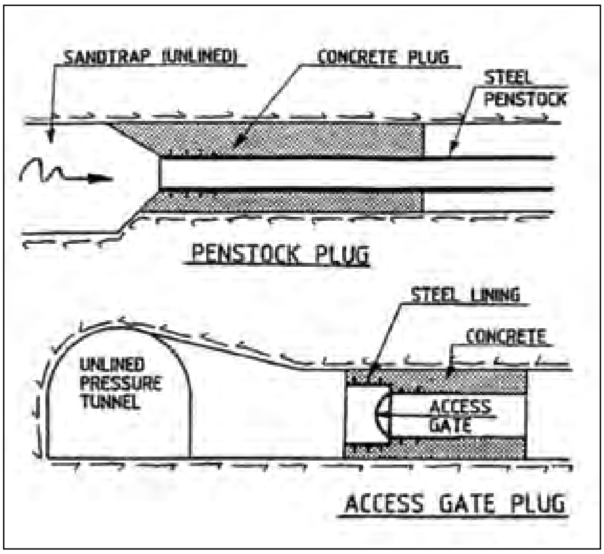


Figure 1 - General layout of penstock plug and access gate plug

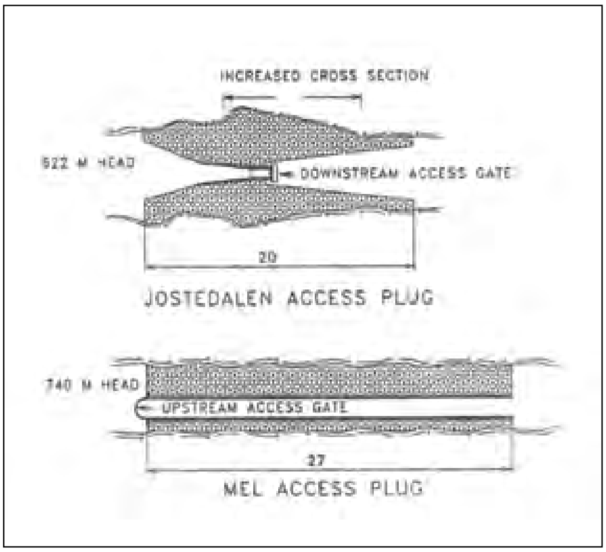


Figure 2 - Sketch of Mel and Jostedalen access plugs.

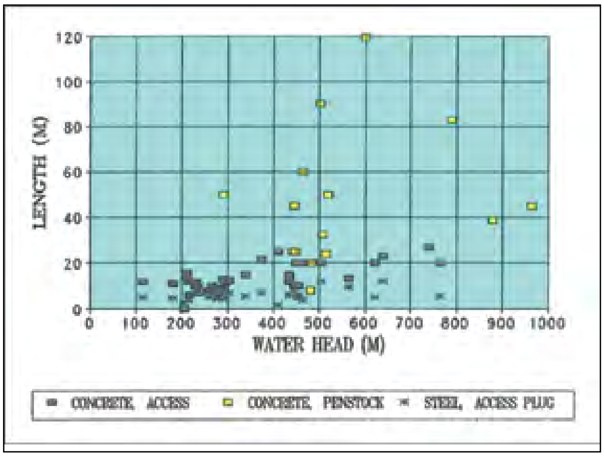


Figure 3 - Length of steel lining and concrete structure vs. static water head.

from 20 to 50 for the tunnel cross sections in question. This complies with a traditional rule of thumb for plug design in Norway, which is based on the assumption that higher gradients may lead to unacceptable high leakage. This gradient criterion may be considered radical. Benson (1989) has for instance suggested that the maximum hy-draulic gradient should be as low as 20 for massive, hard and widely jointed rock types.

In reality, the uniform shear distribution presupposed in this design principle is not valid. Numerical modelling carried out during the research project showed that the shear stress will be concentrated to the first five meters of the upstream part of the plug (assuming steel gate located upstream so that the water pressure is not acting from inside the plug structure). The shear stresses rapidly decrease further downstream along the plug. Therefore, if one considers the actual stress distribution within the con-crete body as calculated by numerical methods, relatively short plug lengths could be allowed. In prac-tical design, however, one should also consider the three dimensional water flow regime and the limitations with respect to grouting. In this context, it is the authors' opinion that the minimum plug length for high pressure plugs that are supposed to act as water tight constructions should never be less than five meters.

5 CONSTRUCTION AND OPERATION

There has never been reported any plug malfunction or failure related to overloading in Norwegian hydropower projects. The only "failure" experienced is unacceptable high leakage. Normally, remedial grouting will be carried out during the first water filling or at a later stage. But the criterion for reme-dial grouting used to depend on the actual owners.

The consumed grout mass as documented for some plugs is shown in Figure 4. As can be seen, several tonnes of (fine grained) cement are normally injected. Most of the cement mass is used to fill the voids that normally will develop in the contact zone between the rock and the concrete at the tunnel roof. If cement grouting is neglected or not well performed, large quantities of the far more expensive chemicals will be needed.

Often one will observe that the plug is constructed at the very latest stage before the power plant is put into operation. The plug construction period must therefore be short. The cast concrete temperature will often rise to about 60 to 70o during the curing period. The plug will then cool down gradually, but slowly. Efficient grouting must not take place too early. It must be delayed until the concrete temperature has reached an acceptable low level. Because the construction of the plug is on the critical path of the overall timetable, it is a trend that grout-ing takes place too soon. Both the tightness of the plug and the grouting expenses will suffer. Careful planning and control with the concrete temperature is the solution of this problem.

The efficiency of the grouting works is believed to depend on the grouting pressure in relation to the water head and the rock stresses. For several plugs, the grouting pressure has been con-siderably higher than the water pressure. Figure 5 shows how the grouting pressure for some plugs is related to the static water pressure.

At Torpa and Sørfjord, the grouting pressure was higher than the minor principal rock stress as indicated by overcoring measurements. At Torpa, the grouting pressure was even higher than the hydraulic jacking pressure measured at the plug location.

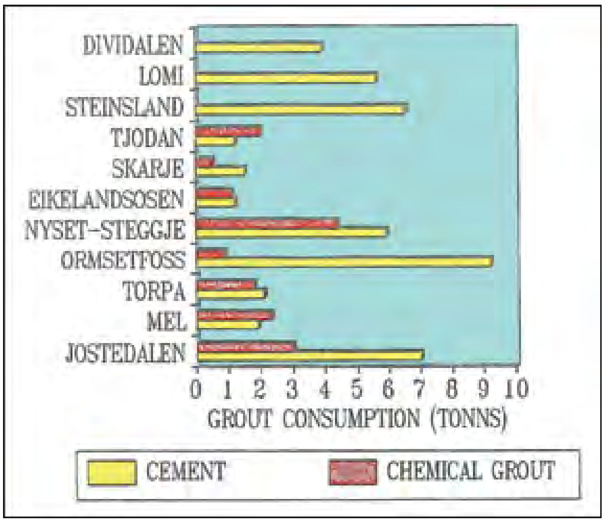


Figure 4 - Grout consumption at some concrete plugs.

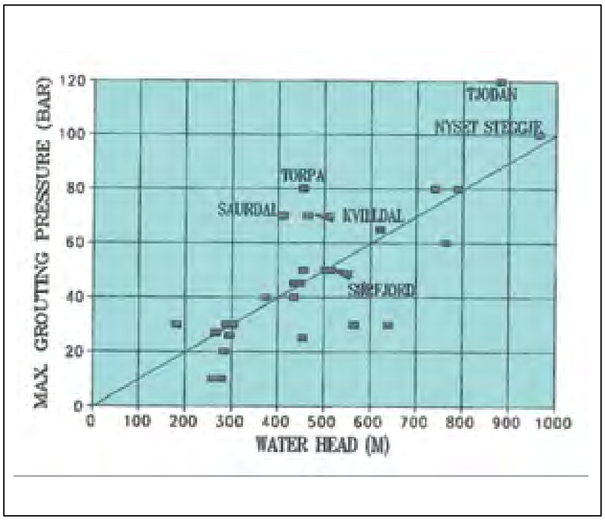


Figure 5 - Grouting pressure vs. static water head.

Leakages
When relating the water leakages to the water head, there apparently is no connection. In the-ory, the leakage should decrease with decreasing pressure gradient (Darcy). However, linear regression analysis does not correlate the leakage to the hydraulic gradient (Figure 6). No cor-rela-tion between the leakage and the length of the steel lining or the linear gradient at the steel lining was observed.

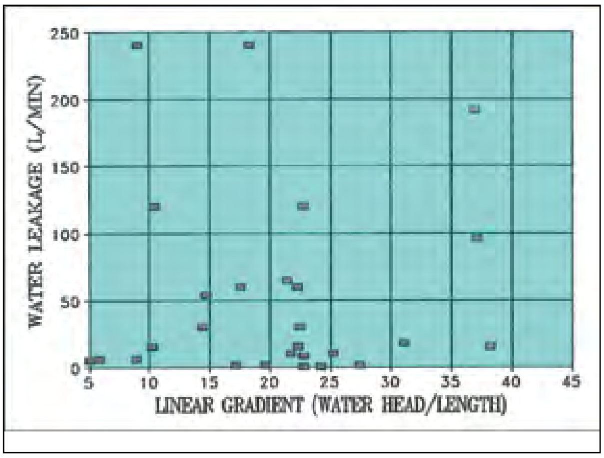


Figure 6 - Leakage vs. linear hydraulic gradient.

To illustrate the latter, the Saurdal hydropower project access plug, with a steel lining of only 1.5 meter at a water head of 410 m (gradient 273) has a leakage of 15 l/min. In comparison, the Sildvik hydropower project plug has a leakage of about 240 l/min. at a gradient of 53 (wa-ter head 640 meter and 12 meter steel lining).

The leakage is best correlated to the year of construc-tion. The modern plugs apparently are better sealed than the older ones. This is a consequence of the introduction of high pressure chemical grouting in plug construction.

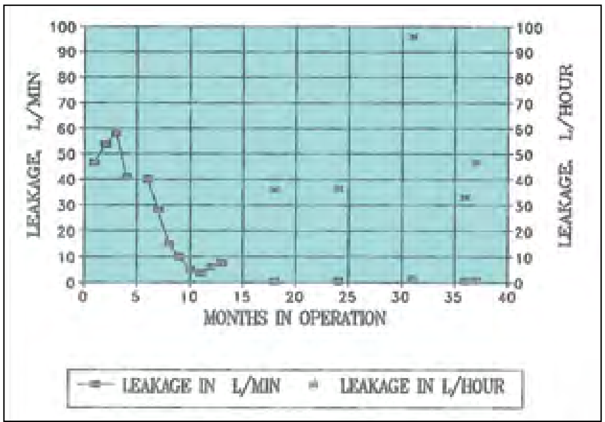


Figure 7 - Leakages at Tjodan (880 meter water head) 1984-1987.

The leakage changes with time. Detailed information is given from Saurdal, Tjodan and Tafjord. At Saurdal, the leakage was about 140 l/min. after the first filling. Additional grout-ing by polyurethane at a pressure of 6 MPa (410 m water head) through a curtain of drillholes from the downstream end about two weeks after filling reduced the leakage to about 15 l/min. Later on, the leak-age decreased further by 60 to 70% within the next year.

Even stronger reduction of the leakages occurred at Tjodan (Figure 7). No remedial grouting has been carried out. The initial leakage after the first water filling was about 50 l/min., which was reduced to about 5 l/min. during the first year of operation. In the next four years, the leakage decreased further, and was only one per cent of the initial leakage at the beginning of 1990. During the first seven years of operation, the pressure shaft was emptied twice. The owner believes that because of the emptying, suspen-sions with fine grained materials may have infiltrated the plug and caused the self sealing that have been observed. The process of leakage reduction during first period of operation (self sealing) is interesting. In addition to fine material infiltration, excess calcium from the concrete ("stalagtite process") may influence the process positively.

6 THE NEW BJØLVO HYDROPOWER PROJECT

History
The New Bjølvo Hydropower Plant is located in Ålvik on the northern side of the Hardanger Fjord, about 100 km east of the City of Bergen, West Norway. The first con-struction phase of the old Bjølvo Hydropower Plant was completed in 1918, followed by 2nd and 3rd construction phases in 1938 and 1972 respectively. The plant primarily provided power to an adjacent fer-ro-alloy electric smelter. The old plant was a conventional above ground plant with forge-welded steel penstocks down the mountain side.

In 1993 Norwegian authorities from safety reasons stat-ed a time limit to take the old pen-stocks out of service. Subsequently planning of necessary upgrading of the power plant was started. In early 2000 Statkraft received the governmental concession for developing the New Bjølvo Hydropower plant, which was to be constructed as an underground system with un-lined pressure shaft and headrace tunnel, and with the powerhouse situated deep into the rock mass.

Project summary
Intake in the reservoir, a vertical shaft of 615 meters, 1350 meters inclined (1:6) headrace tunnel, both unlined. Powerhouse cavern, tailrace and access tunnel each of 1200 meters.

High pressure concrete plug summary
The rock in the actual area consists of competent gneiss with minor intrusives of amphibolite. Stress testing showed σmin=11.3 MPa, thus a safety factor of 1.2-1.4 against hydraulic fractur-ing. Prior to installing the steel penstock the surrounding rock mass in the plug area was sup-ported through three grout curtains, spacing between curtains 5 meters and length of boreholes = 25 meters. For the waterloss testing of the holes following criteria were used:

- If the observed leakage was above 2 Lugeon (L) when water test pressure used equalled pore water pressure + 10 bars, the holes were grouted using rapid cement.
- For holes with leakage below 2 L a grout mix of micro cement with additives was used.
- Grouting to take place in two sequences.
- Grouting criteria were satisfied at grout take ≤10 kg per m. under max pressure.
- A number of test holes were installed.

After assembly of the penstock, grouting hoses were attached to the tunnel circumference and the penstock. The penstock was then embedded using 3 sections, verti-cally divided, each 1/3 of the length of the plug. After the concrete had cooled down sufficiently, contact grouting in the transi-tion rock/concrete and concrete/penstock was performed. Some problems with grouting material leaking into the hoses next to those used for grouting were solved. The final control and contact injection of epoxy material took place through holes in the penstock that had been prepared during prefabrication. The total length of the plug is 26.5 metres. The observed plug leak-age during first filling of water was ≈ 10 l/minute.

Technical data
- **Grout pressure and the placing of packers:** For first sequence 100 bar with packers 2.5 m. from rock sur-face. For second sequence max pressure from 2 to 40 bar with packers 0.5 from rock surface.
- **Grout mix:** (i) Microcement. (ii) Microcement with 3 % microsilica. (iii) Ultrafine mi-crocement with 2 % microsilica
- **Consumption during contact injection:** 3050 kg microcement; 600 kg polyurethane; 1750 kg epoxy
- **Injection tubes:** 200 – 100 and 450 metres respectively

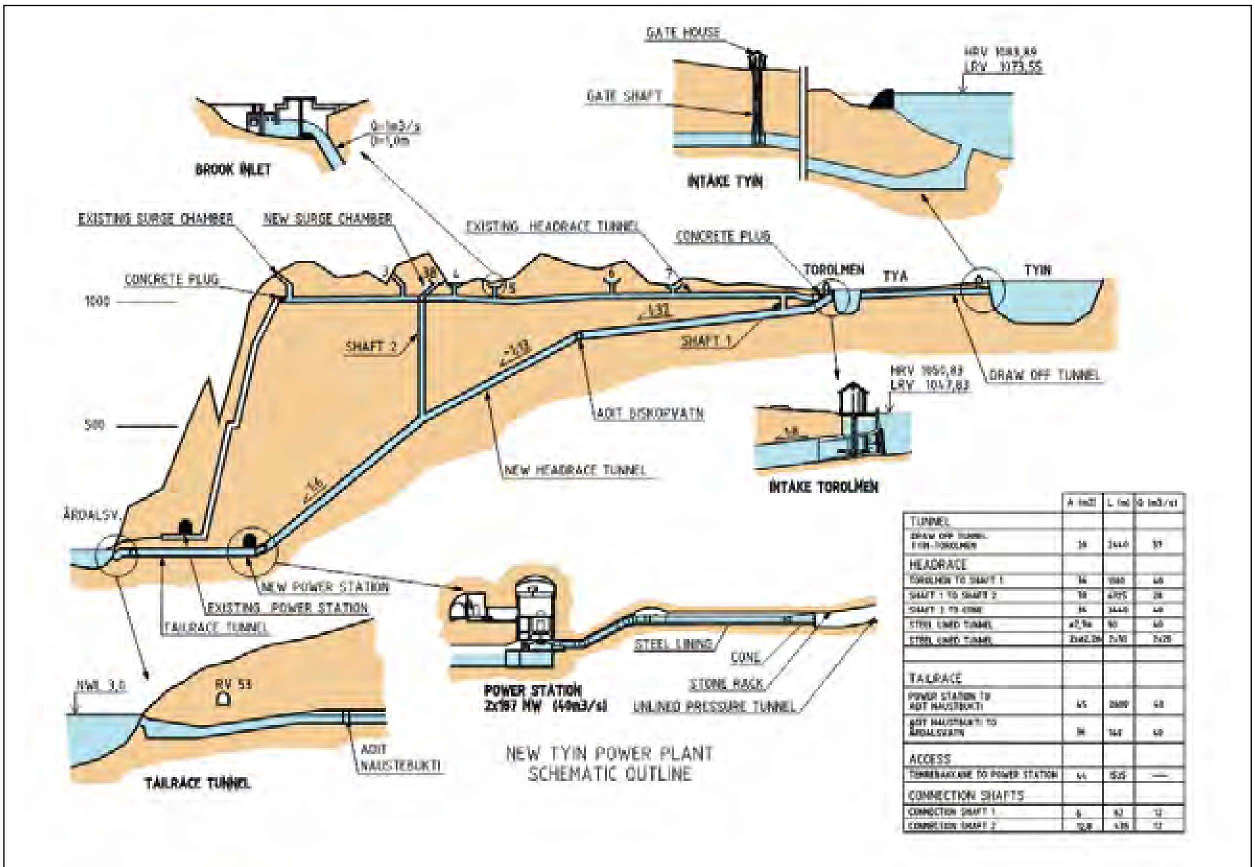


Figure 1 – The new TYIN. All parts are new save the old headrace tunnel that is now used for eight brook inlets, adding water to the system. Courtesy Norsk Hydro/Knut Helgesen

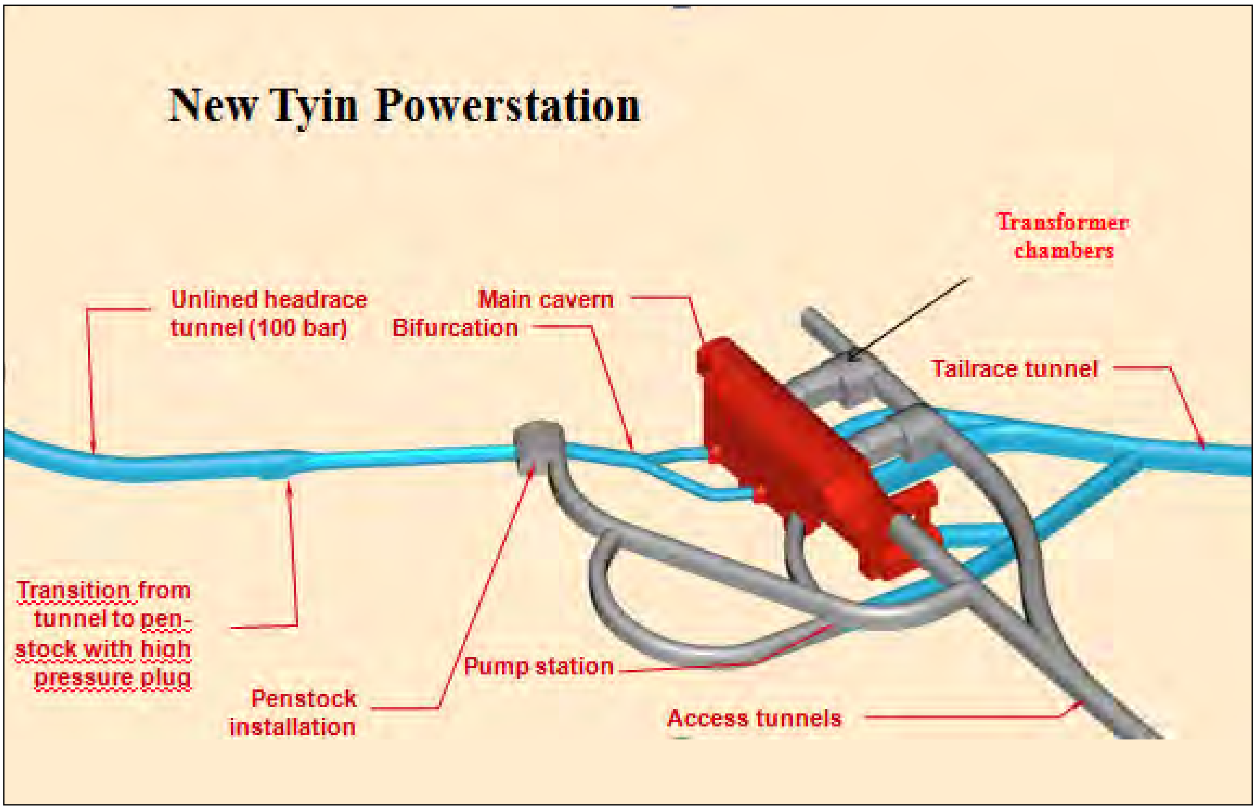


Figure 2- The new powerstation. Penstock installation: area with special section for pipe access and control. Pump station: concerns cooling water supply to nearby industrial facility. Courtesy Norsk Hydro/Knut Helgesen

7 THE NEW TYIN HYDROPOWER PROJECT

History
The Tyin hydropower development started in 1910 during a period with numerous plans and initiatives for industrial development. The lack of funding, sufficient available electric energy in the market and the First World War lead to slow progress. Activities during the twenties were somewhat better and by 1930 the penstock had been installed, however still no need for electricity. The Second World War intensified demand for industrial output and caused ac-tion. By 1944 the first unit was ready for operation. In 1946 further four units had been in-stalled. Around 1990 NVE decided that the sixty year old steel penstocks did not satisfy the safety requirements. The owners were advised accordingly. Undertaken analyses concluded that the upgrading of the existing facilities not to be the best option.

The new plant
The Tyin Power Plant, commissioned 2004, includes 21 km tunnels, intake, outlet, power-house complex, surge shaft and several creek inlets. The water head could be increased with 35 m, energy losses in the system decreased with 63 and the annual output increased with 18 %. The old project produced electricity through the entire construction period. In line with traditional Norwegian hydropower design, the aim was to optimize the location of the under-ground powerhouse complex exploiting the mechanical properties of the rock.

The plug (on elevation ≈ 0)
Early site investigations to establish the actual geology, foliation, stress situation and joints included core drilling with 500+ m long holes. The position of the powerhouse complex was open for modifications to match observations during the initial construction period. During the excavation of the access tunnel hydraulic fracturing and 3-dimensional stress tests were performed. In the end a repositioning of caverns 120 metres towards the tailrace outlet was decided.

Prior to the installation of the penstock in the plug area 4 grout curtains were established. 2 of these with 34 meter long holes and 2 curtains with 17 meter long holes. Bore hole tests showed marginal water loss, hence the planned cement grouting was cancelled and epoxy injection directly performed.

A complex geology requires tests.

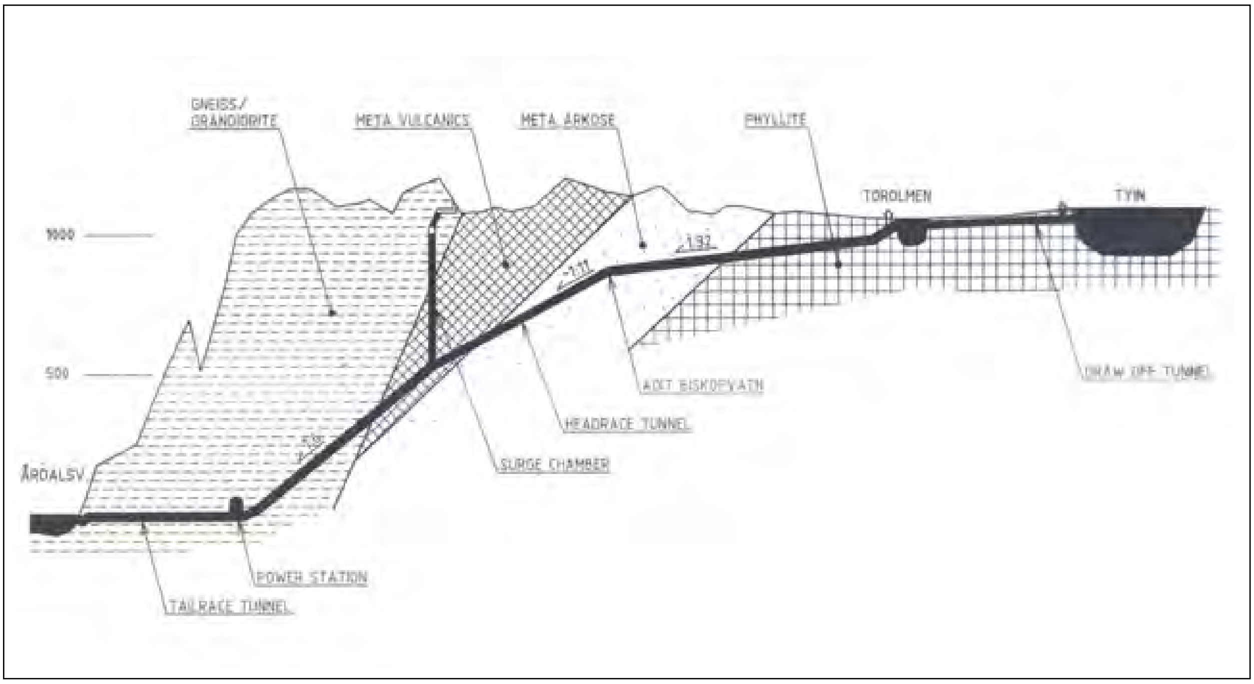


Figure 3 - The geology. The plug described below is installed close to the power station

Contact injection between rock and concrete over the entire length though injection tubes with polyurethane at both ends and epoxy injections between. Contact injection between pen-stock and concrete also through injection tubes. Finally control injections from inside of the penstock in form of 4 curtains with 10 meter holes also by means of epoxy.

Consumed was 1970 kg epoxy for deep injection and approximately 9000 kg for contact and control injec-tion. Leakage control immediately after the first water filling was 0.3 litres per minute.

8 CONCLUSIONS

Analysis and observations from the design, construc-tion and operation of 150 high pressure concrete plugs in Norwegian hydropower projects have shown that the traditional design basis work well. For plugs located in tunnels with cross sections up to 50 m^2 a total plug length between 2 and 5% of the static water head may be recommended.

The final leakage through the plug will to large extent depend on the quality of the concrete and the grouting work. Most of the leakage occurs along the rock to con-crete contact zone and mainly in the roof section. The layout and design of the concrete and the steel lining will in-fluence the plug behaviour and hence the extent of the grouting and construction costs.

In conclusion, the current design, construction and grouting technique of plugs for Norwegian hydropower plants have proven successful for operational pressures of 100 Bar - (1050m wa-ter head).

The current grouting practice may be summarised to:

- Grout curtains for rock grouting - high pressure - cement/microsement.
- Void grouting plug crown area through pipes/hoses. (cement, low pressure).
- Contact grouting construction joints, rock/concrete and concrete/steel. Polyurethane is used at upstream/downstream end to create barriers, and then epoxy is used for the main contact grouting. High pressure at the rock/concrete interface.
- Control holes drilled through the concrete into rock. |Grouting usually with cement, some cases epoxy. High pressure.

Sincere thanks to Johannes Hope for data from Bjølvo respectively to Morten Lund for data from Tyin.

08. ARTIFICIAL GROUND FREEZING TO AID MAINTENANCE OF WATERWAYS

BERGGREN, Anne-Lise

1 INTRODUCTION

GEOFROST had the idea to cut off a water bearing tunnel by freezing, and by this method seal it completely. The idea has been developed through trial and research, and finally used commercially. The system makes it possible to close a water filled tunnel, where there are no upstream gates or other easy ways to cut off the water. In this way refurbishment work of many old power plants may be performed in a safe and cost effective manner.

The ice plug concept is based on two facts: 1) that frozen materials have great strength when temperature is lowered sufficiently, and 2) the sealing may easily be removed after use. Thus the concept is ideal for temporary construction in connection with maintenance and refurbishment works in water filled tunnels of different kinds like hydropower plants, water treatment plants and waste water plants. It is an environmentally friendly method.

The first full scale test was performed in 1989 at the Røssåga waterway in northern Norway, - see Figure 1. In a bypass tunnel between one of the Røssåga lakes and the river down stream, it was possible to carry out a test without stopping the energy production or risking harming a power plant station. In this tunnel, with a cross section of 50 m^2, a 6 m long artificially frozen ice plug was holding back the water reservoir with a water head of 20 m. When the tunnel was emptied, the plug was loaded by more than 1 million kg at on side. This load test lasted for a month. Then the ice plug was removed. (Berggren & Sandvold 1995)

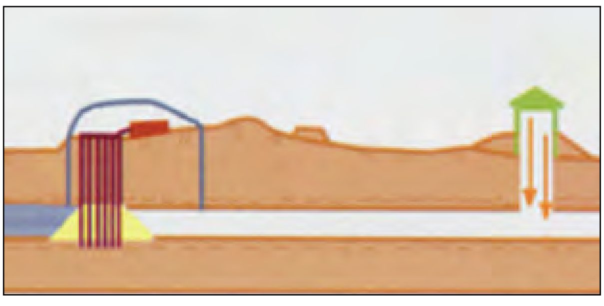


Figure 1 - An ice plug is holding back the water reservoir. Pumping water and gravel fill neutralizes the water movements during initial freezing. Water downstream can then be emptied and work can be done on "dry land".

2 ICE PLUG IN SKODDEBERG HEDRACE TUNNEL

2.1 Achieving the contract and preparations before stopping the hydro power plant

In 2007 Hålogaland Kraft AS planned maintenance works for the Skoddeberg HPP constructed in the 1950s. To be able to carry out these works, the headrace tunnel (approximately 8 m^2) had to be emptied. The gateway in the headrace tunnel was, however, leaking severely. The first approach was to lower the water level in the reservoir, build a coffer dam and get the works done during the winter season when the water flow normally is at its minimum. A crucial question was, however, the safety of this solution, as during the last winter seasons a midwinter flooding had occurred. The water level in the reservoir might then rise by a meter per day, and a reasonable coffer dam might be overrun quite fast.

While searching for other solutions, among many other ideas the ice plug concept appeared. The reason was that one of the involved persons had heard about the test project carried out many years earlier. GEOFROST was contacted and asked about the possibilities and thereafter to tender for the ice plug option. Two different alternatives were proposed, one including brine freezing and one with nitrogen freezing. Even though more expensive, the more rapid nitrogen freezing was chosen in order to minimize the stop in energy production from the hydro power plant.

When planning all activities that had to be coordinated, the date for closing down the hydro power plant was the basis. This date was set based on expected low flow of water combined with minimum reservoir level, so as to minimize the loss of potential power during the stop of the hydro power plant.

Figure 2 - Drilling into the water filled tunnel.

Figure 3 - Freezing pipes ready for installation. Gatehouse in the background.

Before shut down of the hydro power plant all necessary holes had to be drilled and freezing pipes installed. While drilling, it was found that the tunnel was somewhat wider then the theoretical width of 2.5 m. As load would then increase, the design of the ice plug had to be redone, resulting in drilling of some extra holes.

2.2 Neutralizing water flow and currents

In order to be able to create an ice plug, the water must be transformed into ice. The water molecules have to stay close to the heat-removing freezing pipes long enough to let this happen. Both vertical water currents caused by the density differences when temperature change during cooling, and horizontal currents due to the leaking gateway and fissures in the unlined tunnel would prevent this from happening. These water movements are the largest challenges in creating the ice plug. To reduce the currents caused by the temperature gradient and thus reducing the necessary number of freezing pipes to an acceptable amount, gravel was filled in the tunnel. Separate holes on each side of the ice plug area were drilled. To neutralize the longitudinal currents, and thereby also make the gravel stay in place, water was pumped in a bypass pipe, from the upstream side to the downstream side of the ice plug area.

2.3 Freezing and emptying the tunnel

The filling with gravel and the freezing itself had to wait until the hydro power plant had been shut down. In order to generate the ice plug as quickly as possible, liquid nitrogen (LIN) was used for the initial freezing. LIN has a temperature of -197 °C at atmospheric pressure. The nitrogen is delivered by lorries, boils in the freezing pipes and are then let back to the atmosphere. For economical reasons freezing method was changed from nitrogen freezing to brine freezing for the ice plug maintenance. Brine at a temperature between -30 °C and -40 °C was circulating in a closed system, delivering heat to an electrical powered freezing plant.

When the ice plug had achieved the required temperature and thickness: a minimum of 0.8 m at -10 °C, loading could start. To avoid a too rapid loading of the 15 m water head from the reservoir, backpressure was released

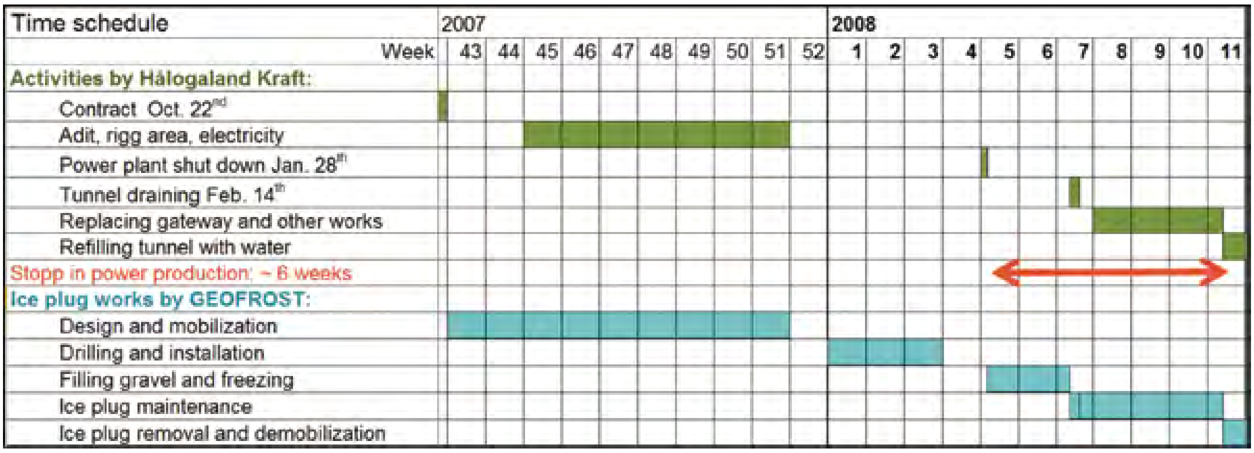


Figure 4 - Timeconsumption for different project activities.

during one day and night. First the water pumping was turned off. The leaks were large enough to thereby lowering the water level downstream in the gateway shaft and the surge chamber. When water surface reached the tunnel level and thus a much larger volume of water was to be removed, a downstream gate was partly opened to speed up the emptying, but not the loading rate.

Besides that the leaking gateway was to be replaced, some rock support work in the surge chamber had to be done, and an extra turbine was to be installed in the hydro power station. Due to the new turbine, a branching of the penstock also had to be built. All these works were coordinated to be performed as quickly as possible as soon as the ice plug was developed and water emptied from the tunnel downstream.

To assure the ice plug was developing as designed, temperature measurements were performed and compared with design calculations. Temperature measurements were continued throughout the maintenance period as well. During loading period deformation measurements were performed by inclinometer. Deformations were insignificant and hardly measurable. They were performed as a quality assurance.

2.3 Time table and economy

The work was carried out during January, February and March 2008. The cold winter months north of the polar circle revealed demanding working conditions. However, works ran smoothly. The extra time allowed for during the preparatory works was thus not used. As shown in the time schedule (Figure 3), the power plant interruption was 6 weeks. Only 2 of these weeks were needed for the establishing and removal of the ice plug.

The cost was approximately 330 000 Euro. The cofferdam solution would have been approximately 3 times as much. In addition weather conditions that winter turned out so that the cofferdam solution would have been impossible to carry through. Heavy rain in late autumn resulted in a full reservoir when it was supposed to be at the lower regulation level.

The greatest benefit for the client however, was the 3 weeks reduced stop of the hydro power production, compared to the preplans.

3 CONCLUSION

GEOFROST has undertaken research work and tested the result in the field. The developed design theories proved to work in full scale. The research project proved that the concept was feasible and this case history has proven the commercial benefit. As an alternative to coffer dams, sealing the intake, emptying the reservoir or run a parallel tunnel, the ice plug method apart for being a safe and environmentally friendly, the method is predictable in time and cost.

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Ringedalsfossen. The waterfall is now reduced to a seldom used spillway for Ringedalsvatn, the lake serving as reservoir for the Oksla power station. Photo:Statkraft.

09. WATERPROOF UNDERGROUND CAVERNS

HANSTVEDT, Alv
REITE, Jørn

INTRODUCTION

The use of the underground, for various purposes, has become more and more interesting over the years. Today we find caverns hosting many facilities from halls for sport activities, car parking, water-treatment plant to low nuclear waste storage. On the surface it is getting more and more crowded and the ground is valuable for development of urban areas. On the assumption that the rock conditions are sufficient for underground construction we can exploit the rock with unlimited possibilities. However, it is of decisive importance that the humidity inside the cavern is taken care of in order to get a dry and controlled environment. The corrosion can, over years, be a challenge in all rock cavern if certain preventive effort is not been carried out.

The critical factor in determining the use of a rock cavern is the control of dripping water and high humidity. Water will occur either as condensation on cold rock surface or as underground water leakage through cracks in the rock. Water veins with large quantities can also create problems. Mechanical, electrical and electronic equipment will over years be destroyed by corrosion in a humid environment. A rock cavern usually has the following basic properties, like constant temperature, complete darkness, and water/humidity problems.

The WG Tunnel Sealing System (WGTS) was invented in1980 and was first installed in a rock cavern for the military in Norway. The requirement was to install a large tent-like storage building (rub-hall) inside a rock cavern. Instead of using the normal structure of pipework inside the building we used the rock ceiling to anchor the rock bolts and hold the PVC fabric in place from the outside.

MATERIAL PROPERTIES AND INSTALLATION

The WGTS is a waterproof system, specially developed to avoid humidity and dripping water in rock caverns, shafts, access tunnels, caverns for storage, etc.

In order to establish humidity control we have to isolate the humidity. The fabric is hung up underneath the rock surface by use of steel wire ropes and 16mm rebar bolts anchored in the cavern ceiling. All fabric joints are

Figures 1 and 2: The picture to the left shows a typical situation in a pipe access tunnel were you have a wet and humid environment that gives corrosion problems to steel pipes and electrical equipment. To the right we see a picture of a complete different result after the installation of WGTS. The humidity is now under control and the lift time for the assets has increased dramatically.

safely closed by hot air welding and pressure tested to make a 100% water proof umbrella system. The WGTS is installed in a way that all water is safely lead down to the drainage system.

The WGTS system is not affected by humidity. The estimated lifetime of WGTS in tunnel and cavern is minimum 50 years. The installation is carried out after the cavern is finished excavated, but can also be completed during rehabilitation of existing tunnels and caverns.

To get the best result, a concrete floor is constructed and a good drainage system is essential for good moisture and humidity control in the final facility. WGTS consists of a PVC fabric, rock bolts, and steel wire rope. Existing and new M&E bolts will be incorporated in the system. WGTS has a special solution to seal around the bolt in order to prevent leakage of water. The system combined with a dehumidifier, gives a complete control of the environment inside the rock cavity, and gives a cost-effective and an energy saving solution.

The fabric is made of fiber re-enforcement PVC at approximate 1,0 mm thickness. The fabric itself is not particular flexible but it's on the other hand very strong product with a tensile test strength of 2600 N/50mm. The WGTS is known for the adaptability and tailor made solution at all required profiles. It can be mounted quickly and to a lower cost then most other solution including shot concrete. The WGTS has no limitations when it comes to adaptability or dimension like length, width or high of the cavern or tunnel.

Figure 3 shows the installation of the WGTS in a rock cavern. From a final product at the far end to the on going installation work, using electrical driven lifts in the front. At the top of the picture we can see the rock bolts and the steel wire rope running between the rock bolts.

REFERENCES

The WGTS fabric is approved by several fire authorities around the World, like the SINTEF NBL in Norway, SITAC in Sweden and the Technical Research Centre of Finland (VTT). It is also tested regarding ISO 9705 and has a Euro classification. The material is self-extinguishing and will never spread or maintain a fire.

The WGTS system has, for more than 30 years, been used with great success in numerous underground facilities like power stations, cavern for low nuclear waste, underground archives, cold and frozen food stores, parking areas, rescue centers, emergency hospitals, oil and gas stores, military facilities etc.

For reference the following geographical location can be mentioned:

- Zimbabwe
- Norway
- Iceland
- Nepal
- Switzerland
- Scotland
- Singapore
- Greenland
- France - Canada
- Sweden
- Finland
- The Faeroe Iceland
- India
- Pakistan
- Italy
- South Korea
- The Philippines
- Spitsbergen

The WG Tunnel Sealing System provides a dry environment inside the cavern, protect your assets, and gives a long lasting and cost-effective solution.

The Tysso Hydropower Project with some 1200 metres head was developed in stages over a long period of time. Construction of the first stage utilizing a head of 410 metres started 1906. After a construction period of 18 months only, the first six units were put into operation. Further development took decades, depending on economy and the power demand. In the end 15 units were in operation. In 1975 plans for new hydropower facilities were adopted. Tysso 1 was closed down in 1989. The old power house is later completely restored, refurbished and reopened for the public as a hydropower museum.
Photo: NVIM

10. RAISE DRILLING OF SHAFTS AND TUNNELS

ØISETH, Trond

INTRODUCTION

Raise Drilling is an often used method for making shafts and small tunnels in Norway. The method is suitable both in hard and soft rock. The flexibility in different angles and diameters is a great advantage compared to conventional shaft excavation (drill and blast) such as:

- Simpler construction and more accurate area for the specific project.
- Very high safety factor for the operators sitting in an operator cabin without any risk for air pollution or rock fall from weak rock zones.
- Limited rock support work.
- Lower construction cost.
- Reduced time consumption.

RAISE BORING PROJECTS FULFILLED IN NORWAY

Already back in 1970 the first raise bore project was fulfilled in Norway. Since then a considerable experience has been gained with this technique. Hundreds of shafts have been drilled up to now.

The raise drill method has fulfilled a lot of hydro-electric power projects with a minimum of damage to the nature. All drilling equipment can be taken apart and lifted to the site with helicopter. No need for road building, just small "needle stick" in the nature.

Raise boring is achieved by drilling a pilot hole with a diameter at 11" or 12 ¼" inches. After completed the pilot hole, the reamer is mounted and the machine pulls and rotate the reamer back, - see Figure 1.

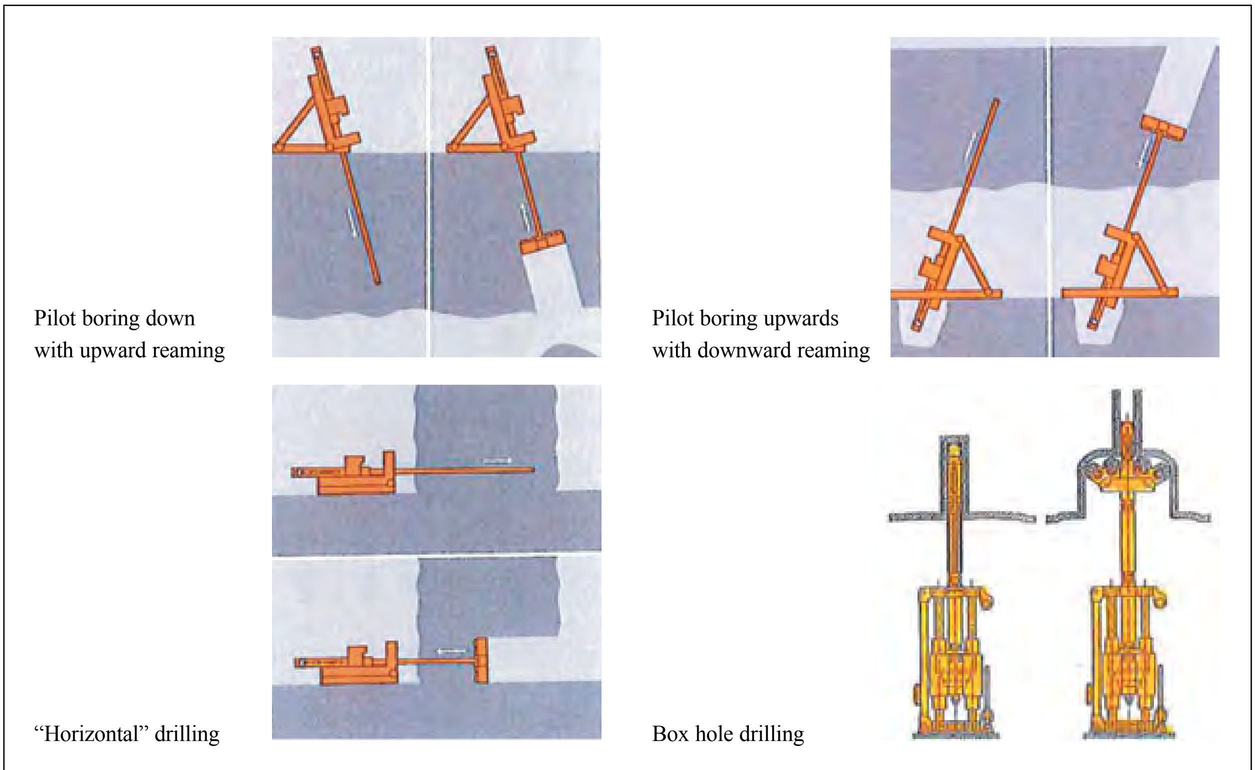


Figure 1 – Examples showing different use of the raise drilling

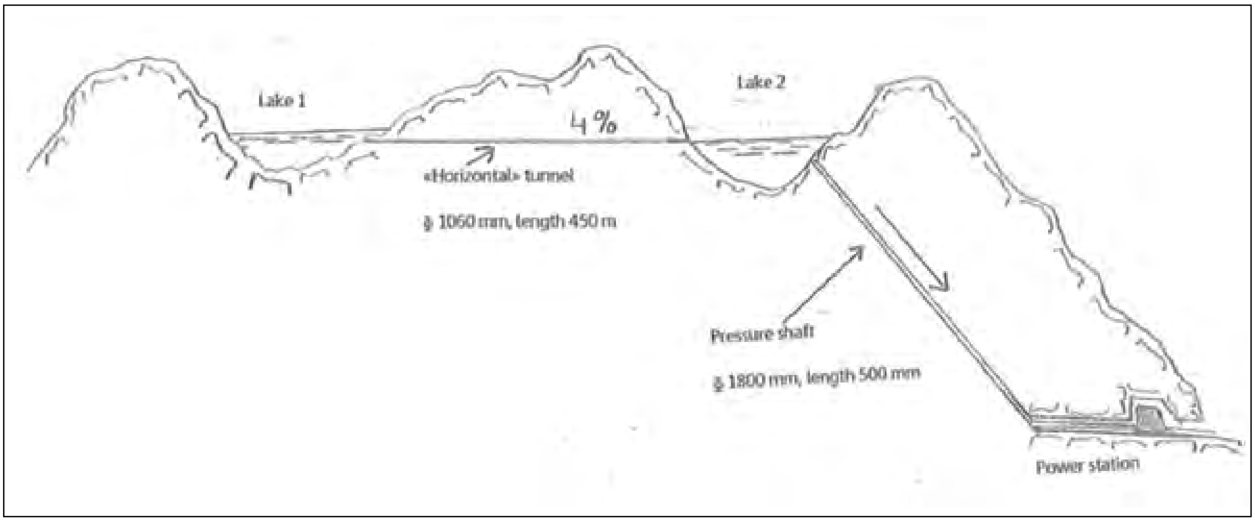


Figure 2 - Hydroelectric power project where it is possible to use the raise drill equipment

Figure 2 shows a typical small Norwegian hydroelectric power project where it is possible to use the raise drill equipment both for horizontal and angle drilled holes. All transport by helicopter.

The method has been used in different types of projects in Norway, such as:

- Surge and pressure shafts in hydro power projects.
- Ventilation and rescue shafts for highway/railway tunnels and underground car parking.
- Service and ventilation shafts for mining industry.
- Horizontal and low angled raise boring for water and sewage transport.
- Filling and pumping shafts for underground oil storage.
- Pumping/dewatering shafts for subsea highway tunnels.

Figure 3 - Gryto Hydropowerplant, Hardanger, Length = 500 m. Diameter ϕ 770 mm, inclination 45˚.

TECHNICAL SPECIFICATIONS, CAPACITY AND EXECUTION

The rig up time for raise drilling compared to conventional shaft excavation is much shorter and needs a limited area for the operation. The normal pilot hole size varies from 9 7/8" (250 mm) to 12 ¼" (311 mm). The penetration rate depends on rock conditions and varies between 1 and 3 m per hour.

The reaming capacity is normally between 0,5 and 2,0 m per hour in hard rock. Compared to the old method, drilling and blasting, the raise drill method can save the project many hours. The safety aspect for the operators is much better with a minimum of risk for accidents. As an average time a 400 m long shaft, ϕ 1060 mm, can be fulfilled in three months included rip up and demobilization.

A pilot hole up to 700 m and reaming the same length is possible with diameters less than 2 m in homogenous solid rock. Also larger diameters can be drilled with increasing the machine capacity (larger equipment) and the drill string diameter.

Shaft lengths more than 600 m has been fulfilled in Norway, Bjølvo Hydro Electric Power Plant, with a 2,1 m reamer head.

The raise drill equipment can be operated by one man, but due to safety regulations (procedure) it is normally two operators on the drill rig. In addition to these two shovels with one man take care of the muck from the shaft. But it is important that no one works under the reaming head while the drilling operation is going on.

Figure 4 - Hesthaugen, Sandvika. Horizontal small tunnel, ϕ 1850 mm, length 295 m, inclination = 0,07 % for leading surface/ waste water away from the railway lines.

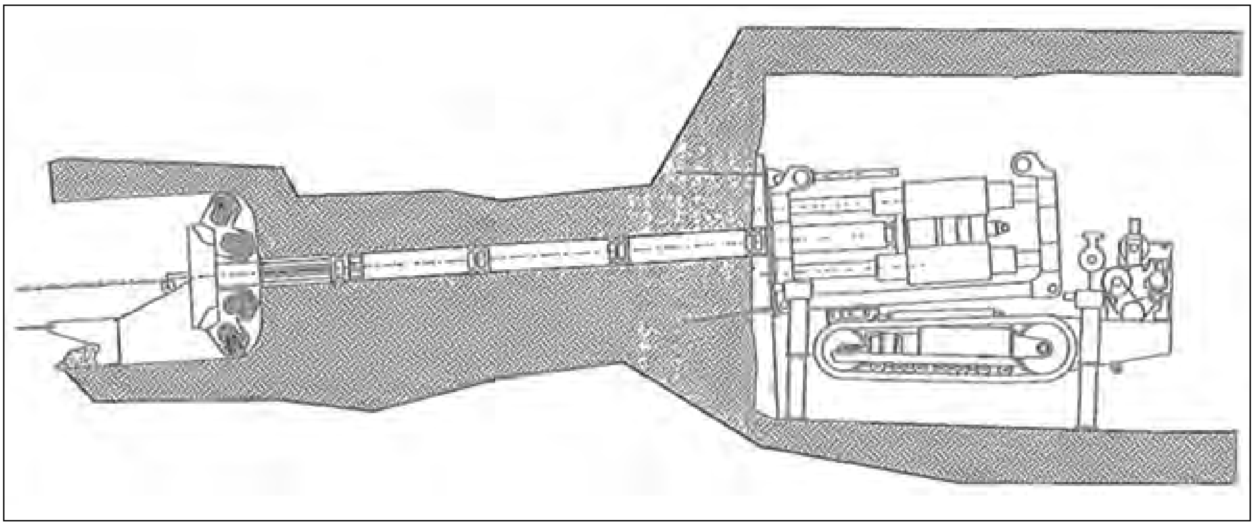


Fig. 5 Sketch of Low Angled Raise Boring.

All the raise drill equipment is compact and can be mobilized and ready to drill in less than one week. If the access to the drill site is situated up in the mountains without a road connection, the equipment can be lifted by helicopter.

Most of the raise drill equipment in Norway can be dismantled in smaller parts with a maximum weight of approximately 3-4 tons. Table 1 gives some example of raise drill shafts the last two years in Norway.

PILOT HOLE DEVIATION

The raise drill string has different diameters depending on reamer head diameter and hole length. Behind the three cone pilot bit is mounted "rod stabilizers" with hard faced steelribs, diameter 2-3 mm less than the pilot bit diameter. Numbers of stabilizers depend on inclination and rock conditions, but normally three to four, with a total length of 4,5 – 6 m. Deviation of the pilot hole is most often less than 2 % of the drilling length. Normally increases the deviation with the pilot hole length. When only small deviations can be tolerated, the drilling accuracy can be improved by increasing or reducing the rotation speed, thrust and the numbers of stabilizers.

It is also possible to drill with a turbine/motor solution which can be directed to the target with high accuracy. This, however, increases the pilot hole cost. Today the pilot hole deviation can be measured constantly with different methods. The most advanced instrument is the Gyro Smart type, which gives a very accurate position of the drill bit. Among many different factors to reach the target, the most important is to have good experienced operators.

COSTS

The cost of raise drilled shafts and Low Angled micro tunnels are depending of:

- Length and diameter of the shaft
- Rock types
- Inclination
- Location of site

The cost per meter will increase when the inclination is near to horizontal because it will need some extra efforts to pump, flash or use a scraper system to remove the muck. However, it is still a cost effective solution for small tunnels compared with the drill and blast method. Also a strict accuracy for the target will increase the costs.

Blind hole drilling is more costly, because the pilot hole must be drilled first, and when the reaming starts, after pulling back the drill string, the string has to be stabilized with non-rotating stabilizers during the reaming process.

Raisedrilled pressure shafts for hydropower projects 2010 – 2012				
Project	Length m	Diameter mm	Inclination	Rock type
Kveaså HPP,	481	1060	43˚	Granite
Holsbru HPP,	5 x~60	770 2400	40˚ - 90˚	Syenite/Quartzite
Søberg HPP,	316		20˚	Granite
Hovland HPP	115	1060	32˚	Micaschist/Gneiss
Jøssang HPP	290	1060	45˚	Gneiss
Folkedal HPP	330	1800	66˚	Gneiss
Eldrevatn HPP	50+	1400 + 1400	3˚	Granite
Haukeli HPP	312	1600	45˚	Gneiss/Granite
Mygland HPP	232	1600	4˚	Granite
Vemork HPP	2 x 76	3100	31˚	Metarhyolite
Suldal 2 HPP	210	4500	90˚	Granite
Sauda HPP	437	1060	3,5 - 4˚	Granite/Metabasalt
Rendalen HPP	104	1540	90˚	Sandstone

Table 1 - Some examples of reamed shafts completed in Norway 2010 – 2012.

11. DIRECTIONAL CONTROLLED DRILLING OF SHAFTS

TONSTAD, Askjell

INTRODUCTION
Based on new patented solutions, the Norwegian Hard Rock Drilling AS – Norhard - now offers solutions for assignments all over Norway. Drilling of Pressure Tunnels for Small Hydro Power Plants has been the main focus area, but technology and equipment is also well adapted for other infrastructure solutions and will make new kind of solutions possible within many areas. Business idea for the company established in 2007:

- In House Technology Development and Equipment Production. Full service provider for environmental friendly and cost efficient drilling solutions onshore.

TRADITIONAL SOLUTIONS
Traditional technology development for full profile drilling of micro tunnels has been driven by the needs from the mining industry. Raise drilling in combination with drill and blast has been the mostly used method for building of the bigger high head small hydro power plants waterways. Drill and blast of pressure shafts have become a too expensive and a too risky method for use when building small hydro power plants, and even also for building of larger ones.

LIMITATIONS
Established technology for drilling of micro tunnels is based on concepts with extension pipes, machinery for rotation in open air, rotating drill string and limited communication between equipment in the hole and equipment/staff outside. Limitations for established on-shore equipment are mainly poor possibilities for directional control and limitations in length.

NEW TECHNOLOGY
Norwegian Hard Rock Drilling AS – Norhard AS, has focused on development of a complete new technology for full profile drilling of tunnels. The basic idea when starting the development was to develop cost efficient and environmental solutions for drilling of pressure tunnels for small hydro power plants. By the concession authorities in Norway – NVE (Norwegian Water Resources and Energy Directorate) - it was also expressed that new environmental friendly solutions would be highly appreciated when preparing concessions making it possible to utilize the huge potential for small hydro power plants and more renewable energy. Main characteristics for the first two machines now in operation are:

- Diameter 700 mm in one operation
- Fully electric operated
- Long distance drilling
- High capacity communication via optical fiber
- None rotating drill string
- Continuous direction- and position control
- Trajectory steered
- Online control and surveillance of drilling process

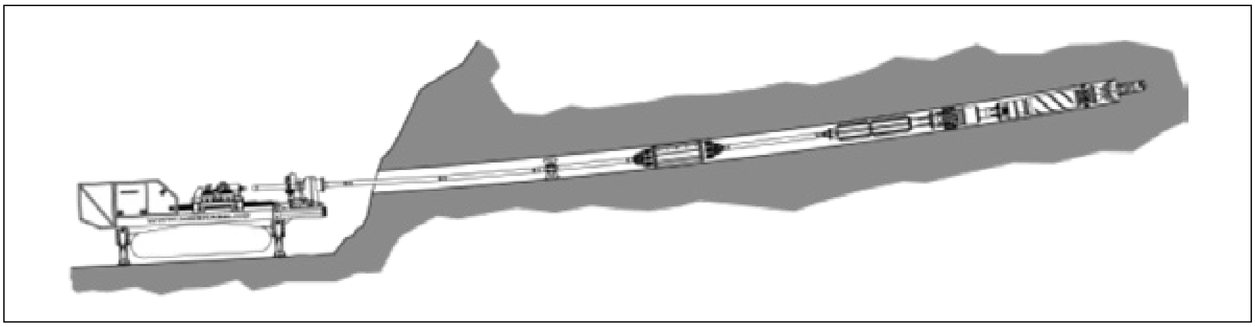


Figure 1: NDL700HR for Ø700mm

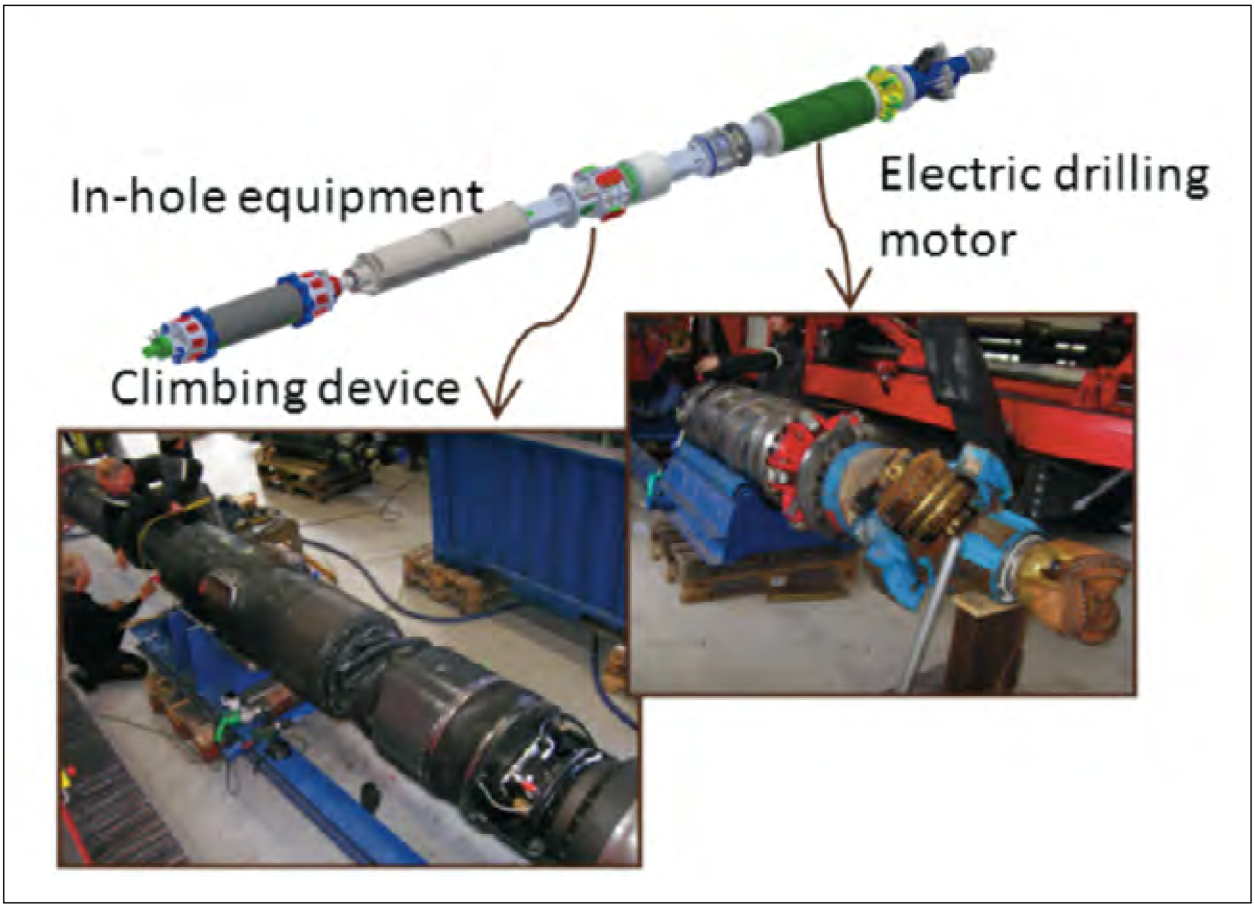


Figure 2: NDL700HR - equipment travelling in rock

- Environmentally friendly solutions
  - Drilling from bottom – No need for helicopter or road at the top end
  - Noiseless
  - No polluting waste
  - Low energy demand

FOUR HYDRO POWER PROJECTS IN OPERATION

Four small hydro power plants in Norway are now in operation with pressure shafts drilled with Norhard technology. Three of these pressure shafts are drilled over lengths in the range of 700 m to 750 m. The shafts are drilled in curves adapted to terrain and location of intake and also supplied with internal lining if necessary due to geological conditions.

Tunnels can be drilled in curves following a circle with radius down to approximately 200 m. For drilling of pressure tunnels also the needs of lining must be taken in account to prevent hydrojacking or in case of very bad rock conditions where there is a risk of block outfall in the shaft. Given ideal topographical conditions and good geological conditions, pressure tunnels can be drilled close to horizontal from the bottom, in order to limit the extent of lining, and to have sufficient rock cover for unlined solutions over the upper part of the tunnel.

The extent of lining is depending on the rock cover along the tunnel, compared to the pressure and actual rock conditions. So far solutions with welded steel pipes have been used. Research works are ongoing also to utilize other materials and methods for lining. In some cases it should be possible and advantageous to utilize flexible materials and take advantage of possible support in strength from surrounding rock. Technologies for such solutions are available within other areas, but practical solutions and theoretical documentation needs to be investigated further before such solutions eventually can be used in pressure tunnels for hydro power plants.

The penstock for Eitro Hydro Power Plant was finished September 2012. The penstock consists of a full profile drilled tunnel of 750 m in length over a difference in height of 430 m starting at an angle of 22 degrees from the horizontal in the bottom and ending up at an angle of

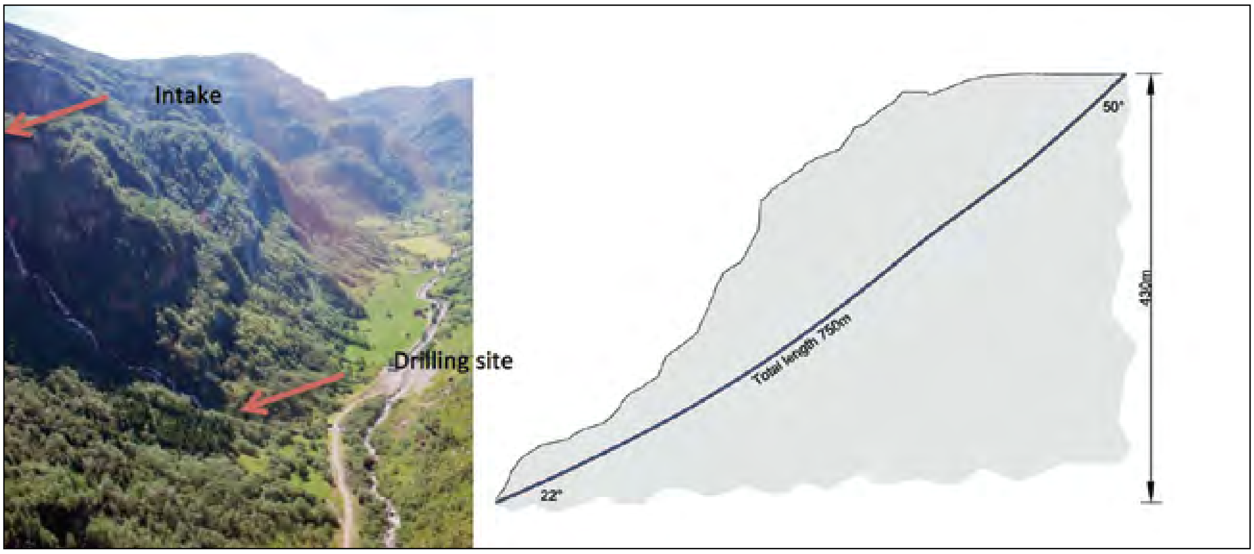


Figure 3: Eitro Power Plant

50 degrees at the top. The drilled tunnel was after drilling lined with steel pipe over its full length. Ø610 mm and 12 m long steel pipes were welded and successively pushed from the bottom end by the use of equipment from Norhard designed for such purposes.

The 680 m long tunnel for Kvangreelva Powerplant is drilled with a relatively horizontal length at an angle down to a few degrees in the bottom before bending up and ending up at an angle of 57 degrees at the top. The tunnel is supplied with steel lining over a length of 325 m at the horizontal part in the bottom. The inner part of the lining is supplied with a plug for fixing and sealing between the lining and the rock.

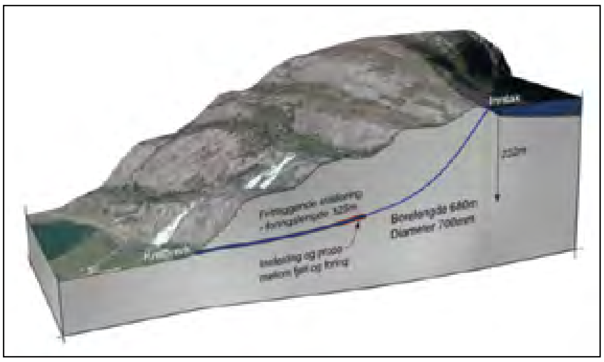


Figure 4: Kvangreelva Power Plant

15 TO 40 M DAILY ADVANCE RATE

By doing these projects, the Norhard technology has proven its capability to drill over long distances and to control the steering and positioning while drilling. For all the projects, the point of breakthrough for the tunnels has been within 1 - 2 m diameter from the target. Rate of penetration is depending on the quality of rock. For the projects delivered so far the average performance has been in the range of 15 m to 40 m a day when working on shift with two operators for operating the machine. The technology is capable of higher efficiency and work is ongoing also to adjust the equipment for better performances. Best performance is obtained when drilling in phyllite and other soft rocks. The lowest rate of penetration and also the highest rate of wear and tear on the drill bit are experienced when drilling in granite, gneiss and breccia. So far, limited resources have been available for analysis of performance data, but the evaluation of data done so far indicates considerably lower rates for wear and tear on drill bits when drilling with Norhard technology compared to other conventional technologies. As part of the preparations for drilling a project in breccia a prognoses based on available international statistics was made by NTNU. Based on the results an improvement by a factor of four was experienced for expected lifetime of drill bit. It is likely to believe that the capability of controlling the process based on readings of vibrations, weight on bit etc. and the capability to be able to adjust the admissions combined with the fact that the climbing device is establishing the drill bit when drilling, gives such important bonuses.

What also is experienced is that when drilling in rock with changes in quality and structures, the directional performance is influenced. The possibility of having on line follow up of position and to be able to adjust the steering while drilling is of great importance in order to keep a stable course and to follow up a planned trajectory for the tunnel.

Figure 5: NDL700HR in operation at Muiodejohka Power plant

CHALLENGES AND FURTHER DEVELOPMENT

Some of the main challenges during the processes needed to establish commercial operation of a complete new and ambitious drilling platform have been:

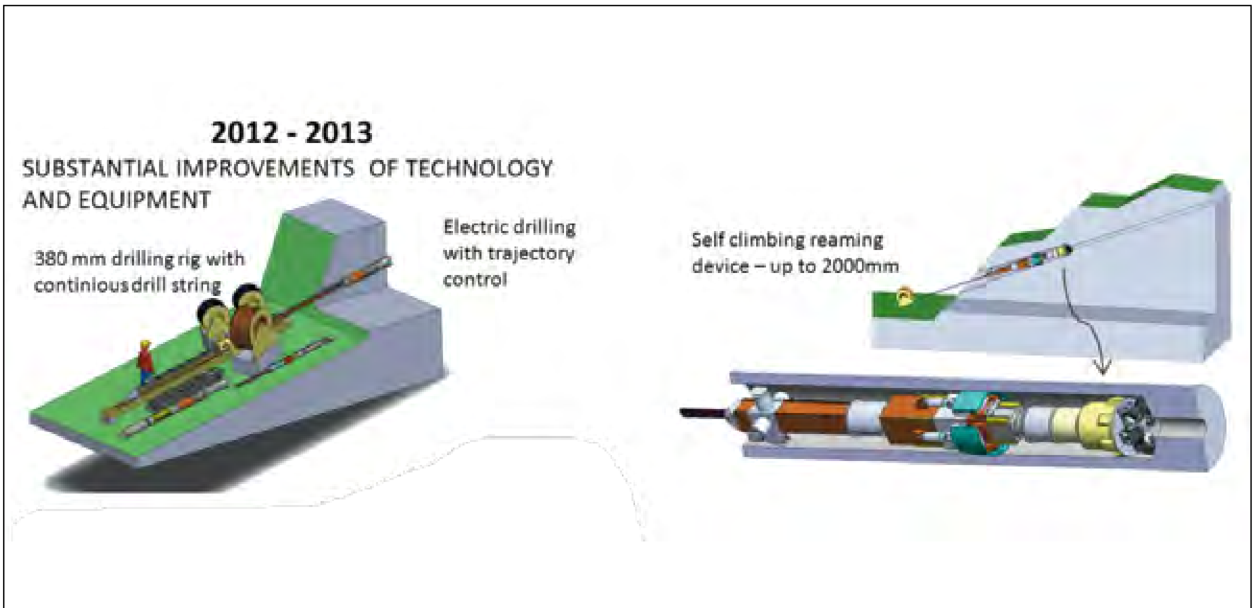
- To develop a strong mechanical design of the critical parts of the system within limited space.
- To control the orientation in various quality of rock
- To establish sufficient reliability of critical components and auxiliary systems for remotely operated equipment travelling underground
- To handle cuttings when drilling on slopes close to horizontal.
- To supply penstocks for high head hydro power plants it has also been necessary to develop, methods and technology for tunnel lining.

FUTURE DEVELOPMENT

Gained experience shows that drilling with existing equipment can be done over distances up to 1 km with upward inclination at a minimum of approximately 4 degrees , and with Ø700mm diameter. Further development is ongoing to modify equipment and technology in order to reach lengths up to 2 km. Equipment will be prepared for horizontal and downward drilling and adapted for all kinds of on-shore infrastructure applications. Also for geothermal wells, oil and gas applications research and development activities are ongoing to utilize and take advantage of the established technology.

Ongoing activities are focused on:

- To increase possible drilling distance up to several km
- Diameter solutions in steps up to several meters



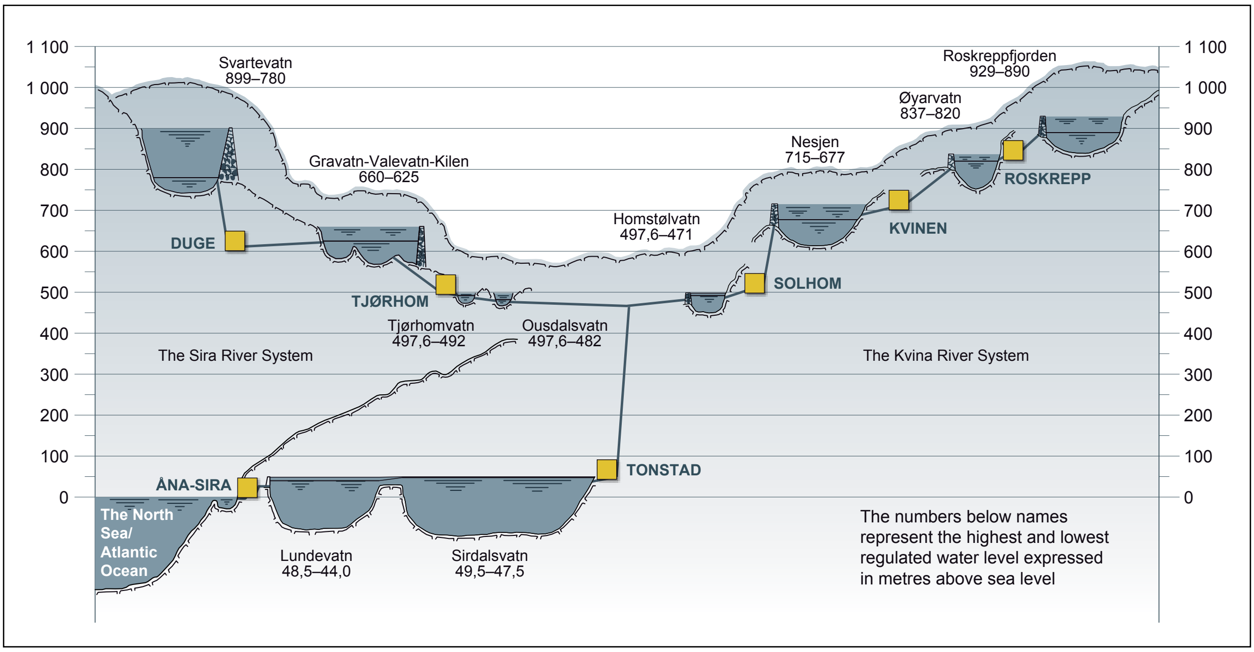
6 Equipment under construction - NDL380HR and self climbing reamer NR1200HR

- Horizontal drilling
- Declined drilling
- New drill string solutions included flexible coil
- Light weight equipment based on modules for making drilling solutions in difficult terrain available without need of road access. Modules with weights limited to 1000 kg are aimed.
- Further development of systems with flexible continuous drill string and continuous circulation. When supplying drill fluids (water or mud) inside detached drill pipes the circulation is broken when pipes are connected/disconnected. In new solutions mud and/or water will be supplied over hoses attached to the drill pipe in the same way as cables for electric power supply and communication.
- High capacity communication and solutions to integrate seismological equipment for geological survey while drilling.
- Adapt the technology for geothermal wells and for the Oil and Gas industry for on-shore as well as for offshore applications.

FUTURE EQUIPMENT FOR HYDROPOWER PROJECTS

To meet the coming challenges and opportunities, a basic change is done in the new machinery concept now under construction. Future equipment for onshore drilling operations is based on a method for drilling where tunnels with diameter exceeding 380 mm will be drilled in two operations, and with two different machines. In a first operation a tunnel of Ø380mm will be drilled. Reaming to required diameter will be done in a second and separate operation. Many of the challenging parts of a drilling operation, like orientation and positioning control will be done when drilling the pilot hole. The main part of the geological risk in a project will also be exposed after drilling the pilot hole. Reamers will be built for several diameter steps, but all equipment needed can be based on the technology already developed. The first reamer now under construction is built for reaming to Ø1200mm.

The main on-shore marked for Norhard is considered to be drilling operations in the diameter range up to approximately Ø2000 mm and within lengths up to approximately 2 km. For for high voltage infrastructure solutions etc. it is considered to be a practical solution to connect several tunnels from one point to another. Nevertheless, for tunnels with steep inclinations, as for example penstocks for bigger hydro power plants, also reamers for diameters up to several meters will be considered.



The Norwegian hydropower industry is based on numerous powerstations, most with moderate capacity. One of the larger developments is demonstrated above. Sira and Kvina are two rivers draining valleys with the same names. The hydropower potential, initially owned by several municipalities eventually agreed to join forces, has been developed through the Sira-Kvina Power Company. Seven powerstations, long tunnels, dams – mainly rockfill - create large capacity reservoirs, hence reliable production also during years with low precipitation. Photo: Sira-Kvina kraftselskap

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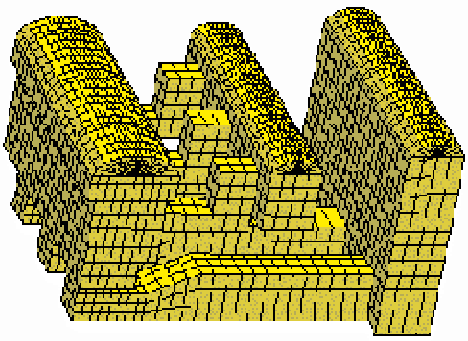

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12. SMALL HYDROPOWER IN NORWAY

LØVÅS, Mona
NAMDAL, Arne

1 TUNNEL PUBLICATION

1.1 About Småkraft

Småkraft AS is a development and production company that was established in 2002. It is owned by four companies; Skagerak Energi AS, Agder Energi AS, BKK Produksjon AS and Statkraft AS. Småkraft AS is headquartered in Bergen and builds hydropower plants (hpp) all across the country.

Småkraft is responsible for all aspects of the projects, including design, financing, construction and operation. We normally sign a 40 year agreement on a lease with the local landowners. Once this is done, we prepare an application for license which is sent NVE where we expect a process period of 5-7 years. When license is granted we estimate about one year of preparation in which all conditions up to investment are mapped, including method, time, cost and quality. Subsequently the investment decision it's time for the approximately 2 years long construction phase. After completion, Småkraft is responsible for plant operation for a minimum of 40 years.

Småkraft has by now put 34 power plants into operation, where 9 have tunnel, shaft or both. Five of these are further described below. We have also 15 new tunnel/ shaft projects with licenses under planning to be built during the next 4-6 years.

The following projects are further described in this article:

Project	County	GWh	MW	Head	Construction Start	Construction Finished	Project manager
Steinsvik	Møre og Romsdal	35,7	8,2	694	2006	2009	Bård Aspen
Rasdalen	Hordaland	17,9	4,3	280	2008	2009	Otto Løkkebø
Kveaså	Aust-Agder	12,8	5,3	370	2010	2011	Mona Løvås
Dokkelva	Møre og Romsdal	18,5	5,5	140	2010	2012	Trygve Matthiessen
Muoidejohka	Nordland	18,2	5,5	481	2011	2013	Bård Storlid Kvinge

2 STEINSVIK

2.1 The plant in general

Allowance date:	May 5th 2006
Date of construction start:	Oct. 15th 2006
Startup date:	Jan. 29th 2009
Pipe / waterway:	ø600 and 700mm ductile cast iron pipe. 1220m, 400m dug down in tunnel
Tunnel	cross section 18m², length 500m, elevation 1:7
Shaft:	diameter 1060mm, length 500m, elevation 520

Distance	Type	Length[m]	Diameter[m]
Pipeline entrance-kt. 330	Profile drilled shaft	500	1,06
Kt.330-323	Tunnel	52	A=12,3m^2
Kt.323-249	ductile cast iron pipe K12	453	0,6
Kt.249-210	ductile cast iron pipe K13	90	0,6
Kt.210-157	ductile cast iron pipe K14	102	0,6
Kt.157-118	ductile cast iron pipe K15	150	0,6
Kt.118-56	ductile cast iron pipe K17	192	0,7
Kt.56 - turbine inlet	ductile cast iron pipe K18	288	0,7

2.2 Feasibility studies and design
It was performed geotechnical inspection of the penstock route and tunnel portal. The conclusion was that the pipeline and portal location was imprudent to carry out following the original design, because of risk of landslides and unstable soil conditions. It was decided to move the portal further north and slightly lower than originally designed. The portal and pipeline was then moved away from the landslide and unstable areas. The impact was, however, extended tunnel and shaft, but slightly shorter road and penstock. Penstock, tunnel and shaft had to be redesigned.

2.3 Tunnel execution / deviations
Construction road was quickly built up to the tunnel portal so tunneling could begin.

Tunnel work began in early November 2006. The first two months had poor progress, 15 m per week. In addition, the direction and elevation was wrong. The directional error proved to be due to a misunderstanding between the consultant and the contractor (oral communication).

It came to an agreement and the contractor made necessary adjustments to rectify the tunnel. He also hired expertise to assist the rest of the tunneling, which was performed adequately. The rock quality was good and only a dozen rock bolts were used to secure the tunnel. The tunnel was completed April / May 2007.

2.4 Shaft - planning and operation
The contract for drilling of 500 m shaft was signed on the basis of a frame agreement with the contractor. There were several delays and the startup date was moved to June 12th 2007. Foundation for the drilling rig and housing barracks should have been ready by then. For various reasons the startup was delayed an additional week. Småkraft was responsible for the interfaces between the contracts. Pilot drilling of the shaft went smoothly, but the pilot did not hit the tunnel. Location was calculated based on the borehole log. However, after several days of searching it turned out that the pilot hole had ended up on the opposite side of the tunnel compared to calculations. Later it is shown that these problems were due to errors in launching the pilot combined with improper logging. The extra tunnel work and delays resulted in significant additional costs. The reaming was completed without any more problems and the contractor was dismantled by Oct. 8th 2007.

2.5 Plug in tunnel
Placement of concrete plug in the tunnel was determined after split tests performed by SINTEF. It is located in an area with good rock quality, nevertheless, deep injection as well as contact injection was performed. Upstream the concrete plug it was established a sand trap by casting a 1.5 m high barrier in the tunnel. 600mm inlet pipe and a 150mm drain pipe with a blind flange go through the plug, and can only be opened with an emptied shaft.

There is some leakage through the plug, most of it in the transition between concrete and rock. The leakage is of no economic significance, it is also decreasing.

2.6 Operating Experience
For emergency shut-down purposes it is installed a guard valve just downstream the concrete plug. The guard valve and associated equipment are sensitive to corrosion and began to rust in the corrosive environment inside the tunnel.

We decided to add membrane to the tunnel and dehumidification to establish a valve chamber to protect the equipment against humidity. The solution is from your point of view adequate.

3 RASDALEN

3.1 The plant in general 4.4 Shaft - planning

Allowance date:	Mar. 29th 2007
Date of construction start:	May 5th 2008
Startup date:	Oct. 15th 2009
Pipe / waterway:	DN900 ductile cast iron pipe. 780m dug down
Shaft:	diameter 1060mm, length 305m, elevation 1:4,8, 50 m DN800 GRP

Rasdalen hpp. is located in Voss, Hordaland and has a gross head of 286m.

3.2 Feasibility studies and design

It was considered both traditional tunnel and pressure shaft with pilot and reaming. Terrain and elevation are suitable for both options. Boreholes were chosen from an overall evaluation of technology, finance and risk. Borehole acquire significantly smaller cross section (0.8 m^2 versus $20m^2$), which reduces the mass transportation and disposal of tunnel masses. It also offers significantly lower concrete consumption to mention some criteria.

Geologist was used for evaluation of the rock quality in the portal areas, expected location of the plug and lining length in the downstream end.

3.3 Shaft - planning and operation

The pilots drilling initiating point and foundation for the drill rig was placed and secured 20 m downstream the intake. The shaft contractor installed the pilot rig to drill on the foundation Småkraft was responsible for. Both parties expected that the other were going to correct for inaccuracy in this setting. The contract was reviewed and we stated that this was Småkraft's responsibility.

The water pressure in the pilot pipe was regularly checked, and the calculation of any vertical deviation was made. Horizontal deviations cannot be measured in this manner, and by 60% bore length it was decided to take up the drill string to log horizontal deviation with gyro. The measurements showed a smaller horizontal deviation and we decided to continue even if the result indicated that the hole would exit slightly more left then intended.

We retained water pressure on the pilot until a few meters before impact. This suggested good rock quality and limited lining length, which now could be calculated. The point of impact, however, was of by 14m. The initial direction and the curvature deviation (rotation) had the same direction of deviation. This variance accelerated over the last 40% of the borehole. It was later determined that half of the deviation was a result of incorrect starting direction, and the other half by curvature deviation. The vertical deviation was only 1.5 m.

Before the pilot was drilled, it was prepared an exit spot in the center of an old rock fall (scree). When the pilot showed up 14m from expected, we chose to enter from the cutting with a 10m long collaring towards the hole. This way we could avoid extensive work in the rock scree.

The pilot jammed in the scree and lost flush water. Contractor kept on for several days to pull the pilot drill string back out.

The drill string broke under reaming, which caused some delays. Otherwise, the reaming went according to plan.

3.4 Plug in shaft

The shaft diameter was about one meter. This made it possible to manually seal a clay zone. This was done by first removing loose clay, secondly place arched reinforcement to fit throughout the circle, third a layer of mortar and chicken wire and finally brushed.

A few meters upstream, where the plug was supposed to be, a 2m long and 40cm deep sand trap was blown out. The sand trap was cleaned and the shaft was then rinsed to get rid of masses after reaming and blasting. A curved steel plate was made as a cover on the lower part of the sand trap for capturing stone. See picture below.

The lower 45m of the 305m long shaft was lined with GRP pipes DN800. On this section was 2" x 4" lumber used as skid for the pipe installation, and a block anchored further up. Thereto a drain pipe with intent to avoid wading during assembly / casting.

Then lining pipes was pulled in one by one a bit past the final position. Installation and refilling with concrete took place in stages from the bottom up. The concrete

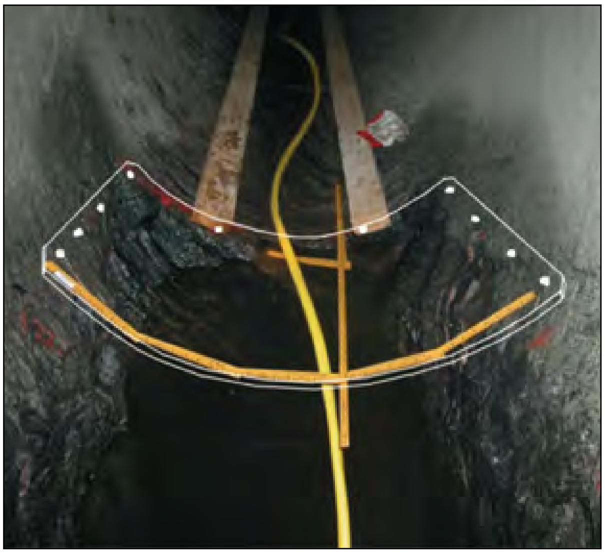

Figure 1 Sand trap and plate upstream plug

Figure 2 Surface

Figure 3 Portal before, Kveså

Figure 4 Portal after, Kveså

Figure 5 Under construction, Dokkelva

Figure 6 Portal complete, Dokkelva

was pumped into tubes placed inside the pipe and then led back between the pipe and rock.

Due to the low injection pressure pipe collapse was considered unlikely, thus was no measures in that matter performed (60m water pressure, 7-8 bar injection pressure). At higher injection pressures measures should be considered.

3.5 Operating Experience

There is some leakage in the bore hole, but this has over time declined. Sand trap seems to work as intended.

4 KVEASÅ

4.1 The plant in general

Allowance date:	May 5th 2006
Date of construction start:	Apr. 19th 2007
Startup date:	Nov. 2011
Pipe / waterway:	DN800 ductile cast iron pipe in the tunnel. Ø1200 ulined shaft. GRP DN800 up stream shaft.
Tunnel	cross section 18m^2, length 408m, elevation 1°
Shaft:	diameter 1200mm, length 339m, elevation 44°

4.2 Feasibility studies and design
There was done evaluation by geologist who recommended the tunnel and shaft geometry, as well as the portal's location. Because of the portal's minimal cover, plus the fact that the old postal road passes just above the portal, it had to be secured.

4.3 Tunnel execution / deviations
The tunnel work started with a cutting which also would form the future facade towards the outside of the power station. Because of the minimal cover, a relatively strong portal was necessary.

A lot of safety measures were not required, shotcrete was only necessary on a minor section. There was a smaller fault zone in the tunnel that had to be secured.

The tunneling took place between April and August 2010.

4.4 Shaft - planning and operation
The contract for the drilling of 339 m shaft was signed on the basis of a framework agreement with the contractor. Pilot drilling of the shaft went smoothly. Logging of the borehole was not performed during drilling, and series of detonations had to be done in the far end of the tunnel before the pilot was found.

Reaming of the pilot hole went without problems and the contractor was finished dismantling by the fall of 2010.

4.5 Plug and pipes in tunnel
Placement of concrete plug in the tunnel was determined after a split test was done.

There was good quality rock in the concrete plug area, nevertheless, deep injection as well as contact injection was performed.

There is some leakage through the plug, but it is of no significance and is also decreasing.

800mm cast iron pipes were laid on prefab concrete foundation in the tunnel. Evaluation has shown that placing the pipes in gravel is advantageous, both in terms of construction and later maintenance of the tunnel.

5 DOKKELVA

5.1 The plant in general

Allowance date:	Feb. 2008
Date of construction start:	Feb. 2010
Startup date:	Mar. 2012
Pipe / waterway:	800m tunnel, hence 150 m DN1200 ductile cast iron pipe and 650m penstock. DN1200 ulined shaft. GRP DN120 up stream tunnel

Dokkelva hpp. is located in Eresfjord in Nesset, Møre og Romsdal, exploiting a precipitation area of 41km^2. Gross head 123 m.

5.2 Feasibility studies and design
Tunnel design was carried out by consultants and geologist. The project was initially planned with tunnel and shaft, but Småkraft concluded that it for financial and implementation purposes was beneficial to have tunnel all the way.

5.3 Tunnel execution / deviations
The cross section is about 22 m².
The carving was “given by nature,” so the tunneling began right on the hillside.

It was first tunneled with a gentle elevation for about 150 m where there was performed a split test in terms of placing the plug. The split test gave satisfying results, and the tunnel continued somewhat steeper, at an elevation of 1:5,5.

The rock quality was very good so the need for securing was almost non-existent.

5.4 Plug in tunnel
It was constructed two plugs in the tunnel. One, 7 m long plug, 150 m from the portal and one, 4 m long plug, in the upstream end where the pipe enters the tunnel. Here it was performed deep injection as well as contact injection.

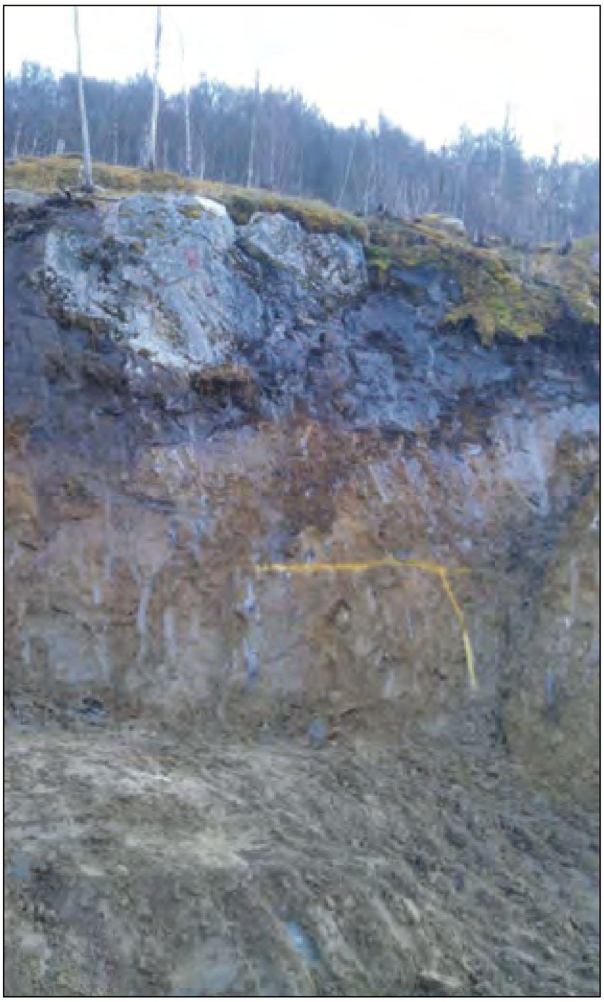

Figure 7 Potal before tunneling

6 MUOIDEJOHKA KRAFVERK

6.1 The plant in general

Allowance date:	May 5th 2006
Date of construction start:	Apr. 19th 2007
Startup date:	Nov. 2011
Pipe / waterway:	DN800 ductile cast iron pipe in the tunnel. Ø1200 ulined shaft. GRP DN800 up stream shaft.
Tunnel	cross section 18m², length 408m, elevation 1°
Shaft:	diameter 1200mm, length 339m, elevation 44°

6.2 Feasibility studies and design
Geologist evaluated the area and recommended tunnel and shaft geometry.

6.3 Tunnel execution / deviations
Tunnel work started with spiling and careful detonations to keep the rock contour at the portal. The first 10 m of the tunnel performed by split detonations of the two cross section halves. The next 10 m was tunneled as one cross section, but blasted with half hole depths. The remaining tunnel was continued the same, only with full hole depths. It was a difficult start with too little progress due to poor rock quality and a lot of fine matter in tunnel masses.

6.4 Shaft operation
It was recommended by a geologist to drill the pilot bottom-up as geological maps showed we had to drill through a 50 m deep limestone marble layer. Top-down piloting could lead to leakage of flush water and even a jam.

In that context, it was made a chamber in the end of the tunnel with enough space for a drilling rig. The drilling began by the fall of 2012, the after 2 weeks 540 m was drilled (270 m a week) with shaft diameter 700mm.

Figure 8 Securing and spiling

Figure 9 20 m tunneled

Bird's-eye view into the main cavern of the New Bjølvo power station. The waterhead is 872 metres. The first Bjølvo station from year 1918 was constructed with surface steel penstock. For improved safety and efficiency a complete new underground replacement was constructed. The New Bjølvo started operation in 2003.
Photo: Statkraft.

13. UNDERGROUND HYDROPOWER CONSTRUCTION UNDER HIGH ROCK STRESS CONDITIONS – CASE: EIRIKSDAL POWER PLANT, HØYANGER, NORWAY

Johannes HOPE
Arne M. MYRVANG
Freyr PÁLSSON

ABSTRACT

Eiriksdal power plant is a new 80 MW underground hydro power plant built to replace two old above ground plants with surface steel penstocks. Eiriksdal power plant is scheduled to be commissioned during the summer of 2013. The project's tunnels and openings are excavated in Precambrian rocks in western Norway. The underground tunnelling system consists of about 4,8 km of tunnels, thereof about 3 km long unlined headrace tunnel and an underground power house (with a L x W x H = 51 x 14 x 34 m). Precambrian rocks in this part of Norway are often imprinted with high horizontal stresses, resulting in heavy spalling and rock bursts during excavation of tunnels and underground structures. During the excavation of the tailrace tunnel there were carried out both hydraulic fracturing rock stress tests and 3D overcoring tests to determine the final placement of the powerhouse cavern and the inlet cone. The tests revealed high rock stresses, which permitted a repositioning of the power station and the inlet cone 150 m closer to the valley side without risking a hydraulic fracturing in the rock mass. Rock reinforcement of the power house cavern was designed before and adjusted during the excavation of the cavern to comprehend against the high horizontal rock stresses. The rock reinforcement was solely in form of rock bolts and fibre reinforced shotcrete. A monitoring scheme was set up in the powerhouse cavern's roof, walls and floor to measure deformations over the construction period and showed measured deformations of up to 25 mm in one of the side walls.

1 INTRODUCTION

Three hydropower stations owned by Statkraft Energi AS (Statkraft) named K2, K3 and K4, located in the Municipality of Høyanger in Sogn and Fjordane County were commissioned in 1938/1955 and 1958 respectively. These plants are conventional above ground plants with surface steel penstocks down the mountain side.

The Norwegian authorities from safety reasons stated a time limit to take one of the old pen-stocks at K2 out of service. Subsequently planning of necessary upgrading of the power plants was started. These studies concluded that the most economical and environmental feasible solution was to replace K2, K3 and K4 by two new power plants.

In 2008 Statkraft was by Royal decree awarded the licence to construct new Eiriksdal power plant and new Makkoren Power Plant, both with underground powerhouses and un-lined pressure shafts and headrace tunnels, see Figure 1.

The construction work started in 2010. Makkoren power plant was commissioned in October 2012, and Eiriksdal power plant is scheduled to be commissioned during the summer of 2013.

To be taken out of service			
Power plant	Capacity (MW)	H_{max} (m)	Production (GWh/ year)
K2	12 + 13	495	115
K3	4	66	20
K4	2	93	10
Sum	31	654	145

New Power Plants			
Power plant	Capacity (MW)	H_{max} (m)	Production (GWh/ year)
Eiriksdal	30 + 50	572	320
Makkoren	4	103	18
Sum	84	675	338

Table 1. Main project data.

This paper covers the design procedure of Eiriksdal powerhouse cavern and inlet cone; the design of rock reinforcement in the powerhouse cavern and a monitoring scheme of deformations in the cavern's surrounding rock walls.

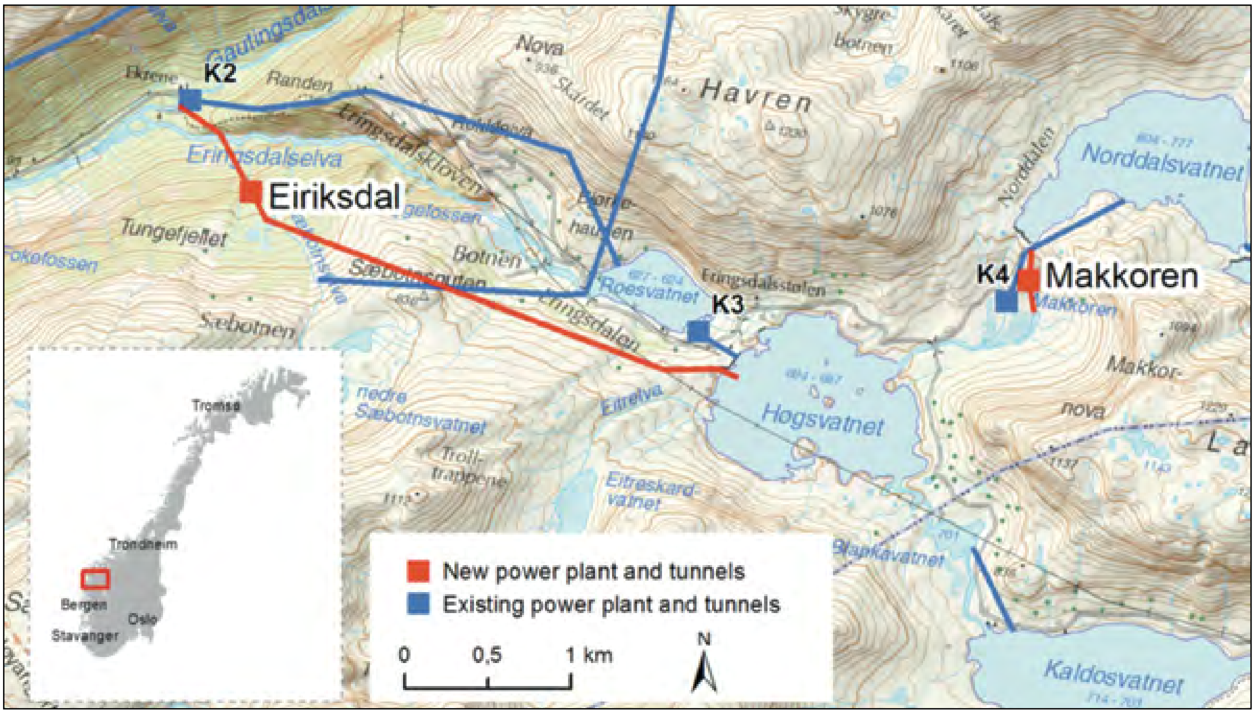

Figure 1. Outline project area.

The main underground tunnels and openings of Eiriksdal power plant consist of an about 630 m long access tunnel, a 650 m long tailrace tunnel and an about 3 km long headrace tunnel. The rock cavern for the powerhouse is 51 m long, 14 m wide and 34 m high. The transformer cavern is 24 m long, 11 m high and 8 m wide. All tunnels and caverns were excavated with conventional drilling and blasting.

2 GEOLOGICAL SETTINGS AND REGIONAL ROCK STRESSES

The rock mass of the project area consists of folded Precambrian granitic to dioritic gneisses with frequently appearing amphibolites lenses. The rock mass encountered during tunnelling at the Eiriksdal hydropower plant was in general massive with one dominating joint set along the rock foliation, having a strike of approximately N 50 E and a dip of 40° – 70° towards SE.

The area is part of an extended Precambrian region, and over the years a large number of underground hydropower plants and highway tunnels have been constructed within the region. A majority of the constructions have experienced considerable rock stress problems as heavy spalling or rock bursts in the roof of tunnels. Due to this, in-situ 3D rock stress measurements have been carried out in many locations. They almost invariably show that both the major and intermediate principal stresses are semi-horizontal with the minor principal stress approximately vertical at some distance from valley sides. Horizontal stresses up to 45 MPa have been measured. In a majority of cases the major principal stress seems to be oriented approximately NW, but sometimes also NE. Near valley sides, the directions will be influenced, often with the major principal stress semi-horizontal parallel with the mountain side, the intermediate principal stress steeply dipping parallel with mountain side and the minor principal stress normal to the mountain side.

Modern Norwegian hydropower projects include unlined headrace tunnels and/or pressure shafts. A crucial requirement in this connection is that the minor principal rock stress is higher than the water head. Therefore, as mentioned below, in-situ rock stress measurements are normally carried out from the access tunnel during construction of the tunnel before the final location of the inlet cone of the steel penstock is determined.

3 DESIGN PROCESS OF INLET CONE PLACEMENT AND POWERSTATION CAVERN

For a successful unlined pressure shaft the minor principal rock stress, σ_3, must be higher than the highest static water head, *P*, in the pressure tunnel (Headrace tunnel). During the initial design phases, the location of the inlet cone and power station cavern was established after an assessment of following:

- The project areas topography, geomorphology and geology.
- A simple limit equilibrium method which accounts for gravitational rock stresses and static water head in unlined pressure shaft (Broch, 2000).
- The maximum water pressure in the unlined pressure tunnel is 5.6 MPa. To obtain a factor of safety of minimum 1.3, σ_3 needed to be 7.3 MPa or higher. $\frac{\sigma_3}{P} \geq 1{,}3$
- Gained experience of tunnel conditions from a nearby road tunnel, Høyangertunnel (opened in 1982), that encountered some heavy rock spalling during excavation due to high horizontal stresses (Myrvang and Grimstad,1984).

Taking these things into consideration the inlet cone and power station was placed about 600 meters and 550 meters inside the mountain from the river Eiriksdalselva, see Figure 2.

An in-situ rock stress measurement program was set up to confirm that the inlet cone location had a sufficient minor principal stress, σ_3, to obtain an acceptable factor of safety against hydraulic fracturing in the rock mass; a scenario that could cause considerable loss of water during the production period. These tests would also be useful when optimising the orientation of the power station cavern's axe. All in-situ rock mass tests had to be done from the excavating tunnels, and could not be done beforehand from surface due to difficult mountainous topography.

3.1 Indications of high rock stresses during tunnelling

Excavation of the tunnelling system started with the tailrace tunnel, which was then followed by the access tunnel once the tailrace tunnel had been driven under the valley Eiriksdal. After the tailrace tunnel passed under the Eiriksdal valley an intense rock spalling in the tunnel roof was experienced and all water dripping into to tunnel ceased. The rock spalling continued along the rest of tailrace tunnel, but the spalling intensity decreased slightly as the tunnel approached the power station area. The same conditions were experienced in the access tunnel. This indicates high anisotropic stresses in the rock mass under the valley side that became more isotropic as the tunnels were driven further into the mountain.

3.2 Rock stress measurements in tailrace tunnel

During tunnelling of the tailrace tunnel following tests were carried out during the weekends when the tunnel excavation was minimal.

Hydraulic fracturing

A hydraulic fracturing test was carried out the 21st to 22nd of August 2010 in four boreholes bored from the Headrace tunnel walls at chainage 525. The hydraulic fracturing test measures the instantaneous shut-in pressure, P_{isi} which corresponds to the minimal principal stress σ3, when the measuring borehole is orientated in the same direction as either the major principal stress, σ1, or the intermediate principal stress, σ_2. The measuring boreholes where therefor drilled with the same orientation as the Eiriksdal valley as this often represents either σ_1 or σ_2.

The results gave a range for the minimal principal stress, σ_3, of 7.3 – 13 MPa.

In-Situ 3D rock stress measurements by over-coring

The following weekend, 28th to 30th of August 2010 a 3D overcoring rock stress test was carried out from the tunnel face at chainage 610. The results gave high rock stress values for the principal stresses, see Table 2.

These results show that the major principal stress, σ_1, has an orientation normal to the orientation of Eiriksdal valley. The intermediate principal stress component σ_2 has an orientation parallel to the valley. The minor principal stress component represents higher stress levels then the theoretical gravitational stresses ($\sigma_{v\ theoretical}$ = 7 MPa).

Principal stresses	Measured Principal stresses, [MPa]	Orientation	Dip
Major Principal Stress, σ_1	24.2 ± 3.4	N345	12°
Intermediate Principal Stress, σ_2	15.4 ± 3.3	N075	4°
Minimal Principal Stress, σ_3	12.4 ± 2.8	N183	77°

Table 2: Measured principal stresses at chainage 610, tailrace tunnel.

Following rock mechanical properties were measured in a laboratory from rock cores gained from the test, see Table 3.

E-mod [GPa]	Poissons ratio, v	UCS [MPa]	Failure plane [°]	Sound velocity [m/s]	Density [kg/m3]
334	0.143	231.3	23	4050	2671

Table 3: Rock mechanical properties of intact rock taken from the 3D overcoring test site.

3.3 Optimised design of powerhouse cavern and location of inlet cone

3.3.1 Powerhouse cavern moved and its length axe turned

Preliminary results of the principal stresses from the 3D over-coring tests were given, the morning 31st of August. The results permitted a repositioning of the power station cavern and the inlet cone closer to the valley side. The power station was moved the same day to where the tunnel face of the Tailrace tunnel was at that time. Or about 150 m from its original positioning, see Figure 2 and Figure 3.

Additionally the orientation of power station length axe was turned from having an orientation of N330 to N010, a rotation of about 40°. This was done to achieve a more optimal orientation of the power station length axe against the major and intermediate principal stresses, σ1 and σ_2. The angle between the power station length axe and σ1 is about 25° in plan after the rotation.

3.3.2 Final placement of inlet cone

The final placement of the inlet cone was verified by one last hydraulic fracturing test 30 meters away from the new inlet cone position. The test results gave an average minimal principal stress, σ3, of 11.2 MPa, the lowest measured σ3 being 8.6 MPa and the highest measured σ3 of 15.8 MPa.

The maximum static water pressure of the inlet cone, P, is 5.6 MPa. The factor of safety against hydraulic fracturing in the surrounding rock of the inlet cone area is therefore 1.54:

$$Fos = \frac{\sigma_3}{P} = \frac{8.6}{5.6} = 1.54$$

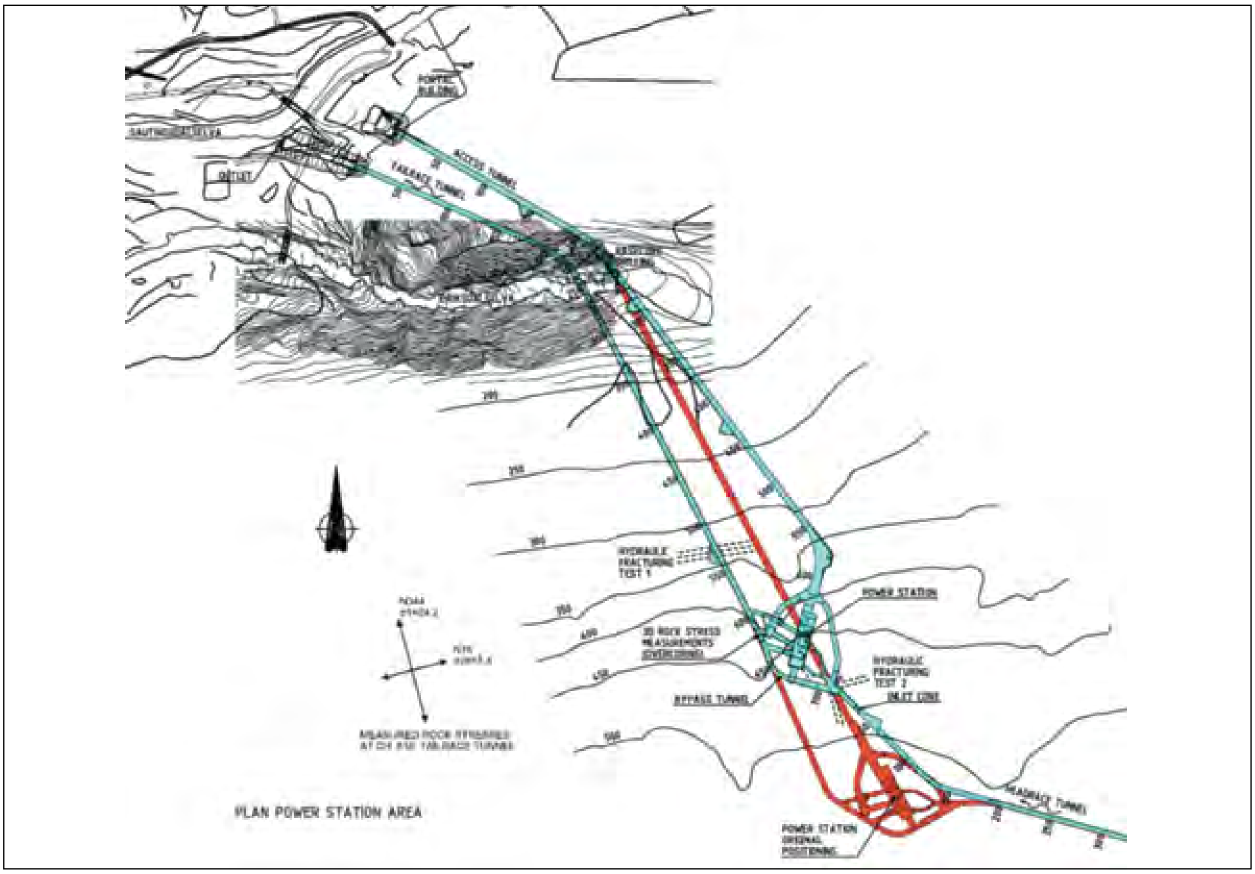

Figure 2. Original and final location of the power station area.

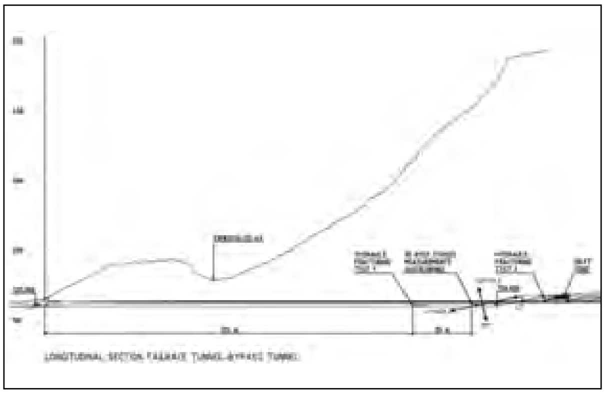

Figure 3. Longitudinal section of tailrace tunnel and bypass tunnel with final positioning of inlet cone.

4 DESIGN OF ROCK REINFORCEMENT OF POWERHOUSE CAVERN

Based on the rock mass conditions and the high rock stresses encountered in access- and tailrace tunnel a permanent rock reinforcement system for the cavern was designed with the help of the Q-system and numerical modelling (Phase^2). The rock reinforcement was further adjusted during excavation. Before the excavation started it was known that the high vertical walls of the power station cavern would experience some amount of deformations due to the high horizontal stresses. Reinforcing the rock against these was one of the most challenging factors when designing the permanent reinforcement system.

4.1 Finite element analyses (Phase^2)

Before excavation of the cavern there was carried out a 2 dimensional finite element analyses of the rock mass around the cavern, using the program Phase^2. The purpose was to get an idea of following:

- The rock stress state changes around the cavern with excavation.
- The amount of total deformations
- The depth of the shear and tensile failure zone in the surrounding sound rock walls without any installed rock support.

The results gave a highest total deformations in the roof of 32 mm, 28 mm in the floor, 53 mm in the downstream wall and 70 mm in the upstream wall (Figure 4).

The failures zone of intact rock was modelled to reach about 1.3 to 2.2 m in the cavern roof and up to about 3 m in the middle of the side walls and the cavern floor.

4.2 Rock support

The installed rock reinforcement was in form of 4 – 6 m long resin anchored rock bolts and 130 – 150 mm thick steel fibre reinforced shotcrete. The rock bolts were tensioned to 25 % of their capacity to provide a known load with a reserve in case of additional load being induced by displacements in the rock mass. The permanent rock reinforcement was installed subsequently with the excavation of the cavern. Table 4 lists the principal installed rock reinforcement in the caverns roof, walls and floor.

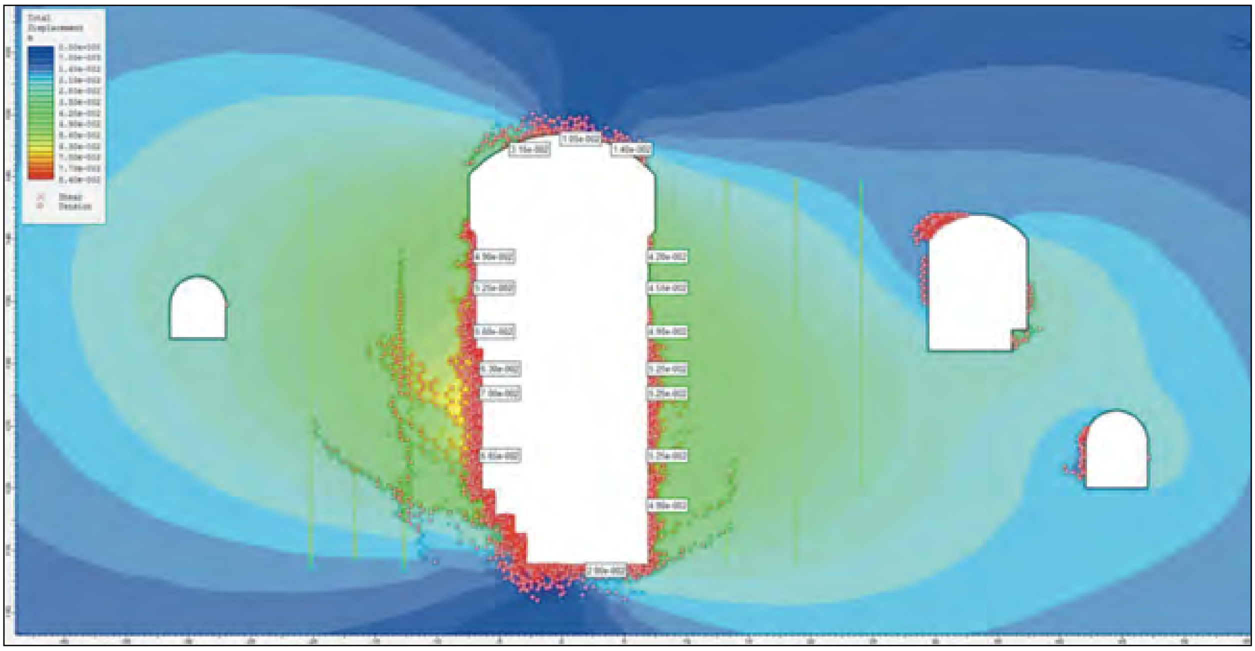

Figure 4 presents the results of deformations of the caverns rock surface and the depth of the failure zone.

Power station caver	Rock reinforcement		
	4 m long rock bolts	6 m long rock bolts	Steel fibre reinforced shotcrete, thickness
Roof	c/c 2 x 1.5	-	130 mm
Walls	c/c 2 x 2, installed before shotcrete	Between elevations 140–136 and 124–120: c/c 2 x 2 m, installed after shotcrete. Between elevations 134 – 126: c/c 1 x 2 m (that is spacing of 1m between rock bolts every 2 m height), installed after shotcrete.	150 mm
Floor	c/c 1 x 1 around generators, fully grouted rock bolts	c/c 1 x 1 beneath generators, fully grouted rock bolts	-

Table 4. The main rock reinforcement installed in the powerhouse cavern.

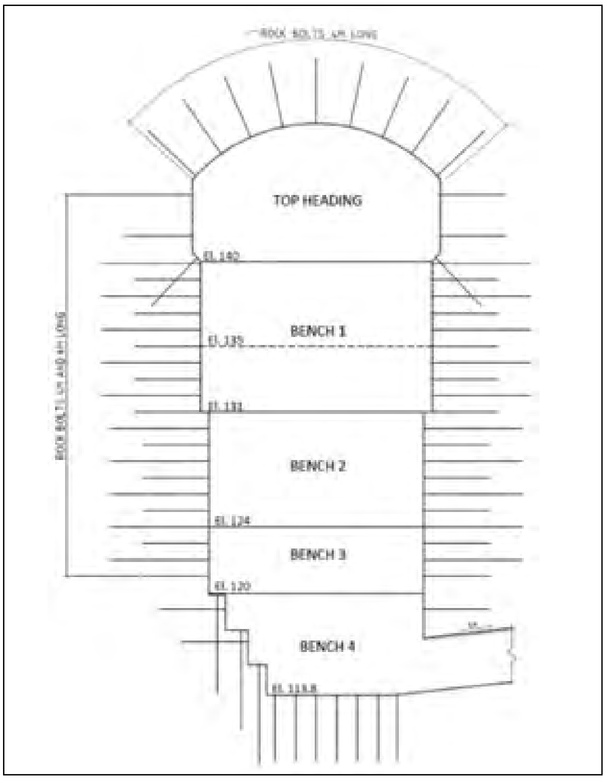

Figure 5 shows principle installed rock bolts in power station cavern and stages of excavation.

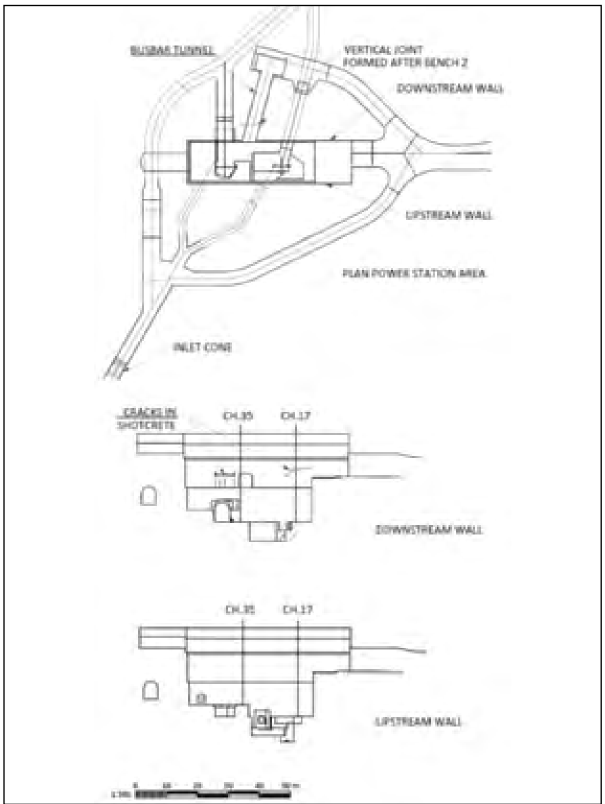

Figure 5 shows principle installed rock bolts in power station cavern and stages of excavation.

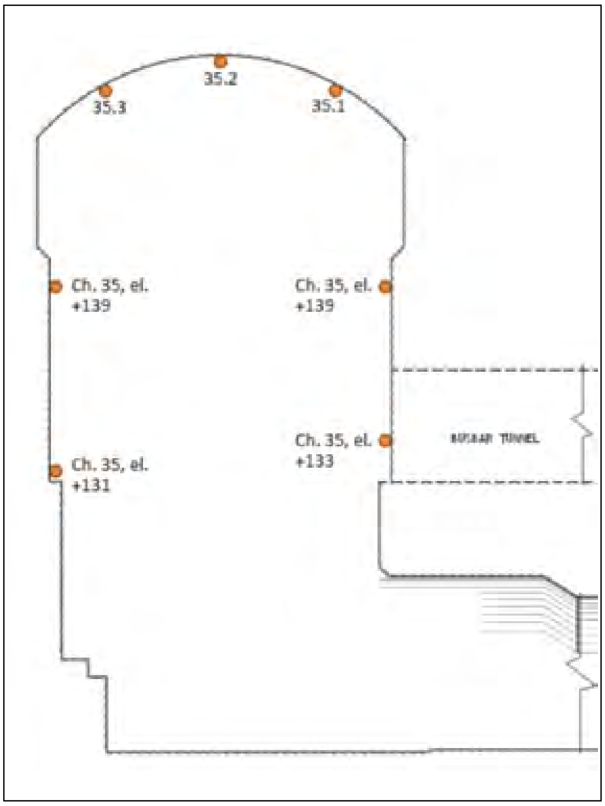

Figure 7. Convergence points installed in chainage 35, similar setup was used in chainage 17.

Figure 5 shows a cross section of the cavern where it's the deepest and the principal rock bolt system.

5 DEFORMATION MEASUREMENTS AND OBSERVATIONS IN POWERHOUSE

5.1 Convergence measurements, placements and monitoring program

There were installed in total 17 measuring points on the cavern roof, the high side walls and the floor to monitor the deformations in the powerhouse cavern. Those were in form of small prisms mounted on the shotcrete. The measuring points were put in two profiles at cavern chainage 17 and 35. Additionally, one measuring point was installed in middle of each of the longitudinal walls at chainage 24, and one at the cavern's floor in chainage 24, see Figure 6. The measuring prisms were installed shortly after excavation and reinforcement of each bench and before the subsequent bench was blasted. Prisms at chainage 17, el. 133 in up- and downstream wall were shot down during benching of bench 2, and those were not replaced.

After the measuring points were installed, they were measured two times a week for a period of 12 – 15 months. Inaccuracy of the measurements are in the scale of ±3 mm but was experienced to be up to 10 mm in some cases.

5.1.1 Results

Measured deformations in the cavern's roof indicated an upward movement of about 4 – 7 mm during excavation of the cavern. Figure 9 and Figure 10 show measured deformations in the roof with time at chainage 17 and chainage 35. The upstream longitudinal wall showed measured deformations of about 0 – 5 mm, see Figure 11. The downstream longitudinal wall showed measured deformations of 12 – 25 mm, with the greatest deformations at chainage 35 which is in close proximity to an intersecting busbar tunnel, see Figure 12. There were no registered deformations in the cavern floor.

5.2 Direct observations of deformations

During benching/excavation of the power station cavern rock spalling was observed in all the cavern's walls. These formed secondary joint sets that broke through sound rock and had a strike with an angle of about

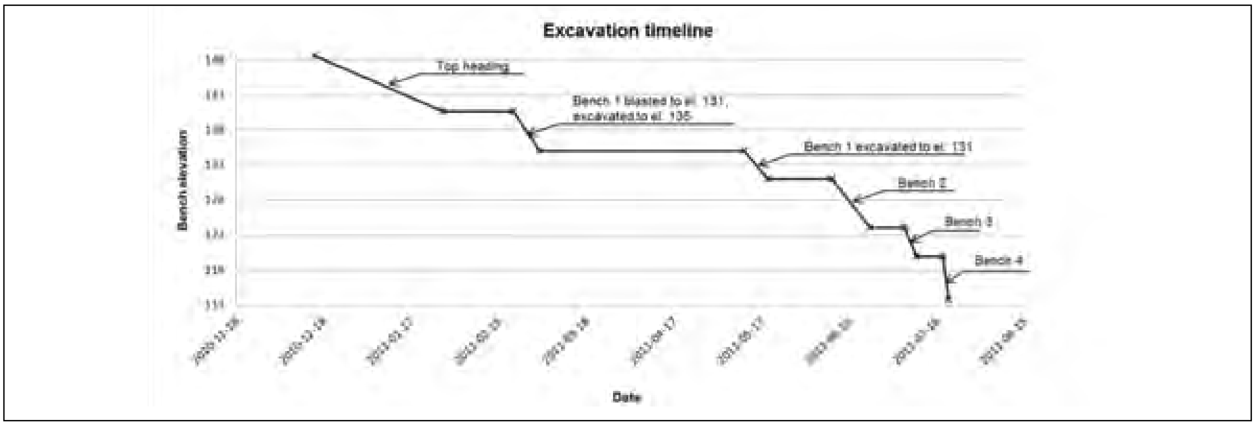

Figure 8. Excavation progress of the cavern with time

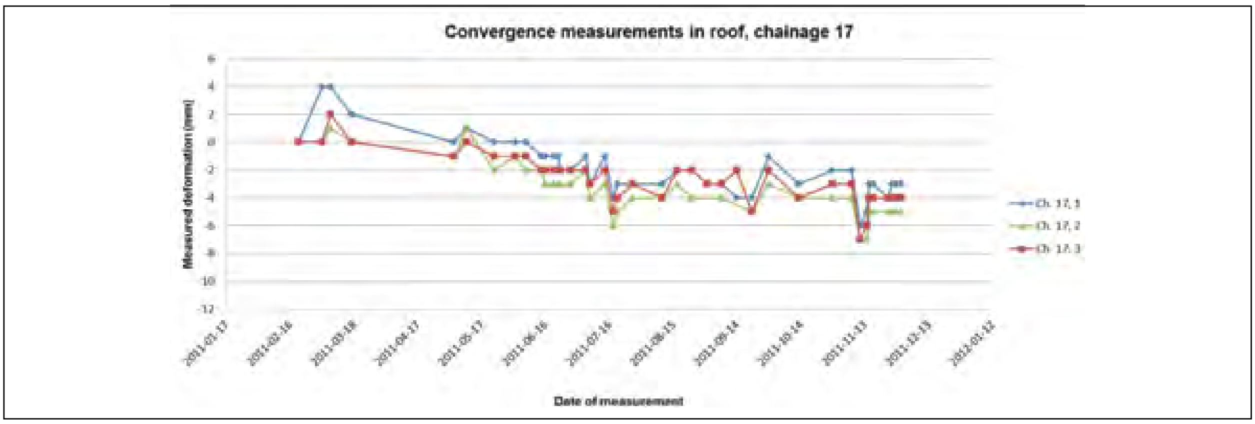

Figure 9 shows measured deformations with time in the cavern roof at chainage 17.

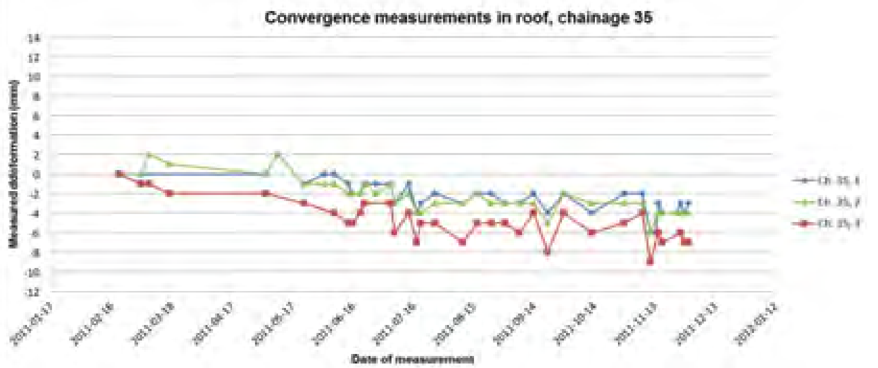

Figure 10 shows measured deformations with time in the cavern roof at chainage 35.

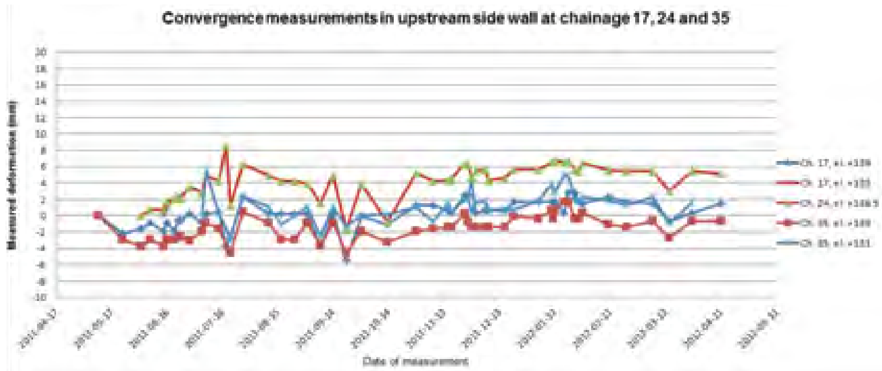

Figure 11 shows measured deformations with time in upstream side wall.

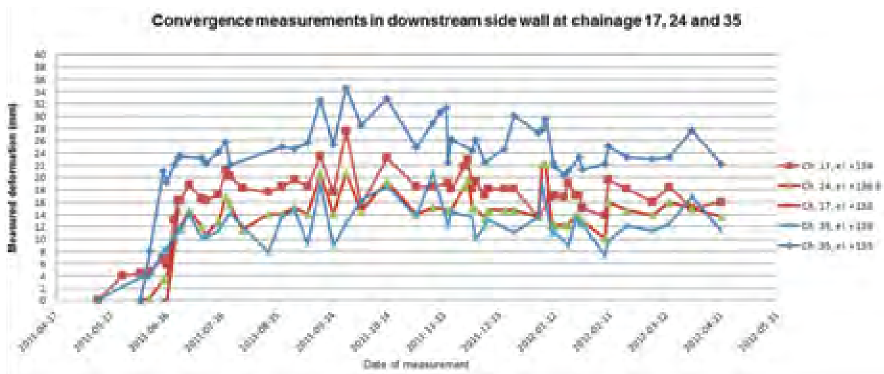

Figure 12 shows measured deformations with time in the downstream side wall.

15-20° from the cavern's axe and a spacing of 0.1 – 0.4 m. Figure 13 shows a great example of this secondary jointing in the upstream wall.

After benching down to elevation 123 m.s.l. a vertical joint formed in intersecting busbar tunnel. This new joint crossed the busbar tunnel's profile at about 5.3 m from the downstream longitudinal cavern wall and had a strike parallel to the power station cavern axe, Figure 6. The joint's opening was measured to be 6 – 8 mm. This joint was bolted together with 4 m long fully grouted rock bolts from the busbar tunnel.

In September 2011, after full excavation of the powerhouse cavern, a crack in the shotcrete was observed in the downstream wall. Deformations of surrounding rock bolt faceplates were also observed.

In April 2012 there were observed 4 vertical cracks in the shotcrete 1 – 7 m away from the intersecting busbar tunnel and above an intersecting run-off tunnel in the downstream side wall. See Figure 6.

No sign of deformations in other walls than the downstream wall has been detected visually, and in December

Figure 13: Shows secondary joint sets sub-parallell to the walls, that formed after blasting due to high rock stresses.

2012, no further sign of deformations has been observed after April 2012.

5.3 Discussion

In a hard stiff rock mass like the one encountered in the power station, area most of the deformations occur quickly after blasting and excavation. The deformations that occurred before measuring equipment was installed could therefore not be comprehended and only deformations as a result of subsequent benching could be grasped.

The convergence measurements in the roof show displacement of 4 – 7 mm upwards. This tendency does not comply with the finite numerical analyses, but it is natural that the roof moves upwards due to the high horizontal stresses.

The upstream wall experienced only minor displacement, opposite to the downstream wall where there were measured displacements of 15 – 25 mm. Most of the downstream deformations measured occurred after benching of the subsequent bench, bench 2. The reason for the higher deformations in the downstream wall is probably due to higher stress concentration in the downstream wall. The downstream wall has three intersecting tunnels that all have bigger diameter then the two smaller intersecting tunnels of the upstream wall. One can also argue that the distance between the busbar tunnel and the run-off tunnel is too little.

The numerical analyses showed higher total deformations in the side walls than was measured with the prisms but are probably comparable with the total deformations the walls experienced. The joint in busbar tunnel indicates that the rock damage zone reach a depth of 5.3 m in the downstream wall, no such joint was observed in any of the intersecting upstream wall tunnels.

6 CONCLUSIONS

The new position of the power station resulted in a reduced tunnel length of both the tailrace tunnel and the access tunnel but increased tunnel length of the headrace tunnel. The tailrace tunnel was shortened by approximately 150 meters and the access tunnel was shortened by 110 meters. The headrace tunnel was correspondently lengthened by 160 m. The reduced length of the access tunnel was the most economically beneficial as it is the most technically equipped tunnel of the three.

The success of moving the power station cavern and the inlet cone shows how important it is to have a well-planned rock stress test scheme beforehand and a design that is flexible to such changes during the construction period.

The rock reinforcement system, used in the cavern, consisting of rock bolts and shotcrete was a success. The flexibility of the resin anchored rock bolts was vital to allow for some amount of deformations in the roof and walls. Even though some signs of deformations in the downstream sidewall were observed the resin anchored rock bolts still had capacity to take up load. The stiff rock bolts installed in the cavern's floor manage halt any deformations in the floor, which is vital as the turbines and generators are sensitive to any displacements.

Unfortunately the monitoring scheme was marked by inaccuracy in the registrations and there could have been done a better job. However the results show a distinct deformation curve that evens out with time. The monitoring was an important tool to see how the rock mass behaved during and after excavation of the cavern. The results were used to revaluate and adjust the rock reinforcement under excavation of the cavern and construction of the powerhouse and to confirm that the installed reinforcement was sufficient. Utterly the monitoring results indicated that the deformations around the crane beam had halted before the crane was put up.

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14. STABILITY EVALUATION OF A LARGE UNDERGROUND POWERHOUSE IN THE HIMALAYAS

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PABST, T.
LI, Charlie

1 INTRODUCTION

1.1 Type area

Hydro Power is the backbone of the Bhutanese economy and Tala hydroelectric project is currently the biggest operating hydro power project in Bhutan. The 1020 MW hydroelectric project is a joint project between India and Bhutan generating 4865 GWh/yr. It is located on the Wangchu River in the western Himalayan Kingdom of Bhutan. By virtue of its ge-ographical location on the Southern slope of the Eastern Himalayas, Bhutan is blessed by nature with altitudinal varying land mass with good vegetation cover, perennial flow of water in the swift flowing rivers and fair climatic conditions. Bhutan is a land-locked country bordering China in the North and In-dia in the West, South and East. It covers an area of 38 394 km^2, roughly measuring 140 km North to the South and 275 km East to West (Figure 1). It is es-timated that over 72% of the land is under the vege-tative cover with altitudes varying from 100 metres above sea level (masl) in the Southern sub-tropical region, to 7550 masl in the Northern alpine region. Bhutan receives fair amount of annual rainfall vary-ing from 500 mm in the North to 5000 mm in the South. Thus, Bhutan is endowed with rich develop-ment potential for harnessing hydropower. Most of the schemes identified are run-of-the river types and they are found to be techno economically least-cost and environment-friendly. Few reservoir schemes are also identified with limited and/or no environ-ment impact in the Southern belt before the Bhutan-ese rivers fan-out and enter the Indian plains. Bhutan has an estimated hydropower potential of 30 000 MW and 120 TWh mean annual energy gen-eration indicating an average development potential of 781 kW in a square kilometre of area of land (catchment). So far 23 760 MW have been identified and assessed to be technically feasible (Tshering and Tamang, 2004).

Due to the complex geological conditions in the area the commissioning of the Tala plant was delayed from 2005 to 2008. Druk Green Power Corporation (DGPC) assumed control of Tala in April 2009. The total cost of the project was about 1 billion USD. The salient features of the project include the fol-lowing:

- A concrete gravity dam 92 m high and 130 m long at the top located at Wangkha, about 3 km downstream of the existing Chukha powerhouse.
- Three underground desilting chambers 250 x 13.9 x 18.5 m size for removal of sus-pended sediments of 0.2 mm size and above.
- Head race tunnel of 6.8 m diameter and 23 km length.
- Surge shaft 15/12 m diameter and 184 m in height.
- Two pressure shafts of 4 m diameter and 1.1 km long each trifurcating into penstocks of 2.3 m diameter.
- Machine hall cavern of 206 x 20.4 x 44.5 m to house six Pelton turbo generators of 170 MW capacity each (1020 MW).
- Transformer hall cavern 190 x 16 x 26.5 m.
- Tail race tunnel 7.75 m diameter and 3.1 km length.
- Two double circuit 400 kV transmission lines from the Tala Powerhouse to the Indo-Bhutan border of 140 km circuit length.

2 GEOLOGY OF THE POWERHOUSE AREA

The project has an underground powerhouse in Tala village with six 170 MW generators. The pillar width with adjoining transformer hall is 39 m. The geological formations at the powerhouse area consist of bedded sequences of quartzite and amphibolites schist partings. These rocks are highly deformed and folded into tight synforms and antiforms. The gen-eral foliation trend is N 49oE – S49oW with dip in N41oW direction. The rock mass rating (RMR) var-ies from 19 to 50 and the rock mass quality Q ranges from 0.11 to 14. This classifies the rock mass as very poor to good.

The powerhouse is located close to a major thrust zone called MCT (Main Central Thrust) which is marking the boundary between the Lesser and High-er Himalayas. It is a major tectonic feature and is the single largest structure within the Indian plate that has accommodated Indian-Asian convergence. It ex-tends for nearly 2500 km along strike and is a zone of more or less parallel thrust planes along which the rocks of the Central Crystallines have moved south-wards against and over

Figure 1. Map of Bhutan, with the location of some of the largest hydropower plants (DGPC).

the younger sedimentary and metasedimentary rocks. Although no conclusive link has been established between this thrust feature and the instability experi-enced in powerhouse cavern, this feature is of concern for the long-term stability of the cavern.

The National Institute of Rock Mechanics (NIRM) in India had carried out the in-situ stress measure-ments by the classical hydrofracture method and the-se were estimated as follows: vertical stress σv = 10.9 MPa (from overburden, approximately 400 m rock cover) and minimum horizontal stress σh = 9.5 MPA (approxi-mately normal to the cavern axis) and maximum hori-zontal stress σH = 14.2 MPa (approximately parallel to the cavern axis). The un-confined compressive strength of the intact rock was reported to be about 63 MPa.

3 ROCK SUPPORT SYSTEM IN THE POWERHOUSE

The rock support system consists of 26.5 mm diame-ter and 12 m long fully grouted tensioned Dywidag bolts. The yield strength of the bolts was 1033 N/mm^2 (i.e. 571 KN for 26.5 mm diameter rock bolts) and the percentage elongation was 8% (WAPCOS, 2011). The rock bolts were installed at a spacing of 1.5m c/c within the rows and also be-tween the rows (staggered rows). The rock bolts started breaking from early 2003 (when power house was under construction) and since then, about 200 bolts are reported to have failed in the machine hall (upstream 'U/S' and downstream 'D/S' walls), trans-former hall and Cable end wall. The locations of failed rock bolts in the machine hall U/S wall are shown in Figure 2. When these rock bolts break they produce a high decibel sound and at times come out of the holes. The length of broken portion of rock bolts varied from a few centimetres to a few meters. This has caused concerns to the Project Authorities because there are possibilities those broken portions of the rock bolts may hit the persons work-ing nearby or may hit the electrical panels leading to break down of the Generator-Turbine set. In addition, it may also lead to instability of the cavern walls.

The convergence of upstream and downstream walls is still continuing, it means that the load on the rock bolts is still increasing due to the movement of the rock mass surrounding it. This movement may be due to the defor-mations occurring along the joint planes in the rock mass thereby increasing the load on the rock bolts. The dynamic movement of the rock mass due to the presence of MCT cannot be ruled out. Hence, for long term stability, the strengthening of the walls of the powerhouse is war-ranted.

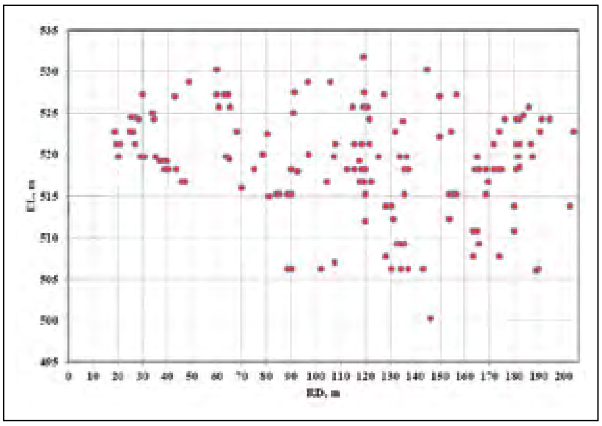

Figure 2. Location (red dots) of failed rock bolts on the upstream (U/S) wall of the machine hall cavern (Naik et al., 2011a). EL: Elevation (m); RD: position along the cavern (which is approximately 200 m long).

4 NUMERICAL ANALYSIS

Numerical simulations were carried out to better understand the reasons of the bolts failures and the overall behaviour of the rock mass. Once verified and calibrated, numerical models can help in giving some recommendations to prevent further instabili-ties. Because the machine hall is longer than 200 m, it was assumed that a 2D model could be used to simulate fairly well the problem (this is however not completely true, considering the differences in the convergences measured along the machine hall, see Singh, 2005, for instance).

Several approaches and numerical codes can be used to simulate the powerhouse cavern hall. Phase2 (Rocscience Inc.) is a finite element code commonly used for 2D numerical analysis of rock support (e.g. Kveldsvik et al., 2011). UDEC (Universal Distinct Element Code), including the Barton-Bandis (BB) non linear joint behaviour option (Barton and Ban-dis, 1990), is a distinct element model used to simu-late blocky rock structures where mechanical dis-continuities control the overall deformation (e.g. Bhasin et al., 1995, 1996; Bhasin et Høeg, 1997, 1998).

Input data were obtained from the numerous publi-cations available about Tala project since 2003. Ge-ometry of the cavern hall (see Figure 3), rock and joints characteristics, and stresses (Table 1) were measured at the beginning of the project and imple-mented in the models. The failure criterion chosen here is Mohr-Coulomb (based on numerical simula-tions already carried out by Venugopala Rao et al., 2003a, b). Some remaining uncertainties (including joints sets configurations, joints lengths, support properties or relaxation) were partly overcome by calibrating the models on the base of measured dis-placements (convergence measured at elevations 506, 515, 520 and 525; see Figure 3 for locations).

Due to the limited number of data available and the complexity of the system, the goal of the simulations is to gain a better understanding of the behaviour of the rock mass and bolt failures. Table 1 shows the input data used for the numerical simulations.

Convergences measured in the cavern hall (Singh, 2005; Sripad et al., 2003) were compared to simulat-ed displacements in UDEC. Results are shown in Figure 4. Several convergence measurements were taken for each elevation at different location along the cavern length. As a consequence, an interval is shown here. Also, two sets of values are reported. One measured after 150 to 250 days (Sripad et al., 2003) and a second one obtained after 500 to 770 days (Singh, 2005). These values are distinguished in Figure 4.

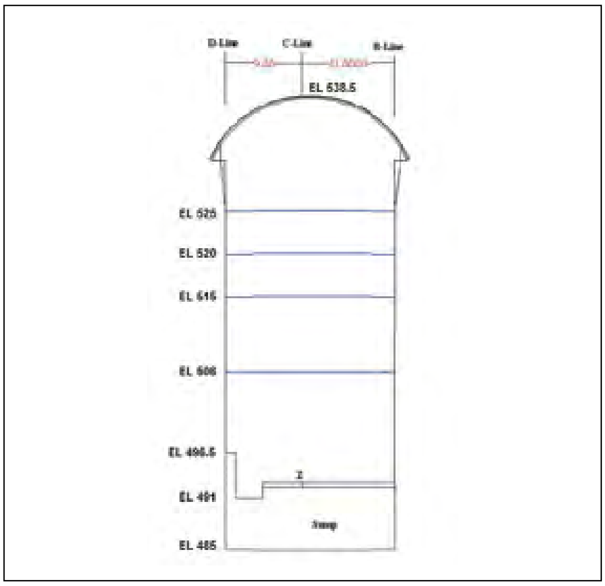

Figure 3. Section of the Machine Hall Cavern, showing gen-eral geometry (dimensions and elevations are in meter) and the locations where convergences were measured (Naik et al., 2011b).

Figure 4 first shows that the simulation results fall into the envelope drawn by experimental results, in-dicating that general behaviour is representatively reproduced by the model. The first, quick, increase of the solid curve (simulated convergence) is due to relaxation (before support is installed). There is not clear link with real time in UDEC, and relaxation was consequently stopped after around 10 cm of convergence and to fall around the measurements made after 200 days (that is, short term). Once the support has been installed (bolts and liner), conver-gence increases more slowly. The additional conver-gence during this phase is generally around 5 cm. It can also be noted that the higher the elevation is, the larger is the convergence (which is also the case in the actual displacements measurements in the ma-chine hall).

Rock mass properties (Sengupta et al., 2007)	
Density	2650 kg/m^3
Young's modulus	6,5 GPa
Poisson's ratio	0,355
Cohesion	2,16 MPa
Friction angle	43°
Field stresses (Sengupta et al., 2007)	
σ^1	10 MPa
σ^2	10 MPa
σ^z	15 MPa
Joints properties (Venugopala Rao et al., 2003a)	
Shear stiffness JKs	10 GPa/m
Normal stiffness JKn	0,97 GPa/m
Cohesion	0,0 Mpa
Friction	25°

Table 1. Input data used in the numerical simulations.

Figure 5 shows maximum total displacements in the cavern, for both finite element (Phase2) and distinct element (UDEC) simulations, once convergence has been reached. The first observation that can be made is that the results obtained with Phase2 and UDEC are fairly comparable. The total maximum displace-ment is around 15 cm in the first case, and around 16 cm in the second case (including the displace-ment during relaxa-tion period in both cases). A trend in the general direc-tion of the maximum displace-ment, that is from top left to bottom right is also ob-servable in the two cases.

In both cases, the crown seems rather stable, while maximum displacement occurs in the upstream and downstream walls of the cavern.

In Phase2, the system seems more stabilized, but the bolts capacity is almost reached in some cases (566 kN for one for instance, while their maximum capacity is 570 kN).
In UDEC, bolts failing on the upstream and down-stream walls are much more numerous and account for about 30% of the total bolts installed (in the 2D plan). This is higher than what has been observed in the cav-ern, but 3D simulations have shown that the proportion could indeed be around 25% failure (Naik et al., 2011b). The differences between the two models (one is con-tin-uous, the other is discrete) and the way support is simu-lated (rather differently in the two models) could explain the slight discrepancies observed. De-tailed comparison between the two codes (both in approach and results) will be addressed subsequently (Bhasin and Pabst, 2013).

Reports dealing with the stability have proposed that fail-ure and high convergence may be due to bolts failures, due themselves to bad grouting. Another reason for this, according to the simulation, may be that the bolts may not be long enough to encompass all the affected area (excava-tion disturbed zone) (Figure 7). Open joints reach distances more than 20 m far from the excavation walls.
In the crown, fewer open joints are observable, which con-firms what was said before, that is there is less movement in the crown. Open joints also don't go as far as on the walls (that is only a dozen of me-ters from the excavation). Results from UDEC (not shown) indicate the same trend.

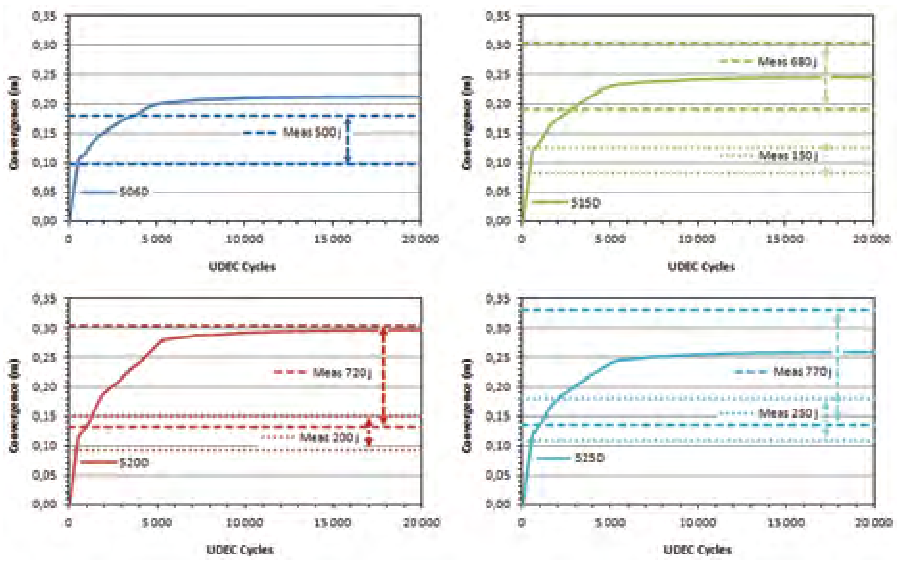

Figure 4. Comparison between convergence measurements (dashed lines; maximum and minimum convergence measured at one or two dates, at different positions along the tunnel) and simulated movements (solid line) for different elevations in the cavern.

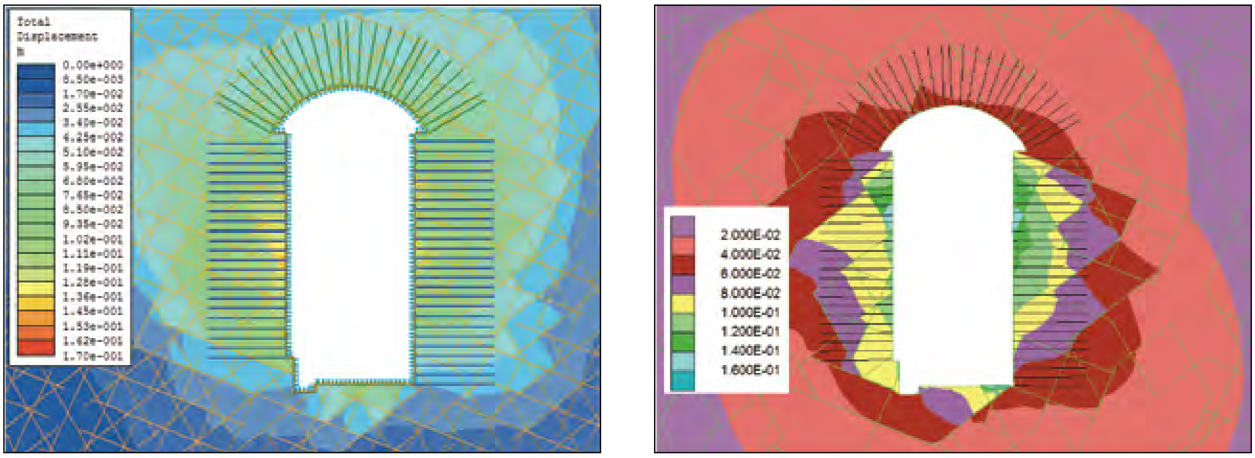

Figure 5. Total maximum displacement (m) around the cavern hall, simulated with Phase2 (left) and UDEC (right). Scales are indi-cated in Figure 3.

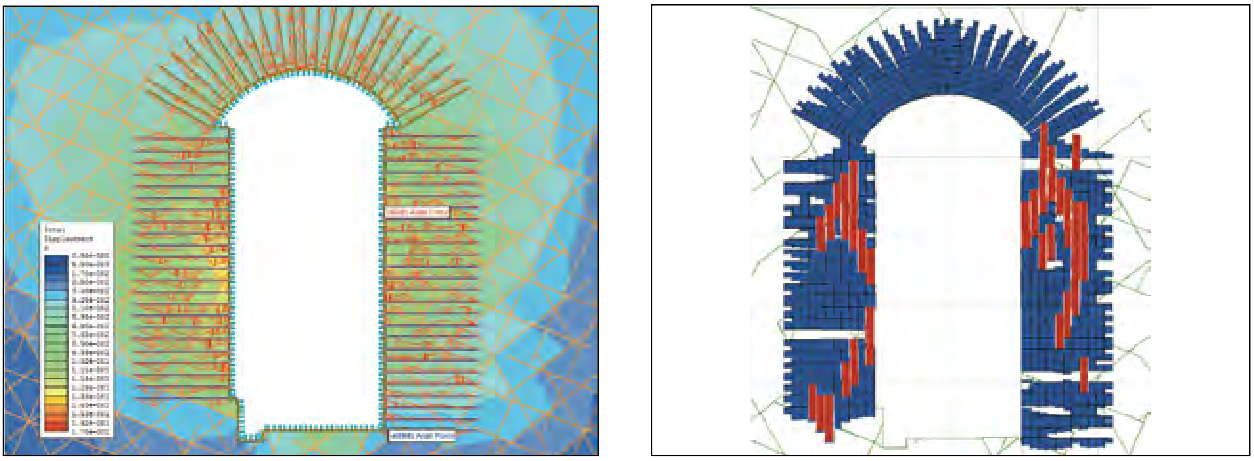

Figure 6. Maximum axial strength on bolts on the walls and on the crown of the excavation, obtained from simulations with Phase2 (left; the minimal and maximal values are indicated in blue and red respectively) and UDEC (right; failed bolt segments are indi-cated with a red line).

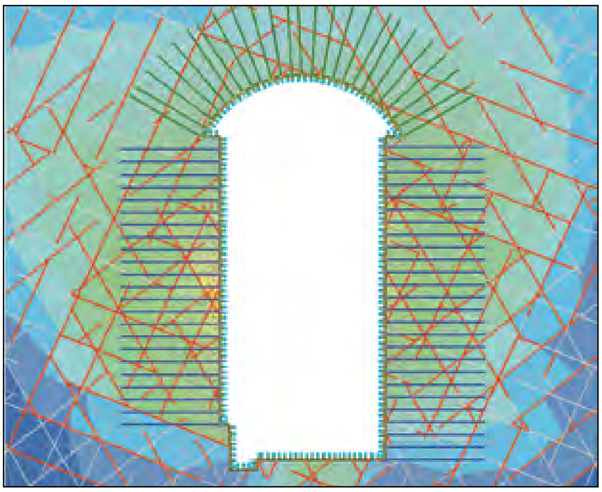

Figure 7. Joints in red are those opened by movement accord-ing to the simulations carried out with Phase2. It appears that bolts may be a bit too small and do not reach the area unyield-ed.

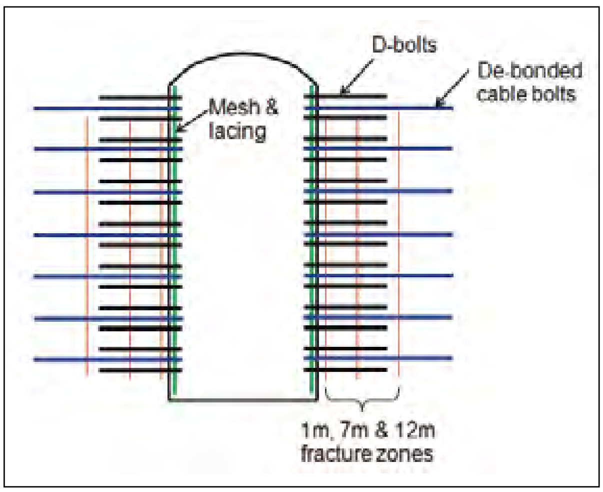

Figure 8. Yieldable support system.

Note that 20 m is also the proposed distance the bolts should reach to assure stability (because of the possible presence of a weakness zone). The exten-someter and con-vergence measurements show that about 80% of the total rock deformation occurs in the first 20 m of the wall rock.

5 RECOMMENDATIONS
This section provides some recommendations for sta-bilizing the instability experienced in the power-house. These recommendations are based on a pre-liminary assessment and may need to be altered as more detailed analysis and information is obtained. An energy-absor-bent support system is recommend-ed for the rehabilita-tion of the side walls of the cav-erns (see Figure 8). The support system may be composed of three layers:

- Layer 1: chain mesh and lacing
- Layer 2: yieldable D-bolts (Li, 2010)
- Layer 3: de-bonded cable bolts

The D-bolts take care of the fractures zones at 1 m and 7 m depths. The cable bolts secure the fracture zone at 12 m depth.

Mesh and lacing
Mesh & lacing as shown in Figure 9 would provide a good surface containment to the side walls.

D-bolts
A D-bolt is made of a smooth steel bar that has a number of anchors spaced along the length of the bar (see Figure 10). The sections between two adjacent anchors are usually designed to be 1 m long. The bolt is fully encap-sulated in a borehole with either cement or resin grout. The anchors are fixed in the grout, while the smooth and straight bar sections have very weak or no bonding to the grout. When the rock dilates, the anchors restrain rock defor-mation so that a tensile load is induced in the straight bar section. The section yields after a small amount of elastic deformation. After that, it plas-tically elongates at the level of the yield/tensile strength until the ultimate strain limit is reached. The bolt absorbs energy through fully mobilising both the strength and deforma-tion capacities of the bolt steel. Every bar section can ultimately elongate about 15% of its length. Thus, the bolt can absorb a significant amount of energy prior to failure under both static and dynamic loading conditions.

The length of D-bolts is recommended 10-12 m. The task of the bolts is to provide reinforcement to the fractured rock in the near-field of the side walls. Their main duty is to accommodate the creeping rock deformation, particular-ly in the 1-m and 7-m fracture zones, but at the same time the bolts provide high support loads to the creeping rock mass. The anchor-between bolt sections are designed 1 m long, but the bolt sections overriding the fracture zones can be made longer than 1 m. Pattern-installed D-bolts form a strong and deformable support shield around the cavern.

Figure 9. Mesh and lacing.

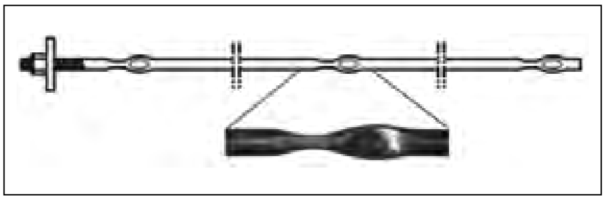

Figure 10. Layout of the D-bolt.

Application of cable bolts
The main tasks of cable bolts are to reinforce the fracture zone at 12 m depth and to provide a suspen-sion to the D-bolt reinforced zone. They also provide reinforcement to the fracture zone at 7 m depth. The layout of the de-bonded cable bolt is sketched in in Fig. 11. The length of cable bolts is recommended 20 m. The middle portion of the cable strands is sleeved to de-bond it from the grout. The near-end and far-end portions are bonded to the grout. Bulges may be made in those portions to enhance the bond.

6 CONCLUDING REMARKS
A certain number of instabilities were observed dur-ing and after construction of the machine hall of the Tala hydro-electric power house. Approximately 5% of the bolts in the powerhouse are reported to have failed and the walls of the cavern are continuing to converge, at a slow rate. Plans are underway to sta-bilize this important underground structure. Numeri-cal simulations, based on two codes, have confirmed what was observed in situ. Convergences have been fairly well reproduced and some information was obtained regarding bolts failures. Observations in the cavern show that the expulsed bolts were free of grouting, indicating that maybe the reason of these failures may be the not so good installation.Based on these considerations, recommendations have been proposed to stabilize the walls of the powerhouse cavern using chain mesh and lacing, yieldable D-bolts and de-bonded cable.

7 ACKNOWLEDGEMENTS
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Brattset power station, part of the Orkla river development.
Photo: Trønderenergi.

15. NORWEGIAN HYDROPOWER TECHNOLOGY APPLIED IN VIETNAM

TRINH, Nghia
GRØV, Eivind

1 GENERAL INTRODUCTION

Vietnam, a country with a population of approximately 90 million has 1,110 hydropower projects in operation with a total installed capacity of appr. 25 G. The installed capacity of these operating hydropower projects varies significantly and ranges from just a few kW up to 2400 MW, the latter being the Son-La HPP which entered into operation on the 23rd of December 2012. As a comparison Norway with its population of 5 million has a total installed capacity of 30GW.

The geology of Vietnam is in short divided into five structural blocks: Northeast, Northwest, Truongson, Kontum and Nambo. The NE block consist of igneous rocks which have been found dating from the Early Paleozoic to the Quaternary. The NW and Truongson blocks are regarded as NW-SE trending Paleozoic folded systems filled with Paleozoic formations of thickness >12000 m. Precambrian strata are widespread in the Red River fault zone and Fansipan range in the NW block, and in the Kontum block. Archean rocks are found only in the Kontum block, which is regarded as a stable massif without Paleozoic sedimentary rocks. The Nambo block is covered with a very thick (>6000 m) sequence of Cenozoic formations deposited in a continental rift. As a tropical country, the rock in Vietnam is normally covered with in-situ weathered (soil) material.

The natural circumstances described in very brief terms above are to some extent quite similar to those of Norway, except the in-situ weathered soil and young sedimentary rock types. Thus, they are well fit for development of hydroelectric power production. The most updated plan has been approved by the Prime Minister of Vietnam and dates back to as recently as 21st of July 2011. According to this plan, there are about 70 large, 38 medium, and a large number of small hydropower projects which are planned to be developed during the period of 2011 - 2030.

The design concept of hydropower projects in Vietnam is mainly based on a number of Vietnamese standards and sub-standards. In addition some foreign guidelines are also available and can be used in situations where the local Vietnamese standards are not fully appropriate, or in cases where new technologies are considered being required. A typical set of the design standards for a medium hydropower can consist of more than 20 standards covering many aspects of a hydropower project.

2 INTRODUCTION OF THE NORWEGIAN CONCEPT FOR HYDROPOWER PROJECTS IN VIETNAM

The Norwegian hydropower concept was first introduced systematically to selected Vietnamese hydropower engineers who participated in the Hydropower Development Programme at the Norwegian University of Technology and Science (NTNU). This programme is tailored to fit the need of young international engineers within the field of hydropower development.

According to the introductory description, the MSc.-course in Hydropower Development at NTNU, known as the HPD programme, is a two-year long international Master's programme in hydropower planning. The first year consists of a series of 6 basic courses and a desk study of a relevant project where the students work in groups applying knowledge from other courses by conducting a pre-study of the development alternatives in a Norwegian water system. This involves learning how to combine techniques, environment and economy to secure success. The final year consists of four compulsory advanced courses in the autumn, while the entire spring semester is dedicated to the master thesis.

Since 1996 a total of 12 Vietnamese engineers have graduated from this HPD Programme. The students are given scholarships from NORAD (Norwegian governmental aid programme) to enhance development in some selected countries, and one of these is Vietnam. A part of the agreement is that these professionals shall perform their skills in their country of origin following graduation to MSc. As a direct result from this programme the Norwegian hydropower concept is gradually being acknowledged and applied in Vietnam.

Through this programme Norwegian hydropower tunnelling technology has been introduced to young engineers. They again bring confidence in this technology to Vietnam to motivate it to be applied in the Vietnamese hydropower industry.

Norwegian experience in rock tunnelling has been brought to Vietnam through the knowledge obtained by the students from the HDP-programme described above. However, so far the knowledge of Norwegian concepts has been materialised in just a limited way in three or four hydropower projects. The application has typically been limited to:

- Plan, design, and construct the headrace tunnels as unlined tunnels;
- Change tunnel design from concrete lined tunnels to unlined tunnels to reduce the cost implications and construction time;

Water conveyance in a high head hydropower project normally consists of 3 main components: low water pressure tunnel (up to e.g. 150 m head), a surge chamber/shaft and high pressure shaft (vertical or inclined) upstream a tunnel leading to the power station. Construction of vertical or inclined pressure shafts appears to be the most challenging task and currently this is the main obstacle in improving the layout of hydropower projects in Vietnam.

Whilst in Norway, construction of shafts was earlier carried out with the use of the Alimak system (drilling and blasting platform climbing upward on a rail system). The Alimak system is today considered to be out of date in construction of hydropower shafts (and shafts in general) due to the inherent safety risk associated with the method. Implementing today's technology the construction of shafts can be carried out in a safe manner with the use of raise boring technique. Thus there are two particular challenges to be dealt with when implementing this technology in Vietnam:

- The available capacity of contractors in Vietnam in excavating vertical shafts is at present shorter than a length of 170 m;
- Shafts higher than this will require international contractors. Such contractors are difficult to hire in this region of the world.
- The difficulty in finding a reliable contractor with a reasonable price level affects heavily the cost aspect of the shaft alternative. Thus, under such circumstances unlined pressure shafts become less competitive, the longer these shafts are the less competitive.
- One of the solutions that have been introduced is to divide the shafts into several sections of less than 170 m, but this solution requires additional access tunnels to these sections and thus increasing the total cost of the solution.

Another solution would be to apply TBMs or drill and blast tunnel with an even and steady inclination from the intake to the powerhouse with the use of air-cushion surge chamber. Such a solution may imply that the tunnel inclination becomes steep and thus requiring specialist contractors employing dedicated equipment. Such special requirements would affect the construction cost and time.

The application of Norwegian tunnelling technology will be described in more details in the following chapter. The chapter will also discuss experienced difficulties in applying Norwegian tunnelling technology in hydropower tunnels in Vietnam.

3 SOME PRACTICAL APPLICATIONS OF THE NORWEGIAN TUNNELLING TECHNOLOGY TO HYDROPWER TUNNELS IN VIETNAM

As mentioned above, Norwegian tunnelling technology has been introduced to some hydropower projects in Vietnam, and probably the potential exists for its increased influence in this industry. It would be way too comprehensive for an article like this to enter into all the relevant details from these projects. Therefore we have chosen to describe some typical examples herein and present some details of the projects and the way Norwegian experience has been applied.

Zahung HPP, 30 MW, H= 66 m, unlined tunnel L= 1.5 km

This hydropower project is located in the central part of Vietnam, with an installed capacity of 30 MW. Its location is about 100 km west of Da Nang city, along the A-Vuong river. The headrace tunnel is a normal horseshoe shaped tunnel of 6 m width and about 1.4 km length in granitic rock. The tunnel had been designed as unlined tunnel following the Norwegian experience in hydropower tunnel. The design team for the project was headed by a professional who achieved his MSc-degree at the mentioned HPD-programme. An optimum cross section, rock cover and rock mass quality as well as rock support were designed accordingly. The major part of the tunnel was classified as "Good" to "Extremely Good" rock mass quality, - see Figure 1. In this part of the tunnel, only scaling and spot bolting was needed for

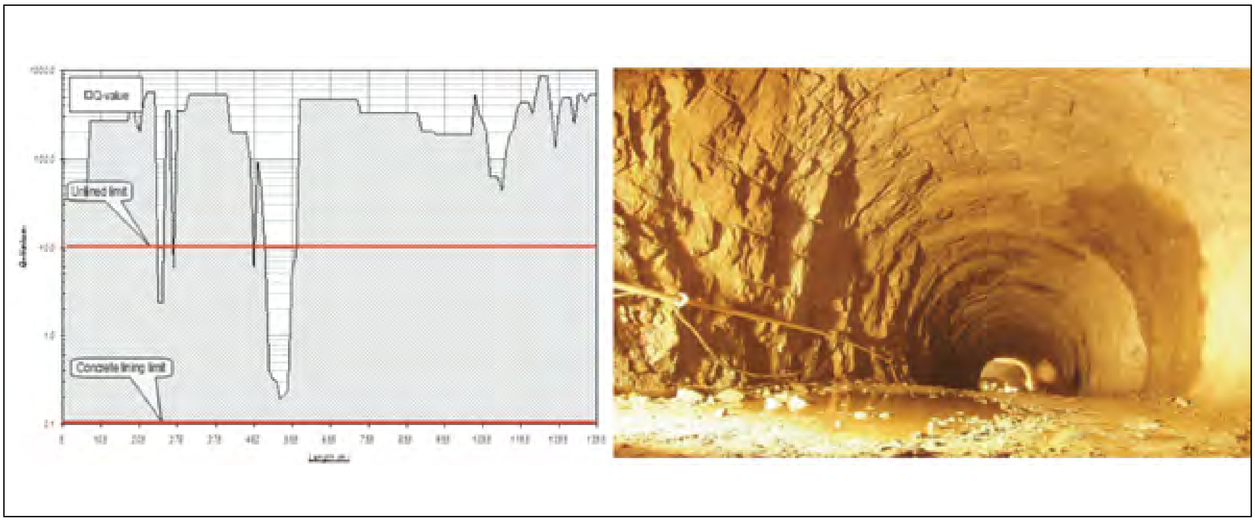

Figure 1 - Q-value distribution along the Zahung headrace tunnel and a typical tunnel section with "Extremely Good" rock mass quality.

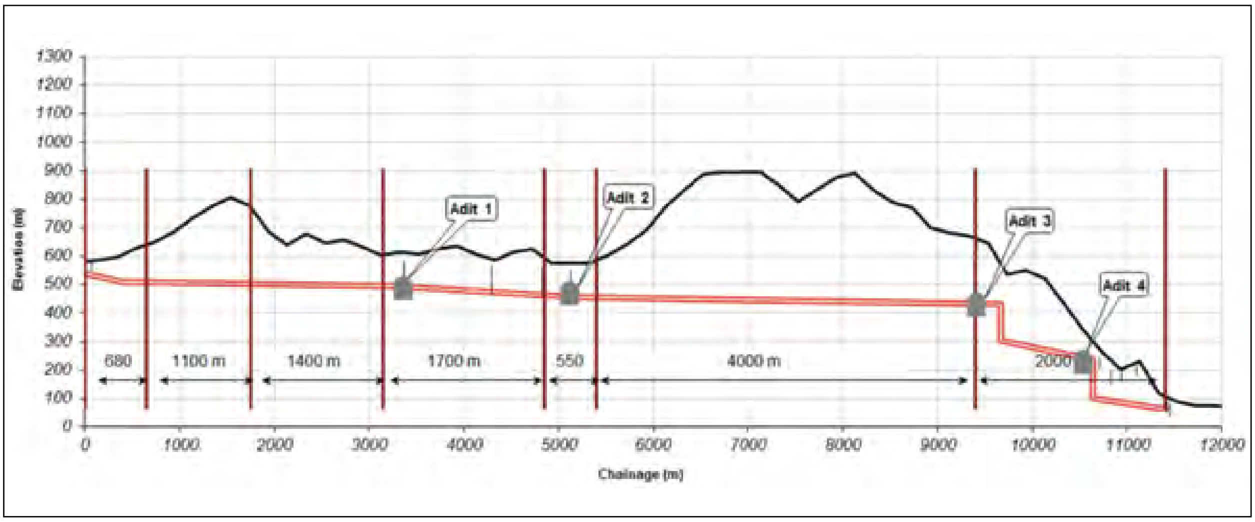

Figure 2 - Longitudinal section of the A-Luoi HPP.

supporting the tunnel. In one particular section of the tunnel a crushed zone was encountered. The crushed zone was mapped and classified as "Poor" rock mass quality and required rock support as systematic bolting and 10 cm of sprayed concrete reinforced with steel mesh. The construction was completed and put into operation in 2009. The remaining of the tunnel held typically good rock mass and was supported with occasional installation of sprayed concrete and rock bolts. Since the commissioning of the Zahung hydropower project it has been operating successfully without any problems or interruptions in operation.

A-Luoi HPP, 170 MW, H=456 m, lined/unlined tunnel L= 12 km

This hydropower project is located in the Hue province in central Vietnam. The project has an installed capacity of 170 MW with average static water head of about 456 m. The head race tunnel is approximately 12 km long. The longitudinal section of the tunnel is presented in Figure 2. The project is designed to use Pelton turbines.

In the design stage, a section of 4 km tunnel was designed as unlined tunnel in accordance with the Norwegian tunnelling concept. The remaining part of the tunnel was designed with concrete lining. During construction, it was found that the rock mass at some sections of the tunnel was better than anticipated and thus a large part of the concrete lining tunnel was removed from the design and changed to be constructed as unlined.

It is noteworthy to study the last portion of the head race tunnel which was excavated from two adits and holds a series of shorter vertical and inclined tunnel sections. This is indeed not a modern solution as described above. A Norwegian design approach may have included an even inclination from one end to the other of the head race tunnel with the possibility of avoiding several adits, or alternatively an inclined/ vertical shaft with a full length being placed deeper into the rock mass to withstand the inner pressure without steel lining.

The geological conditions along the tunnel are considered as complicated. The upstream half of the tunnel is located in sedimentary rock, and the second half of the tunnel (from about chainage 5000) is located in granitic rock. The geology in the tunnel was mapped and the Q-values are found in Figure 3. The rock mass in the sedimentary section varies from “Extremely Poor” to “Fair” quality, whilst “Poor” rock mass quality was observed to be crushed and weathered rock. In combination with groundwater this rock mass becomes very unstable. During construction spiling bolts and steel ribs embedded in shotcrete were used as temporary rock support in many locations to enable safe construction in these zones. The zones were permanently supported with concrete lining. In the “Poor” to “Fair” rock mass quality the temporary rock support consisted of scaling and spot bolting. The permanent rock support in the “Poor” to “Fair” rock quality was a combination of rock bolts and shotcrete. Typical conditions of the rock mass in this section are presented in Figure 4.
The second half of the tunnel is located in granite. The rock mass varies from “Fair” to “Extremely Good” quality. Temporary and permanent rock support in such areas consisted of scaling and spot bolting. Several crushed and sheared zones were encountered which varied from less than a meter to a few meters

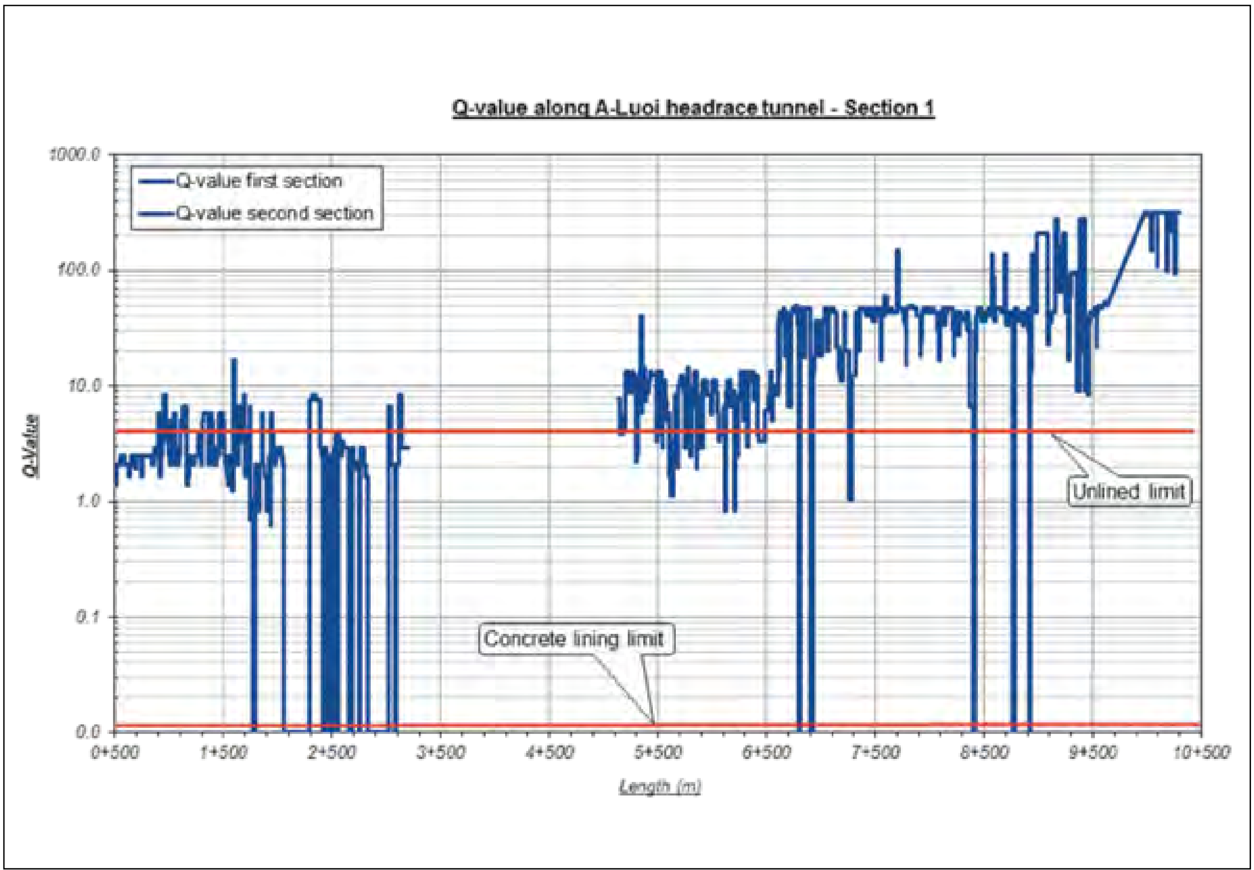

Figure 3 - Q-value distribution along the headrace tunnel of the A-Luoi HPP.

Figure 4 - Typical "Extremely Poor" (left hand side) and "Fair" (right hand side) rock mass conditions.

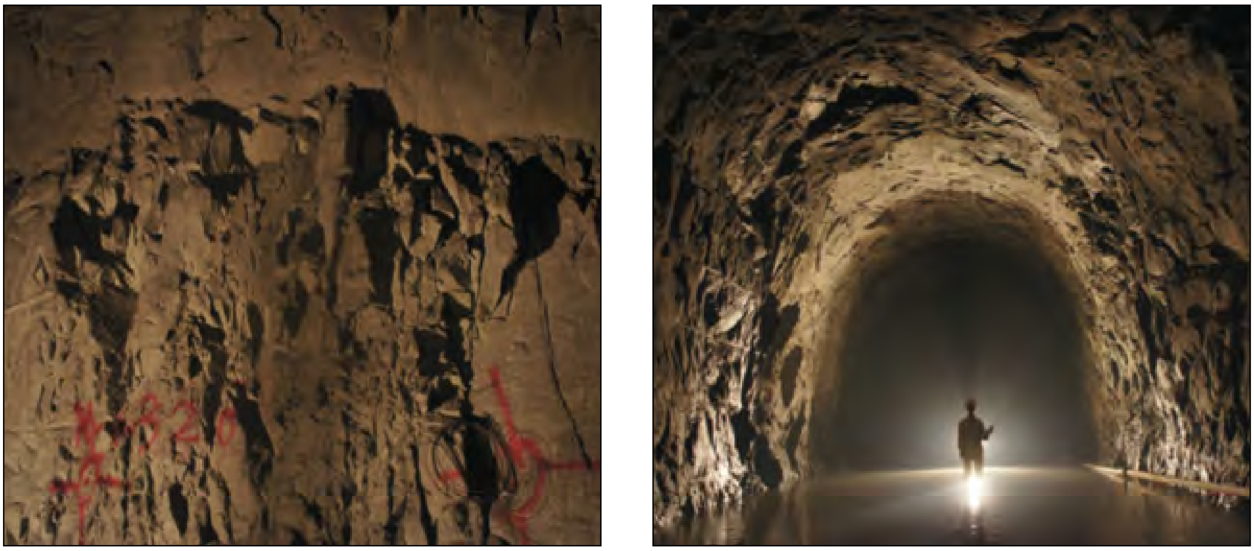

Figure 5 - Typical "Crushed zone" (left hand side) and "Extremely Good" (right hand side) rock mass conditions.

thickness. Typical filling material in such zones was weathered rock into crushed stone and clay. The presence of groundwater was sometimes associated with these crushed and sheared zones and caused severe challenges during the excavation phase. Spiling bolts and steel ribs embedded in shotcrete were normally used as temporary support in these zones whilst permanent rock support consisted of systematic bolting with steel mesh in 10 cm of shotcrete were normally used. Concrete lining for permanent rock support was used to a limited extent in one single large zone which yielded "Extremely Poor" rock mass conditions. Typical conditions of the rock mass in this section are presented in Figure 5.

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16. COLLAPSE AND BURST DEBRIS FLOOD AT SVANDALSFLONA PRESSURE SHAFT

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1 INTRODUCTION

The adopted construction philosophy for hydropower tunnels in Norway gives reduced capital investment costs due to reduced construction period and less cost for the permanent tunnel support of the entire scheme. This Norwegian approach of keeping pressure tunnels and shafts unlined has become acceptable in many other countries with the conditions that the geology favors (Neupane and Panthi, 2012).

However, during the last 5 years at least three cases of tunnels and shafts built for hydropower plants have experienced partial collapses in Norway. Most of these collapses happened along the waterway systems consisting headrace tunnels and shafts that had been under operation at full hydrostatic pressure for many years. These collapses mainly occurred in the weakness zones where either compromises may have been made on the applied support during construction caused by difficult location and time pressure, or in-adequate rock support may have been applied due to un-careful mapping of the geological conditions. It is important to be noted that the compromise made on long-term stability may result in severe economic consequences (loss of revenues), which may not be acceptable for the industry and to the society at large.

This paper is a summary of a detailed analyses report (Panthi, 2009) and the article published in Rock Mechanics and Rock Engineering (Panthi, 2012). The paper mainly discusses reasons behind the collapse and burst debris flood that took place at the inclined pressure shaft of Svandalsflona Hydropower Plant during clearing collapsed rock mass from the shaft bottom.

2 THE PROJECT

Svandalsflona hydropower plant located at Odda came in operation in 1978 and is one of the several hydropower plants developed under Røldal/Suldal hydropower development scheme. The Røldal/Suldal scheme all together utilizes 793 square kilometres catchment area, was developed in the 1970s and utilizes reservoir capacity of about 830 million m3 to generate 2757 GWh energy annually. One of the projects within this scheme "the Svandalsflona hydropower plant" has an installed capacity of 20 MW and produces 36 GWh energy annually. The project utilizes water from three small lake reservoirs consisting vestre (west) Middyr, østre (east) Myddyr and Stutakvelven lakes. Stutakvelven lake also functions as up surge facility and is connected with the headrace system through a 47 degree inclined pressure/surge shaft. The inclined pressure shaft begins at approximate waterway chainage of 1620 m. The total length of main waterway system consists of approximately 4000 m and passes through varying geological conditions (Fig.1).

3 PROJECT GEOLOGY

Svandalsflona hydropower plant has a complex geological set-up. The project is situated mainly within three different geological formations belonging to; (1) Precambrian basement older than 700 million years, (2) Cambro-Silurian basement with an age between 350 and 570 million years and (3) Overthrust basement (nappes) of Caledonian orogeny with an age of approximately 350 million years. The downstream part of the headrace tunnel up to approximate chainage 900 m, tailrace tunnel, access tunnel and underground powerhouse are aligned within greenstone and green schist rocks. The middle part of the headrace tunnel from chainage 900 to 2750 m, where Stutakvelven inclined shaft and reservoir are located, passes through schistose phyllite and quartzite of Cambro-Silurian basement rocks. Within this section, a small band of greenstone is also intruded between approximate cahinage 2170 and 2300 m. The upstream part of the headrace tunnel from approximate chainage 2750 m and further upstream is aligned through quartzitic gneiss representing Overthrust basement rocks (Fig. 2).

Like other waterways system of the power plants in Norway, the headrace tunnel of Svandalsflona was constructed without any extensive use of rock support measures. Therefore, the tunnel systems were mostly left unlined. The major weakness zones were stabilized with full concrete lining. Low density bolting, occa-

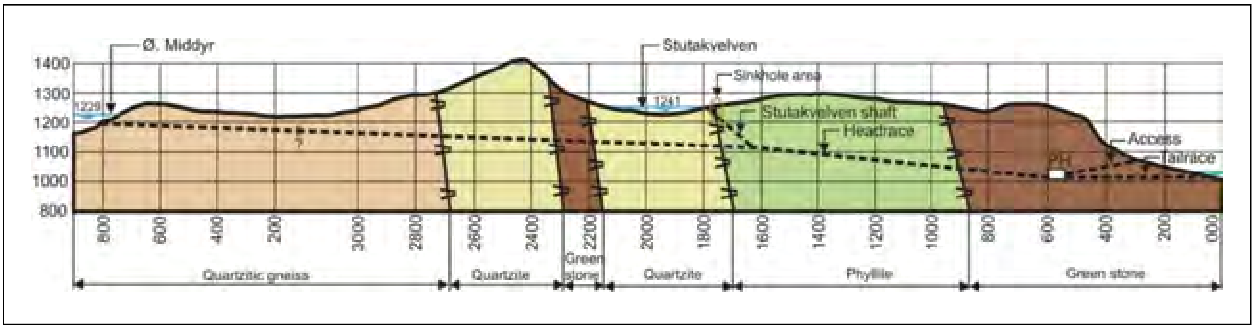

Fig. 1 - Longitudinal profile and geological set-up along the Svandalsflona (Panthi, 2012)

sional unreinforced and mesh-reinforced shotcrete were also used to satisfy stability along the headrace tunnel. A weakness zone crosses the Stutakvelven inclined shaft at an approximate elevation of 1212 m (Fig. 1). The weakness zone is described as the zone consisting highly fractured (cubical like) rock material mixed with silt and sand. The rock mass of the weakness zone has low frictional and cohesion properties giving very low self-supporting capability.

The inclined shaft was excavated in 1978 from bottom to top using Alimak raise climber. This was possible until the boundary weakness zone was hit at an elevation of approximately 1212 m (Fig. 1). Due to difficulty in securing the weakness zone from Alimak raise climber, the excavation was stopped from down to top. The remaining 40 meters length of the inclined shaft was therefore excavated from the top using traditional shaft sinking method. The weakness zone was permanently supported using concrete lining to the side wall and roof. The concrete walls were braced using in-situ concrete bracing beams installed at the floor of the inclined shaft.

After almost 30 years of successful operation of the inclined shaft, a major rock slide took place at this weakness zone in 2008 and a sinkhole appeared all the way to the surface topography (Fig. 1 and 2). The chronology of the events after the inclined shaft collapse is detailed in the following chapter.

4 CHRONOLOGY OF EVENTS AFTER COLLAPSE

In May 2008, during routine inspection, it was noticed that the water level at the Stutakvelven reservoir was 40 meters higher than the water level at other two intake reservoirs (V. Middyr and Ø. Middyr), even though the intake gate was fully opened. This was a clear indication that there must be a blockage within the inclined shaft. This blockage was only possible if a rock slide had occurred inside the inclined shaft and filled it with slide (collapsed) rock mass. After more investigation on the surrounding topography, a sinkhole was found near the intake gatehouse. The diameter of the sinkhole was approximately 5 m in the rock mass and approximately 8 m at the topographic surface (Fig. 2).

Left photo (Fig. 2a) was taken after closedown of the power plant and emptying the waterway system for inspection from the headrace tunnel in July 2008. Similarly, the right photo (Fig. 2b) was taken in early

Fig. 2 - Sinkhole nearby Stutakvelven intake gate house in fully drained (a) and fully water filled (b) conditions (Courtesy: Norsk Hydro).

September 2008 after the waterway system was once again filled for normal operation. No water in the sinkhole (Fig. 2a) indicated that the collapsed material deposited and clogged the shaft had higher permeability than the quantity of leakage (discharge) through the stoplogs of the closed intake gate and groundwater inflow (if any) from the rock mass. The important chronological events that were taken after finding the sinkhole at the Stutakvelven surface are summarized in Table 1.

After burst (flash) flood that took place on 9th Many 2009 and two workers lost their life, series of investigations and inspections were carried by Norsk Hydro using various experts. Finally, Norsk Hydro also wished to obtain independent external expert's opinion on this case. This author was involved as a single member independent external expert to evaluate the dynamics and factors that may have triggered the burst flood of the 9th May 2009.

IMPORTANT DATES	DESCRIPTION OF THE EVENTS / COURSE OF ACTIONS
Late May 2008	Abnormal water level at the Stutakvelven reservoir was noticed and a sinkhole nearby stream (bruck) intake gate was found on 3rd June 2008.
June / July 2008	ROV inspection in early June confirmed water level inside the inclined shaft at 1241.6masl and slide deposit up to 1184masl. After ROV inspection stoplogs at the intake gate closed and headrace tunnel was drained. Inspection inside headrace tunnel and at the bottom of the inclined shaft was carried out in late June 2008. It was observed that the leaked water from stoplogs was flowing freely through slide deposit. After inspection, headrace tunnel was re-filled, stoplogs at the inlet gate were opened and plant came in full operation until the middle of April 2009.
August to October 2008	Loose glacial deposit was removed from the sinkhole area. Approximately 1000 m3 loose material was excavated. One meter high concrete safety wall was constructed at the top of the sinkhole excluding hillside where bedrock was exposed. On 12th September stoplogs at inlet gate were closed once again. Approximately 30 meter deep sinkhole was then secured with shotcrete and bolting. In late October an inspection was carried out from the top of the inclined shaft up to the bottom of the sinkhole and sinkhole top was covered by corrugated sheet for winter close down.
Middle April to early May 2009	The waterway system was once again emptied, inspection at the headrace tunnel and bottom of the inclined shaft was carried out. Excavation and removal of rock slide deposit started from the shaft bottom. Considerable volume of slide rock mass (approximately 840 cubic meter loose rock mass) was removed by 8th May 2009.
9th May 2009	At 13:23 a sudden burst (flash) flood mixed with loose rock mass and sediment came down from the inclined shaft, which swapped away two workers engaged in removing slide rock mass from the shaft bottom.
10th May 2009 and on ward	Series of investigations and inspections took place to find out reasons for the burst flood including video inspection into the shaft.

Table 1 - Main events after identification of the inclined shaft blockage (Panthi, 2009 and 2012).

5 FINDINGS

Remotely operated vehicle (ROV) was used to carry out inspections through water filled inclined shaft from the top in early June 2008 and September 2008 to identify approximate elevation up to where slide rock mass was deposited into the shaft. Inspections confirmed that a huge overbreak took place from the lower end of the approximately 10 m long concrete lining section at the weakness/fracture zone located between elevations 1222 and 1215 m (Fig. 1 and 5). The slide rock mass fragments were visible at an elevation of 1196 m and large block of rock mass fragments and broken concrete blocks were identified at the shaft invert at and after approximate elevation of 1189 m.

Investigation confirmed that an approximately 93 m shaft below elevation 1184 m (including 6 m upper part with 13 m2 theoretical cross-section and 87 m bottom part with 4 m^2 theoretical cross-section) and 16 m long Alimak chamber with 16 m^2 theoretical cross-section were filled with rock slide deposit. The inclined shaft was excavated using Alimak Raise Climber and drill and blast method of excavation. In general, inclined shafts of this nature excavated using Alimak Raise Climber get over excavation of approximately 50 percent. Estimated slide rock mass that came down from the weakness zone collapse and deposited in the shaft is presented in Table 2.

Location descriptions	Cross-section area (m2)		Length (m)	Volume (m^3)
	Theoretical	Over excavated		
Inclined shaft above 1180 m elevation	13	20	6	120
Inclined shaft below 1180 m elevation	4	6	87	520
Alimak Chamber (25 % over excavation)	16	20	16	320
Estimated slide deposit above 1184 m	-	-	20	60
Total loose volume slide rock mass				1020

Table 2 - Estimated loose volume of slide rock mass from the sinkhole (Panthi, 2012).

Total loose volume of slide material (Table 2) also consist considerable volume of glacial sediment deposit that fell down from the upper part of the sinkhole. Depth of this sediment deposit was approximately 3 m and the sinkhole diameter at surface topography was about 10 m (Fig. 2b). With an average diameter of 8 m, the total sediment volume that came into the shaft was estimated to approximately 150 m3. This gives total loose volume of the slide rock mass to approximately 870 m3. With an expansion co-efficient of 1.6, the solid volume of the rock mass that fell down from the sinkhole was thus consisted to about 540 m3. As shown in Fig. 5, the total depth of the sinkhole in the rock mass is approximately 30 m (1242 – 1212). This gives an average cross-section of the sinkhole (Fig. 2a) within the rock mass to approximately 18 m2, which indicates an average diameter of the sinkhole to about 4.75 m.

Slide rock mass filled Alimak chamber, part of the transport tunnel and lower narrow part (below elevation 1180) of the inclined shaft, which could accommodate loose slide volume of approximately 840 m3. Remaining loose volume was deposited to the upper wider part of inclined shaft above elevation 1180 m. This uppermost part of the slide material in principle must include rock mass from the upper part of the sinkhole and overburden glacial sediments from the top of the sinkhole consisting clay, silt, sand and boulders (Fig. 2 and 5). It was quite obvious that the finest clay and silt particles were settled between elevation 1185 and 1212 m. More than 30 % of glacial sediments consisted of particle size less than 1 mm and approximately 15 % sediments contains fine clay with a particle size less than 0.063 mm. This means considerable quantity of silt and fine clay deposited above elevation 1184 m (Panthi, 2012).

On 12th September 2008, stoplogs at the inlet gate of the Stutakvelven shaft were closed after excavation and removal of the glacial sediment deposit at the sinkhole top was completed (Table 1). Accordingly, rock support (shotcrete and bolts) was applied inside the upper part of the sinkhole to secure stability. Construction of an approximately 1 m high concrete protection wall was completed in three side of the sinkhole top excluding steep hill side (Fig. 4) where rock mass was exposed. The main purpose of this concrete protection wall was to increase safety against people and animals but not to block (hinder) surface runoff (drainage) into the shaft through sinkhole in case of rainfall or snow meltdown. The drainage catchment of the sinkhole is approximately 3000 m2 (Panthi, 2009 and 2012). Water level at the sinkhole was at an approximate elevation of 1141 m (Fig. 2b) before stoplogs were lowered down.

However, once inlet gate was closed, water level from the sinkhole top suddenly dropped and disappeared. After having noticed this, the leaked water through inlet gate to the shaft was measured and recorded as approximately 250 l/min. In addition to this leakage, there could be some minor inflow from open joints of the rock mass. Therefore, total discharge from the inclined shaft might have been more than 250 l/min. Hence, the drainage capacity (permeability) of the slide rock mass deposited in the shaft was greater than the discharge consisting leakage from inlet gates and groundwater inflow (if any) from the rock mass from above elevation 1184 m. Since vestre and østre Middyr lake reservoirs have maximum operation level of 1213 and 1229 m, the water level in inclined shaft most likely was below either of these two elevations depending upon existing water level in one of these two reservoirs.

Assessment on air temperature and precipitation conditions at the Stutakvelven reservoir area was carried out for the period from 15th March to 9th May (before burst flood of the 9th May 2009). This assessment was crucial since there was a need to check environment conducive for snow meltdown and rainfall. Since no air temperature and hydrological gauging stations were located directly at the Stutakvelven area (sinkhole area), records from nearby Kaldevatn gauging station having similar characteristics and similar elevation of approximately 1200 m were used.

Temperature records indicated that winter months (December 2008 and January to March 2009) were fairly stable and had temperature below zero degrees Celsius. However, temperature record between 15th March and 9th May 2009 showed that the daily mean temperature during most of month April was above zero degrees Celsius excluding 5th to 10th April. Air temperature reached its maximum to 7.6 and 6.26 degrees Celsius on 4th and 30th April, respectively. Precipitation condition in the form of snow pillow in equivalent water column is presented for the period between 15th March and 9th May 2009 (Fig. 3).

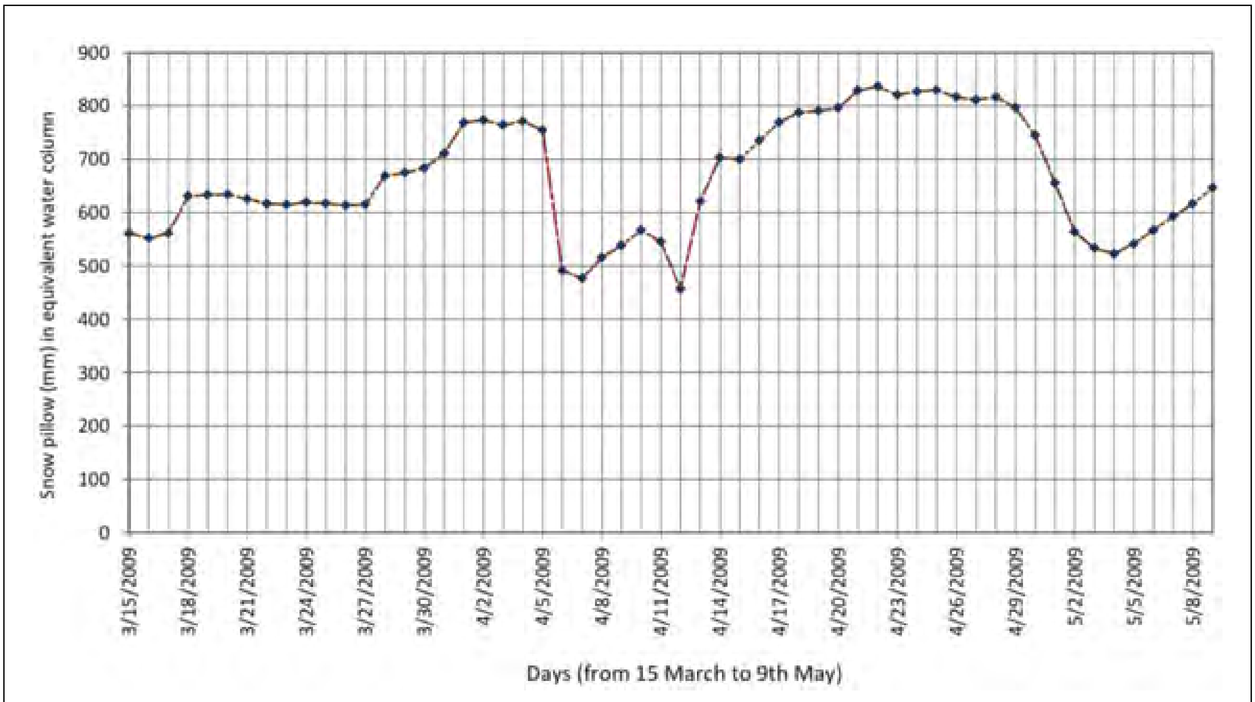

Fig. 3: Daily snow pillow in equivalent water column at the Kaldevatn from 15th March to 9th May 2009 (Panthi, 2009 and 2012).

The figure indicates that there was considerable snowfall from 27th March to 3rd April, from 12th to 22nd April, and after 5th May. Similarly, there were two periods in April with significant snow meltdown and these were the periods from 5th to 7th April and from 29th April to 3rd May 2009. These periods were the warmest period of the month and snow meltdown was quite logical.

Similarly, the headrace tunnel drainage began on 15th April, inspection on the bottom of the shaft and along the headrace tunnel was carried out on 20th April and excavation and removal of slide rock mass from the shaft bottom commenced on 22nd April 2009 (Table 1). It was a surprising coincidence that the air temperature increased considerably from the same day (22nd April) and reached its maximum to 6.28 degrees on the 30th April. Note that the mean temperature between 28th April and 2nd May was always above 3 degrees Celsius (Panthi, 2009 and Panthi, 2012).

6 DISCUSSIONS AND CONCLUSIONS

Snowmelt that took place before headrace tunnel was drained (before 15th April 2009) had little influence for scoring fine clay that was laid at the invert of the shaft between elevations 1184 and 1212 m. Because the power plant was in full operation, water level (at motionless state) inside inclined shaft was somewhere between elevations 1190 m (minimum operational reservoir level) and 1212 m (bottom of the sinkhole). However, during snow meltdown between 29th April and 3rd May 2009 (Fig. 3), the headrace tunnel had different environment since it was completely drained down. It is likely that the slide rock mass deposit in the shaft had drainage capacity (permeability) exceeding 250 l/min. before excavation and removal operation started. The water that was accumulated above elevation 1184 m during plant operation (July 2008 to April 2009) was also drained parallel with headrace tunnel drainage operation in the middle of April. As per the information received, the crew involved in the removal of slid rock mass deposit reported no change in the drainage flow that came from the shaft. They seemed very confident that the flow was not less than 250 l/min., the leaked water from stoplog gate.

Fully drained condition in the shaft and snow meltdown activity that took place at the sinkhole catchment was conducive for scoring of fine clay deposit to take place from above elevation 1184 m. One meter high concrete protection wall constructed at the top of the sinkhole (Fig. 4b) could not hinder the runoff that came down from the sinkhole catchment. As shown in Fig. 3, snow meltdown of approximately 290 mm equivalent water column took place from 29th April to 3rd May 2009. This is equivalent to approximately 870 m3 of water that could have melted and flown down to the shaft from the sinkhole catchment during four melting days. Approximately 200 mm equivalent water column of snow melted within 32 hours of the warmest period when temperature exceeded 4 degrees from 13:00 the 30th April to 20:00 of the 1st May. This meltdown only gives equivalent water volume of approximately 600 m3, which is considerable extra discharge into the inclined shaft. Hence, the combined discharge in the shaft during this period certainly exceeded 550 l/min. This could have been catalyst for scoring and transport of fine clay from the shaft invert above elevation 1184 m. The clay, silt and sand that came down reduced drainage capacity of the slide rock mass between elevation 1180 and 1184 m considerably.

Fig. 4: Clearance and concrete protection wall at the sinkhole (Courtesy: Norsk Hydro).

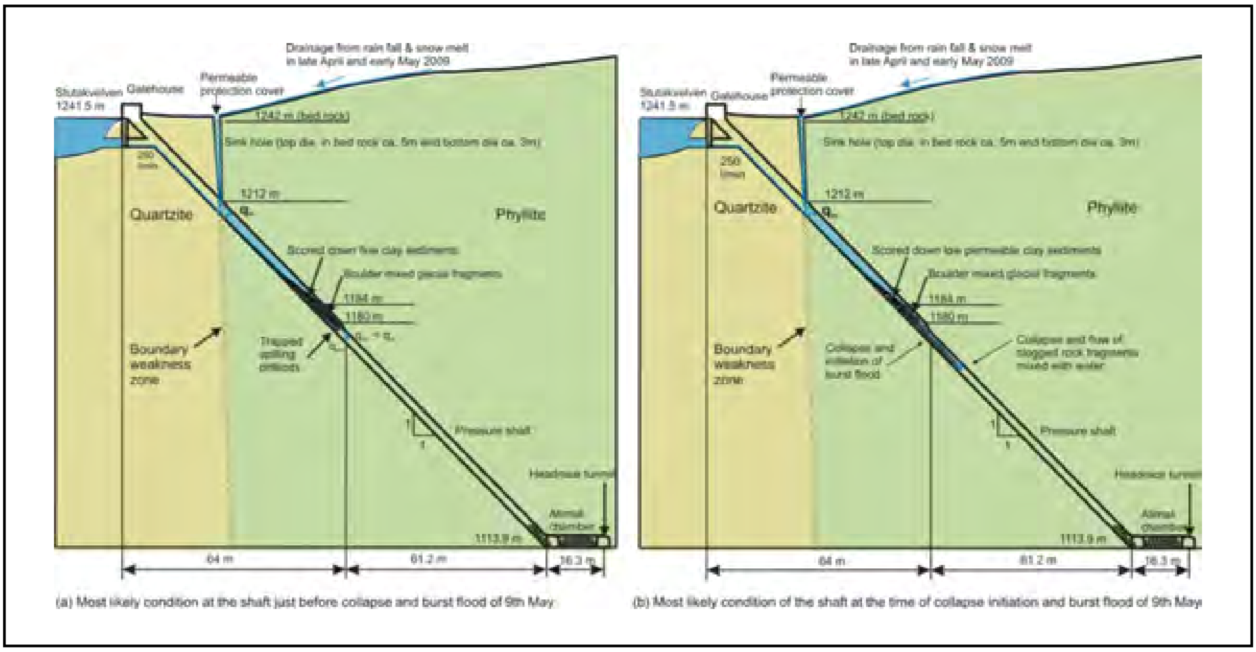

Fig. 5: Interpreted conditions of the Stutakvelven inclined shaft just before (a) and after the collapse of the clogged slide material (b) (Panthi, 2009 and 2012).

The video inspection report after the burst flood of 9th May emphasized that at about 91 m from the inlet gatehouse (top of the shaft) video inspection device had to make forceful drop indicating remains of clogged slide material. This location is in fact the end of transition between lower narrow part of the shaft with 4 m2 theoretical cross-section and upper wider part of the shaft with 13 m2 theoretical cross-section. Large sized rock boulders (exceeding 500 mm) that slide down from the sinkhole top were clearly visible in the photographs of the video inspections.

Interlocking of these large rock boulders, hindrance made by drill rods used as rock bolts during shaft excavation and detached reinforcement came down from the concrete lined section of the weakness zone are believed helped blockage of inclined shaft at this transition location between elevations 1180 and 1184 m.

It was likely that scored and transported fine clay deposits down to this transition zone were not capable to completely seal the slide deposit above 1180 m elevation due to existence of rock boulders (fragment) laying at the invert of the shaft. Hence, considerable water discharge continued draining through the slide material. This could have made it difficult for the crew working at the shaft bottom to be suspicious about. However, it is for certain that the drainage capacity of the interlocked and clogged slide material at this transition zone was reduced considerably. Similarly, significant volume of runoff from the sinkhole catchment discharged into the shaft during warmest period and accumulated above elevation 1184 m of the shaft. The pressure built up for more than a week (between 3rd to 8th May) above clogged slide deposit and weight of the clogged slide deposit above elevation 1180 m was no more capable holding itself at the 47 degrees inclined shaft. Therefore, sudden collapse of the slide debris clogged above elevation 1180 through the empty shaft below elevation 1180 m was eminent (Fig. 5b).

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Appendix I.

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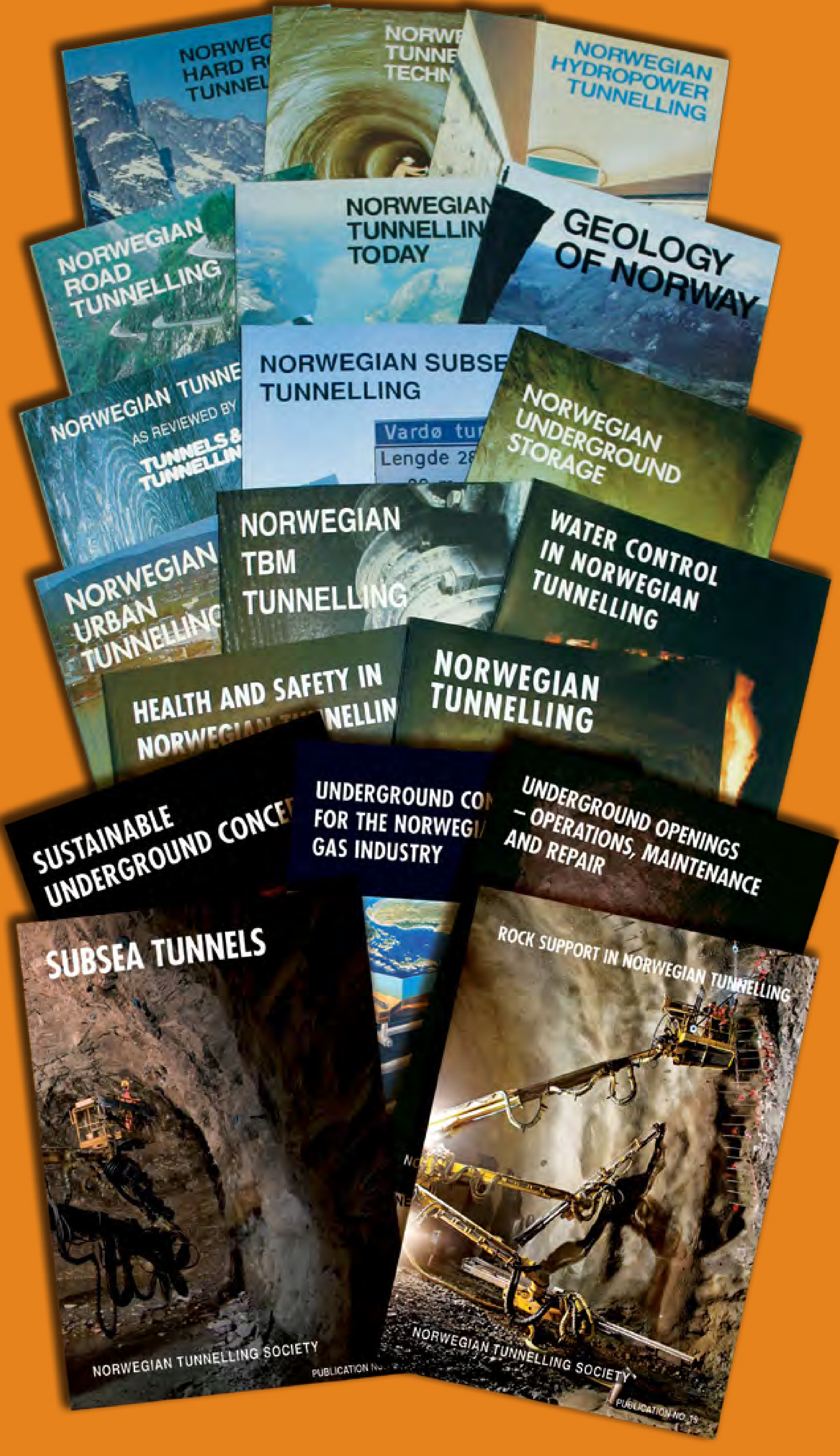
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