

MUIR WOOD LECTURE 2010

TUNNELS AND UNDERGROUND WORKS FOR HYDROPOWER PROJECTS

Lessons learned in home country and from projects worldwide.

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MUIR WOOD LECTURE 2010

TUNNELS AND UNDERGROUND WORKS FOR HYDROPOWER PROJECTS

Lessons learned in home country and from projects worldwide.

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Since ITA was established 36 years ago there has, on a worldwide scale, been considerable development in tunnelling technology and an increase in the use of the underground for various purposes. This has had an important influence not least on the hydropower industry. All over the world it is now common to locate powerhouses underground if possible, and longer and longer tunnels are being excavated to convey more and more the water to the turbines.

In the author's home country 99% of all electricity is generated from hydropower. More than 200 powerhouses are now located underground in Norway and the total length of tunnels connected to the powerhouses is in the order 4000 km. The paper describes the design of the cost saving unlined high pressure tunnels and shafts. Also the technology behind the unlined air cushion replacing the surge chamber is described and the potential for applying this technology for underground gas storage is shown. The use of heavy rock anchors for roof stabilization in underground powerhouses is discussed based on theoretical studies and real cases. Selected examples of stability problems in tunnels caused by slaking basalts, friable sandstones and swelling shales are described.

The concluding remarks demonstrate that with a good understanding of rock masses and their behaviour, there are considerable advantages in using the underground for hydropower projects as well as for other projects. Structures should be made safe enough for their purpose, but overly conservative support should be avoided as this adds unnecessary costs to the projects.

1 >> INTRODUCTION

1.1. SIR ALAN MUIR WOOD – A LONG TERM FRIEND

First of all: I was very pleased when I learned that the Executive Council of ITA had decided to establish an annual lecture to honour and remember our first president and life time honorary president, Sir Alan Muir Wood. Secondly: I was very honoured and grateful when I one year ago was invited to give the first Muir Wood Lecture. I assume that one reason for the invitation was my long time friendship with Sir Alan. I therefore feel it natural to summarize some of this as an introduction to my lecture.

The first time I met Alan Muir Wood was at the inaugural meeting of the International Tunnelling Association in Oslo in May 1974. I remember him sitting at the head of the table leading the negotiations in a very quiet and orderly way. If things became a little tense or tricky, he just sucked his pipe very slowly a couple of extra times and then came up with a suggestion that was almost always immediately accepted by all participants. As far as I remember, there was no discussion about who should be the first president of this new association, - Alan was the obvious candidate.

Since the establishing of ITA, with only two or three exceptions, we met every year at the General Assemblies and the combined congresses or symposia as it was called in the early days. And during my 13 years in the Executive Council we met several times per year in different places around the world. So the situation developed in a natural way from a professional relationship to a personal friendship, - which also included our wives. Thus they were guests in our home and

we in their home. My last pictures from ITA meetings that include Alan were taken in Milan in 2001 and is shown in Figure 1.

In February last year I not only lost a highly respected colleague, but also a dear friend. "Father of modern tunnelling" was a term which was used in the many civil engineering and tunnelling related journals and websites in the weeks after his death. In the many obituaries Sir Alan's professional life was described in some detail and I will not repeat them, only refer you to the ITA website. Most of the tunnel projects he was involved in were for natural reasons related to communication like railway, metro and road tunnels, - one example being the famous Channel Tunnel. As major part of my tunnelling experience has been related to hydropower, we had some professional contact on a couple of projects, but only really met professionally on one project, - the Strategic Sewage Disposal Scheme in Hong Kong. For this project we met in the arbitration court as expert witnesses, - Sir Alan for the Contractor and I for the Client.

My last contact with Sir Alan was only few weeks before he died when we exchanged e-mails concerning his participation in the annual British Tunnelling Society meeting in December 2009 to debate tunnelling contracts. The debate is reported in great detail in *Tunnels&Tunnelling*, March 2009. This issue also contains a comprehensive obituary.

1.2. THE INTENTION OF THIS LECTURE AND LAY OUT OF THIS PAPER.

After this special introduction I will turn to the tunnelling and underground space

subject that has been a major part of my professional life, - namely tunnels and underground structures related to the development of hydropower. I will start with the development in my home country of Norway and will then add some experience learned from hydropower projects in several other countries where I have been involved. As my profession is geological engineering, the cases I will describe and discuss are, of course, related to various problems caused by the types of rock and the geological and topographical conditions. Inspired by Sir Alan's devotion to all aspects of tunnelling and to use the underground for all it is worth, it is my aim to try to point out how lessons learned during the development of hydropower projects may be applied in other aspects of tunnelling and the use of the underground, and vice versa, how lessons learned from other projects may be useful for hydropower projects.

The first and probably the most important lessons, I have, of course, learned in my own home country. It is thus natural to start this lecture/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 is a technology that can be used for storage of compressed gas and will be discussed in Chapter 3. Having been involved in different ways on hydropower projects in many countries around the world, I will also include some lessons learned from some selected projects and express some opinions about the planning and design. Chapter 4 is devoted to roof stability in large power houses, and references are made to experience from the support of the largest public mountain hall in the world. In Chapter 5 some selected samples of problems caused by special types of rock masses and stress conditions in water conveying tunnels are discussed.



Figure 1. Pictures taken at the ITA/World Tunnel Congress in Milan in 2001

2 >> THE DEVELOPMENT OF UNDERGROUND HYDROPOWER PROJECT 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 125 TWh of electric energy in Norway is generated from hydropower. Figure 2 shows the installed production capacity of Norwegian hydroelectric power stations. It is interesting to note that, since 1950, underground powerhouses 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 2 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,



Figure 3. The Aura underground hydropower station, commissioned in 1952

(Edvardsson & Broch, 2002). Example of an underground powerhouse from the early 1950s is shown in Figure 3. 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 4.



Figure 4. Åna-Sira (up) and Tafjord K-5 (down) underground powerhouses

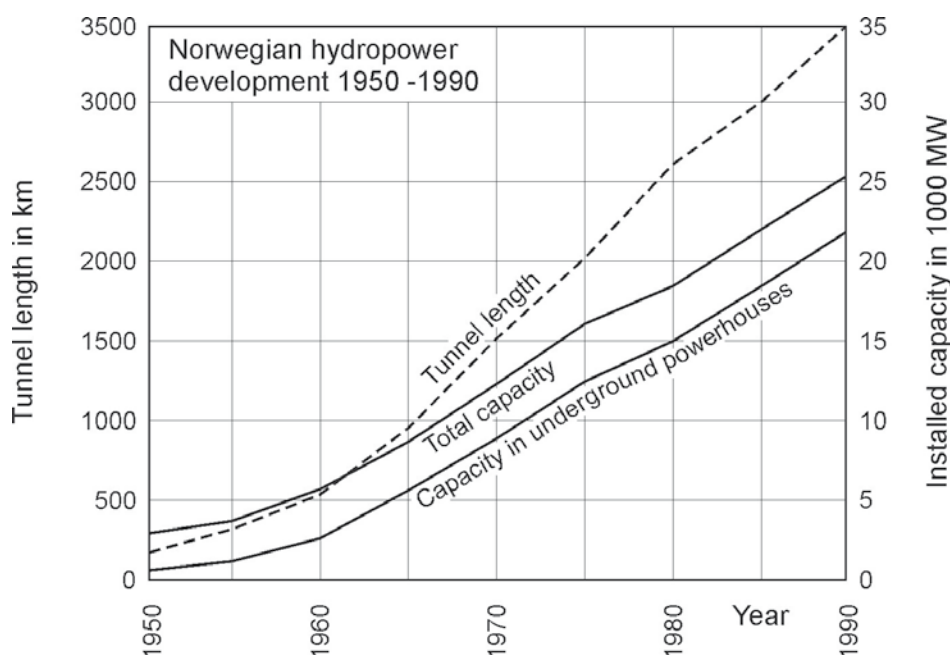


Figure 2. The development of Norwegian hydroelectric power production capacity and the accumulated length of tunnels excavated for the period 1950 -1990.

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)

2 >> THE DEVELOPMENT OF UNDERGROUND HYDROPOWER PROJECT IN NORWAY

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 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 5. 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 5 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 almost 100 unlined high-pressure shafts or tunnels with water heads above 150 m are successfully operating in Norway, the highest head being almost 1000 m. Figure 6 clearly demonstrates that increasing water heads reflect an increasing confidence in unlined pressure shafts.

The bottom sketch in Figure 5 shows how this new design philosophy influenced the general layout of a hydropower project. The steeply inclined pressure shaft, normally at 45°, 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 hydraulic 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. This will be discussed in some detail in the next chapter.

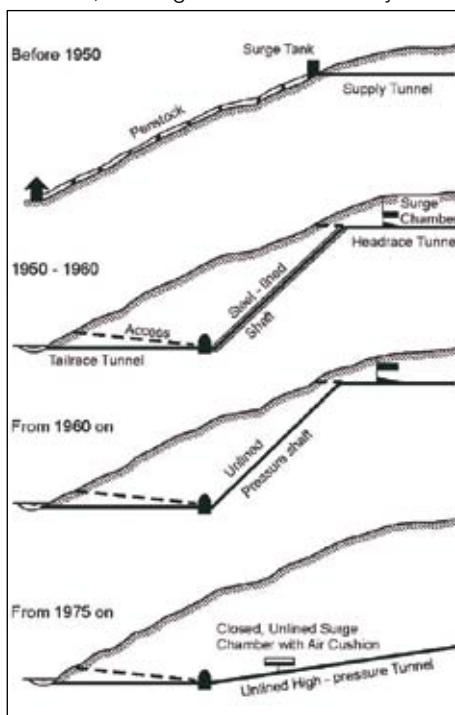


Figure 5. The development of the general layout of hydroelectric plants in Norway

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).

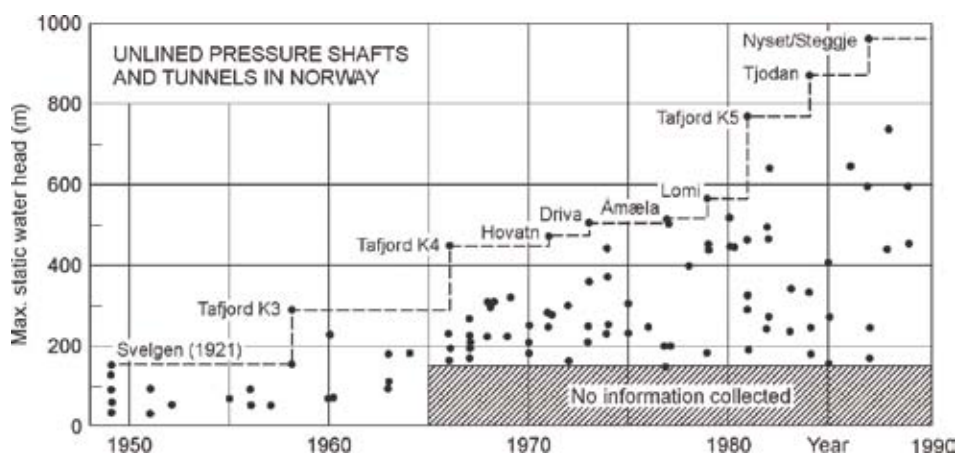


Figure 6. The development of unlined pressure shafts and tunnels in Norway.

3 >> APPLYING THE EXPERIENCE FROM AIR CUSHIONS TO UNDERGROUND GAS STORAGE CAVERNS

The development of underground storage of gas in unlined caverns in Norway took place in parallel with the development in the other Nordic countries. The Norwegian experience base included unlined pressure shafts in hydropower projects with very high pressures (up to 1000m water head), utilising the rock stress to prevent hydraulic splitting. This was followed by the innovative air cushion surge chambers that used the hydrostatic head of the groundwater in the rock fissures as confinement.

The first containment principle for storing of air or gas in unlined rock caverns is that any internal storage pressure must be sustained by the minimum in-situ rock stress 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 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 requires 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 gas 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.

In addition to the above design principles, temperature effects must be considered. Gas can be stored at natural rock temperatures under high pressure, or chilled at lower pressures. The latter storage mode causes a series of effects, such as contraction of the

rock and concrete plugs, opening of existing joints or creation of new cracks etc. Carefully planned procedures for gradually cooling down the caverns to operation temperatures are of utmost importance. Unexpected performance is often connected to the effects of sub-zero temperatures.

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 2,000m³ to 120,000m³ 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 5 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

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

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

3 >> APPLYING THE EXPERIENCE FROM AIR CUSHIONS TO UNDERGROUND GAS STORAGE CAVERNS

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 7. 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 400Nm³/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 gas caverns in the form of so-called air cushions confirms (Blindheim et al., 2004) the following:

- 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 gas 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.

In Norway, feasibility studies for LNG storage as compressed gas at natural rock temperature have been performed. One potential project was an underground buffer storage of 100 millions m³ (total volume of caverns ~1 million m³) at a depth of 1000m in gneiss utilising a water curtain. This storage would feed a 1600MW underground power-plant at Vikna on the Mid Norwegian coast, based on gas from the offshore Haltenbanken petroleum field (Thidemann, 1989). The project was believed to be technically and economically feasible, but was not realised as the gas was used otherwise.

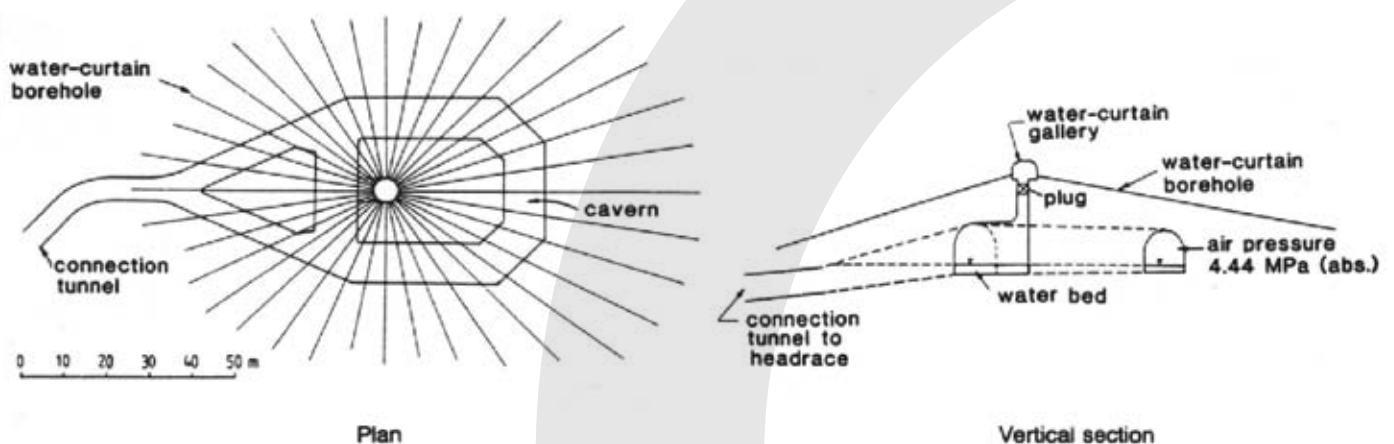


Figure 7. Air cushion surge chamber with water infiltration curtain at Torpa (Kjølberg, 1989)

4 >>> ROOF STABILITY OF LARGE UNDERGROUND POWERHOUSE

The large **Xiaolangdi Multipurpose Dam Project** on the Yellow River in P.R. China was completed ten years ago. The underground powerhouse has a length of 250 m, a span of 26 m and a maximum height of 58 m and is thus among the largest powerhouses in the world. The installed capacity is 1800 MW.

The rocks in the area are sandstones and shales from the Triassic era. They are fairly flat lying with a gentle dip of approximately 10 degrees in the downstream direction. The sandstones, which are the dominating rocks in the powerhouse area, are strong and of good quality, - Q = 8-12, RMR = 59-66. They are, however, interbedded with thin seams of clay, so-called clay intercalations, which often is the case with young sandstones. The rock overburden is 85 to 115 m, and the vertical to horizontal in-situ stress ratio is about 0.8.

The crown of the Powerhouse was originally designed with 6 and 8 m long rock bolts at 1.5 m spacing and a 20 cm thick reinforced shotcrete layer, see Figure 8. After having inspected an exploratory gallery along the whole length of the crown without seeing a single fault, the author felt that this was an adequate support. Based on results from numerical model the Designer decided to extend the length of the bolts to 8 and 12 m. New models, where the horizontal clay intercalations were included, gave a poor result such that it was decided to install 345 25m long tendon anchors with a capacity of 1500 kN in the roof as shown in Figure 9. This caused a delay in the construction of 4-5 months and added considerable costs to the project.

As a member of the Panel of Experts the author strongly objected the installation of these tendon anchors and particularly the fact that they were stressed with a force of 1500 kN (150 tons). First of all, based on the observation of the rock mass, the author was convinced that the original support was more than adequate, and secondly was afraid that by tensioning these tendon anchors, the natural arching effect in the roof may be disturbed. Thus, a Ph.D.-study was initiated to model the influence that tensioned tendon anchors may have on the stability of large cavern roofs in general and on the roof of the Xiaolangdi powerhouse in particular, Huang (2001).

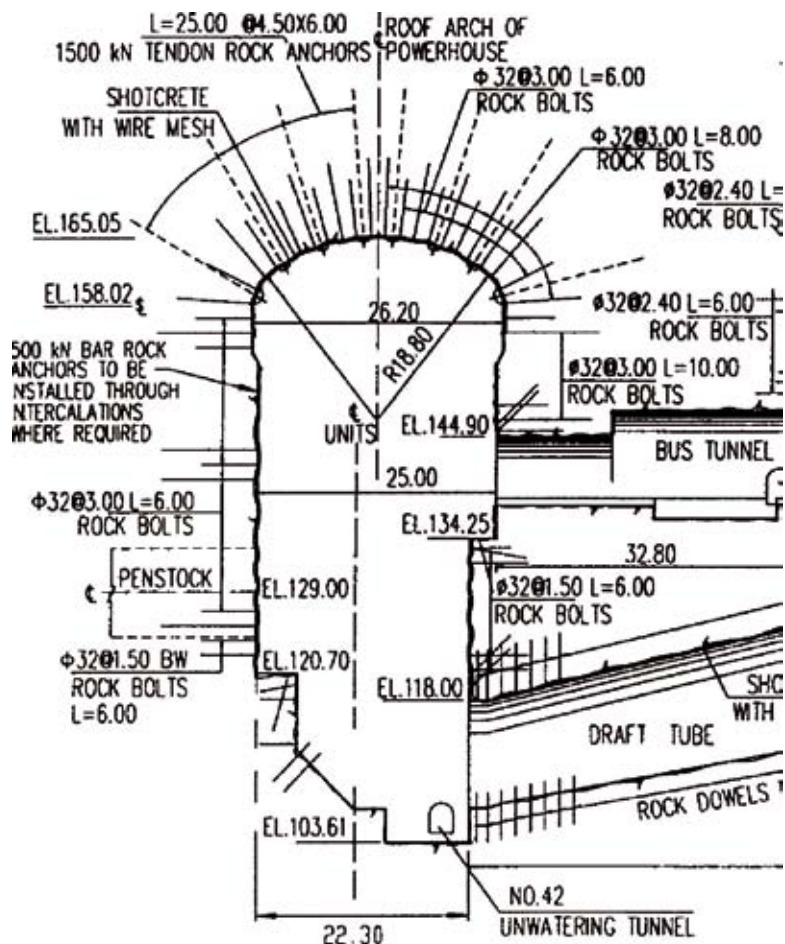


Figure 8. Powerhouse machine hall with rock reinforcement (from Wang and Solymar 1997).



Figure 9. Tendon anchors in the roof of the Xiaolangdi underground powerhouse

In the stability study of the Xiaolangdi powerhouse cavern carried out by Huang as part of his Ph.D., the results from the DIANA and UDEC modeling were found to be close to the monitored data (Huang 2001). Conclusions were thus drawn as follows, Huang et al. (2002):

The displacements in the walls are greater than in the roof. The arch shaped roof of the powerhouse fits the elliptical rock ring surrounding the cavern and thus leaves a limited plastic or tensile zone of about 5 m thickness. A larger plastic or tensile zone extends to a maximum depth of about 20 m in the walls. The plastic or tensile zone is in coordination with the major stress variation.

A natural crown arch with a thickness of about 5 m likely would form in the Xiaolangdi powerhouse crown due to favourable horizontal in-situ stress and the good quality of the rock mass. The visualization of the principal stress variation, strength/stress contours and the stress state in the rock mass showed evidence of such a crown arch forming.

4 >> ROOF STABILITY OF LARGE UNDERGROUND POWERHOUSE

Grouted rock bolts contribute to a reduction of roof subsidence and wall displacement and control the closure of the clay intercalations. Only small loads are built up within limited lengths of the bolts from the surface in the roof. Only minor stress changes were observed in the roof during benching. This may indicate that a self-supported natural crown arch is established. Load built up along the whole length of rock bolts in walls means long rock bolts are needed there.

After systematic rock bolting of a natural crown arch, the tensioned cables had only a marginal effect on reducing the roof settlement. The cables in the crown can not provide immediate shear resistance to rock joints/clay intercalations. Furthermore, the modelling suggested that the tensioned cables may actually increase wall displacements as it triggers the mechanism of the coupling of roof deformation with wall deformation for a cavern with high walls. The upward support force induced by the extra support pressure on the roof may rotate and move rock blocks in a way that counteracts the forming of an arch and thus induces sliding along joints and increases tensile stress, which turns out contrary to the expectations of the Designer. It may thus be concluded that to stabilize an intrinsically stable crown arch of a large span underground excavation, passive rock bolts

may be more effective than tensioned cable anchors. Such anchors may, in fact, have a negative influence on the arch stability.

One example proving that strong rock anchors in the roof are not needed if the rock mass is of reasonable quality and the cavern is oriented correctly in relation to the stress situation, is the **Gjøvik Olympic Mountain Hall** in Norway, (Broch et al, 1996). This hall (or cavern) has a span of 61 m, a length of 91 m and a maximum height of 25 m. The rock cover varies between 25 m and 50 m, that is considerably less than the span. It was primarily built for ice hockey games, and is by far the largest man-made cavern for public use in the world. The rock at the site is Precambrian gneiss and the Q-value is typically 30 for the best and 1 for the poorest quality of the rock mass with 12 as an average Q-value.

Of particular interest to rock mechanics and civil engineers is, of course, the large span, as the span is normally regarded as the critical dimension of a cavern. The cross section in Figure 10 shows the excavation sequences and the roof bolts. The roof was excavated to its full width from a centre adit by side stopping. The final height was then blasted by benching in two steps.

The systematic permanent bolting consists of alternate rows of fully grouted 6 m rebar bolts and 12 m twin steel strand cables in a 2.5 m



Figure 11. The Gjøvik Mountain Hall during excavation, - span 61m

x 2.5 m pattern. The former have a diameter of 25 mm and capacity of 22 tonnes, while the latter have a diameter of 12.5 mm and a capacity (for each strand) of 16.7 tonnes at yield. In general the 6 m bolts were placed before the 12 m cables in both the 10 m span pilot tunnel, in the primary 38 m span top heading and in the 6 m span abutment tunnels. After blasting of the remaining rib pillars between the abutment tunnels and the top heading, the final bolts were put in.

50 mm of fibre reinforced shotcrete was sprayed first, followed by another 50 mm to make the final 100 mm thickness. The shotcreting was by the wet mix process, using 50 kg/m³ of 25 mm long EE steel fibres, and with a concrete quality of 35 MPa. This is in accordance with modern Norwegian practice, where the wet mix process is virtually the only technique used today.

It was an interesting observation that the instrumented, fully grouted rockbolts in most cases showed very low to zero load, and furthermore that when load build-up occurred, it was in the lower end of the bolt, i.e. close to the cavern surface. This clearly indicates that the roof is an over all self-supporting structure because of the arching effect and the favourable horizontal stresses. Observations have also indicated that the need for the 12 m long grouted bolts is questionable. The author tend to believe that in a rock mass like in Gjøvik, grouted rockbolts with lengths of 3 - 4 m set in a pattern of 2 - 3 m together with 100 mm steelfibre reinforced shotcrete will do the supporting job, - even for spans of 50 - 60 m.

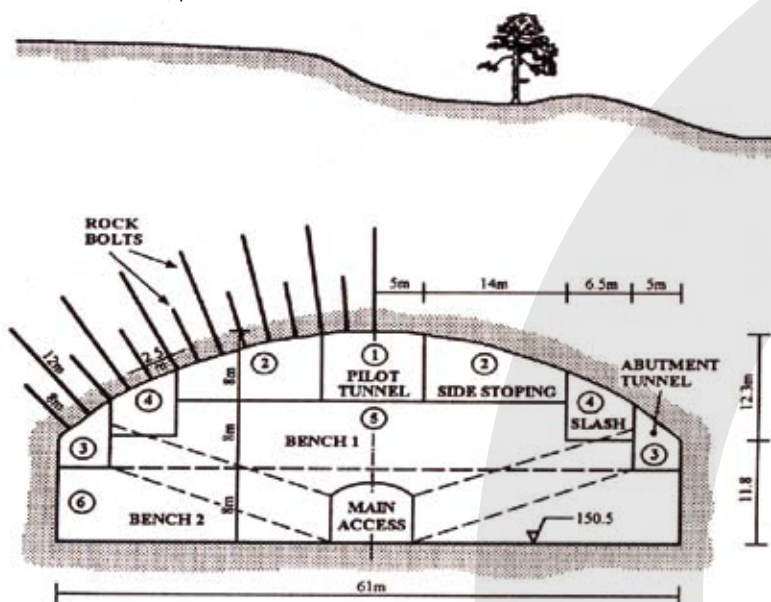


Fig.10 Excavation sequences and roof bolts for the Gjøvik Olympic Mountain Hall.
(Notice the low rock overburden)

5 >> LESSONS LEARNED FROM WATER TUNNELS AROUND THE WORLD

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 around the world will be discussed.

5.1. TUNNELLING IN "CRAZING" BASALTIC ROCKS

Figure 12 shows that the 45 km long headrace tunnel for the Muela hydropower station, also referred to as the Transfer tunnel, goes through basalts for its entire length, Broch, (1996). 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 disintegrate, see Figure 13.

In addition, active zeolites, in particular laumontite, caused fine fracturing in weak,



Figure 13. "Crazing" due to weathering in amygdaloidal basalts in the Transfer tunnel in Lesotho.

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.

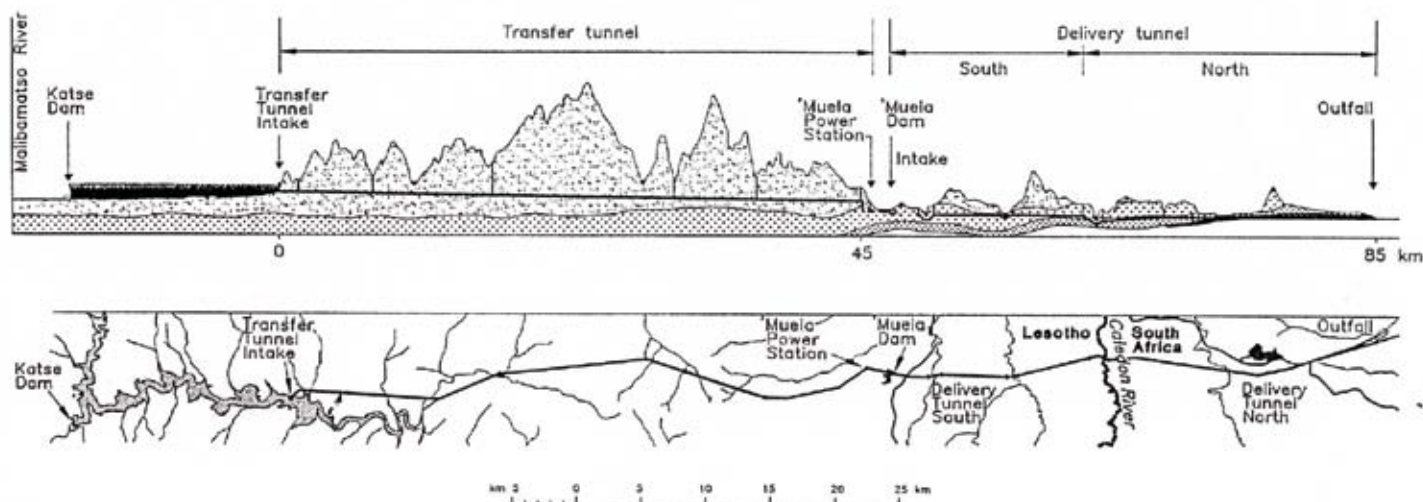


Figure 12. Plan and longitudinal section of the Lesotho Highlands Water Project. Upper geological formation: basalt, intermediate formation: Clarens sandstone.

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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 a 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.

5.2. TUNNELLING IN FRIABLE SANDSTONE.

5.2.1. CASE 1 – Guavio hydropower project

On November 7, 1983 the excavation from the downstream adit of the 5.25 km long, 65 m² tailrace tunnel for the **Guavio Hydropower Project** (8 x 200 MW) in Colombia, had reached Station K4+ 567 as shown in Figure 4 , 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 12. 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/

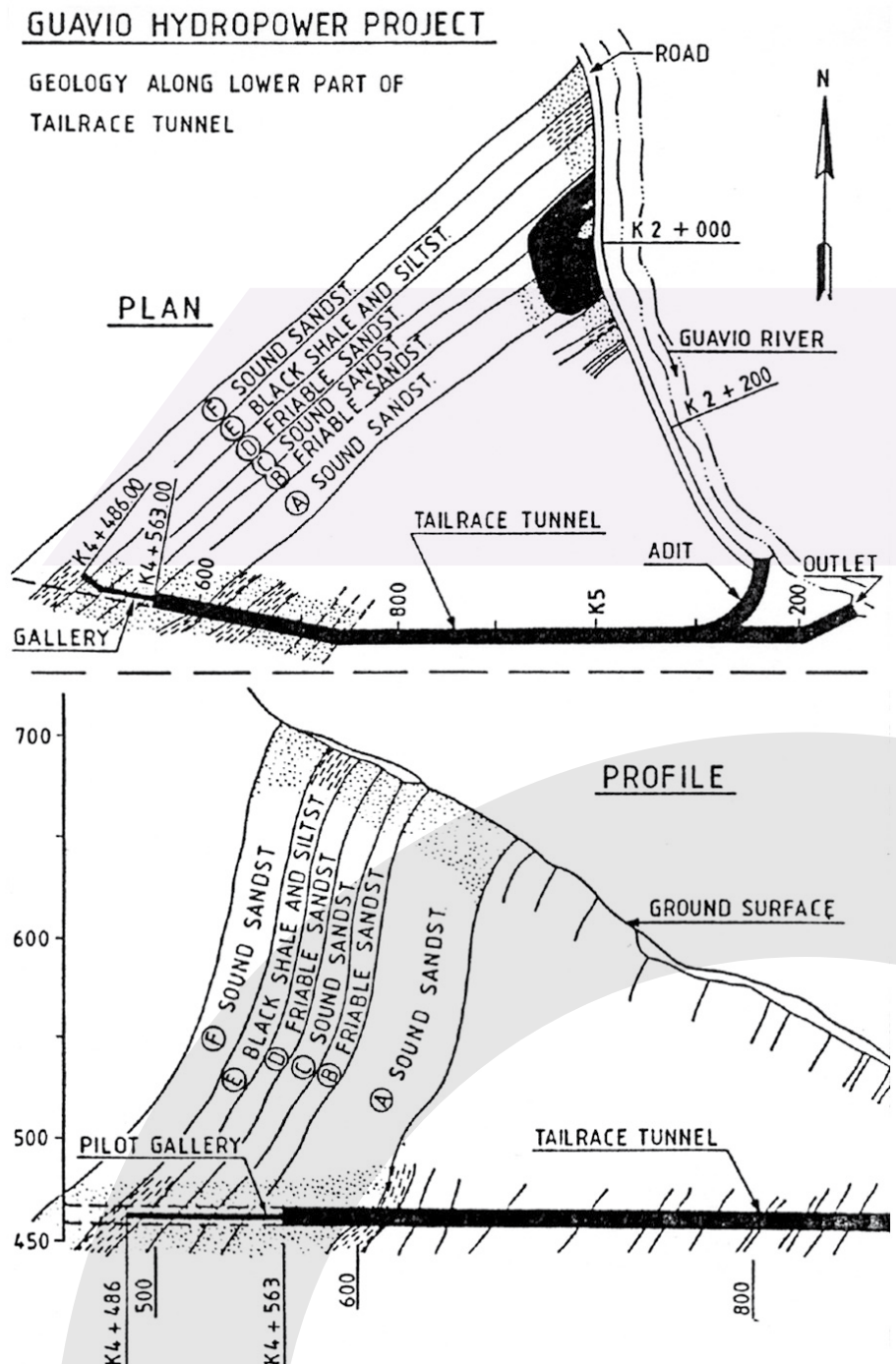


Figure 12 Geology along the lower part of the Guavio tailrace tunnel.

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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 m³ 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.

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 m³ 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 m³ 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 13. 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 were 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 m³ of grout used for the pilot tunnel 2250 m³ of cement/bentonite and 6250 m³ of silicate were used for the radial grouting. This gives 11500 m³ grout for a 77 m long tunnel, or 150 m³ 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.

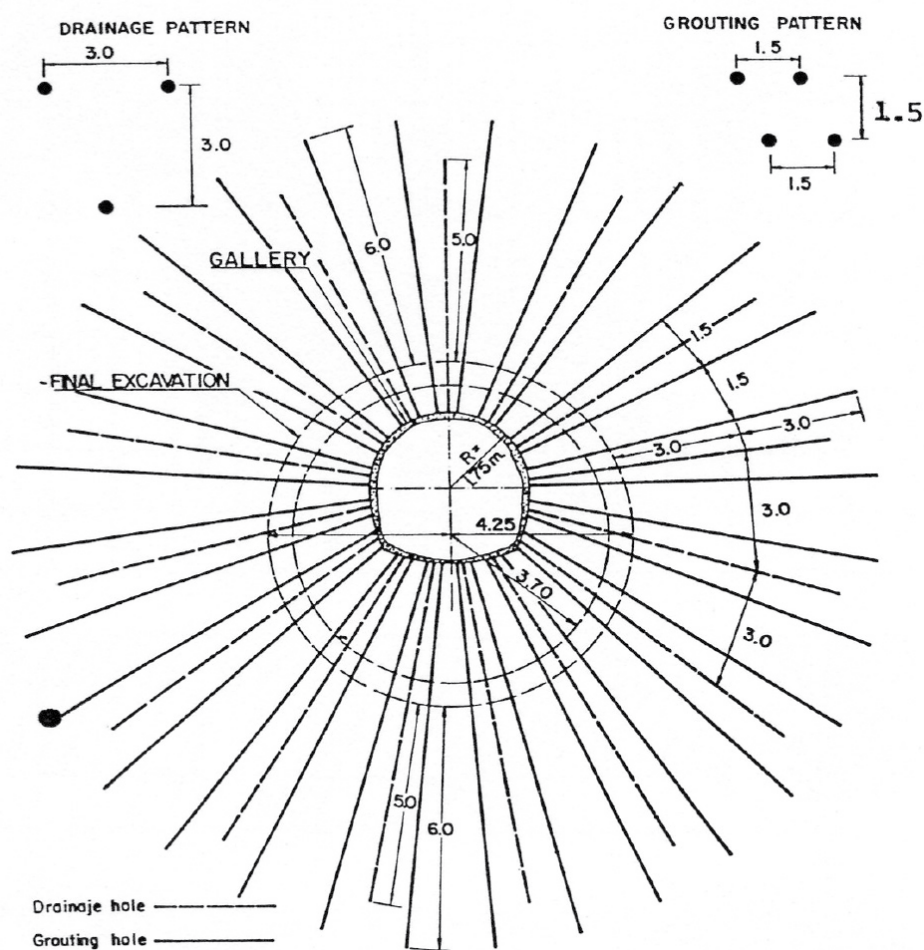


Figure 13 Pattern for the radial grouting and drainage for enlargement of the pilot tunnel of the Guavio tailrace tunnel.

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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.

5.2.2 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, - see Figure 12. 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 oversteering 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 14.

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 oversteering always occurred where the ratio was lower than 2.5. There were indications that time dependent oversteering 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.

5.3 TUNNEL COLLAPSES in shales – the Chingaza Project in Colombia

In the early 1970s the need for fresh water in the city of Bogota in Colombia was increasing, and the Chingaza project was started. After a construction period of approximately 10 years the project was completed in 1982. 38km of tunnel had been excavated through different types of sedimentary rock of Cretaceous and Tertiary age. The longest tunnel, which is 28.4km

and has a diameter of 3.7m, was filled with water for the first time in September 1983.

A short time after the tunnel had been put in operation, the capacity started decreasing. In the beginning of January 1984, after four months of operation, it was obvious that the tunnel was about to become blocked. The water flow had almost totally stopped, and the tunnel was taken out of operation. After a complicated clean-up operation, more than 40 fall-outs and slides from the tunnel periphery were observed. Some of them had completely filled up parts of the tunnel. Figure 15 shows the distribution of slides in the upstream part of the tunnel.

In Figure 16 cross sections of the upper 15 slides are shown. The majority of the fall-outs occurred in the Fomeque-formation. This formation is dominated by shales with interbedded siltstones and limestones. The rocks are partly folded and sheared along the layers. In some places the deformation has resulted in slicken-sided joints, and in other places crushing and fracturing.

Of the 4250m tunnel through the Fomeque-formation, 2000m was lined with a shotcrete layer of 5 to 15cm thickness, 2200m was lined with circular unreinforced concrete, and 64m was covered with four short steel linings. All the fall-outs and slides occurred in the shotcrete lined parts, despite the fact that the shotcrete layer had been applied several years before the tunnel was filled with water, and no clear signs of weakening or cracking had been observed.

In a preliminary study several hypotheses regarding the reasons for the fall-outs in the Fomeque-formation were discussed. In the actual area the tunnel had been ventilated for several years before it was taken into operation. Since the fall-outs only occurred after the tunnel had been filled with water, it seemed obvious that wetting of the rocks had to be an important factor affecting the stability. Water causes a reduction in the rock strength, but it seemed unlikely that this alone could explain the cracking of the shotcrete and the many fall-outs and slides from the tunnel periphery.



Figure 14 “Dog-earing” in sandstone due to high vertical stresses in the Delivery Tunnel in Lesotho

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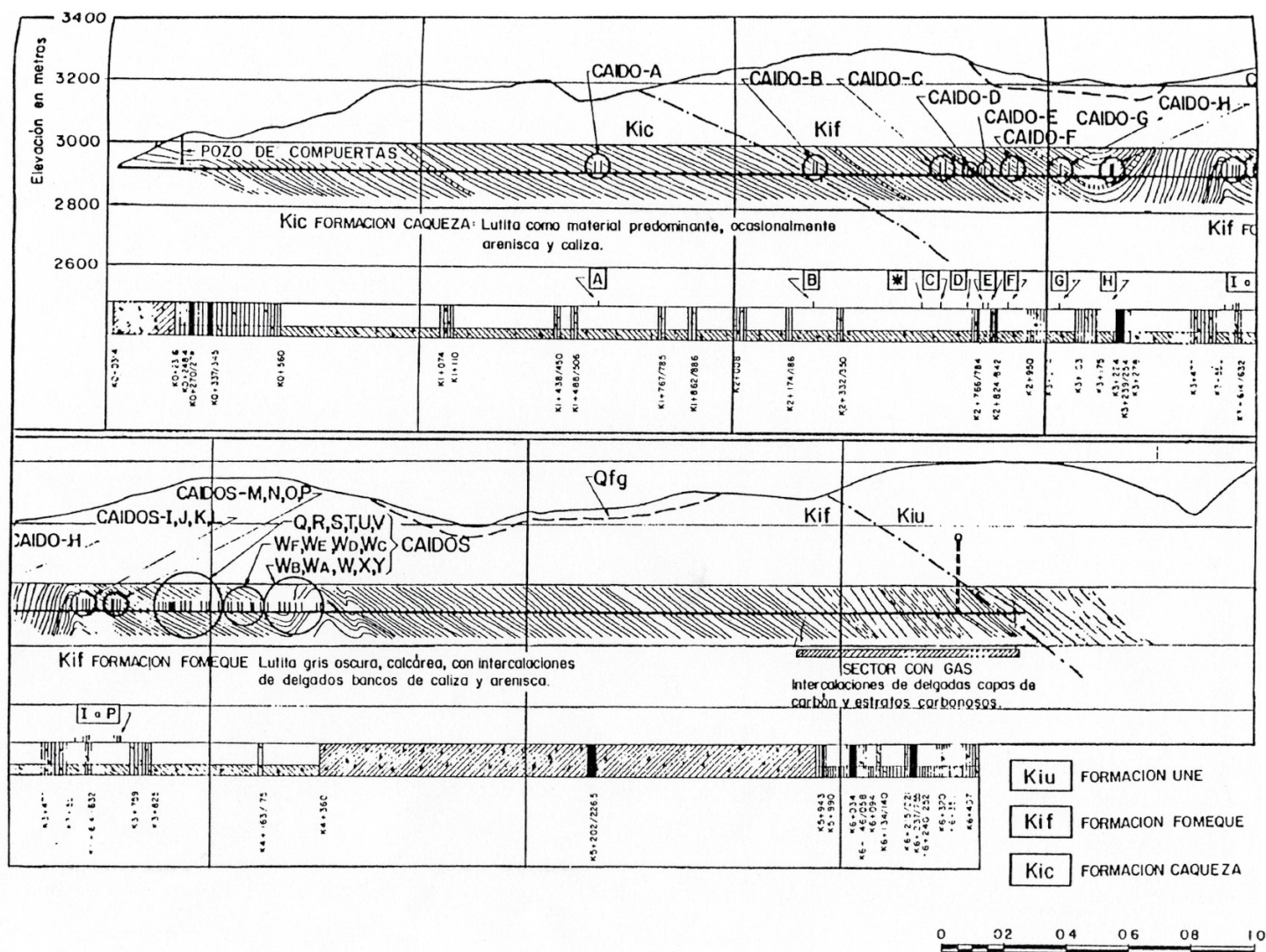


Figure 15 Geologic profile along the Chingaza tunnel from the intake down to the El Diamante ventilation shaft. Positions of the fall-outs and slides (CAIDO) are shown.

In order to understand what had happened rock samples from the slides were collected and analysed with respect to mineralogy and texture and swellability. The results from these analyses are discussed in detail in Brattli and Broch (1995), and the interested reader is referred to that paper. In this paper only the conclusions are presented. Based on observations in the tunnel, evaluation of the swelling tests and the observed reactions of the rocks when submerged in water as well as the

mineralogical/petrographical analyses, the following explanation of the slides in the Chingaza tunnel is suggested:

i) Failures occurred in the Fomeque-formation where the tunnel periphery was lined with shotcrete, normally of 5 to 15cm thickness. The tunnel had in the actual area been ventilated for several years before being filled with water. As a shotcrete lining is not water or air tight, it is believed that an

extensive draining and drying-out of the rock masses along the tunnel periphery took place.

ii) The drying-out process gave rise to local contraction stresses in the rocks near the tunnel periphery. This resulted in a heavy microfissuring along the bedding and shear planes of the shales, while for the stronger and more massive siltstone very few new fissures were created, if any.

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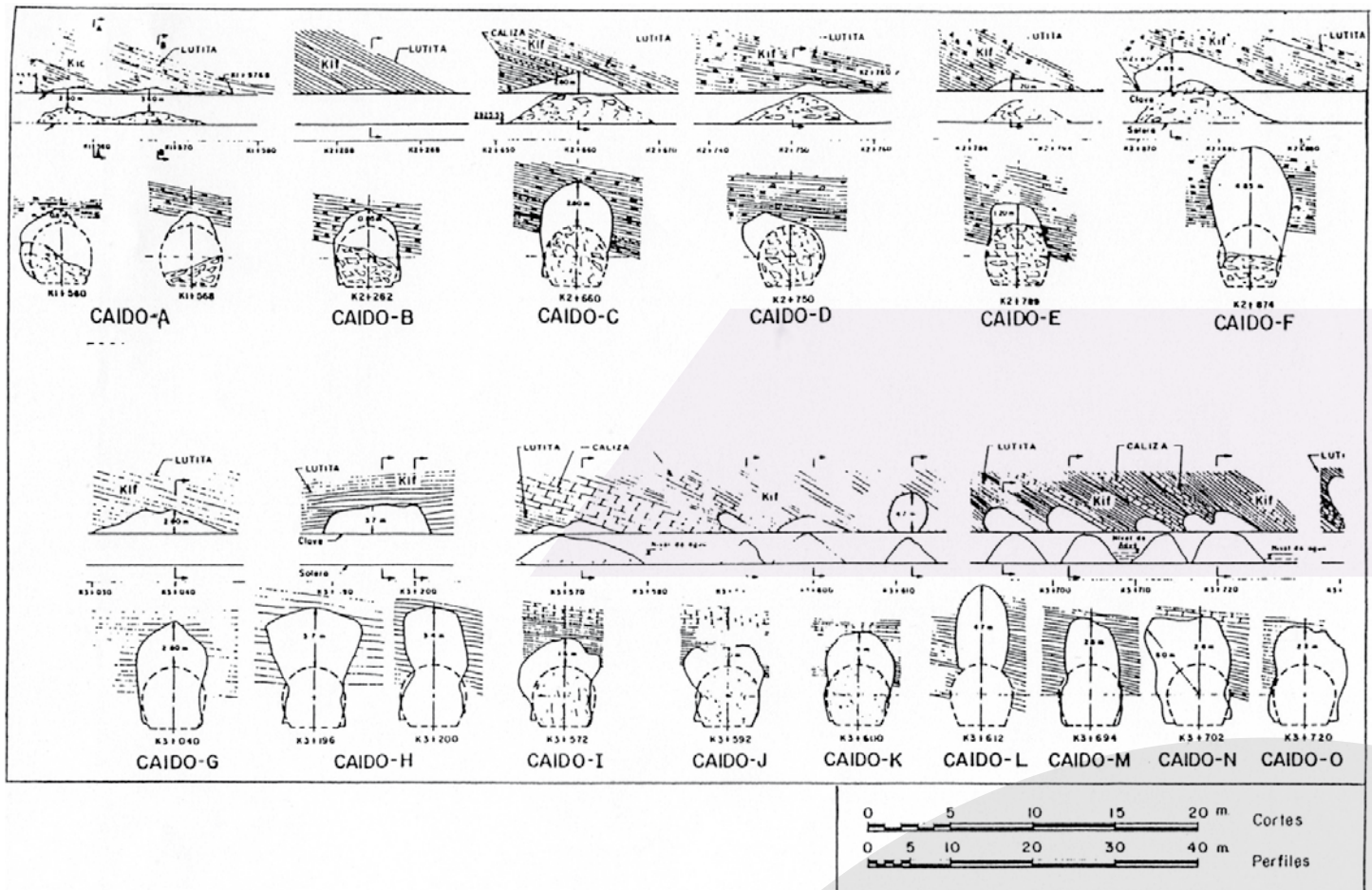


Figure 16 Cross sections of the slides A to O in the Chingaza tunnel

- iii) The microfissuring of the shale not only reduced the general strength of these rocks, but more importantly caused a considerable increase in the permeability and the exposed rock surface area. Some time (a few days or weeks) after the tunnel had been filled with water, the water would have penetrated through the shotcrete and entered into the numerous fissures in the shale.
- iv) The increased water content will in itself cause a reduction in the strength of the rock mass. It is, however, unlikely that this alone can explain the slides, fall-outs and cracking of the shotcrete. Stronger forces seem necessary to initiate this process.
- v) When water enters into all the new microfissures in the shales, the exposed and partly dehydrated illite/smectite minerals will adsorb water and start swelling. The swelling pressure measured on discs of the shale in the laboratory clearly indicate pressures of a magnitude that easily cracks a normal shotcrete lining and thus initiates the fall-outs and slides.
- vi) The swelling of the illite/smectite minerals in the strongly fissured shale is therefore believed to be the initiating factor causing most of the fall-outs and slides in the Chingaza tunnel. The problems have been enhanced by the reduction in rock strength as a result of the many cracks and fissures caused by the draining and drying of the rocks during the long construction period.

6 >> 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. One problem with hydropower tunnels is that during construction when they are filled with air, they may give the impression of being stable. When filled with water, the environmental situation for the rock mass in the periphery of the tunnel is changed, and the strength of some rocks may be radically reduced. Also gouge material in weakness zones may be activated and cause major fall outs if not properly supported. 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 up to 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.

And finally: The design and operation of the unlined air cushion surge chambers should be regarded as unique full scale tests. This experience may be applied for other aspects of the use of the underground, - one example being the storage of gas. Only the fantasy will set limits to other examples of how this technology may be utilized.

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