USE OF THE ROCK MASS AS A STRUCTURAL COMPONENT IN UNDERGROUND PROJECTS

Erik Tøndevold
Norwegian Water Resources and Electricity Board
Oslo
Norway

Subterranean location gives an additional degree of freedom compared to location on the surface. From far back engineers have understood this basic fact, sometimes digging tunnels instead of using canals or pipes. However, when power development started a hundred years ago, although tunnels were sometimes used, power stations etc. were located on the surface.
Gradually the rock blasting technology was improved, and more and more constructions were located underground. The additional degree of freedom resulted in shorter waterways and other advantages.
This development started by using surface designs in a «hole in the rock». The rock was only a nuisance to be got rid of. (Analogy: the first cars were horse wagons, and the only change was to replace the horse by a motor).
Gradually engineers understood that the rock was not an «enemy». It was really a «friend». Rock can be utilized as an integral part of the constructions. The following articles describe examples of such integral design: unlined high-pressure tunnels and shafts, air cushion surge chamber and a special crane beam.
A typical element of many Norwegian schemes is to utilize deep lakes as reservoirs. The traditional way of creating a reservoir is to build a dam. However, in many lakes nature itself has a «dam» at the outlet. By piercing this «dam» one obtains a reservoir below the lake's natural surface.
Most engineers are conservative (« you don't repair something that is all right»). In general it is sound practice to use designs that have merits and from which all the snags have been removed. On the other hand, some significant improvements are still possible. Some new designs save money, and now and then they even result in something that was impossible with conventional design. For the too «trigger happy» modernists a warning is warranted: «One pilot project is fine, but ten similar pilot projects at the same time are a disaster .» Usually the first project is implemented too late, and the second, third etc. are implemented too early.
In pilot projects it is important to report the experiences. The Norwegian power industry is very decentralized, and the engineering staff consists mostly of «doers», not «thinkers». Therefore not all the experiences have been put down on paper, unfortunately.
As an example, let us follow the evolution towards the high head unlined pressure shafts.
Originally the water was sent down to the power station in penstocks situated on the surface. To bring the water underground was a long procedure:

1. At Såheim (1916) the penstocks had to be placed underground in shafts (9 penstocks in 3 shafts) because of the topography. This design was used in many schemes until the 1950's.
2. The steel shortage after the second world war led engineers to use a steel-saving design. At Lyse (1953) steel linings were used, and part of the water pressure was transferred to the rock by means of pouring concrete into the gap between lining and rock.
3. One then started to omit the steel lining altogether for some low pressure schemes, and gradually unlined shafts have been used for higher and higher heads. This

development was possible thanks to close cooperation between geologists and engineers (applied geology).

In shafts the differences between the two designs (2 and 3) are obvious. A small diameter shaft is excavated, a steel lining erected, and the space between lining and rock wall is filled with concrete. This concrete is to «put the rock back». On the drawing one sees a nice and narrow gap to fill. But blasting a shaft is not that easy. Therefore much more rock is removed, and the concrete quantity increases. This problem is well known, and everybody accuses others of doing a sloppy job.
An unlined shaft is more robust in that aspect. Deviations from a theoretical shaft are of no consequence, because no lining shall be erected.
Of course, the choice of the unlined alternative is not so clear cut. Several articles describe how to cope with the problems of deciding where to locate the shaft and how to handle the uncertainties. It is a balance between shallow location (cost) and deep location (rock overburden).
Steel linings are used to sustain part of the water pressure and to avoid leakages. However, they also reduce the head loss, notwithstanding the fact that the waterway's cross section is reduced by concrete. The reason: smooth walls instead of rough rock walls.
However, in recent years rock engineering technology has developed full face boring equipment, mainly for cost saving purposes. When this equipment is used in shafts, it looks even sillier to line them, because the rock walls already are quite smooth.
Above is an example of new technology and new equipment changing old priorities. Below is another example, where the new design «makes the impossible possible».
To be able to operate satisfactorily a hydro power station needs a surge chamber dose to the turbine. A common design is to have a surge shaft at the top of the pressure shaft. However, this requires that the rock surface is situated at the same level as the intake. Without such favourable topography one had to develop the head in two steps instead of one, or to move the power station closer to the intake, thereby moving it to a place where a longer access tunnel etc. was necessary.
The invention of the air cushion surge chamber gave more flexible lay-outs, because the above-mentioned topographical requirement was not necessary any more. As a fringe benefit, the new design also reduces the environmental impacts. By using an air cushion surge chamber and an inclined headrace tunnel, one avoids an access road and a rock spoil deposit situated high up on the side of the valley for everybody to see. As far as costs are concerned, it is advantageous to concentrate the work as much as possible.
In the following articles you will find examples on how Norwegian engineers have carried out their schemes and also why.

COMPACT DESIGN OF CAVERNS FOR HYDRO-POWER STATIONS IN NORWAY.

I. TVEITAN & B. BUEN
Ing. A. B. Berdal A/S. Norway

SUMMARY
Norway relays heavily on hydro power for energy. This has made large in- vestments necessary and compact and economic design is a virtue made out of a necessity. An essential part of the design is to determine the optimum Bite, and the orientation and layout of the cavern and tunnels with respect to the geological conditions.
Compact design gives reduced excavated volume, reduced volume of construction materials and reduced rock reinforcement. This is achieved through an arrangement of generators, turbines, valves and transformers in a space saving way. The span width of a cavern is usually determined by the generator shaft diameter. As a rule of thumb: span width = generator shaft diameter + 3.0- 3.5 m. Simple shapes of tunnels and caverns, and layout that makes construction easy and gives low excavation costs are sought for.
Six examples of design practice covering the evolution in the period from 1945 until today are given.

INTRODUCTION
The first underground hydro-power plant in Norway, Bjørkåsen south of Narvik, was built in the years 1919-1923. Up to 1940 power plants were only exceptionally built underground, and by today's standard, with low outputs.
Most of the power plants built in Norway after the war. are under- ground constructions with the power-house in rock. In the years 1970- 1980 about 90 power plants with a total output of 6350 MW were put in operation. 60 of them are underground constructions with a production of 31.5 TWh a year. In order to make these plants feasible. it was necessary to use economical compact designs.

METHODS AND POSSIBILITIES IN COMPACT DESIGN
The topography of Norway favours the construction of underground hydropower plants. By going underground the length of the water conduits can be shortened considerably compared to above ground channels or penstocks. By using unlined tunnels and shafts as the practice is today, an underground power plant is as a rule considerably more economic than a comparable surface plant. At the beginning it was the shortening of the penstocks that made the profit. Later on when the penstocks were embedded in concrete the pressure was transferred to the rock, and thus a great deal of steel was saved. The next step in the development was unlined pressure shafts, were only the last 20-50 m were lined with steel.
With regard to need of space and environment protection the under- ground construction, concerning design, is favourable. Underground construction has lower maintenance and repair costs than constructions above ground.

In a country with steep valley sides as in Norway, the hazard due to rock falls, slides and snow avalanches must always be reckoned with. A subsurface power station can therefore be much safer than one situated above ground. Also the wartime security aspects are in favour of a subsurface power station.

The intention of the underground power plants' compact design is to reduce building costs. The savings derive directly from the reduced volume that is caved out, and therefore lower building costs and shorter construction time is obtained. The work with rock support will also be cheaper, as the support costs depend on the span and height of the cavern.
For a given rock quality it is always an upper limit for the size of an unsupported cavern. When this limit is passed, the support costs increases rapidly. The principle of rock support as a function of cavern size is illustrated on fig. l.

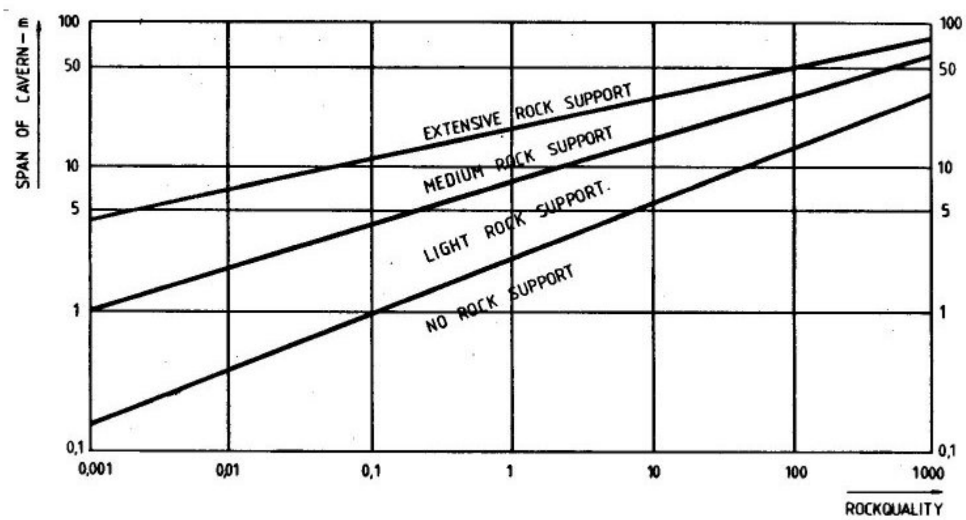


Fig. l. Rock support as a function of cavern size and rock quality (Barton et Al. 1974)

Compact design of power plants in rock has been made possible through the technical development of the different components in the production system. The development towards more efficient units, better quality of building materials and better arrangements has made the need of space per unit output less.
A similar development has taken place in power plant design. The older ones have the same equipment and layout as the construction above ground.
But one has come to realize that rock is a three-dimensional construction material, where the need of space is decisive for the volume that needs to be caved out.

ENGINEERING GEOLOGY IN HYDRO POWER CONSTRUCTION
Engineering geology as a subject in Norway has mainly developed from the requirement in hydro construction. In several of the early projects problems with weathering, discontinuities and faults occurred.
Today there is always done through a registration of the geological conditions to serve as basis for the design of a power plant.

The registrations cover:
- Rock types and their mechanical properties
- Distance between joints, strike and dip
- Faults, their character and orientation
- Primary state of stresses
- Groundwater conditions

Based on this the layout for the project is optimised with regard to the prevalent geological conditions.
If it is possible in one way or another the cavern, tunnel or shaft is given such an orientation, based on the direction of discontinuities and rock stresses, that the stability problems and loose rock outside the contour is minimised. The cross-sections of shafts and caverns are given a shape that is adjusted to the mechanical proper- ties of the rock, the discontinuities and the rock stresses. Depending on the rock conditions one gives the profile a flat or a high roof arch and curved walls, or an asymmetric shape.
If several tunnels or caverns lie close to each other, the stress concentration in the pillars between the openings can give rise to rock- bursts or spalling. The spacing of openings and also the sequence of excavation are therefore important parameters in the design.
A comprehensive and in detail description of the role of engineering geology in design is given by Selmer-Olsen and Broch (1978).
The permanent rock support is as a rule based on a classification of rock quality done in the caverns and tunnels by an engineering geologist. The engineering geologist is, however, also involved in pre- paring the tender documents and in site supervision and through this an optimal combination of temporary and permanent rock support is achieved.
The development in excavation technique and concrete technology has had a favourable influence on the work with rock support.
Modern methods for contour blasting give a moderate cracking of the rock surface, and the necessary support is reduced.
Carefully blasted rock surfaces can be of aesthetic value, and rock walls are often used as permanent walls in dry caverns.
Use of anchors and reinforced shotcrete as support has in many cases replaced the earlier much used reinforced concrete arches. The height in the power plant caverns can therefore be reduced.

EXAMPLES ON THE DEVELOPMENT OF POWER PLANT DESIGN

The older medium to high head power plants were equipped with Pelton turbines. Suitable Francis turbines for high heads were not yet avail- able. To avoid large spans in the caverns the turbines, generators and transformers were placed in line.
Åbjøra hydropower plant built in the years 1947-51 is an example of this design, see fig. 2.
The Åbjøra plant with a head of 430 m has 3 horizontal Pelton turbines each rated at 27 MW. As evident from the plan on fig. 2 a cavern will be rather long with this type of arrangement.
Turbines with vertical axis requires less width and a reduction in excavated volume of about 20% compared to a design with horizontal Pelton turbines can be achieved.

The Hjartdøla power plant shown on fig. 3, utilizes vertical Pelton turbines. The 2 aggregates rated at 52 MW each operates at a head of 590 m.

In above ground power plants an arrangement with generators and trans- formers placed side by side is quite common. This arrangement has also been adopted underground. For large aggregates this arrangement requires a very wide cavern. This layout therefore is best suited for very good rock conditions and even then the rock support may be significant.
The Tokke power plant fig. 4t commissioned in 1962 features the "side by side" layout. The head is 390 m and the cavern has 4 vertical Francis turbines each rated at 110 MW.
The design with the transformer in a cavern parallel to the machine hall gives a short distance between generator and transformer and the danger of fire is reduced.

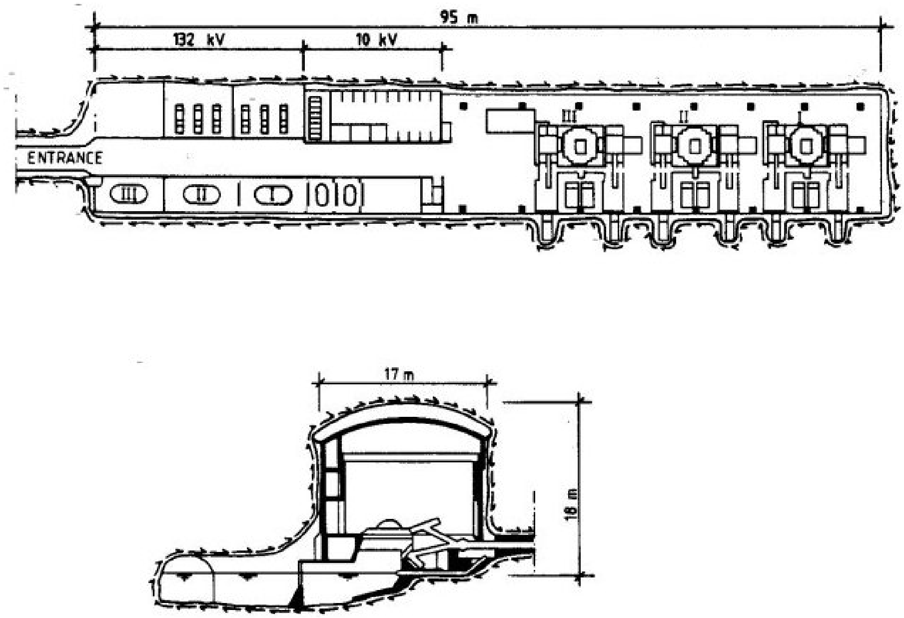


Fig. 2. Åbjøra power plant with "in line" arrangement and horizontal Pelton turbines (3 x 27 MW, head 430 m)

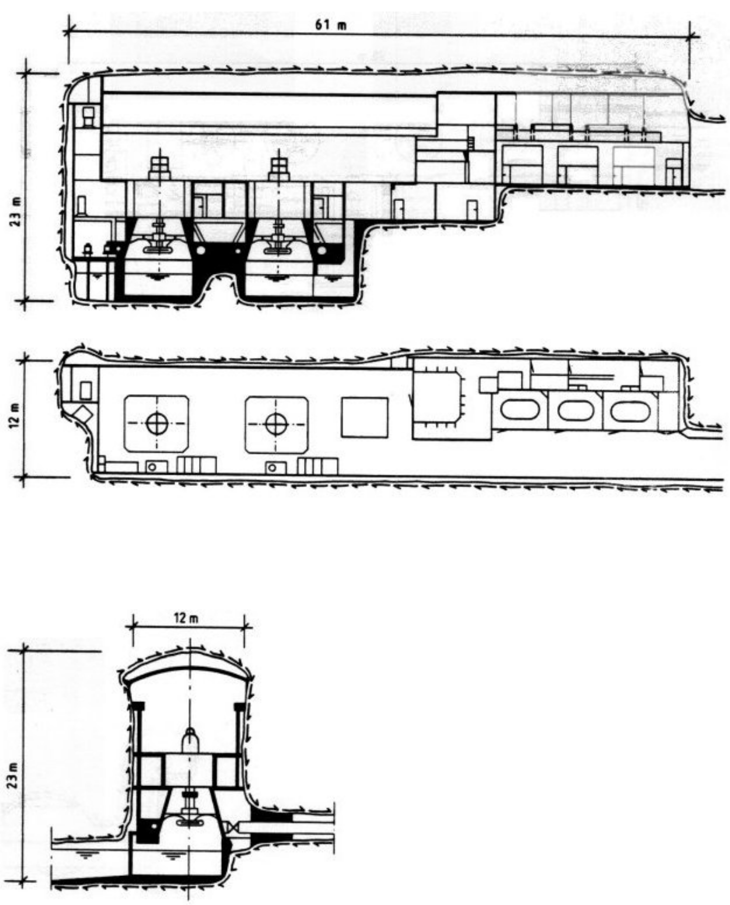


Fig. 3. Hjartdøla power plant with "in-line" arrangement and vertical Pelton turbines. (2 x 52 MW, head 590 m)

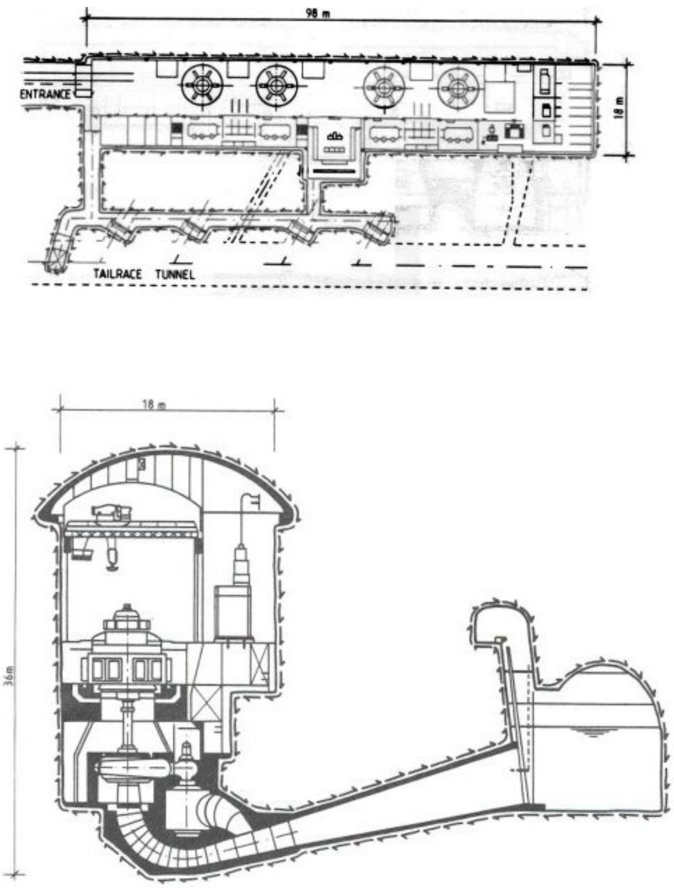


Fig. 4.
Tokke power plant with "side by side" arrangement and vertical Francis turbines. (4 x 110 MW, head 390 m)

From a stability point of view this can be a good arrangement because the use of 2 small caverns instead of 1 large is possible. The stress concentration in the pillar between the caverns may, however, increase the need for rock support. The total excavated volume both with regard to cavern volume and to necessary access tunnels will also be higher for the two-cavern layout.
The Brokke power plant, fig. 5, is an example of the two-cavern layout. The head is 280 m and 4 vertical Francis turbines with an output of about 75 MW each are in operation.
The latest development is an arrangement with the transformers between the units, below the floor of the machine hall. The distance between the units has to be increased. The span of the cavern, however, is limited to a minimum by the other components of the units. It is the necessary dimensions of the generator excavations and the valves that determine the span. The shortest span is w hen the inlet makes an angle of 60 to the axis of the cavern.
The modified in line arrangement was us ed at the Borgund power plant commissioned in 1974. fig. 6. This plant features 2 vertical Pelton turbines rated at 80 MW each. The head is 870 m.
Arøy power plant was commissioned in 1983. Here a rock support by bolts and reinforced shotcrete are used in stead of the traditional reinforced concrete arch, se fig. 7. A light roof consisting of corrugated sheeting is used to ensure clean and dry conditions.
The Arøy plant features 2 vertical Francis turbines, one small of 20 MW arid one larger of 70 MW .

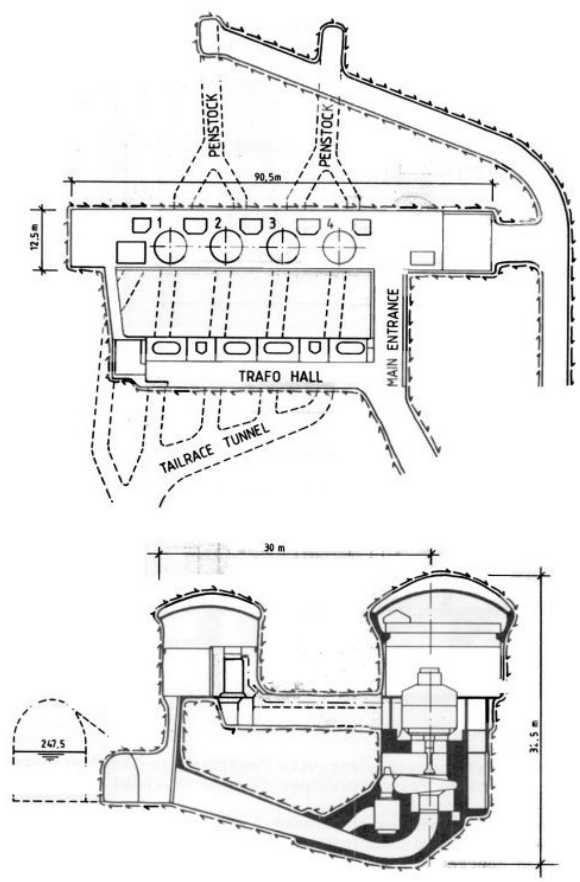
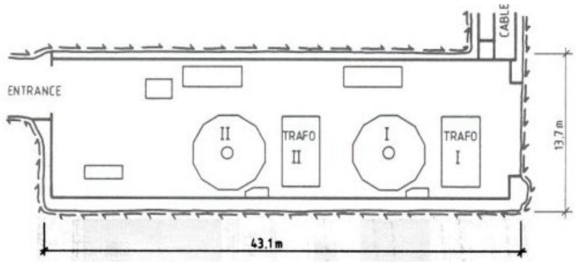


Fig. 5. Brokke power plant with ”two-cavern arrangement and vertical Francis turbines. (4 x 75 MW, Head 280 m))



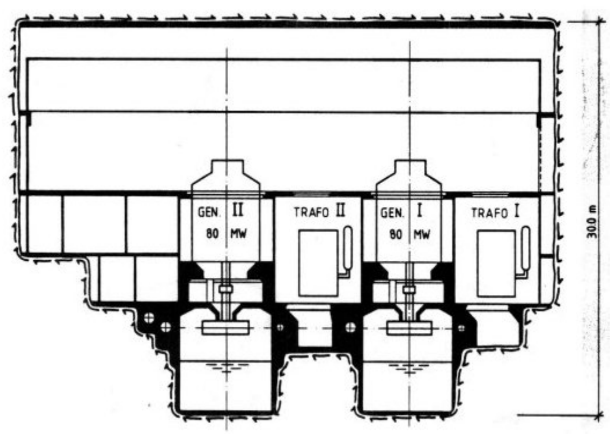
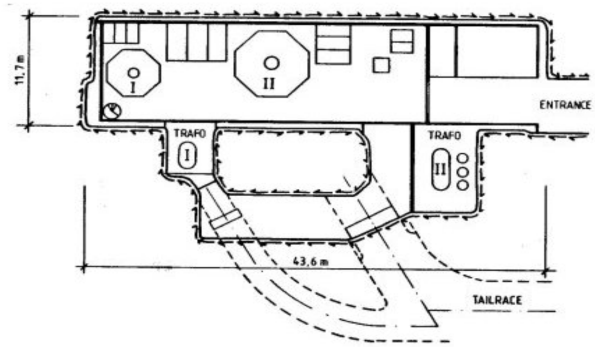


Fig. 6 Borgund power plant with "modified in-line" arrangement and vertical Pelton turbines (2 x 80 MW, head 870 m)



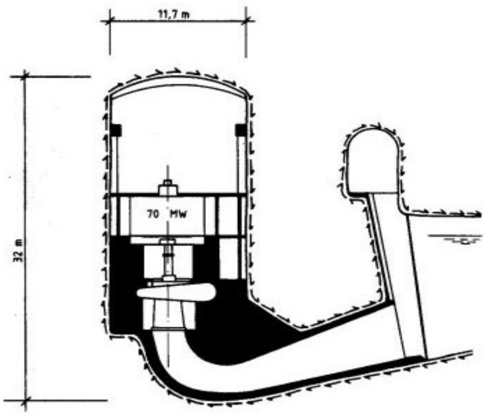


Fig. 7. Årøy power plant with "modified side by side" arrangement and vertical Francis turbines. (1 x 70 MW, 1 x 20 MW, head 146 m)

The disparity in aggregate size is due to the large fluctuations in flow for this run of the river scheme. The net head is 146 meters.
By using shotcrete and bolts as roof support a marginal reduction of roof height is possible. The main saving is, however, a significant reduction of construction time compared to the cast in place concrete arch solution.

CONCLUSIONS

Experience has shown that correct location and orientation of a power station and auxiliary tunnels with respect to the prevalent geological conditions are important for keeping down excavation and rock support costs.
By carefully considering the placing of the turbines, valves and transformers one can achieve minimum cavern widths. The dimensions of the generator excavations usually determine the span of the cavern. In most cases it is possible to achieve a span of the cavern that is 3- 3.5 m wider than the measure of the generator casing. Besides it might be necessary with local slashing to make room for the valves etc.
By applying the rule of thumb that span width equals generator casing diameter + 3.5 m it seems evident that for the majority of machine halls a span of 12- 13 m will be sufficient. By further using this rule of thumb on existing power stations it seems like a number of these are roomier than

strictly necessary. There may be a number of valid reasons for making a large cavern, but in addition to higher excavation costs there will be added costs for rock support and for structural concrete.
The height of the caverns is decided by the technical installations and dimensions of the crane. A slight reduction can be obtained by using rockbolts and shotcrete in the roof instead of cast in place concrete arches. One can also achieve savings by deliberately making a flatter roof and compensate with longer rockbolts.
Date on excavated volume compared to effect, type of turbine and head is given in Table I below:

Table 1.

POWER STATION	YEAR OF COMMIS-SONING	HEAD IN METERS	TURBINE TYPE *)	EFFECT MW	EXCAVATED VOLUME m^3	VOLUME-EFFECT RATED m^3/MW
Åbjøra	**1951**	**430**	**P**	**3 x 27**	**16,500**	**204**
Daja	1955	155	F	24	12,000	490
Hjartdøla	**1958**	**590**	**P**	**2 x 52**	**14,500**	**140**
Tokke	1962	390	F	4 x 110	43,600	99
Brokke	**1964**	**280**	**F**	**4 x 75**	**40,000**	**133**
Jørundland	1969	280	F	55	6,100	110
Sundsbarm	**1969**	**480**	**F**	**200**	**15,000**	**75**
Hovatn	1972	490	F	44	4,500	102
Ylja	**1972**	**685**	**P**	**65**	**6,500**	**100**
Borgund	1974	870	P	2 x 80	14,000	75
Fagerli	**1975**	**230**	**F**	**72**	**12,500**	**174**
Leirdøla	1975	450	F	100	10,800	108
Skibotn	**1977**	**440**	**F**	**72**	**6,550**	**91**
Lomi	1979	560	F	120	12,000	100
Lomen	**1982**	**310**	**F**	**54**	**10,400**	**193**
Ustekveikja	1982	110	F	36	7,500	208
Årøy	**1983**	**146**	**F**	**70 + 20**	**14,500**	**183**
Sjønstå	1984	120	F	63	13,000	206
Tjodan	**1984**	**896**	**P**	**110**	**11,500**	**104**

*) TURBINE TYPE
P=PELTON
F=FRANCIS

**) EXCAVATION VOLUME OF CAVERN OR CAVERNS USED FOR TURBINES, GENERATORS AND TRANSFORMERS

Tentative conclusions based on table 1 is that for small low head plants a volume effect ratio of about 200 m^3/MW can be achieved. For medium size high head plants a ratio of 75 to 100 m^3/MW can be obtained. Data on large high head plants is not available but by extrapolating the values taken from the smaller plants a ratio of 50 m^3/MW should be well within range.
As a cost reducing factor a small cavern is important. One should, however, regard the cavern and its auxiliary tunnels as a whole and much effort should be put into achieving a simple layout. From a construction point of view a simple layout and the lack of intricate geometry means simple and low cost excavation.

The necessary transport tunnels should of course as far as possible be utilized for functions in the power station. An example is the transport tunnel for the tailrace tunnel used as surge chamber.

REFERENCES

BARTON et al. "Analysis of rock mass quality and support practice in tunnelling, and a guide for estimating support requirements". Report no. 54206. Norwegian Geotechnical Institute, Oslo, 1974.

SELMER-OLSEN, R, & BROCH, E, "General design procedure for underground openings in Norway". Proceedings Rockstore 1977. London 1978. .

SOLEM, A. VOIGHT, F. "Norwegian Hydro Power Stations", Teknisk Ukeblads Forlag 1966. (Norwegian text)

Development of Unlined Pressure Shafts And Tunnels in Norway

Einar Broch
University of Trondheim

Topographical conditions in Norway are especially favourable for the development of hydroelectric energy. More than 99% of a total annual production of 90 TWh of electric energy is generated from hydro power. Figure 1 shows the installed production capacity of Norwegian hydroelectric power stations. It is interesting to note that, since 1950, underground powerhouses are predominant. In fact, of the world's 300-500 underground powerhouses almost one-half are located in Norway. Another proof that the Norwegian electricity industry is an "underground industry" is that it has approximately 2,500 km of tunnels.

Through the design, construction, and operation of all these tunnels and underground powerhouses, valuable experience has been gained. Also, special techniques and design concepts have been developed over the years. One such Norwegian specialty is the un- lined, high-pressure tunnels and shafts which this paper describes.

It should be mentioned as a preliminary matter that the rock of Norway is of Precambrian and Paleozoic age. Although there is a wide variety of rock types, highly metamorphic rocks predominate. From an engineering point of view they may in general be classified as typical hard rock. Most of our hydropower tunnels have only 2-4% concrete or shotcrete lining. Only in a few cases has it been necessary to increase this to 30-60%. The low percentage of lining is due not only to favourable tunnelling conditions; it is also the consequence of a support philosophy which accepts some falling rock during the operation period of a water tunnel. As long as rock falls in certain parts of the tunnel don't develop considerably and increase the head loss, a reasonable number of small blockages spread out along the tunnel will not harm the tunnel or disturb the operation of the hydro power station. If necessary, they may be removed during later inspection and maintenance.

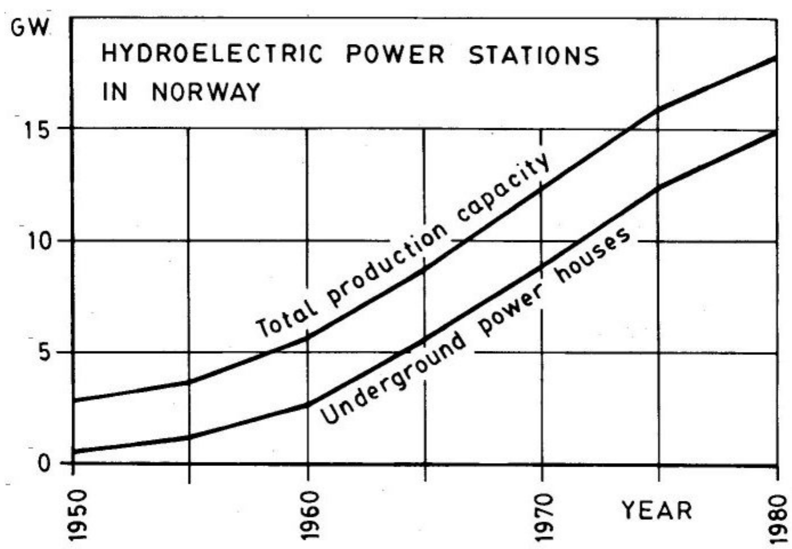


Figure 1. The development of hydroelectric power production in Norway. (Myrset 1980)

Early Unlined Pressure Shafts

During and shortly after the First World War there was a shortage of steel leading to uncertain delivery and very high prices. As a result, four Norwegian hydro power stations with unlined pressure shafts were put into operation (Table l).

As early as 1922, three pressure shafts were described in detail in a publication from the Norwegian Geological Survey (Vogt 1922). The Herlands-foss, Svelgen, and Toklev shafts were later investigated and described by Broch and Christensen (1962), and Herlands-foss was discussed by Selmer- Olsen (1970).

Table 1. The first unlined pressure shafts in Norway.

Name	Year	Water Head (m)	Diameter (m)	Rock Type	Experience
Herlandsfoss	1919	136	3.20	Mica-schist	Partly failed
Skar	1920	129		Gneiss-granite	Completely failed
Svelgen	1921	152	2.40	Sandstone	Minor leakage
Toklev	1921	72	2.50	Monzonite	No leakage

The pressure shaft at Herlands-foss is shown in Figure 2. According to the original design the penstock and the concrete plug were placed only 50 m from the turbine, leaving 150 m of the high-pressure tunnel unlined. During the first filling of the tunnel and the shaft, increasing leakage through the mica-schist layer was observed. The tunnel was then emptied and a 60-m- long reinforced concrete lining was placed inside the penstock. After two months of operation, rapidly increasing leakage was again observed. Inspection of the emptied tunnel revealed open cracks in the concrete on both sides of the tunnel near the springline. After this failure the pen- stock was extended through the whole tunnel to the foot of the shaft (Fig. 2). No leakage from the shaft has been observed since, and the power station has operated without unplanned stops for 60 years.

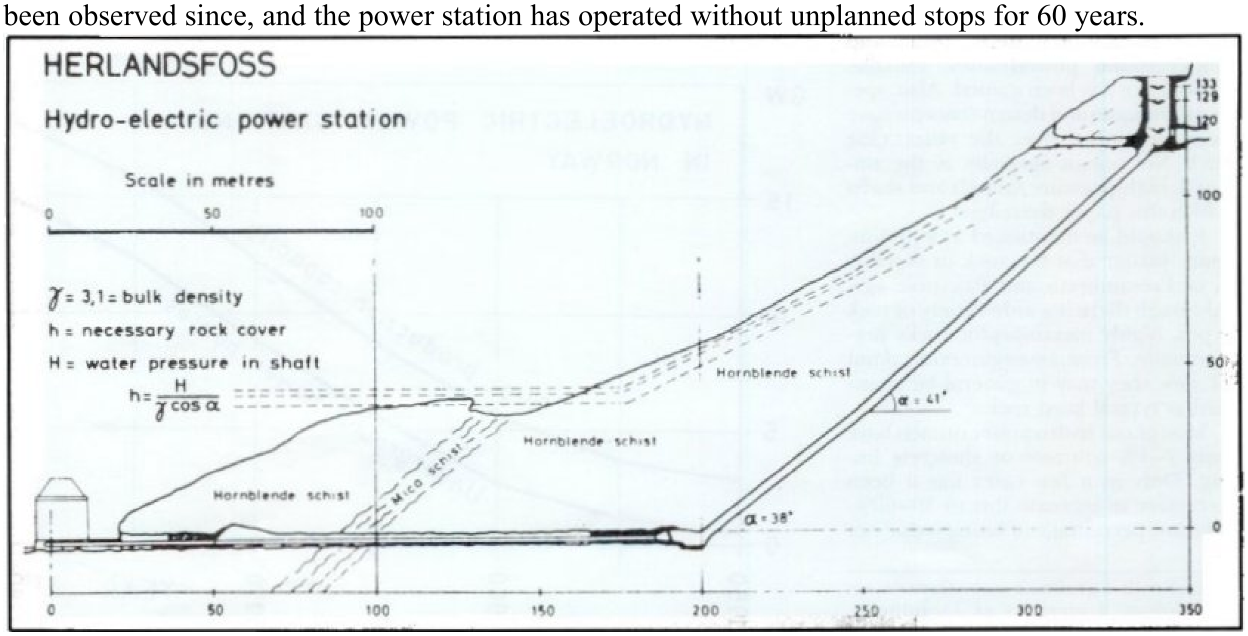


Figure 2. The Herlandsfossen hydroelectric power station, in operation since 1919. (Selmer-Olsen 1970.)

The pressure shaft at Skar was, to make a long and miserable story short, a complete failure and was replaced by an ordinary penstock (except for the upper part of the tunnel). The primary reason for the unacceptable leakage was the low overburden of rock, only 22 m where the water head was 116 m.

At Svelgen, leakage of 3-5 l/sec was observed as two small polluted streams during the first filling of the pressure shaft. A short section of the shaft was lined with concrete and grouted with cement. Since then the shaft has operated without problems.

The Toklev pressure shaft has functioned without any reported problems since it was put into operation.

Development of the General Plant Layout

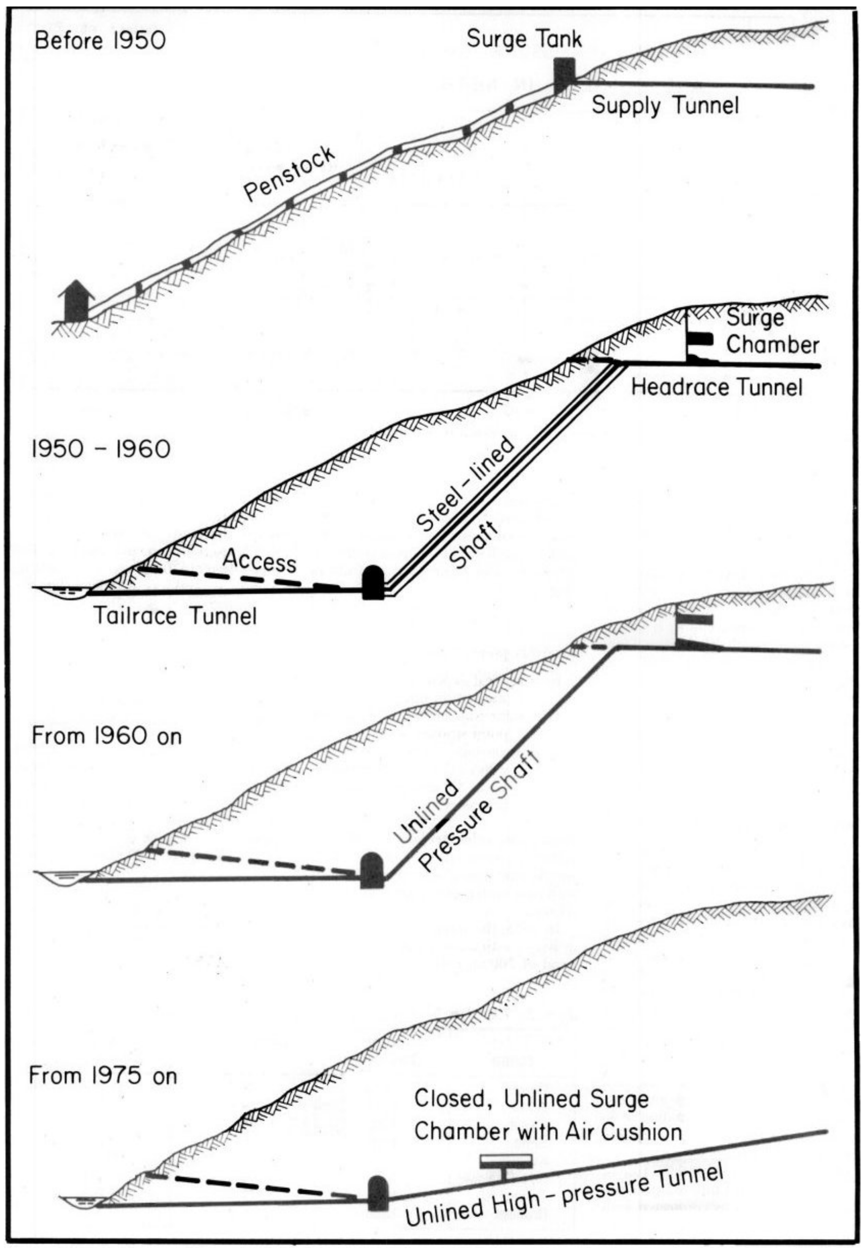


Figure 3.
The development of the general layout of hydroelectric plants in Norway.

Although three out of four pressure shafts constructed around 1920 were operating without problems after some initial problems had been solved, it took almost 40 years for the world record of 152 m of water head in unlined rock at Svelgen to be beaten. Through 1958, nine more unlined pressure shafts were constructed, but all had water heads below 100 m. Before 1950 the above-ground powerhouse with penstock was the conventional layout for hydropower plants

When the hydropower industry went underground in the early 1950's they brought steel pipes with them. Thus, for a decade or so most pressure shafts were steel-lined. During the period

1950-65, a total of 36 steel-lined shafts with heads varying from 50 to 967 m (with an average of 310 m) were constructed.
The new record shaft of 286 m at Tafjord K3, which was put into operation successfully in 1958, gave the industry new confidence in unlined shafts. As Figure 4 shows, new unlined shafts were constructed in the early 1960's. Since 1965, unlined pressure shafts have been the conventional solution for heads up to 600 m. In 1981, the Tafjord Kraftselskap set its third world record by putting into operation an unlined pressure shaft with a water head of 780 m. However, plans are already completed for a new unlined pressure shaft with a water head of 1 ,000 m (Bergh-Christensen and Kjolberg 1982). Altogether, by the end of 1982, 64 un- lined pressure shafts with water heads between 150 and 780 m (with an average of 314 m) were in operation in Norway. Figure 4 clearly demonstrates that increasing water heads reflect an increasing confidence in unlined pressure shafts over the last 20 years.

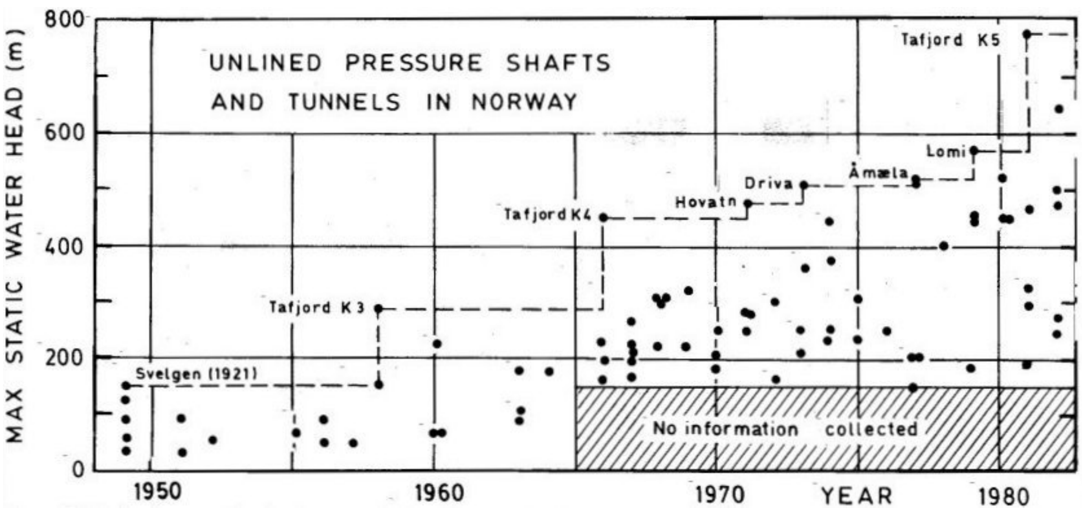


Figure 4. The development of unlined pressure shafts and tunnels in Norway.

This confidence in the tightness of the unlined rock mass increased in 1973 w hen the first closed, unlined surge chamber with an air cushion was successfully put into service at the Driva hydroelectric power plant. This innovation in surge chamber design is de- scribed in detail by L. Rathe (1975). The bottom sketch in Figure 3 shows how the new design influences the general layout of a hydro power plant. The steeply inclined pressure shaft, normally at 45°, is replaced by a slightly inclined tunnel, 1:10-1:15. Instead of the convention al open 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.

Table 2. Closed unlined surge chambers with air cushions in Norway.

Name	Year Completed	Air pressure (bar)	Volume of Chamber (m³)
Driva	1973	42.5	6,000
Jukla	1974	24	6,200
Oksla	1980	46	17,300
Sima	1980	50	7,100
Kvilldal	1981	43	100,000
Nye Osa	1981	18	12,000
Tafjord K5	1981	75	2,000
Brattset	1982	26.5	3,000

After the tunnel system is filled with water, compressed air is pumped into the surge chamber. At Driva, where the water head at the chamber is 425 m, the total volume of the chamber is 6,000 cu m. Of this, 3,000 m is filled with compressed air. 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. Table 2 describes the air cushion surge chambers in service.

The First Design Criteria

In summarizing the experience from early unlined pressure shafts, Vogt (1922) states that the first and foremost requirement for a pressure shaft is that leakage be avoided. He disagrees with those who at that time claimed that an unlined pressure tunnel is safe w hen the weight of the rock overburden is greater than the water pressure. (He even says that to get this idea out of the profession is one of his main reasons for writing the report.) According to Vogt, the main risk for unlined pressure tunnels and shafts is bad rock masses with weathered zones, joints, etc. Hence, the best way of avoiding leakage is to place the tunnel as deep in to the rock mass as possible.

In the years before 1968 the rule of thumb for planning unlined pressure shafts in Norway was connected with the general layout for hydro power plants used at that time (Fig. 3). For construction reasons the inclination of the unlined shafts varied between 31 ° and 47°, with 45° as the most common. The rule was expressed as follows (Fig. 5):

$$h > c \cdot H$$

for every point of the tunnel, where

h = vertical depth of the point studied (in m),
H = static water head (in m) at the point studied, and
c = a constant, which was 0.6 for valley sides with inclinations up to 35° and increased 1.0 for valley sides of 60°.

High valley sides steeper than 60° are rather uncommon in Norway. This simple rule was, of course, to be used with care under special geological conditions.

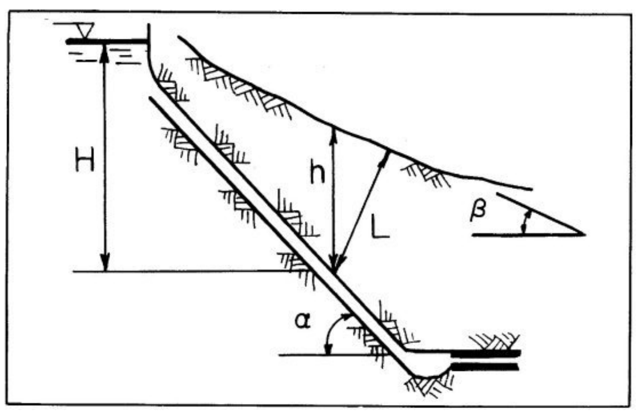


Figure 5. Definitions for the rule of thumb for tunnel design.

In 1968, the unlined pressure shaft at Hyrte, with a maximum static water head of 300 m, failed. The shaft had the uncommon inclination of 60°. A revised rule of thumb which would also cover shafts steeper than the commonly-used 45° was presented by Selmer-Olsen (1970). It was expressed in a more general way as follows (Fig. 5):

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

where γ_w = density of water ,
γ= density of the rock mass,
α = the inclination of the shaft.

The failure of the Byrte shaft was also analysed for the first time with a finite element mod el (Brekke et al. 1970);
In the fall of 1970 another failure occurred at Askora, where an unlined tunnel in sandstone with a water head of approximately 200 m was hydraulically split. The split followed sand-filled, steeply dipping joints with a strike parallel to the very steep valley side (55°) and normal to the tunnel. The failure is described in detail by Bergh-Christensen (1975).

After this failure a new rule of thumb was introduced by Bergh-Christensen and Dannevig (1971) where the inclination of the valley side was taken directly into account (Fig. 5)
$L > \gamma_w \cdot H / \gamma \cdot \cos\beta$

where

L = shortest distance between the surface and the point studied (in m), and
β = average inclination of the valley side.

Based on this formula, a diagram showing existing unlined pressure shafts with or without leakage was presented. This was further supplied with information by the Norwegian Geotechnical Institute (1972) and is shown in a slightly revised version in Figure 6. It is worth noticing that the unlined pressure shafts where leakage is observed are, with the exception of Bjerka, plotted below the curves defined by the rule of thumb. At Bjerka unfavourable geological conditions with steeply dipping, permeable joints with strike parallel to the valley side caused leakage at a distance up to 1 km from the pressure tunnel.

Design Charts Based on Finite Element Models
Parallel with the revisions of the rule of thumb, the search for better and more general design criteria was intensified at the Department of Geology of the University of Trondheim. These criteria were to be valid for unlined pressure shafts and tunnels, and also for unlined surge chambers with com- pressed air cushions. The first hydropower plant with this new design, Driva, was already under construction.
The new design tool was put into use in 1971-72 and is described in detail by Selmer-Olsen (1974)It is based on the use of computerized finite element models (FEM) and the concept that nowhere along an unlined pressure shaft or tunnel should the internal water pressure exceed the minor principal stress in the surrounding rock mass.
Very briefly, the FEM models are based on plain strain analysis. Horizontal stresses (tectonic plus gravitational) in- creasing linearly with depth are applied. Bending forces in the model are avoided by making the valley small in relation to the whole model. If required, clay gouges (crushed zones containing clay) may be introduced.

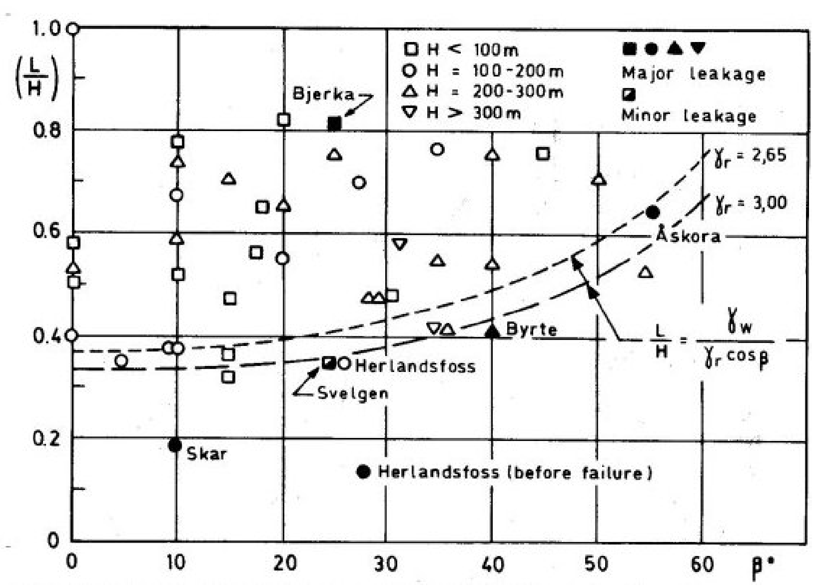


Figure 6. Unlined pressure shafts in valley sides with various inclination, β.

In addition to real cases, a number of idealized but typical valley sides have been analysed. One example of an idealized model is shown in Figure 7. In this case, the inclination of the valley side $\beta = 40°$, the bulk density of the rock mass $\gamma_r = 2.75$, Poisson's ratio $\nu = 0.2$, an the $\sigma_{hor}/\sigma_{vert}$ ratio at a instance of 5 d from the valley is 0.5 (d is the depth of the valley and H the maximum static head; as Young's modulus E is kept constant, it will not influence the results). To make the model dimensionless, the static water pressure is expressed as the ratio H/d, where the water head is expressed as a height in the same units as the valley depth (e.g., in meters). The curved lines run through points where the internal water pressure in a shaft equals the minor principal stresses in the surrounding rock mass ($\sigma_3 = H$).

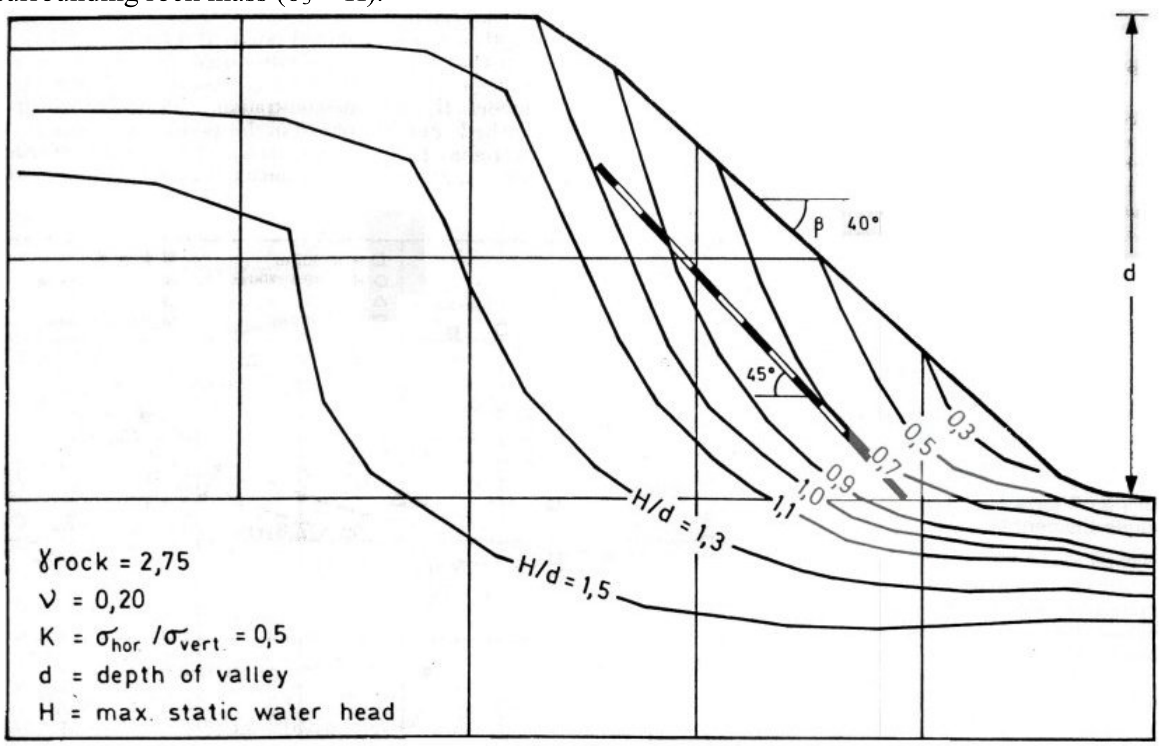


Figure 7. Design chart for unlined pressure shafts based on a finite element model. The curves run through points where the internal water pressure in the shadfy equals the minort principal stresss in the surrounding rock mass, ($H = \sigma_3$).

The use of the design charts can be illustrated by an example. Let the bottom of the valley, where the power station is located, be situated 100 m.a.s.l. and the top of the valley side 600 m.a.s.l. This makes d = 500 m. The maximum water level in the intake reservoir is 390 m.a.s.l. This makes H = 290 and the H/d ratio = 0.58. At all points in- side or below the 0.58 line the minor principal stress in the rock mass exceeds the water pressure in an unlined shaft; hence, no hydraulic splitting should occur. If a factor of safety of 0.2 is introduced, the critical line will be the $1.2 \cdot 0.58 = 0.7$ line. As a demonstration, a 45° inclined shaft is placed in this position in Figure 7.
Design charts for a number of valley side inclinations, !:3, are available. To fit the actual valley side to one of these is normally possible. In this process it is necessary to simplify and idealize the valley side by smoothing out the actual profile and ignoring protruding parts. It is also possible to make interpolations between two standard design charts. It is important that the profiles be made at a right angle to the contour lines of the map. In cases where the pressure shaft is placed in a part of

the valley side which is protruding in the horizontal section, a series of profiles through the protruding part should be studied.

Through a number of analyses, all the factors influencing the results have been carefully evaluated within natural limits. For example, if a measured bulk density, γ_r varies from the standard 2.75, this can be compensated by a correction of the overburden by the ratio 2.75/γr Also, an upper layer of topsoil or weathered rock masses can be compensated for by reducing the thickness of the overburden in accordance with the bulk density of these masses.

Measurements as well as observations indicate that the $\sigma_{hor}/\sigma_{vert}$ ratio in topographically undisturbed areas in Norway normally varies between 0.5 and 1.3, and very seldom exceeds 1.5. As a conservative solution, a ratio of 0.5 is used in the standard charts An increase in Poisson's ratio, ν, will give σ = H lines that go deeper in their valley side. For the standard charts, ν = 0.2 is used.

The shape and width of the valley have a major influence on the stress distribution near and under the bottom of the valley. The analyses, however, have shown that the σ=H lines on levels above the bottom and actual distances from the valley side are not influenced much. On the standard charts the width of the bottom of the valley is normally 1/3 d

So far only two-dimensional models have been used, and the stress perpendicular to the model plane has been assumed to be the intermediate principal stress, σ2. Stress measurements are sometimes carried out as a control, mainly where there is reason to believe that the tectonic stresses are not or- mal. Also, hydraulic splitting tests are sometimes used as a control of the σ = H lines.

For such tests it is important that the boreholes intersect natural joints with unfavourable directions with regard to the possibility of leakage from the unlined shaft. The splitting pressure is raised to 20% above the estimated minor principal stress at the actual location.

Geological Restrictions

The FEM-developed design charts are based on the assumption that the rock mass is homogeneous and continuous, an assumption which cannot be absolutely correct even for massive Precambrian granites and gneisses. However , observations and investigations of stress- induced stability problems such as rock bursts, popping rock, and spalling rock in a large number of tunnels in fjord and valley sides clearly indicate that the natural jointing of rock masses has only minor influence on the distribution of the virgin stresses. The jointing may, however, have a strong influence on the final development of a stability problem as well as on leakage through the rock mass.

As the permeability of rock normally is negligible, it is the jointing and the faulting of the rock mass, and in particular the type and amount of joint infilling material, that is of importance w hen an area is being evaluated. Cal- cite is easily dissolved by cold, acid water, and gouge material like silt and swelling clay are easily eroded. Crossing crushed zones or faults containing these materials should preferably be avoided. If this is not possible, a careful sealing and grouting should be carried out. The grouting is the more important the closer leaking joints are to the power- house and access tunnels and the more their directions point toward these. The same is also valid f()r zones or layers of porous rock or r()Ck that is heavily jointed or broken.

For pressure shafts in valley sides, one should be warned against clay-filled, crushed zones in the area, as they may have an unfavourable influence on the stress distribution. If they have a strike nearly parallel to the valley side with a steep or medium-steep dip towards the valley, they are especially dangerous. Not only may they change the stress distribution; they may also often cause leakage during construction as well as during operation. The hydraulic split- ting of the pressure tunnels at Askora and Bjerka was caused by such unfavourably oriented joints and faults. A careful mapping of all types of discontinuities in the rock mass is therefore an important part of the planning and design of pressure shafts.

During the construction period it is important that changes in all leakage into the shaft or tunnel, even minor dripping or seeping leakage, be observed. For the selection of the places for unlined, closed surge chambers these observations are of crucial value, as these points of water leakage are openings where a loss of compressed air may take place. Core drilling and water-pressing tests in the drill holes are thus normally apart of the investigation for the location of these chambers.

Underground Hydropower Plants With Unlined Waterways

As a final point an example of an underground hydro power plant will be shown and briefly described. Figure 8 shows the simplified plan and cross section of a small hydro power plant with only one turbine. No dimensions are given, as the intention is to show a system rather than give details. Similar layouts can be found for Norwegian plants with water heads in the range of 200-600 m. The figure is to some extent self-explanatory. It should be pointed out, however, that w hen the design charts are used the dimensioning or critical point will normally be where the un- lined pressure shaft ends and the steel lining starts. This is where the selected
σ_3 = H line should intersect the water- way. The elevation of this point and the length of the steel-lined section will vary with the water head, the size and orientation of the powerhouse, and the geological conditions, in particular the character and orientation of joints and fissures. Lengths in the range of 30- 80 m are fairly common.
The access tunnel to the foot of the unlined pressure shaft is finally plugged with concrete and a steel tube with a hatch cover. The length of this plug is normally 10-25 m, depending on the water head and geological conditions. Around the concrete plug and the upper part of the steel-lined shaft a thorough high-pressure grouting is carried out. This avoids leakage into the powerhouse and the access tunnels.

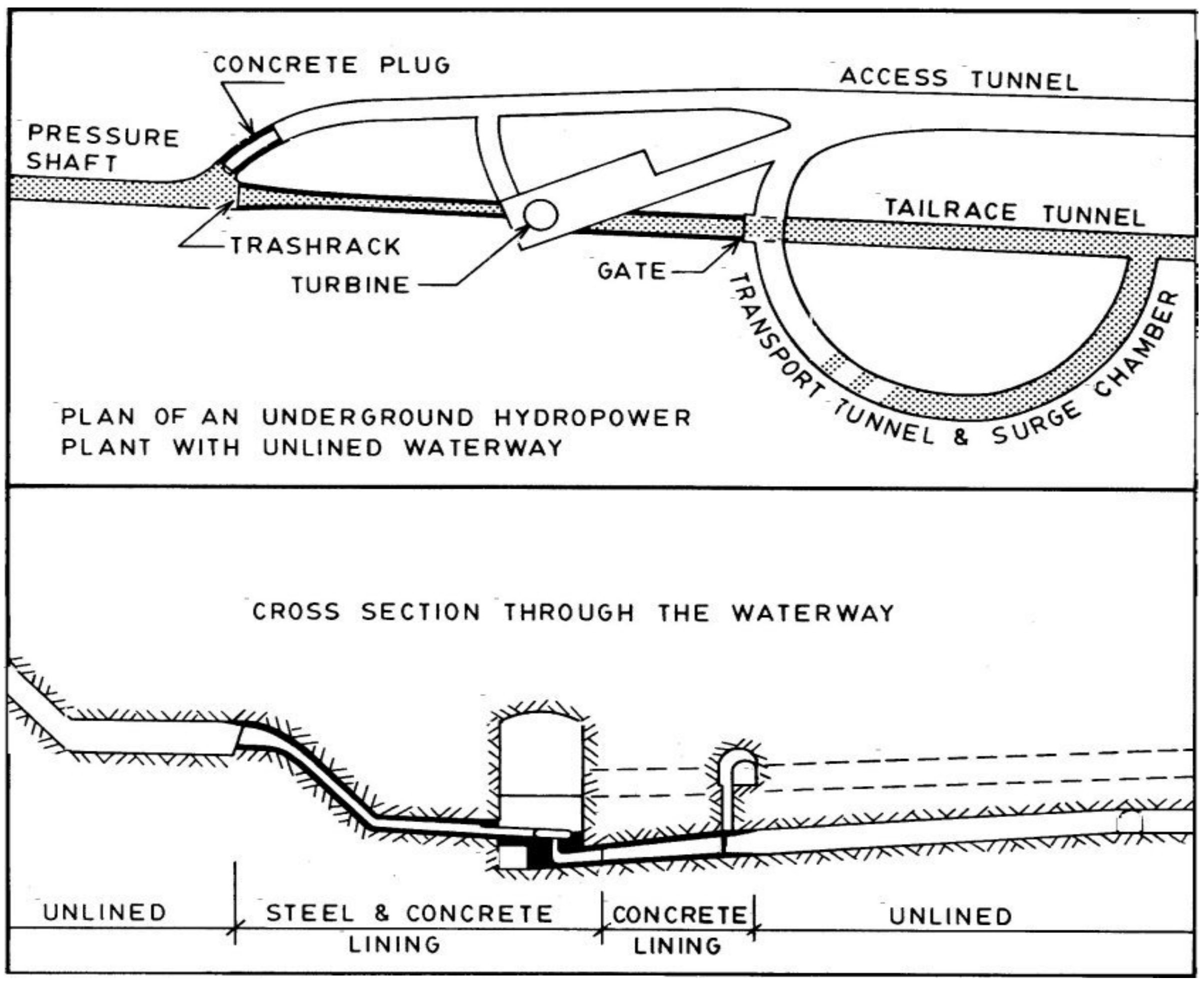


Figure 8. Plan and cross section of an underground hydropower plant with unlined waterways.

Concluding Remarks

Experience from a considerable number of pressure tunnels and shafts, as well as so-called air cushion surge chambers, have been gathered over a long period of time in Norway. These show that, providing certain design rules are followed and certain geological and topographical conditions avoided, un- lined rock masses are able to contain water and air under pressures up to at least 75 bars, equalling 750 m water head. In the future this experience may also be important outside the hydro power industry, for instance in the construction of cheap, unlined storage facilities for different types of gas or liquid under pressure.

References

Bergh-Christensen, J. & Dannevig, N. T !971.
Ingeniørgeologiske vurderinger vedrørende uforet trykksjakt ved Mauranger kraftverk Engineering geological evaluation of the unlined pressure shaft at the Mauranger hydropower plant). Unpublished report. Oslo, Norway: Geoteam A/S.

Bergh-Christensen, J., 1975.
Brudd i uforet trykktunnel ved Askora kraftverk (Failure of the unlined pressure tunnel at the Askora hydropower plant). Fjellsprengningsteknikk-Bergmekanikk 1974 Broch, Heltzen, and Johannessen, editors). Trondheim, Norway: Tapir. Pp 15.1-1~.8.

Bergh-Christensen, J ., and Kjolberg, R. 1982.
Investigations for Norway's longest un-lined pressure shaft. Water Power and Dam Construction 34:4, 31-35.

Bergh-Christensen, J ., and Kjolberg, R. 1982.
Investigations for a 1000 metres head unlined pressure shaft at the Nyset/Stegroe project, Norway. Rock mechanics: caverns and pressure shafts (Wittke, editor). Rotterdam, The Netherlands: A. A. Balkema. Pp 537-543.

Bergh-Christensen, J. 1982.
Design of unlined pressure shaft at Mauranger power plant, Norway. Rock mechanics: caverns and pressure shafts (Wittke, editor). Rotterdam, The Netherlands: A. A. Balkema, Pp. 531-536.

Bergh-Christensen, J. 1982.
Surge chamber design for Jukla. Water Power and Dam Construction 34: 10, 39-41.

Brekke, T. L., Bjørlykke, S., and Blindheim, O. T. 1970.
Finite element analysis of the Byrte unlined pressure shaft failure. Large permanent underground openings (Brekke and Jørstad, editors). Oslo, Norway: Universitetsforlaget. Pp. 337-342.

Broch, E., and Bergh-Christensen, J. 1982.
Undersøkelse verdrørende uforete trykksjakter (Investigations of unlined Pressure shafts). Trondheim, Norway: Inst. for Vassbygging, NTH.

Broch, E. 1982.
Designing and excavating underground power plants. Water Power and Dam Construction 34:4, 19-25. I

Broch, E. 1982.
The development of Unlined pressure shafts and tunnels in Nor- way. Rock mechanics: caverns and pressure shafts (Wittke, editor). Rotterdam, The Netherlands A. A. Balkema. Pp. 545-554.

Broch, E. 1984.
Design of unlined or concrete lined high pressure tunnels in topographically complicated areas. To be published in Water Power and Dam Construction 36: 11.

Buen, B., and Plastron, A. 1982.

Design and supervision of unlined hydropower shafts and tunnels with head up to 590 meters. Rock mechanics: caverns and pres- sure shafts (Wittke, editor). Rotterdam, The Netherlands 1982. A. A. Balkema. Pp. 567-574.

Johansen, P. M., and Vik, G.
Prediction of air leakages from air cushion surge chambers. Rock mechanics: caverns and pressure shafts (Wittke, editor). Rotterdam, The Netherlands: A. A. Balkema, Pp. 935-938.

Myrset, Ø.1980.
Underground hydro-electric power stations in Norway. Subsurface space (S. M. Bergman, editor). Vol. 1. Ox- ford: Pergamon Press. Pp. 691-699.

Myrset, Ø., and Lien, R. 1982.
High pressure tunnel systems at Sima power plant. Rock mechanics: caverns and pressure shafts (Wittke, editor). Rotterdam, The Netherlands: A. A. Balkema. Pp. 667-676.

Norwegian Geotechnical Institute.
Oversikt over norske uforete tunneler og sjakter (A review of Norwegian unlined pressure tunnels and shafts). Unpublished report.

Rathe, L. 1975.
An innovation in surge- chamber design. Water Power 27, 244-248.

Selmer-Olsen, R. 1970.
Experience with unlined pressure shafts in Norway. Large permanent underground openings (Brekke and Jørstad, editors). Oslo, Norway: Universitetsforlaget. Pp. 327-332.

Selmer-Olsen, R. 1974.
Underground openings filled with high-pressure water or air. Bull. Int. Ass. Engineering Geology 9, 91-95.

Vogt, J. H. L. 1922.
Trykktunneler og geologi (Pressure tunnels and geology). Report no. 93. Oslo, Norway: Norges Geol. Undersøkelse.

EXPERIENCE GAINED FROM UNLINED HIGH PRESSURE TUNNELS AND SHAFTS IN HYDRO-ELECTRIC POWER STATIONS IN NORWAY

by R. Selmer-Olsen
A/S Geoteam
Trondheim,
Norway

INTRODUCTION

Unlined shafts and tunnels carrying water at pressures greater than 1.5 MPa have been in use in Norway for more than two decades. At present 66 hydro-electric power stations with such unlined high pressure sections are in operation (January 1983), and 13 more are under construction.
The maximum static head of unlined tunnels and shafts in operation is 790 m. This will increase to 875 m before 1986. The largest cross-sectional area is 135 m^2, in a tunnel with a 460 m static head. Conservatism and the hydrological and topographical conditions of the sites have to a considerable extent been the factors limiting this development, rather than the rock types encountered.
The rocks in which the high pressure sections are situated differ considerably, but they are all Palaeozoic or older and of moderate to high strength. One tunnel and one shaft were initially failures.
They were built before 1970, before the criteria for overburden used today had been established. At nine of the power stations an enclosed unlined surge chamber employing a cushion of compressed air has been preferred to a surge shaft with an open surge chamber. Compressed air volumes ranges from 1,200 to 88,500 m^3 and air pressures from 18 to 78 bars. The first air cushion was put into operation in 1973 and has not required any subsequent refilling of compressed air .
At Osa, air leakage from the cushion after grouting is about 1.2 normal m^3 per minute*. At Tafjord V, the air cushion could not be maintained because of air leakage through a new-built extensive fracture. This fracture is believed to have been opened by the high air pressure, owing to an unusual anisotropy in the tectonic rock stress field. The leakage of air from the other cushions after grouting has been less than 4 normal mJ per hour and can easily be compensated**. The most suitable rock for the cushion is selected during advance at a reasonable distance from the power- house. To get low water flow within the chamber and a negligible solution of air in water flowing past, the cushion is located over and to the side of the headrace tunnel. Of the same reason the length of the adit is made more than three times the span.

SUMMARY OF PROJECTS

Since more detailed information may be of interest for other water power projects, a list of the different cases has been made (Table 1). This states the owners, commissioning date, maximum static head for the unlined section, gradient and cross-sectional area of the tunnel or shaft, and the type of rock along the unlined section. In certain instances the lowest static heads are also listed (in brackets). In most cases, however, the water level at the intake varies by less than 15 m and only the maximum value is quoted. The maximum pressure increase through surging is omitted as it has a short term effect of little consequence for hydraulic fracturing.

* Ref. Edvarson, S. and B. Saetren (1985) -Publ. no. 3 Before repair: 15 Nm^3/min.
After repair: 1.2 Nm^3/min.
** Ref. Pleym, A. and O. Stokkebø (1985) -Publ. no. 3 Before repair: 200-250 Nm^3/hour
After repair: Almost zero

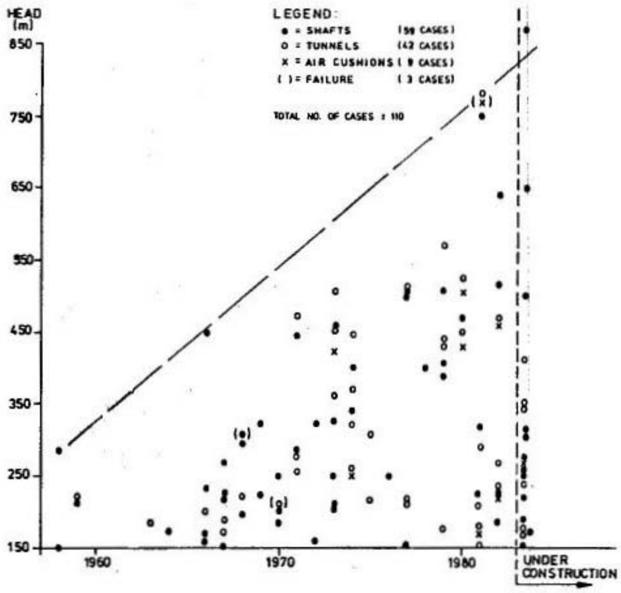


Fig. 1: The development of unlined high pressure sections in hydro-power supply systems in Norway.

The development of unlined high pressure shafts and tunnels with utilizing static heads exceeding 150 m is illustrated in Fig. 1. This shows the increasing confidence in this type of construction. Cases of initial failure are marked with brackets.

Fig. 2 shows the distribution of tunnel and shafts areas and their relation to static heads. There are nine shafts and twenty-seven tunnels with an area greater than 20 m2; for one shaft and three tunnels the area exceeds 60 m^2.

The nine unlined air cushions have cross-sectional areas of between 80 and 350 m^2, and heads of between 180 and 780 m. The one with a 780 m was initially a failure, however .

Nineteen shafts and fourteen tunnels have static heads greater than 400 m, and for five shafts and three tunnels it exceeds 600 m. About 80% of the unlined shafts have areas of less than 40 m2 and gradients between 43° and 45°, The others, mostly older ones, are spread in inclination between 32° and 70°, The shaft with an area of 90 m^2 is vertical,

About 50% of the unlined tunnels slope at 1:8 to 1:10, (A steeper gradient than 1:7 in the carriage-way is seldom used for tunnelling),

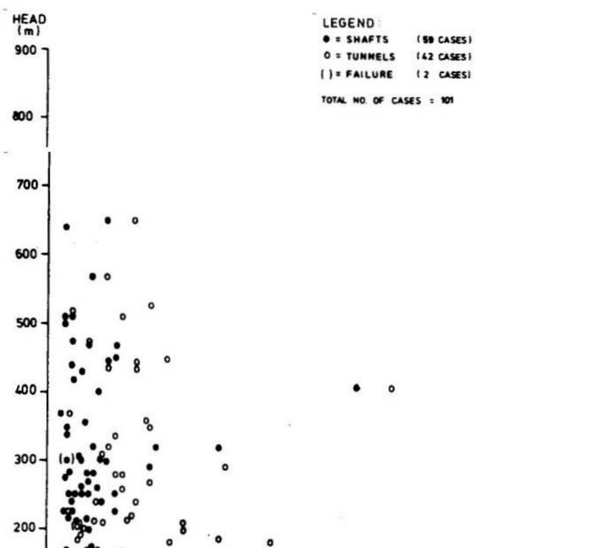


Fig. 2: The distribution of the cross-sections and heads of unlined high pressure shafts and tunnels in Norway.

ROCK TYPES AND STRENGTHS

About half of the unlined high pressure sections listed in Table 1 are situated in high metamorphic Precambrian rock, the other half in Palaeozoic rock. Some of the latter is also of rather high metamorphism, dating from the Caledonian orogeny, while elsewhere it is of low grade metamorphism (green schist facies) which can appear almost un-metamorphosed.

Table II. gives a simplified survey of rock strength properties from Norwegian underground excavations that include the unlined high pressure supply systems.

The lowest measured strength is found in a calcite marble. However, only a few weak rock types from hydro-electric power plants have been measured because the strength has not been considered a limiting factor for the use of unlined high pressure supply systems under Norwegian conditions. The values for stronger rock

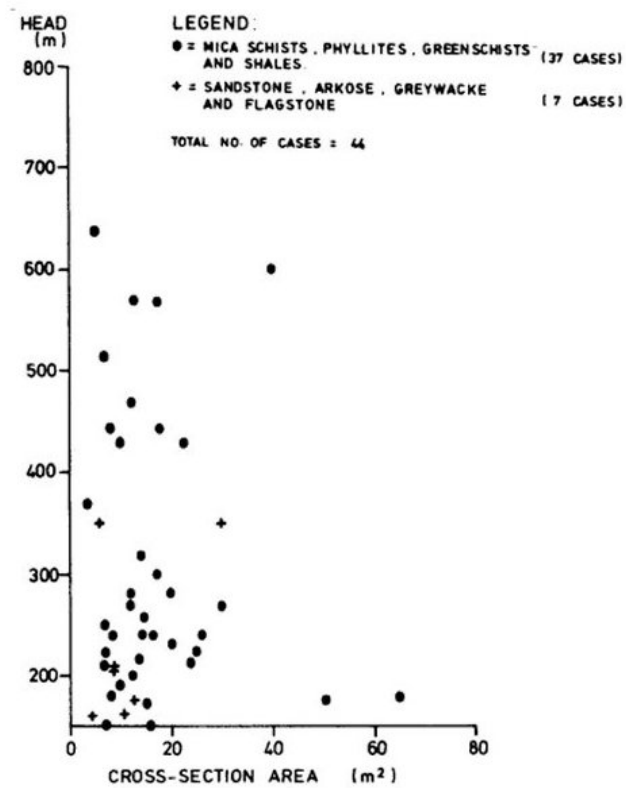


Fig. 3: The distribution of the cross-sections and heads of unlined high pressure shafts and tunnels in Norway situated in Rock with moderate strenght.

types were obtained for other purposes. The porosity as measured in rock specimens is not considered to be a limitation either under Norwegian conditions. In most cases the porosity is less than 1% , only infrequently higher than 2.5%.

Fig. 3 shows the relationship between tunnel and shaft areas and static heads for sites in Caledonian rock. It is evident from a comparison with Fig. 2 that the most extreme water pressures and the very largest cross-sectional areas are to be found in excavations in the strong Precambrian rocks. This is a coincidence, however, in part reflecting the topography and geology of the river system which have been developed.

Table 1. Unlined Shafts/tunnels with a head larger than 150m
See Kap04_Experience_Table1.pdf

	Uniaxial compression strenght (MPa)	**Pointload test I_s 32 mm cores (MPa)**
Precambrian rock	Max value ⊥ ~ 260	Max value // ~ 24
	Bulk value ⊥ = 145 +/- 60	Bulk value // = 18 to 9
	Mica gneiss ⊥ = 70 +/- 15	Bulk value ⊥ = 16 to 6
(Water saturated		Mica gneiss ⊥ = 8 to 4,5
samples)		Anisotrophy < 2
	Moderate content of mica, chlorite and calcite	
Palaeozoic-	Max value ⊥ ~ 300	Max value // ~ 22
Caledonian rock	Bulk value ⊥ = 165 +/- 90	Bulk value // = 18 to 4,5
		Bulk value ⊥ = 15 to 4
		Anisothropy < 2,5
(Water saturated samples)		

	Uniaxial compression strenght (MPa)	Pointload test I_s 32 mm cores (MPa)
	High content of mica, chlorite or calcite	
Palaeozoic-Caledonian rock (Water saturated samples)	Bulk value ⊥ = 70 +/- 35 Min value ⊥ =35	Bulk value // = 12 to 4,5 Min value // = 1 Bulk value ⊥ = 7 to 1,4 Min value ⊥ = 0,4 Anisotrophy : 70% < 2 ~ 30% = 2 to 5 Max = 18

Table II Survey of Norwegian rock strenght properties.
// = loaded paralelle to the foliation ⊥ = loaded in an right angel to the foliation.

STABILITY PROBLEMS AND ROCK SUPPORT

In Norway the most common stability problems in tunnels conveying water, including the high pressure ones, are caused by highly fractured rock, and gouge material with clay in fissures and fracture zones. Most often the clay is a swelling type, but talc, chlorite, kaolinite, etc. occur too. Added to this the occurrence of high and anisotropic rock pressures in relation to the rock strength causes intensive spalling. In some cases intensive local water inflows develop. Squeezing and slaking are rare.

Where power station operation is concerned the most serious problem has been clay, typically requiring lining of between 3% and 10% of the tunnel length with shotcrete or cast in place concrete. However, this varies from site to site between 0% and 60% .

Bolting against extensive spanning, and grouting against water inflow, have often been needed in addition. At some sites up to 50% of the tunnel length has been systematically bolted and partly shotcreted. At one other 3% of tunnel length has been systematically pre-grouted in order to advance through rock with high pore water pressure and heavy leakage.

W hen locating sites for shafts, efforts have usually been made to avoid weakness zones with clay gouges, as well as rock types and structures that indicate potential difficulties.

Nevertheless, a quarter of the shafts have had to be partly supported in the same way and for the same reasons as the tunnels. Grouting has frequently been used in the shafts, rock traps and plugs to reduce leakage to the terrain surface and more particularly to the underground power-house.

Accesses are always constructed to make inspection and maintenance work possible. In a few cases some complementary work has been done but with the exception of the three initial failures no operational problems have been reported later on.

CRITERIA AGAINST HYDRAULIC FRACTURING

To avoid hydraulic fracturing of pressure sections in the supply system which are not steel lined, the water pressure must not exceed the minimum in situ principal stresses in the rock mass at any point. Tensile strength in the rock mass could only be mobilized in an homogeneous rock mass where no unfavourably oriented discontinuity or weakness planes exist. (An unfavourable orientation of a weakness planes is one that is approximately normal to the minimum principal stress). However, such conditions could never be foreseen with any certainty and are consequently not taken into consideration.

The minimum principal stresses are defined by the sum of the gravitational and the horizontal tectonical stresses. The residual stresses that will be released from the rock close to the tunnel surface do not influence the in situ stresses.

As the internal pressure in one direction may exceed the gravitational stresses, the rock might be lifted slightly, causing the water to flood out and rendering the supply system useless. This results in major repair work, with steel lining.

Should the internal pressure exceed a low horizontal principal tectonic stress, caused by an extreme anisotropic tectonic stress, the rock mass will deform. If the bedrock overburden is sufficiently great, the water leakage will be limited and can be repaired by grouting. An air leakage from an air cushion caused in this way might have very serious consequences, however, since air has a conductivity 100 times greater than water in such thin fissures, as well as a very low density that not as water reduce the pore pressure with the height.

Two-dimensional finite element models for a plane at a right angle to the valley side are often used to estimate the gravitational stresses. Varying rock properties, clay gouges and complex rock structures will nevertheless cause difficulties. Other problems such as the usual variations in horizontal stresses have less influence on the mini- mum principal gravitational stresses in a valley side. By selecting unfavourable rock properties as input and increasing the head to 1.1 H, most of the difficulties are neutralized. In this fashion idealized finite element models have been used in the design of unlined supply water systems. Before 1970 the usual vertical overburden at shafts sloping less than 45° was claimed to be greater than 0.6 H. This caused the failures in valley sides steeper than 40° noted in Table I, nos. 18 and 23.

A simple equation is most often used to calculate the minimum gravitational stresses. However, it defines the equilibrium limits between water pressure and rock weight for a potential displacement towards the free surface.

$$L = (H \cdot \gamma w / \gamma r \cdot \cos\beta) \cdot F$$

L= Length of shortest distance from a location in the shaft/tunnel to the free surface

H= Static head of the location
γw= Unit weight of rock mass and water
F =Slope angle of the tangent plane where the shortest distance sphere touches the free surface
γr= Factor of safety

This equation has shown fair agreement both with empirical data and results from idealized finite element analyses. A factor of safety down to 1.05 has often been employed. In cases where an extensive clay-bearing weakness lone outcrops along the valley side dipping towards the valley floor, the above formula must be supplemented to take into account additional criteria to the shear strength and deformation properties of the weakness zone.

For air cushions it is also necessary to measure the local minimum horizontal stresses. These stresses depend on the tectonic forces and their local anisotropy, the creep properties in the rock mass, the orientation of adjacent fault zones, etc. There is often great variation over short distances. The earliest stage at which this measuring can be done, is after the supply tunnel face has passed a potential site for locating the chamber.

A warning must be given in the case of air cushions against relying solely on the estimated gravitational stress. At a depth of 700 m at Tafjord V (Table 1, no 57), the minimum horizontal stresses were clearly less than 7.7 MPa since a vertical hydraulic fracture occurred. The water leakages from the supply tunnel system could be ignored but the air leakages made the air cushion useless. The chamber was dry during the excavation. Small water leakage occurred in several parallel vertical fissures in the vicinity of the chamber and down to the power-house. Spalling was observed within the chamber along planes parallel to these fissures and to the hydraulic fracture that later developed. This vertical fracture in a right angle to the valley side indicated unusual anisotropic horizontal stresses that could not be a result of gravitational forces only.

The fact that a chamber fulfil the equation above, is absolutely free of visible water leakages before being filled with air is besides a high ground-water table insufficient evidence for concluding that the chamber will be air tight under pressure. The air pressure at Osa was about 19 bar. An unexpected air leakage about 15 Nm^3/min was in this case reduced to 1 ,2 Nm^3/min by grouting. (Ref. Edvardson & Sætren 1985.) At Tafjord however new fractures were opened when lowering the water table by air filling after successful grouting of the first one. The gap of the first fracture was about 0.3 mm and no closing was measured by emptying the water system. The air leakage was about 3.8 Nm^3/min at 77 bar. The new parallel fractures gave in average an air leakage of 2.8 Nm3/min at 77.5 bar. At time of fracturing it was for one day up to 4.3 Nm^3/min. At Tafjord the air cushion is not absolutely necessary for normal running of the turbine. An attempt to grout the new fractures will probably be made at a suitable time.

OTHER OBSERVATIONS

As far as Caledonian and older rock in Norway is concerned, high water pressure does not reduce rock stability or damage the usual excavation supports such as linings cast during advance and sealing with shotcrete, grout or rock bolts. In a few cases drainage holes have been drilled as a special precaution where there are long stretches with shotcrete in dried -up rock in shafts. Generally nothing has been done in tunnels to accommodate the high pore pressure behind shotcrete w hen emptying the tunnel system. Sufficient drainage will exist in the invert and through the shotcrete.

Rock support work during advance is fairly similar in both high and low pressure systems. If grouting is done at face or behind a lining, it al- ways has to be done to sufficient depth to prevent hydraulic fracturing by the build up of pore water pressure. No stability problems which are a special phenomenon for high pressure sections have been observed. The swelling pressure of smectite-containing zones depend on the petrography and the in-situ rock stress field. It behave equal in low and in high water pressure systems, if shotcreting is attempted.

On the other hand, more spalling might occur in the high pressure systems because of their frequently deeper location. This often increases the cost of support.

If an inclined supply tunnel is used, most of the loose material lying in the invert is unloaded. The rest is mainly washed down to the rock trap with the initial water filling. The rock trap is then cleaned out once or twice after a few months running. Later on, the transport of material along the tunnel to the rock trap is of little significance as long as sand and gravel are not washed into the supply tunnel at an intake. Almost all abrasion damage to turbines after the running-in period is caused by particles that have entered at an intake.

Where possible, it is advantageous to flush the tunnel before filling it (0.5 m^3 water per m span and 4 hours run per km tunnel is sufficient with a tunnel gradient of 1:10). This is however, seldom or never done today because of possible environmental damage. Only in a few cases is the invert cast in place usually in response to a request for a higher quality on the carriageway w hen advancing through weak rock. In approximately horizontal tunnels the invert usually is left untouched. In inclined supply tunnels a cast invert on levelled debris has to be drained and sufficiently secured against erosion during future filling and emptying the tunnel.

The potential loss of head due to perimeter roughness in unlined blasted tunnels is compensated by having a larger cross-sectional area. This becomes a question of excavation cost versus the cost of casting in place. A reasonable degree of smooth blasting gives a Manning’s ratio of 30-50. By using full-face boring machines (TBMs) the Manning’s ratio can be increased to about 60 and the unlined cross-section correspondingly reduced.

The often short stretch of steel lining from the unlined section down to the underground powerhouse (30-80 m) demands a watertight rock mass. Careful grouting is done at the upstream end of the steel lining, in addition to the grouting of possible leaking fissures directed to powerhouse or access tunnels inside the steel lining. An ad it is often blasted to the steel lining downstream of the plug for draining and grouting.

CONCLUDING REMARKS .
The encouraging financial and technical experience with unlined high pressure supply systems in hydro-electric power plants has result ed in their rapid development in Norway. Teething troubles associated with determining the required rock cover seem to be over. Since 1970 the over- burden design criteria have been applied at over 60 sites in Norway. With the exception of the three failures mentioned, no operational problems have been reported from the 100 or so individual shafts and tunnels with heads higher than 150 m.
The simple design with inclined high pressure supply tunnel and an enclosed unlined surge chamber with a cushion of compressed air has proved a successful solution.

Tabel I : Unlined shafts and tunnels with a head larger than 150 m.

No.	Name of project	Owner	Date of commiss-ioning	Unlined section			
				Max static head	S - shaft T - tunnel Ss - Surge shaft Sc - Surge air cushion TBM - tunnel boring machine Slope - Area - Volume.	Rock type	Geological formation
1	Svelgen I	BrK	1921	152	S - 45º - 4,5 m^2	sandstone	Devonian
2	Balmi	BaK	1958	150	S - 45º - 16 m^2	phyllite	Caledonian
3	Tafjord III	TK	1958	286	S - 32º - 6,2 m^2	gneiss	Precambrian
4	Porsa	PK	1959	222	S - 43º - 7 m^2 T ~ 0º - 7 m^2	greenstone dolomite shale	Caledonian
5	Fortun	ÅSV	1963	186	T ~ 0º - 10 m^2	gneiss	Precambrian
6	Søkna	STK	1964	174	S - 35º - 13 m^2	arkose	Caledonian
7	Straumsmo	NVE	1966	232	S - 45º - 20 m^2	flagstone phyllite gneiss	Caledonian Precambrian
8	Tafjord IV	TK	1966	450	S - 45º - 20 m^2	gneiss	Precambrian
9	Ørteren	HK	1966	160	S - 45º - 5 m^2	gneiss	Precambrian
10	Håen	STK	1966	200	S - 33º - 12 m^2 T - 1:12 - 12 m^2	greenschist phyllite	Caledonian
11	Kalvedalen	OFE	1967	225	S - 45º - 5 m^2	gabbro/quartzdiorite	Caledonian
12	Tysso II	TF	1967	217	S - 35º - 12 m^2	granite	Precambrian
13	Uvdal II	ABK	1967	170	S - 45º - 12,5 m^2 T - 1:9 - 22,5 m^2	quartzite	Precambrian
14	Søa	STK	1967	267	S - 45 º - 10 m^2	gneiss	Precambrian
15	Målset	NVE	1967	190	T ~ 0º - 10 m^2	phyllite	Caledonian
16	Rana	NVE	1968	220	S - 45º - 25 m^2 T - 1:10 - 25 m^2	mica schists	Caledonian
17	Trollheim	NVE	1968	297(252)	S - 45º - 17,3 m^2	augen gneiss	Caledonian

18	Byrte *(failure S)*	NVE	1968	303	(S - 60º - 6 m$^{2)}$	granite gneiss	Precambrian
19	Blåfalli	NVE	1968	302	S - 45º ~ 10 m^2	gneiss	Precambrian
20	Hove	NVE	1969	320	S - 63º - 14 m^2	phyllite gneiss	Caledonian Precambrian
21	Småvatna / Kvevang	KK	1969	220	S -45º - 7 m^2	flagstone phyllite	Caledonian
22	Fjone	VFK	1970	250	S -45º - 10 m^2	granite gneiss	Precambrian
23	Åskåra *(failure T)*	ESF	1970	210	S -40º - 9 m^2 (T - 1:100 - 9 m^2)	sandstone	Devonian
24	Goulasjokka	TFK	1970	180	S - 70º - 8 m^2	mica schists	Caledonian
25	Hovatn	KEV	1971	475	S - 45º - 7 m^2 T - 1:14 - 12 m^2	granite gneiss	Precambrian
26	Jørundland	AAK	1971	280	T - 1:10 - 22 m2 Ss - 60º - 6 m^2	banded gneiss	Precambrian
27	Bogna	NTE	1971	280	S - 45º - 13 m^2	granite gneiss	Caledonian
28	Savalen	OK	1971	257	S - 45º - 15 m^2	mica schists	Caledonian
29	Finndøla	VFK	1972	305 (265)	S ~ 43º - 9,5 m^2	granite gneiss	Precambrian
30	Svelgen IV	BrK	1972	160	S - 45º - 11 m^2	sandstone	Devonian
31	Ylja	OFE	1973	210	S - 45º - 9 m^2	meta-arkose	Caledonian
32	Sjona	HKL	1973	250	S - 45º - 7,1 m^2	mica schists	Caledonian
33	Skjomen	NVE	1973	360 (340)	T - 1:9 - 29 m^2 Ss - 50º - 11m^2	granite gneiss	Precambrian
34	Driva	STE	1973	510	T - 1:12 - 22 m^2 Sc - 6700 m^3 S - 45º - 8 m^2 T - 1% - 8 m^2	paragneiss	Precambrian
35	Dividalen	TFK	1973	268	S - 43º - 8 m^2 T ~ 0º - 8 m^2	mica schists graphite schists	Caledonian
36	Solholm	SKK	1973	206	S - 45º - 16 m^2	gneiss	Precambrian
37	Mauranger	NVE	1974	445 (430)	S - 45º - 18 m^2 T - 1:10 -23 m^2	gneiss	Precambrian
38	Ulvik II	BKK	1974	370	S - 45º - 4 m^2 T - 0,5% - 6,5 m^2	phyllite gneiss	Cambrian Precambrian
39	Olusjøen	ØF	1974	320 (204)	T - 1:14 - 18 m^2	gneissic tectonites	Caledonian
40	Jukla	NVE	1974	260 (80)	T - 1:20 - 22m^2	gneiss	Precambrian
					Sc - 6200 m^3		
41	Grytten	NVE	1975	310	T - 1:10,4 - 16 m^2	gneiss	Precambrian
42	Fagerli	SKS	1975	214	T - 1:10 - 24 m^2	mica schists	Caledonian
43	Vemundsbotn	BKK	1976	250	S - 45º - 12 m^2	gneiss	Precambrian
44	Lassajavri	KK	1977	150	S - 45º - 7 m^2 T ~ 0º - 8 m^2	quartzite phyllite	Caledonian
45	Svandalsflona	NH	1977	215	T ~ 1:15 - 14 m^2	phyllite quartzite	Caledonian
46	Åmela	TuK	1977	500 514	S - 45º - 5 m^2 S - 50º - 5 m^2 T - 0,8% - 7 m^2	banded gneiss	Precambrian
47	Bratsberg	TEV	1977	150	S - 45º - 7 m^2	greenstone	Caledonian

48	Båtsvatn	NVE	1977	210	T - 1:8 - 16 m^2	gneiss	Precambrian
49	Leirdøla	NVE	1978	~400	S - 45º - 15 m^2	granite gneiss	Precambrian
50	Lomi	SKS	1979	570	S - 44º - 13 m^2 T - 1:9 - 17 m^2	mica schists	Caledonian
51	Skibotn	TFK	1979	440	T - 1:10,4 - 18 m^2 Ss - 45º - 7 m^2	mica schists dolomite	Caledonian
52	Kjela	NVE	1979	180	T - 1:9 - 36 m^2	diorotic gneiss	Precambrian
53	Kolsvik	HKL	1979	430 (386)	T - 1:13 - 23 m^2 Ss - 45º - 10,2 m^2	mica schists	Caledonian
54	Lang - Sima	NVE	1980	525 (490)	T - 1:14 - 30 m^2 Sc - 45º - 10000 m^3	granite gneiss	Precambrian
55	Oksla	NVE	1980	450 (440)	T - 1:9 - 35 m^2 Sc - 19000 m^3	granite gneiss amphibolite	Precambrian
56	Steinsland	BKK	1980	470	S - 45º ~ 20 m^2	gneiss	Precambrian
57	Tafjorc V *(failure Sc)*	TK	1981	780	S - 45º - 8 m^2 T - 1:10 - 15 m ^{2} (Sc - 2000 m^3)	gneiss dunites	Precambrian
58	Holen	ØO	1981	320 (180)	S - 45º - 32 m^2 T - 0,12 % - 50 m^2	mica gneiss	Precambrian
59	Sy - Sima	NVE	1981	290	T - 1:14 - 52 m^2 Ss - 45º - 30 m^2	granite gneiss	Precambrian
60	Osa	HE	1981	210	T - 1:10 - 40 m^2 T - 0,2% - 40 m^2 Sc - 12500 m^3	shale granite diorite	Precambrian
61	Kvilldal	NVE	1982	460 (410)	T - 1:8,8 - 135 m^2 Sc - 136000 m^3	gneiss	Precambrian
62	Sildvik	NK	1982	640	S - 45º - 5 m^2 (TBM)	mica schists	Caledonian
63	Grana	KVO	1982	470	S - 45º - 12 m^2	mica schists	Caledonian
64	Litjfossen	KVO	1982	270	T - 1:13 - 30 m^2 Ss - 45º - 12 m^2	phyllite greywacke	Caledonian
65	Brattset	KVO	1982	240	S - 45º - 16 m^2 T - 1:12 - 26 m^2 Sc - 9000 m^3	phyllite greywacke	Caledonian
66	Sørfjord	NK	1982	515	S - 45º - 7 m^2	mica schists	Caledonian
67	Tjodan	LK	Under construction Jan. 1983	875	S - 41º - 7,5 m^2 (TBM)	gneiss	Precambrian
68	Saurdal	NVE		410	T - 1:16 - 100 m^2 Ss - 90º - 90 m^2	gneiss	Precambrian
69	Mesna	MK		340	S - 45º - 6 m^2 (TBM) T - 1:9 - 20 m^2	arkose	Precambrian
70	Skarje	ØO		650	S - 45º - 17 m^2	gneiss	Precambrian
71	Ullset	KVO		280 (230)	S - 45º - 12 m^2 T - 1:10 ~ 20 m^2 Sc ~ 6000 m^3	mica schists	Caledonian

72	Slunkajavrre	NK	Under construction Jan. 1983	240	T - 1:11 - 15 m^2 S - 45º - 8 m^2	phyllite marbel	Caledonian
73	Alta	NVE		180	T - 1:16 - 65 m^2 Ss - 90º - 50 m^2	mica schists	Precambrian
74	Kobbelv	NVE		600 (500)	S - 45º - 40 m^2	granite	Caledonian
75	Mosvik	NTE		150 170 186	S - 45º - 9,6 m^2 Ss - 45º - 6 m^2 T - 0,7% - 9,6 m^2 (TBM)	granite gneiss mica schists	Caledonian
76	Kvittingen	BL		250	S - 45º - 9,6 m^2	metarhyolite	Caledonian
77	Lomen	OFE		~350	S - 45º - 6 m^2 (TBM) T - 1:9 ~ 30 m^2	arkose	Caledonian
78	Myster	BKK		250	S - 45º ~ 20 m^2	gneiss	Precambrian
79	Slind	SKE		168	T - 1:10 - 15 m^2	phyllite	Caledonian

List of abbreviations of the owners etc.	
ÅSV - Årdal og Sunndal Verk	NTE - Nord-Trønderlags Elektristetsverk
AAK - Aust _Agder Kraft selskap	NVE - Norges vassdrags og Elektrisitetsvesen
ABK - Asker og Bærum Kraftselskap	OFE - Oppland Fylkes Elektrisitetsverk
BaK - Balmi Kraftlag	OK - Opplandkraft
BrK - Bremanger Kraftselskap	PK - Porsa Kraftlag
BKK - Bergenhalvøens Komm. Kraftselskap	SHK - Sunnhordland Kraftlag
BL - Bergen Lysverker	SKE - Selbu Komm. Elektrisitetsverk
ESF - El.forsyningen i Sogn og Fjordane	SKK - Sira-Kvia Kraftselskap
HE - Hedmark Energiverk	SKS - Salten Kraftsamband
HK - Hol kommune	TEV - Trondheim Elektrisitetsverk
HKL - Helgeland Kraftlag A/L	TF - Tyssefallene
KEV - Kristiansand Elektrisitetsverk	TFK -Troms Fylkes Kraftselskap
KK - Kvænangen Kraftlag	TK - Tafjord Kraftselskap
KVO - Kraftverkene i Orkla	TuK - Tussa Kraft
LK - Lyse Kraftverk	VFK - Vestfold Fylkes Kraftselskap
MK- Mesna Kraftverk	ØF - Østfold Fylke, Borgund Kraftselskap
NH - Norsk Hydro A/S	ØO - Øvre Otra I/S
NK - Nordkraft	

REFERENCES

1922 VOGT, J.H.L.:
«Trykktunneler og geologi». Norges Geol. Undersøkelse nr. 93, Oslo.

1968 VOGT, F. and SOLEM, R.:
«Norske Kraftverk>.Ingeniørforlaget, Oslo 1968.

1968 GOODMAN, R.E., TAYLOR, R.L. and BREKKE, T.L.:

«A model for the mechanics of jointed rock». Journal of the Soil Mechanics and Foundation Division. Proc. of the American Society of Civil Engineers. Vol. 94, No SM3.

1969 SELMER-OLSEN, R.:
«Experience with unlined pressure shafts in Norway». Proc. Int. Symposium on Large Permanent Underground Openings, 1969.

1969 BREKKE. T.L., BlØRLYKKE. S. and BLINDHEIM, O. T. :
«Finite element analysis of the Byrte unlined pressure shaft failure». Proc. Int. Symposium on Large Permanent Underground Openings, Oslo 1969.

1970 TØNDEVOLD, E. and STOKKEBØ, O.:
«The Folgefonna Project, Water Power». Dec. 1970.

1972
«Oversikt over norske uforede tunneler og sjakter med vanntrykk over 100 m samt enkelte andre med lavere trykk». Report 54402. Norwegian Geotechnical Institute, Oslo 1972.

1972 BARTON, N.:
«A model study of air transport from underground openings situated below ground water level». Proc. Symp. on Percolation through Fissured rock, Stuttgart. T3-A.

1972 DI BIAGIO, E. and MYRVOLL, F.:
«In situ for predicting the air and water permeability of rock masses adjacent to underground openings». Proc. Symp. on Percolation through Fissured rock, Stuttgart. T1-B.

1972 BJØRLYKKE. S. & SELMER-OLSEN, R.:
«Nødvendig overdekning i dalsider ved fjellrom med høyt innvendig vann- eller lufttrykk». Report no. 6, Department of Geology, Technical University of Nor- way.

1973 NTH, NGI & V HL:
«Lukket fordelingsbasseng med luftpute». Samlerapport. NVE. Oslo nov. 1973.

1973 BUEN, B.:
«Ingeniørgeologiske forundersøkelser ved Skjomen Kraftverk». Fjellsprengningsteknikk-Bergmekanikk, Tapir, Trondheim 1973.

1974
«Utforede tunneler. Spesielle problemer ved store tverrsnitt». Report 54 202-2, Norwegian Geotechnical Institute. Oslo 1974.

1974 SELMER-OLSEN, R.:
«Underground Openings filled with High-Pressure Water or Air». Bulletin of the International Association of Engineering Geology, Krefeid. No. 9, pp. 91-95.

1974 SMITH, P. T .:
«The Eidfjord hydro development in western Norway. Water Power». Vol. 26, No. 7, pp. 239-245, 1974.

1974 BERG-CHRISTENSEN. J.:
«Brudd i uforet trykk- tunnel ved Åskåra kraftverk. (Failure of unlined pressure tunnel at Åskåra Power Plant»>. Proc. Bergmekanikkdagen 1974. Tapir, Trondheim.

1975 RATHE, L.:
«An innovation in surge chamber design. Water power». Vol. 27, 244-248. 1975.

1975 STOKKEBØ, O.:
«Svingekammer med luftpute ved Jukla pumpekraftverk. (Air-pressurized surge chamber at the Jukla power and pumped storage plant»). Proc., Begmekanikkdagen, Tapir, Trondheim 1976.

1975 TESSEM, S.:

«Erfaringer med råsprengte trykkluft- magasin). Proc. Bermekanikkdagen. Tapir, Trondheim 1976.
1979 SOLVIK, Ø.:
«Sammenlikning av hydraulisk motstand i vanlig sprengte og fullprofilerte tunneler . (Comparison of hydraulic resistance in blasted and full-face bored tunnels»). Proc. Bergmekanikkdagen. Tapir, Trondheim 1980.
1980 MYRSET, Ø.:
«Underground hydro-electric power stations in Norway». International Symposium (on) Subsurface Space. Rockstore '80. Stockholm 1980. Proceedings, Vol. 2, pp. 691-699.
1981 SELMER-OLSEN, R. :
«Betraktninger over store vannlekkasjer i dyptliggende tunneler». Proc., Bergmekanikkdagen, Tapir Trondheim 1981.
1982 BERGH-CHRISTENSEN, J.:
«Unlined compressed air surge chamber for 24 atmospheres pressure at Jukla Power Plant». Proc. In!. Symp. on Rock Mech. related to Caverns and Pressure Shafts, Aachen 1982.
1982 BERGH-CHRISTENSEN, J.:
«Design of pressure shaft at Mauranger power plan!, Norway». Proc. ISRM Symposium Aachen 1982.
1982 BERGH-CHRISTENSEN, J. and KJØLBERG, R.S.:
«Investigations for a I,(XX) metre dead unlined pressure shaft at the Nyset-Steggje Project, Norway». Proc. Int. Symp. on Rock Mech. related to Caverns and Pressure Shafts, Aachen, Aachen 1982.
1982 BORCH, E.:
«The development of unlined shafts and tunnels in Norway». Proc. In!. Symp. on Rock Mech. related to Caverns and Pressure Shafts, Aachen 1982.
1982 BUEN, B. and PALMSTRØM, A.:
«Design and supervision of unlined hydro power shafts and tunnels with head up to 590 meters». Proc. ISRM Symposium Aachen 1982.

1982 JOHANSEN, P.M. and VIK, G.:
«Prediction of air leakages from air-cushion surge chambers». International Society for Rock Mechanics. Symposium on Rock Mechanics related to Caverns and Pressure Shafts. Aachen 1982.
1982 LUNDE, S. and STOKKEBØ, O.:
«Norway taps folgefonni glacier». Norwegian hard rock tunnel- ling. Tapir, Trondheim 1982.
1982 MYRSET, Ø.:
«Underground hydro-electric power stations in Norway». Norwegian hard rock tunnelling, Tapir, Trondheim 1982.
1982 MYRSET, Ø. and LIEN, R.:
«High pressure tunnel
systems at Sima power plant». Proc. ISRM Symposium Aachen 1982.
1982 SMITH, P. T. :
«A brief description of the Ulla-førre hydropower development». Norwegian hard rock tunnelling. Tapir, Trondheim 1982.
1982 WALBØ S,:

«Air cushion surge chamber in Kvilldal power plant the Ulla-førre development». Norwegian hard rock tunnelling. Tapir, Trondheim 1982.

1984

«Luftputekammer Tafjord K5» Report 8107.04 A/S Geoteam, Trondheim 1984.

Unlined high pressure tunnels in areas of complex topography

by E. Broch

Experience indicates that high pressure tunnels and shafts can be made unlined for water heads of up to 1000 m. A careful evaluation of geological and topographical conditions is required. This article shows how complicated and irregular topography should be revised on the drawing board to give a safe design. An example indicates how over-estimation of the rock mass overburden may cause leakage.

THE NORWEGIAN approach to the design of unlined high pressure shafts and tunnels has been presented in several recent articles in Water Power & Dam Construction-3 and at the International Symposium on Rock Mechanics Related to Large Caverns and Pressure Shafts (Aachen, May 1982)4-8. These articles include general discussions as well as more specific descriptions of a few cases.

To summarize briefly, Norway has 65- 70 unlined pressure shafts and tunnels with static water heads of 150 m or more, the highest head at present in operation being 780 m. Investigations for a 1000 m-head unlined shaft have also been completed. The oldest unlined shafts have been in operation for 60 years.

Two different design criteria are used. One so-called rule-of-thumb criterion introduced by Bergh-Christensen and Dannevig in 19719 is expressed as

$L > H\gamma_w/\gamma_r$ cops,

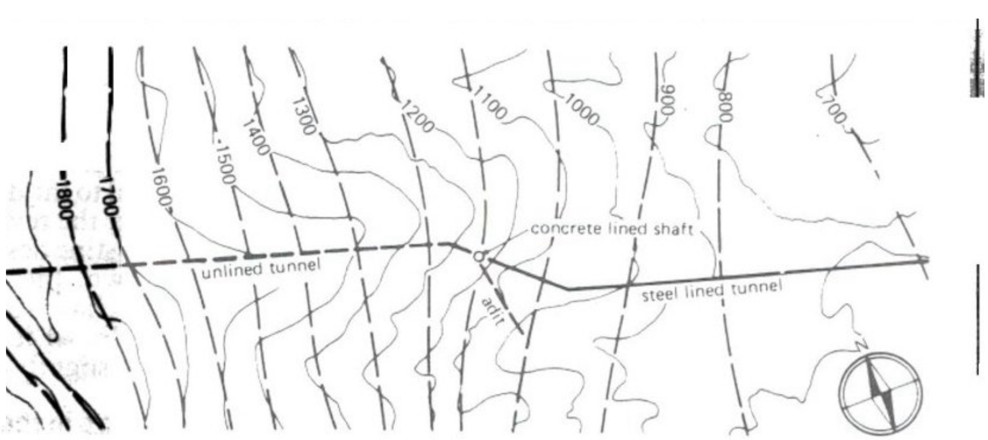
where:

L = shortest distance between the surface and the point of the shaft-tunnel studied (m)
H = maximum static water head at the point studied (m)
γ_w = density of water
γ_r = density of the rock mass
p = average inclination of the valley side.

The criteria are plotted in a L/H versus p-diagram in Fig. 3.

The second criterion was first presented in English by Selmer-Olsen in 1974[10]. It is based on the use of computerized finite element models, and the concept that nowhere along an unlined pressure shaft or tunnel shall the internal water pressure exceed the minor principal stress in the surrounding rock mass. Design charts are developed for a selection of valley sides and can be used to find quick, approximate solutions. An example is shown in Fig. 4.

Fig. 1. Tunnel system in a topograpichally complicated area. The height difference between contour linesis 25 m. dashed lines are revised contour lines for every 100 m.



Evaluation of the topography

Whichever of the two methods is chosen, a careful evaluation of the topography in the vicinity of the pressure tunnel or shaft is necessary .Although this is mentioned in most of the papers, in this author's recent experience there has not been enough emphasis on this very important point.
The explanation for this may be that the Norwegian topography is strongly influenced by recent glaciation. It is thus fairly simple and smooth, as compared with the topography one often finds in non-glaciated, mountainous areas, where streams and creeks have eroded deep and irregular gullies and ravines in the valley sides.
The remaining ridges, or so-called noses, between such deep ravines will, to a large extent, be stress relieved. They should therefore be neglected w hen the necessary overburden for unlined pressure shafts or tunnels is measured. This does not mean that pressure tunnels should not be running under ridges or noses - only that the extra overburden this may give should not be accounted for in the design, unless the stress field is verified through in-situ measurements.

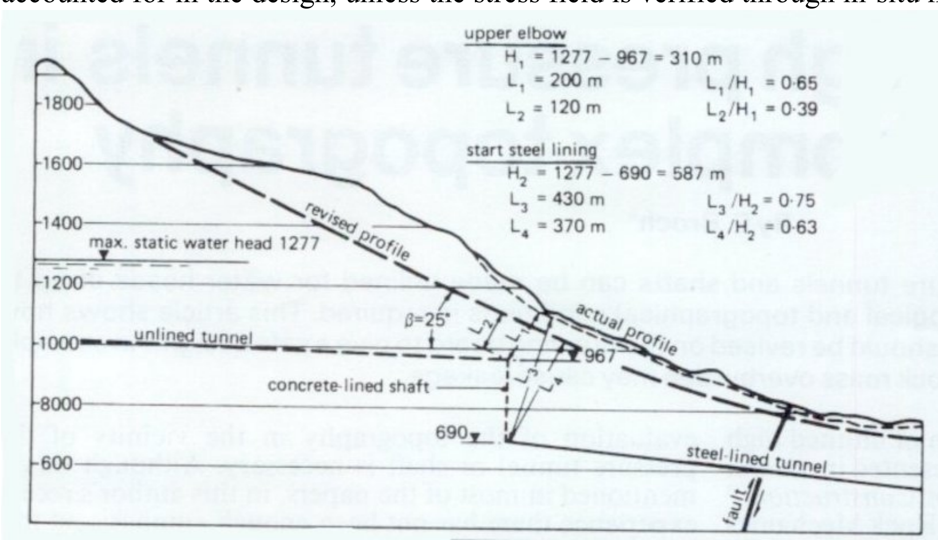


Fig. 2. Vertical cross section through the tunnel system in Fig. 1. The actual topograpical profile as well as the revised profile are shown. Water heads at upper elbow of the shaft, H_1, and at the start of the steel lining, H_2, are given. The shortest distance between these two points to the profiles are shown, $L_1 – L_2$ and L/H –ratio are calculated.

The best way to explain the design approach is probably to describe an example. The simplified map in Fig. I is based on an actual case in a tropical environment. The rocks along the part of the tunnel system shown are of the Mesozoic age. They are sedimentary rocks with a strike approximately normal to the tunnel direction and a steep dip towards the valley side.
The contour lines show parts of a very uneven valley side with a number of small creeks making deep cuts into the top soil as well as into the rock mass. As can be seen on the map, the tunnel system which consists of an upper and a lower headrace tunnel and a vertical shaft, runs, to a large extent, under a ridge. This is a good location for the unlined tunnels, but can, as will be shown, easily lead to an over-estimation of the overburden.
A simplified valley side where all protruding ridges and noses are cut off is demonstrated by the dashed lines on the map. These straightened contour lines are drawn for every 100 m. The result of this revision of the topography is shown in the vertical section through the
tunnel system, Fig. 2. The fully drawn line shows the actual topography, the thin, dashed line represents. the rock surface and the heavy, dashed line is the revised topography.
The inclination of the revised valley side increases from 10. in the lowest part to almost 30. in the upper part. Near the upper elbow of the vertical shaft the inclination is approximately 25..
As indicated on the profile, the upper headrace tunnel is unlined, the vertical shaft is concrete-lined, and the lower headrace is steel-lined, up to 20 m from the lower elbow. The maximum water level in the intake reservoir is at el. 1277.

Application of rule-of-thumb criteria

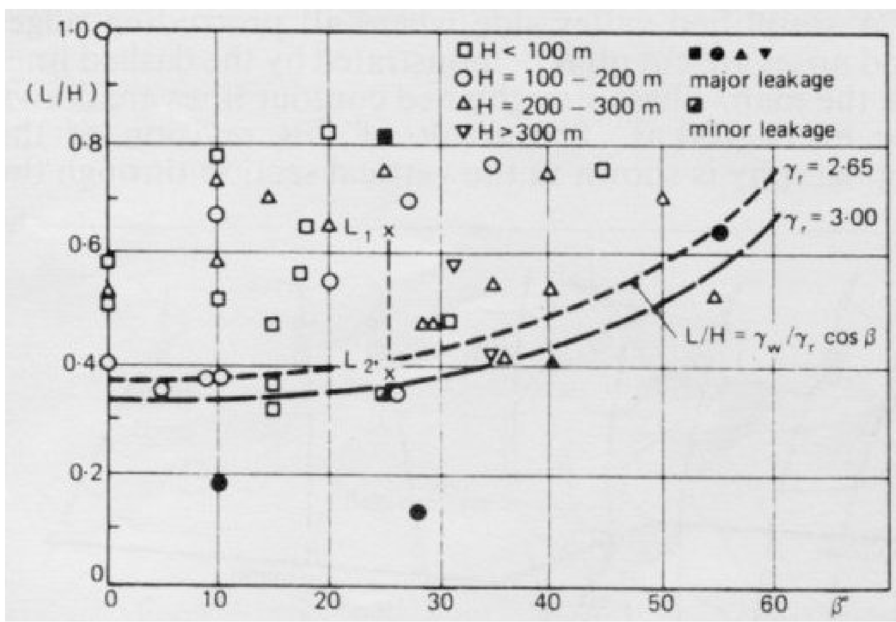
A concrete lining, even when it is reinforced, should not be regarded as a water-tight lining of a tunnel with internal water pressures of the order of magnitude which is encountered in this case. Thus there are two critical points along the waterway which have to be controlled to see that the overburden is in accordance with the criterion. The first is at the upper elbow of the shaft and the second is at the starting point of the steel lining.

At the upper elbow the water head is H 1 = 1277 -967 = 310 m. The shortest distance to the actual rock surface can be measured to L1 = 200 m, .which gives an over-burden ratio L1/H, = 0.65. The Similar distance to the revised profile is measured to only L2 = 120 m, giving an overburden ratio L2/ H, = 0.39. These results are plotted on to the diagram developed by Bergh-Christensen and Dannevig, as shown in Fig. 3, by using a valley side inclination of β=25º.

It is evident from the diagram that the revision of the topography has caused the overburden ratio to move from a fairly safe and acceptable position to a point in the diagram where leakages in unlined pressure tunnels have been experienced.

A similar analysis of the situation at the starting point of the steel lining gives a head H,2 = 1277 -: 690 = 587 m. The shortest distance to the rock surface is L = 430 m, giving L3/H 2 = 0.75. The shortest distance to the revised profile is L4=370 m, giving L4/Hl.=0.63. Both points are well inside the safe area shown in fig. 3.

Fig.3. L/H –ratios for unlined pressure shafts plotted against inclination of valley side, β. Filled points indicate shafts where leakage has been observed – after Bergh-Christensen and Dannevig[9]. Crosses marked L_1 and L_2 refer to the example described in this paper.



Application of design charts

In Fig. 4, the revised profile (dashed line) has been fitted to an existing design chart with a valley side inclination β=25º.. Because of a major fault zone striking parallel to the valley side, the model is stopped at el. 750. From that point the terrain is fairly flat for 700 m along the tunnel before it drops down to below el. 500, where the above- ground powerhouse is situated.

The valley side rises to approximately el. 2000. The uppermost part is, however, an east-west running ridge. This peak has little influence on the stresses developed in the rock masses in the valley side. A cut-off at el. 1750 has thus been provided for this model. This gives a valley with a depth D = 1750 -750 = 1000 m. With a water level in the intake reservoir of 1277 m, the static water head measured with the bottom of the valley as reference datum is H= 1277- 750 = 527 m. This gives an H/d- ratio of 527/1000 = 0.53. The 0.53-line is fitted into the chart. Note that this line

intersects the valley side at el. 1277. Here the water head, as well as the minor principal stress, is zero. Following the 0.53-line downwards, this line defines the balance between the increasing water head and the increasing minor principal stress. Outside the line, the water pressure is higher than the lowest possible stress in the rock mass. Hence, in this area there is a chance that the water may open existing joints with an orientation normal to the direction of the minor principle stress.
After the revised profile of the valley side is fitted in to the existing design chart model, and the actual H/d -line is drawn, the tunnel system can be plotted. This, of course, has to be done taking careful account of the scale. From Fig. 4 it can be seen that the upper elbow of the shaft is slightly outside the 0.53-line.

Leakage in the upper elbow area

As shown on the map in Fig. I, there is a 300 m-long horizontal adit to the upper elbow. W hen the tunnels were first filled, increasing seepage into the adit was observed even though only half of the maximum water pressure at the upper elbow was applied. The tunnel was dewatered, the concrete plug in the adit extended and extensive grouting was carried out. When the tunnel was refilled up to the reservoir level, leakage exceeding 600 l/s was discharging from the adit. Further concrete lining and grouting in the adit tunnel brought the leakage down to below 400 l/s.
The upper headrace tunnel was again dewatered. An inspection of the elbow area revealed several open cracks in the concrete lining on the top of the shaft as well as in the adit inside the concrete plug. The widths of the cracks were up to 10-20 mm. These cracks were clearly formed in tension.
During the subsequent drilling and grouting, adverse geological conditions were found in the elbow area. In some zones, grout takes were very large. There is thus no doubt that such conditions were the cause of some of the leakage which has been experienced. The leakage observed during the first filling, w hen only half of the maximum water pressure was applied, must have been caused solely by the geology, that is, existing joints which were open or only partly filled.
However, there are also good reasons to believe that later leakages are partly a result of an imbalance between the water pressure and the stress situation in the rock masses surrounding the elbow area. The best clue for this assumption lies in the tension cracking which obviously had taken place.
Repairs to the tunnel, which included about 100 m of steel lining at the elbow, were completed in July 1983. Since then it has functioned satisfactorily.

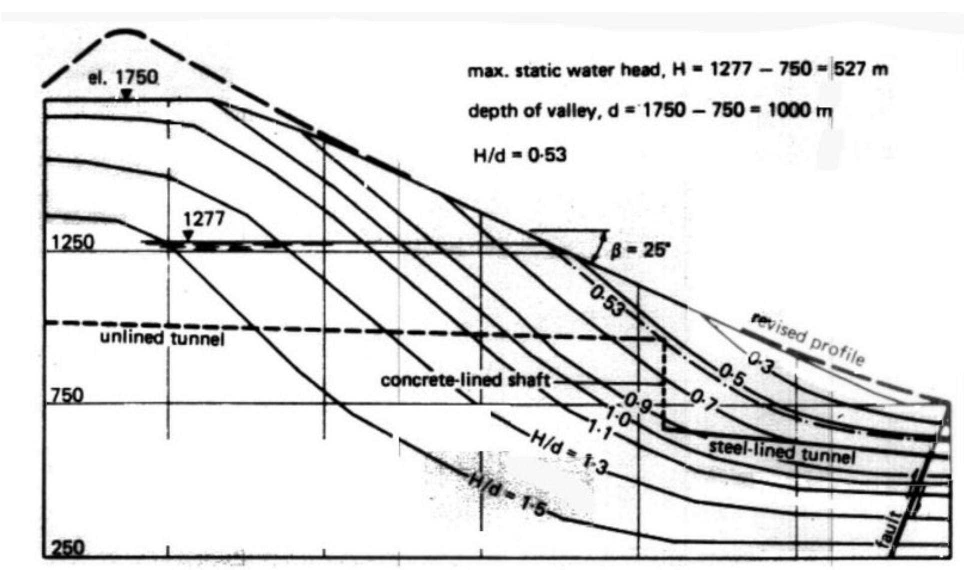


Fig. 4. Design charts for unlined pressure tunnels and shafts based on stress analysis of the valley side by the use of a finite element model . Informations from the example analysed in this article are fitted in ucing correct scale.

Conclusions

In regions with complicated topography one has to be careful when the design criteria for unlined pressure tunnels or shafts are being applied. Protruding ridges and noses do not add much to the stresses in the rock masses in a valley side, and should therefore be neglected. Simplified topographic maps with smooth contour lines, drawn inside such protruding features, should be prepared. Based on these maps, revised vertical profiles can be drawn. These revised profiles should be the basis for the design of high pressure tunnels or shafts without steel linings.
It has been shown that leakage from a tunnel with a water head of 310 m may, in part, be caused by an over-estimation of the overburden under a protruding ridge in a valley in a tropical environment. Open cracks in the concrete lining indicates that a concrete-lined tunnel should, from a design point of view, be treated as an unlined tunnel.

Acknowledgement

To concentrate the reader's attention to the principles, no names have been used in this article. The actual case on which the example is based is the Chivor project in Colombia, where a steel liner had to be introduced to deal with the deficient surface. The author was challenged to write this article by Dr. C. S. Ospina of lngetec, Bogota, consulting engineers to the Chivor project, as well as other similar projects in Colombia. the author is greatly indebted to Dr Ospina and his colleagues for inspiring discussions and help before and during the preparation of the article.

References

l. BROCH, E. "Designing and Excavating Underground Power- plants", Water Power& Dam Construction, No. 4, 1982.
2. BERGH-CHRISTENSEN, J. AND KJØLBERG, R. "lnvestigations for Norway's Longest Unlined Pressure Shaft", Water Power & Dam Construction, Vol.34, No. 4, pp. 31-35, 1982.
3. BERGH-CHRISTENSEN, l. "Surge Chamber Design for Jukla". Water Power & Dam Construction, Vol. 34, No. 10, 1982.
4. BERGH-CHRISTENSEN, l. "Design of Unlined Pressure Shaft at Mauranger Power Plant, Norway", Wittke, W. (ed): "Rock Mechanics: Caverns and Pressure Shafts", A. A. Balkema. Rotterdam, Netherlands; 1982.
5. BERGH-CHRISTENSEN, l. AND KJØLBERG, R. "lnvestigations for a l 000 m Head Unlined Pressure Shaft at the Nyset/Steggje Project, Norway", Wittke, W. (ed): "Rock Mechanics: Caverns and Pressure Shafts", A. A. Balkema, Rotterdam, Netherlands; 1982.
6. BROCH, E. "The Development of Unlined Pressure Shafts and Tunnels in Norway", Wittke, W. (ed): "Rock Mechanics: Caverns and Pressure Shafts", A. A. Balkema, Rotterdam, Netherlands; 1982.
7. BUEN, B. AND PALMSTRØM, A. "Design and Supervision of Unlined Hydro Power Shafts and Tunnels with Head up to 590 m ", Wittke, W. (ed): .'Rock Mechanics: Caverns and Pressure Shafts", A. A. Balkema, Rotterdam, Netherlands; 1982.
8. MYRSET, Ø. AND LIEN, R. "High Pressure Tunnel Systems at Sima Power Plant", Wittke, W. (ed): "Rock Mechanics: Caverns and Pressure Shafts", A. A. Balkema, Rotterdam, Netherlands; 1982.
9. BERGH-CHRISTENSEN, l. AND DANNEVIG, N. T. "Engineering Geological Consideration for an Unlined Pressure Shaft at Mauranger Hydro Power Station", Unpublished report in Norwegian, Geoteam A/S, Oslo, Norway; 1971.
10 SELMER-OLSEN, R. "Underground Openings Filled with High- Pressure Water or Air", Bulletin. International Association Engineering Geology, Vol. 9, 1974.

DESIGN OF UNLINED PRESSURE SHAFT AT MAURANGER POWER PLANT , NORWAY

J .BERGH-CHRISTENSEN
A/S GEOTEAM. Oslo. Norway

SUMMARY
The Mauranger Hydro-Electric Power Plant in Western Norway utilizes a 455 metre static head unlined pressure shaft. The paper describes the investigations performed and the criteria applied for the design of the shaft.

INTRODUCTION
The Folgefonni Hydro-Electric Power Project, developed and owned by the State Power Board, utilizes run-off from the great Folgefonn glacier in Western Norway. The Project comprises the Jukla Pumped- Storage Station (Bergh-Christensen 1982)
and the Mauranger Power Station, as shown in Fig. I.

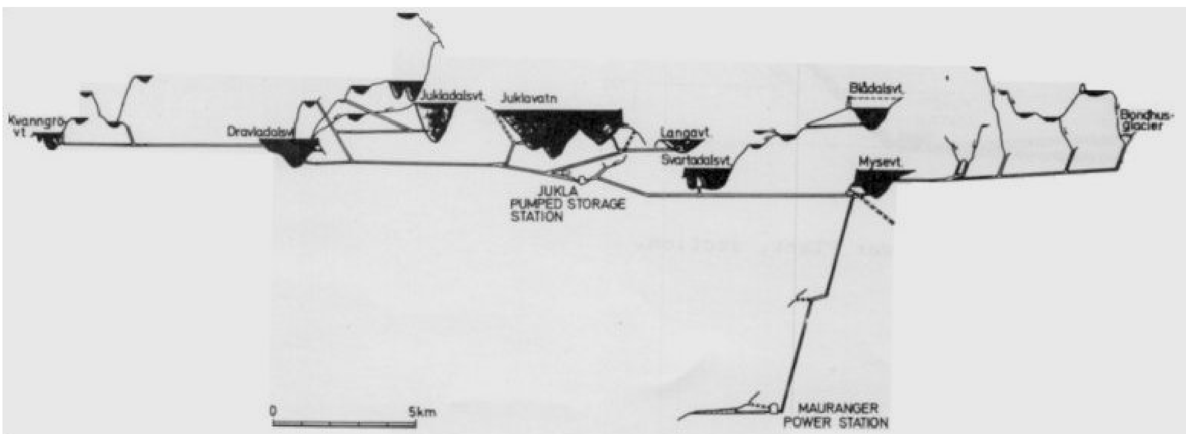


Fig. 1. The Folgefonni Hydro-Electric Power Project.

The general layout of the Mauranger Power Plant is indicated in Figs. 2 and 3. It includes a 20 m2 unlined pressure shaft with a maximum static head of 455 metres.
The utilization of unlined tunnels and pressure shafts in hydro-electric power plants has long traditions in Norway. More than 50 unlined shafts or tunnels with heads higher than 150 metres have been put into operation (Broch and Selmer-Olsen 1982).
Detailed planning of the Mauranger project started in 1970. The failure of a 300 metre static-head unlined pressure shaft at Brokke Power Plant in 1968 (Selmer-Olsen 1970), and then the failure of a 200 metre static-head unlined pressure tunnel at Åskåra Power Plant in 1970 (Bergh- Christensen 1975), indicated the need for detailed geological investigations for this type of project, as well as the need for re-evaluation of the "rule of thumb" design criteria applied up to then for placement of unlined shafts and tunnels.

SITE INVESTIGATIONS
Acting as engineering geological consultant to the State Power Board, A/S GEOTEAM was responsible for all site investigations and rock mechanics design evaluations performed prior to and during the construction of the Mauranger pressure shaft.

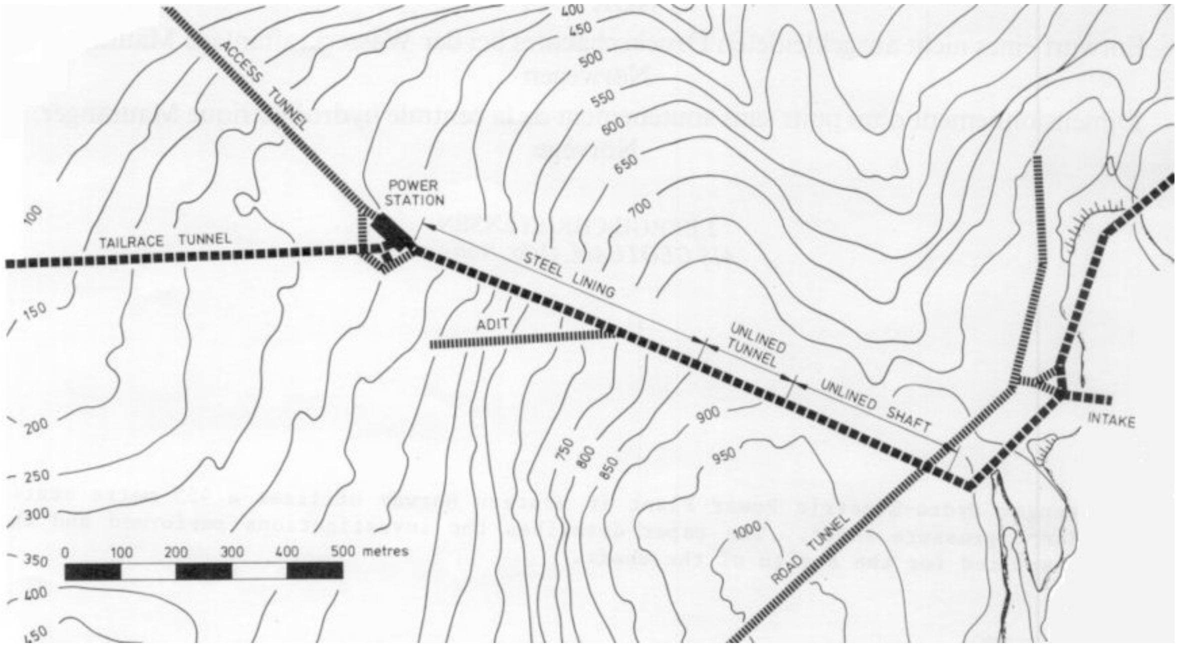


Fig. 2. Mauranger Power Plant, general layout.

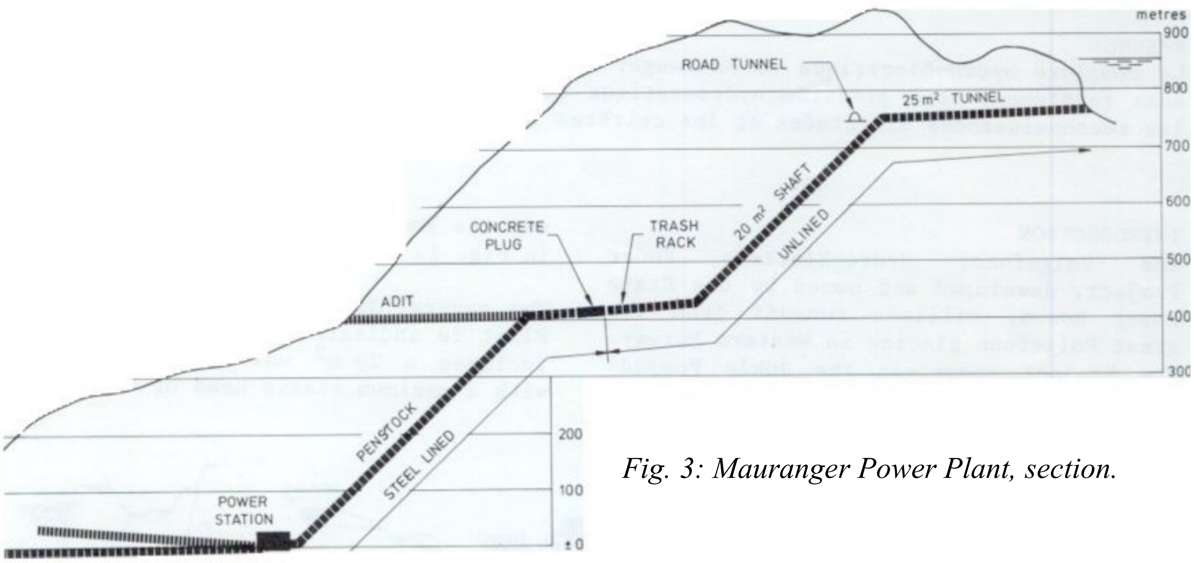


Fig. 3: Mauranger Power Plant, section.

As the first step in the field investigations, a geological survey of the site was performed, followed by detailed geological logging during excavation of tunnels and shafts. The rock is a Pre-Cambrian granitic gneiss. The head-race tunnel and the upper 70 metres of the pressure shaft are cut by a series of faults containing swelling clay gouge. The close proximity of a road tunnel called for extensive sealing and grouting works to he performed in these parts of the project in

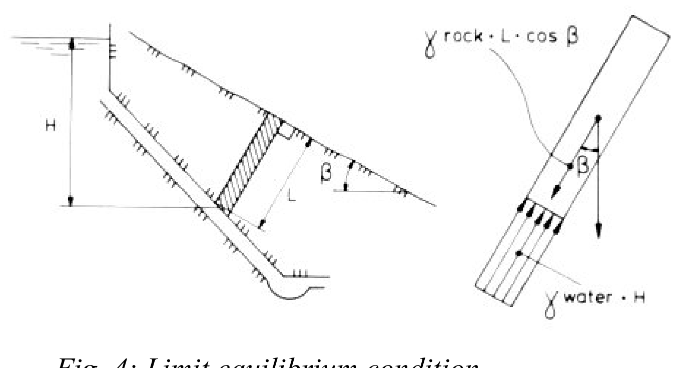


Fig. 4: Limit equilibrium condition.

order to avoid excessive leakages. The main part of the unlined pressure shaft and pressure tunnel designed for a maximum 455 metre static head was, however, placed in moderately jointed rock. For assessment of rock stress conditions, borehole in situ rock stress measurements were performed at two locations in the unlined high pressure tunnel. These measurements showed the rock stress to be highly anisotropic, with δ_3 -values of only 0,5 to 1,2 MPa; i.e. only a fraction of the intended water pressure.

"STATE OF THE ART" STUDY

In order to establish design criteria for the project a "state of the art" study was performed, including:

- Review of proposed design guidelines for unlined pressure shafts.
- Case studies of existing unlined high pressure tunnels and shafts, including known projects where total failure or severe leakages had occurred.
- Theoretical study of potential failure modes, including induced "hydraulic fracturing".
- A review of the failures of unlined shafts/tunnels at the Brokke and Åskåra power plants revealed that the two incidents had certain features in common:
- Both are situated in steep valley slopes.
- Both are cut by steep, permeable joints and weakness zones striking nearly parallel to the slope.
- The "side cover" of the tunnel/shaft, measured as the shortest distance out to the valley slope, is moderate, and lower than the vertical rock cover.

At both sites, the failure mechanism was thought to be as follows:
Leakage from the unlined shaft/tunnel allowed the build-up of high joint water pressures, which in turn caused a deformation of the "side-burden" rock and opening of joints to form a composite failure plane. This in turn caused washing out of gouge material and large scale water out-bursts in the valley slope.
Both incidents illustrated the importance of geological detail, and the importance of adequate rock cover for the safe construction of unlined shafts.

ROCK COVER CONDITIONS

A review was performed of rock cover conditions at the major unlined shafts and tunnels completed in Norway at that time (Bergh-Christensen and Dannevig, 1971), resulting in a diagram, as shown in Fig. 5.
A simple equation defining the limit equilibrium between water pressure and rock weight for a potential displacement towards a free surface as defined in Fig. 4 indicates necessary rock cover $L/H \geq 1/(2{,}65 \cdot \cos\beta)$
When this limit curve is plotted onto Fig. 5, it is seen to give a fair agreement with the empirical data. It is also found to be in agreement with criteria defined by Kieser (1960) for shafts without steel lining.
As shown in Fig. 5 the rock cover at Mauranger should be adequate compared with the above criteria and the empirical data.

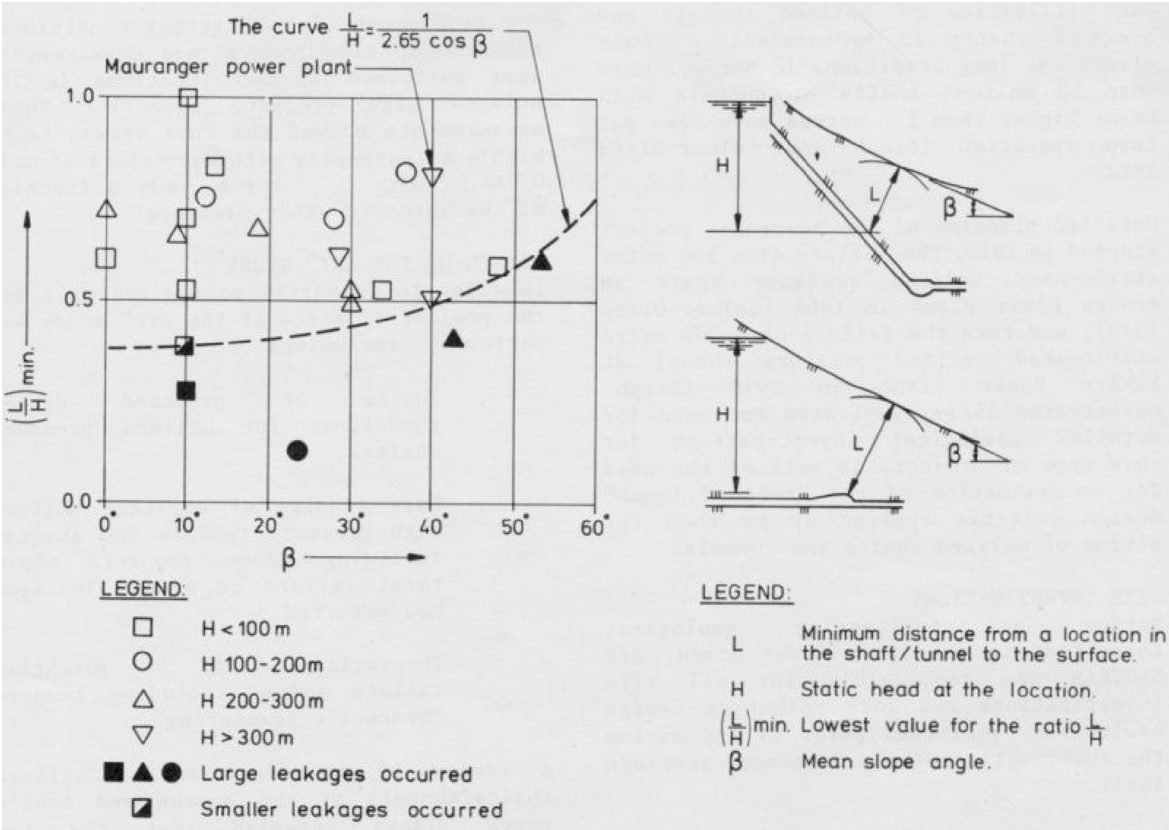


Fig. 5. Rock cover conditions at unlined shafts and tunnels.

HYDRAULIC FRACTURING EVALUATIONS

However, the very low values found by the in situ rock stress measurements necessitated an analysis of the potential for failure by "hydraulic fracturing". Hydraulic fracturing is a term well known in petroleum engineering as a method of increasing well yield. It comprises creating and propagating fractures in the reservoir rock by injecting fluid under pressure through a well bore to overcome native stresses and cause material failure. The same failure mechanisms might under certain conditions apply w hen high water pressures are applied to the large diameter "well-bore" of a pressure shaft or tunnel.

The basic mechanisms of hydraulic fracturing have been discussed by Hubert and Willis (1957), Barenblatt (1962), Geertsma (1966), Howard and Fast (1970), and others.

Some simple conclusions may be drawn from these papers:

a)

The basic condition for the opening up of a permeable joint is that the fluid pressure in the joint exceeds the native in situ normal stress on the joint plane.

b)

When condition a) is fulfilled, the propagation of a fracture may theoretically continue as long as injection of fluid is continued and the fracture acts as a closed system (without drainage or pressure loss).

c)

Field data from hydraulic fracturing of oil wells show that the "breakdown pressure" needed for initiation of fractures from a well is higher than the inferred effective overburden pressure, and also higher than the pressure needed for propagation of the fractures once initiated.

Applying these conditions on the Mauranger shaft, it was assumed that a potential composite failure due to hydraulic fracturing would have to develop along pre- existing joints (tensile strength of the rock being higher than the intended water pressure). The basic question might then be put as: Is the pressure tunnel or shaft intersected by any joint-sets with an orientation such that the native normal stress on the joints is less than the intended water pressure?

A detailed logging of joint-sets was performed in the pressure tunnel and shaft. Based on the in situ rock stress measurements, the orientation of planes with a theoretical normal stress lower than 4,5 MPa was calculated, and compared with the joint orientation data, as visualized in Fig. 6. Based on this analysis it was concluded that none of the native joint sets had an orientation that would facilitate a hydraulic fracturing type large scale failure.

Based on the above evaluations it was concluded that the conditions at Mauranger, as revealed by the site investigations, would allow the pressure shaft and pressure tunnel to remain unlined. However, some modifications in the design were recommended and a detailed programme for systematic grouting of a number of potential leakage section~ were specified.

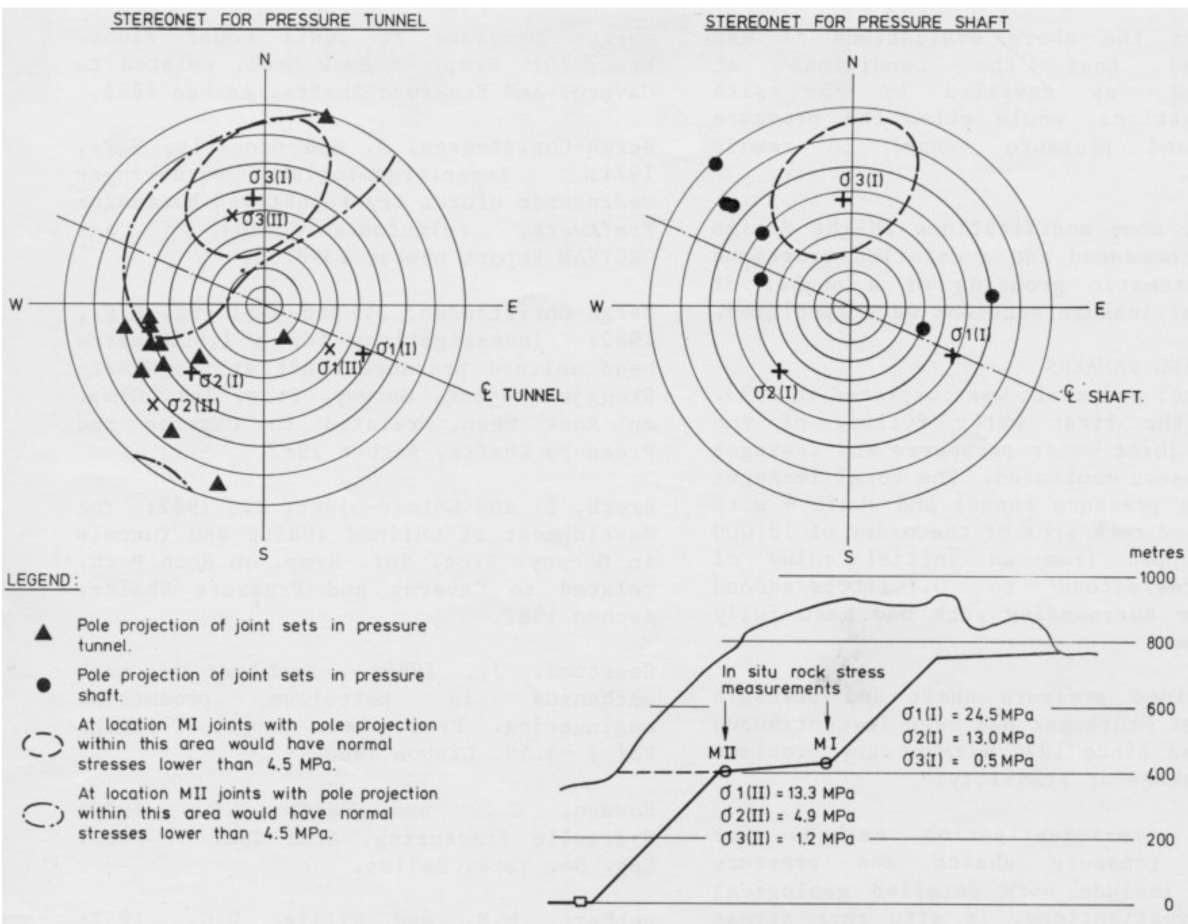


Fig. 6. Comparison of in situ joint data with critical normal stress orientations.

CONCLUDING REMARKS

The tunnel and shaft was completed in 1973. During the first water filling of the system, joint water pressures and leakages were closely monitored. The total leakages from the pressure tunnel and shaft -with an exposed rock area of the order of 13,000 m^2, dropped from an initial value of 0.5 litres/second to 0.1 litres/second when the surrounding rock had been fully saturated.

The unlined pressure shaft and pressure tunnel at Mauranger has been in continuous operation since 1974 without any problems with leakage or stability.

Current pre-investigation methods for unlined pressure shafts and pressure tunnels include both detailed geological site investigations, in situ rock stress measurements and the use of finite-element models for computer analysis of assumed stress conditions, as described by for instance Broch and Selmer-Olsen (1982) and Bergh-Christensen and Kjølberg (1982).

The present paper summarizes some of the investigations that constitute part of the basis for current Norwegian design methods. In particular, the concept of "hydraulic fracturing" related to pressure shafts and pressure tunnels is discussed.

REFERENCES

Barenblatt, G.J., 1962: The mathematical theory of equilibrium cracks in brittle fracture. advances in Applied Mechanics, 7, pp. 55-129.

Bergh-Christensen, J., 1975: Brudd i uforet trykktunnel ved Åskåra kraftverk (Failure of unlined pressure tunnel at Åskåra Power Plant).' Proc. Bergmekanikkdagen 1974. Tapir, Trondheim.

Bergh-Christensen, J. 1982: Unlined compressed air surge chamber for 24 atmospheres pressure at Jukla Power Plant. Proc. Int. Symp. on Rock Mech. related to Caverns and Pressure Shafts, Aachen 1982.

Bergh-Christensen, J. and Dannevig, N.T., 1971: Ingeniørgeologiske vurderinger vedrørende uforet trykksjakt ved Mauranger Kraftverk, Folgefonnanleggene. A/S GEOTEAM Report number 2398.03.

Bergh-Christensen, J. and Kjølberg, R.S., 1982: Investigations for a 1,000 metre head unlined pressure shaft at the Nyset/Steggje Project, Norway. Proc. Int. Symp. on Rock Mech. related to Caverns and Pressure Shafts, Aachen 1982.

Broch, E. and Selmer-Olsen, R., 1982: The development of unlined shafts and tunnels in Norway. Proc. Int. Symp. on Rock Mech. related to Caverns and Pressure Shafts, Aachen 1982.

Geertsma, J., 1966: Problem of rock mechanics in petroleum production engineering. Proc. 1st. Congr. I.S.R.M. Vol. I -3.59. Lisbon 1966.

Howard, G.C. and Fast, C.R., 1970: Hydraulic fracturing. AIME Soc. of Petr. Eng. New York, Dallas.

Hubbert, M.K. and Willis, D.G., 1957: Mechanics of hydraulic fracturing. Trans AIME 210, P. 153.

Kieser, A., 1960: Springer Verlag. Druckstollenbau. Vienna,

Selmer-Olsen, R., 1970: Experience with unlined pressure shafts in Norway. Proc. Int. Symp. on Large Permanent Underground Openings, Oslo 1969.

HIGH PRESSURE TUNNEL SYSTEMS AT SIMA POWER PLANT

Ø.MYRSET
Norwegian State Power System, Oslo
R.LIEN
Norwegian Geotechnical Institute, Oslo

SUMMARY
The article describes geologic conditions, investigations performed and design criteria for unlined high pressure tunnels and air cushion chambers. The economic advantages of unlined pressure tunnels for the upper portion of the head instead of steel lined shafts all the way to the top are discussed. Further, the paper discusses the decision of adopting the air cushion chamber solution for the Lang-Sima system. Problem related to establishing access to unlined high pressure tunnels are also described.

GENERAL DESCRIPTION
The Sima project started in 1920 when Osa Fossekompani A/S began building a power station in Osa, in the municipality of Ulvik, Hardanger. Work was however discontinued then. During World War II, the Germans, planning an aluminium plant in Osa, resumed the work. After a short time the project stopped once more.
In 1962. the State Power System made a fresh start t the project and extended considerably the area covered. In 1973. a reduced plan for development of Eidfjord North was adopted by the Norwegian Parliament Stortinget. The plant was named the Sima Power Plant.

1.1 The Sima Power Plant is today (1982) the largest in Norway
The power station's four generating sets have a total output of 1120 MW and the annual output of power is about 2800 GWh (2800 million kWh). The entire power works
cost 2000 million NOK, exclusive of interest during the construction period.
The Sima power station uses water from two geographically separate areas and properly speaking consists of two stations, Lang-Sima and Sy-Sima, installed in one generating hall.

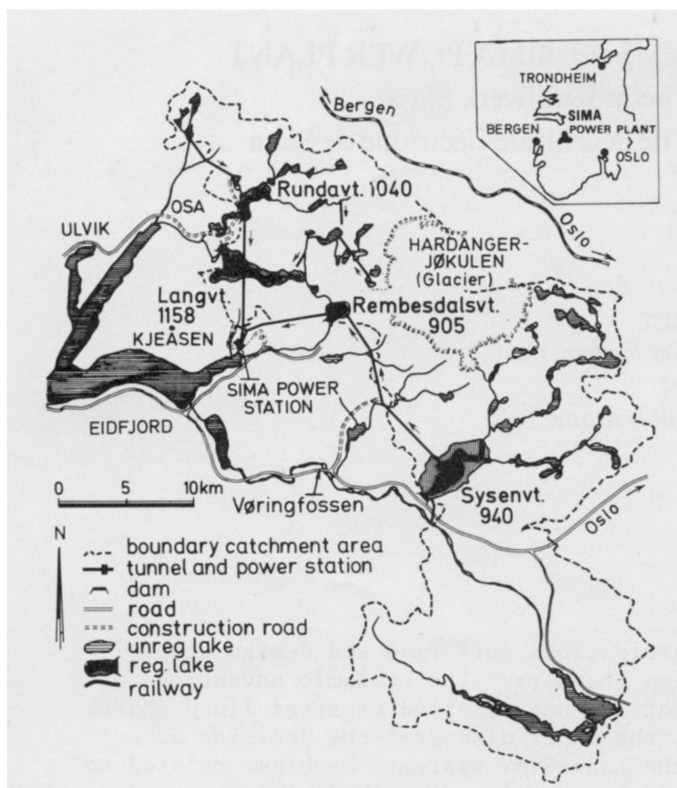


Fig. 1. Sima Power Plant area.

1.2 Ground conditions are favourable
Below 1000 meters, the Sima region consists of Precambrian rocks, mostly gneiss and granite, covered locally by phyllites of Cambro-Silurian age.
Loose material from the late Quaternary age is found on hillsides and valley bottoms. Exposed bedrock occurs frequently in the mountainous area.

1.3 Lang-Sima

The upper parts of the Sima River system, inclusive of Austdøla, are diverted to Langavatn. Langavatn, one of the intake basins, gave rise to the abbreviated name Lang-Sima. Langavatn is dammed up by rock- fill dams. The reservoir has 48 metres of working storage and top water level at 1158 metres above sea level. Nordøla is diverted to Rundavatn, the other intake basin. Rundavatn is dammed up to 1040 metres above sea level by a rock fill dam.

The two Lang-Sima generating sets can run alternately on water from Langavatn or Rundavatn. From the intakes, the water is carried to the valve chamber at Kjeåsen through one common, unlined tunnel, 8 kilometres long with a cross-sectional area of 30 square metres. The Kjeåsen area is 600 metres above sea level. The maximum water pressure in the unlined section of the supply tunnel is 525 m.

Instead of a conventional surge shaft in the water supply system, a comparatively new concept was selected, the so called air cushion chamber. The air volume is approximately 5000 m^3 with 48 atm. maximum air pressure. The pressure, maintained by compressors, is monitored continuously.

From the valve chamber at Kjeåsen, the water is led to the power station through a steel- lined shaft.

1.4 Sy-Sima

Water from Bjoreio is diverted to the main storage in Sysenvatn, which has 66 metres of working storage and a top water level 940 metres above sea level. Sysenvatn is the origin of the abbreviation Sy-Sima. The Sysell dam is also a rock fill dam, with an impervious core of moraine material. With its 3.6 million m^3 of fill, the dam is one of the largest in Norway. The crest of the dam is 1100 metres long and is easily seen from Highway 7 across the plateau.

Water is carried from Sysenvatn to Rembesdalsvatn through a 14 kilometres long tunnel, with a cross-sectional area of 35 m^2.

Rembesdalsvatn is the intake basin for Sy-Sima. Drawing from Sysenvatn can maintain a high water level during most of the generating period. Rembesdalsvatn has 45 metres of working storage and a tap water level 905 metres above sea level.

A tunnel 7 kilometres long with a sectional area of 52 m^2 leads from Rembesdalsvatn to the valve chamber at Kjeåsen. Maximum water pressure in the un- lined tunnel to Sy-Sima is 300 metres. The surge shaft in this supply tunnel also serves as the intake for the Åsåni river.

1.5 Pressure shafts with a total steel lining weight of 7500 tons

From Kjeåsen, the water is carried down to the power station in two steel lined pressure shafts, one for Lang-Sima and one for Sy-Sima. Each of the shafts divides into two branches at the bottom. The gradient of the 850 metres long shafts is 1: 1.

The Lang-Sima shaft has an internal diameter of 3.4 metres at the top and 2.75 metres at the bottom. At its thickest, the lining is 78 mm. Corresponding figures for Sy-Sima are 3.9 -3.0 metres in diameters and 68 mm thickness. The total steel weight is 7500 tons.

1.6 The power station -a large underground chamber

The machine hall situated in Simadalen, about 3 kilometres from Eidfjord, has a 700 metres long access tunnel. The excavated chamber is 200 metres long, 20 metres wide and 40 metres high. High stresses in the rock caused so-called rock burst. To secure the underground rooms against rock fall, 20,000 steel bolts were used in the power station and adjacent tunnels. The total length of these bolts amounted to 100 kilo- metres. Conditions in the rock made it natural to choose a very compact power station layout.

Lang-Sima's two generating sets operate with two gross heads -1152 metres (Langavatn) and 1034 metres (Rundavatn). Each set has a vertical Pelton turbine with 5 jets and a 250 MW generator.

Each of Sy-Sima's two 310 MW generating sets have a 5-jet vertical Pelton turbine. Up to the time of writing (-1982), these are the world's largest Pelton turbines as far as output is concerned.

2 INVESTIGATIONS CARRIED OUT AND USE OF RESULTS

In the area f the Sima Power Plant, bed- rock is expo ed nearly everywhere in valley sides and on mountain plateaux. In this situation, favourable for geological field observations, only geological mapping was performed as pre-investigation. Experience with tunnels and underground chambers in the Precambrian gneissic rocks in the area indicated conditions favourable for tunnelling and excavation of large underground chambers.

The main problem concerned firstly the high stresses in rock masses especially near the valley bottom. where rock burst phenomena could easily be observed on the surface (mountain heights up to 1500). and secondly weakness zones (faults) with gouge material containing swelling clay. Since few faults were observed little or no lining of tunnels was expected necessary. However heavy rock burst problems were expected in the power station and in the tunnels around it. Generally speaking. these observations were confirmed afterwards.

After tunnelling started, geological tunnel mapping was performed continuously. In the access tunnel to the Power Station and in the valve chamber at Kjeåsen (Fig. 2 and 5) rock stress measurements were carried out (Mining Department, The Norwegian Institute of Technology, Trondheim, Norway). Stress measurements were also done in the proposed location of the air cushion chamber. Table 1 shows the stress measurements made:

Table 1. Principal stresses measured.

Location	*Principal stresses. (MPa)*		
	σ_1	σ_2	σ_3
Power station			
Access tunnel	*19,5*	*9,5*	*3,2*
Kjeåsen valve chamber	*10,0*	*6,7*	*4,8*
Air cushion chamber	*13,0*	*10,0*	*6,8*

The data in Table 1 were used to find a proper orientation of the power station! and re-evaluate the necessary lining for the roof and walls. n addition, the data also enabled one to place the cone-shaped end of the steel lining for Sy-Sima and Lang-Sima according to the criterion that water pressure on unlined rock should not exceed the minimum principal stress (σ_3). The air cushion chamber should also follow the same criterion i.e. internal air pressure less than (σ_3). (Broch and Selmer-Olsen, 1982).

The design of unlined pressure tunnels and especially air cushion chambers needs to consider carefully the permeability of the rock. The rock at the cone-shaped end of the steel lining was of very good quality both for Sy-Sima and Lang-Sima and no water leakage into the tunnel was registered.

This was found satisfactory. but due to a fault zone in the rock. the steel lining for Sy-Sima had to be extended such that it ran over the fault zone.

In the area of the air cushion chamber, extensive core drilling and permeability tests were performed to check the imperviousness of the rock mass. As mentioned before, the air cushion chamber site was first of all selected at a location with sufficiently high minimum principal stress (σ_3), but at the same time at a location where a minimum of air leakage could occur. The site chosen showed no water leakage into the tunnel and the rock surface was dry.

The permeability tests showed practically no water flow with pressures up to 60 bars. It was concluded that no injection work was necessary.

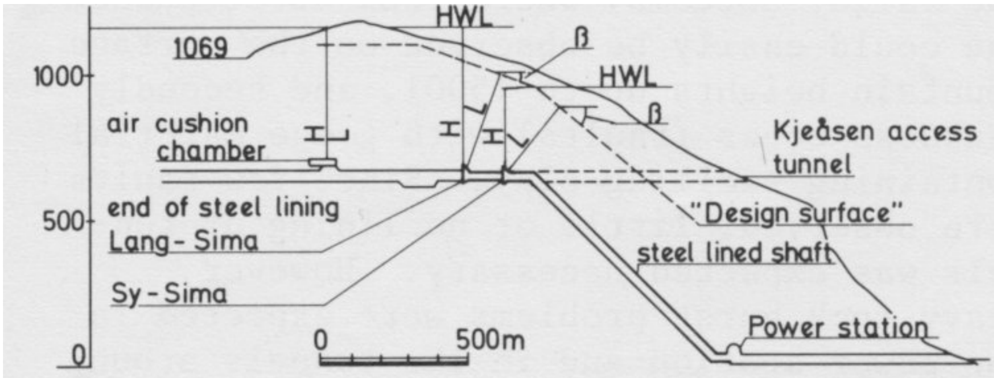


Fig. 2 Longitudinal saction of the Sima Power Plant. See Table 2.

2.1 The overburden criterion for unlined high pressure tunnels seems to give a safe design

Figure 2 illustrates the overburden criterion used at NGI for the design of unlined high pressure tunnels and air chambers: The weight component of the overburden perpendicular to the surface should exceed the water pressure. With reference to Fig. 2 the criterion can be expressed by the formula:

$$L \cdot \cos\beta \cdot \gamma_r \geq H. \cdot \gamma_w$$

Particular attention should be paid to unfavourable joint sets or fault zones and to special topographical conditions. For in- stance, edges or any important convex formation should not be considered as overburden, and a line for the so-called "Design Surface" should be drawn.

See Fig. 2. In the case of the Sima Power Plant this simple criterion agrees well with the results of more sophisticated finite element analyses, described by Broch and Selmer-Olsen (1982).

Table 2 applies the overburden criterion to the Sy-Sima and Lang-Sima pressure tunnels. Figure 3 presents the results from the analysis of 43 high pressure shafts and tunnels. The dashed curve indicates equilibrium between water head (H) and overburden (L) measured normal to the "Design Surface" . The Sy-Sima and Lang-Sima data are also plotted in the diagram. The distance above the dashed curve can be regarded as an apparent safety factor (F). Lang-Sima has been in operation for 2 years and Sy-Sima for 1 year. Inspection after the emptying the Lang-Sima system showed no sign of deformation that could cause either water nor air leakage. (Johansen and Vik, 1982)

Table 2. Data for the overburden criterion at Sy-Sima and Lang-Sima. See Fig. 2.

Location	H	L	L/H	β^0	Min. L/H	Apparent F. .
Sy-Sima						
Cone-shaped end of steel lining	300	300	1,0	40	0,5	2.0
Lang-Sima						
Cone-shaped end of steel lining	520	362	0,7	20	0,4	1.75
Air cushion	490	≈400	≈0,8	0	0,38	≈2.0

H: Water pressure in meters. β^0: Slope angle.
L: Overburden to surf in metres. Min. L/H: Minimum Ration L/H after diagram.

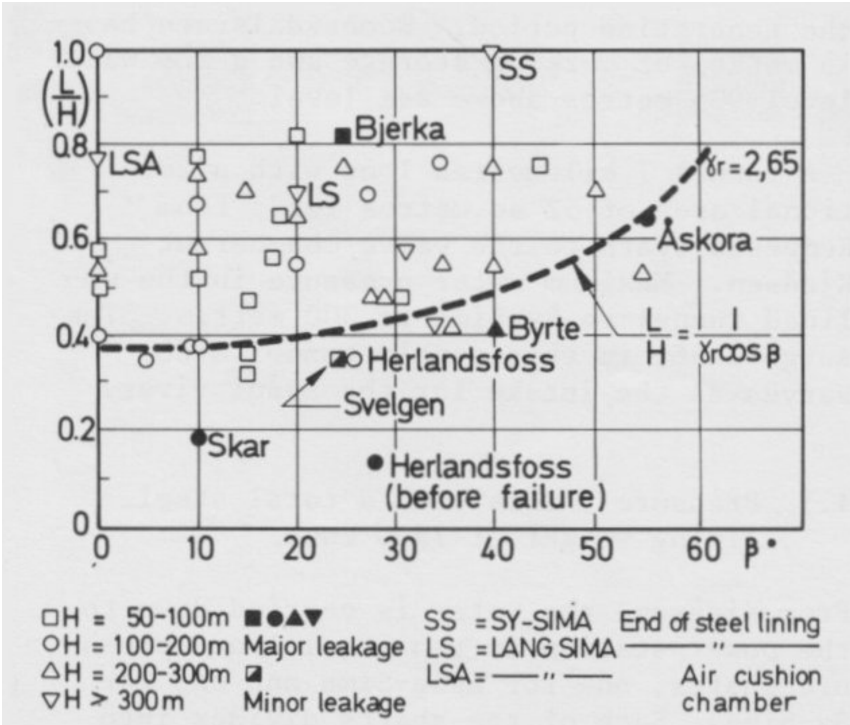


Fig. 3
Diagram for the ratio overburden / water head versus inclination of "Design Surface".

3 ECONOMICAL ADVANTAGES OF UNLINED TUNNELS

Preparatory work on the project started in 1973. The major part of the work took place during the second half of the seventies. Lang-Sima started operation in early 1980 and Sy-Sima in early 1981, both with nearly full storage. A representative starting date can thus be taken as mid-1980.
The costs for the whole power project amounted to about 2,000 million current NOK, exclusive of interests. In reference to an average building index, this amount corresponds to 2,400 million 1980-NOK. If one includes 7% per year in interest over the period the capital is invested, the costs reach 2,800 million NOK (in 1980, 5 Norwegian NOK = US $ 1.00). The total is approximately distributed as follows: 45% on the regulation area (dams and diversion tunnels), 35 to 40% on the power station and 15 to 20%, or more precisely 500 million NOK, on the supply systems (from the power station wall to the intake).
It is of interest to study the distribution of costs within the supply systems one can begin with the unit costs for the most important parts of the project. These unit costs are given as total costs at the time of the starting date, mid-1980. (The same applies to capitalized energy loss.) For both the building and installation works, investment tax (about 13%) , local and central joint costs (auxiliary. constructions and operations, administration, etc.) and an interest rate of 7% have been added to the direct costs. (It is approximately correct to distribute the joint costs proportionally If one increases the quantity of direct works in one location, the contribution of the auxiliary constructions and operations in also increase. In the opposite case the direct work will go slower, thus increasing the direct costs). Together these added costs result in total costs in terms of 1980-NOK, approximately double the current direct costs.
Table 3 lists for Sy-Sima the costs and capitalized value of the energy loss per metre length of shaft or tunnel. The table compares steel-lined and unlined shaft/ tunnel sections. In the unlined tunnel, the stabilization costs are low. On the 7-km stretch fro the cone-shaped end of steel lining to the Rembesdalsvatn intake, about 1% of the length is lined with concrete.
In the Lang Sima supply tunnel, the same favourable rock conditions prevail in the selected tunnel alignment.

Table 3. Sy-Sima supply system. Illustration of costs and capitalized energyloss in kr/m lengt of steel-lined and unlined tunnel.

1 System part / Location	2 Technical data dimensions	3 Cost kr/m	4 Value of energy loss kr/m	5 Total (3 + 4) kr/m
Steel-lined shaft. El.40	Shaft area = 12 m^2 Concrete = 5 m^3/m Lining Ø = 300cm Steel weight= 3000 kg/m	90,000	40,000	130,000
Steel lined shaft El.600	Shaft area = 18 m^2 Concrete = 6 m^3/m Lining Ø = 390cm Steel weight= 3000 kg/m	90,000	10,000	100,000
Steel lined tunnel, from El.600to end of steel lining.	Tunnel area ≈ 22 m^2 Concrete = 10 m^3/m Lining Ø = 390 - 400cm Steel weight= 3000 kg/m	90,000	10,000	100,000
Unlined tunnel, from end of lining to intake	Area = 52 m^2	11,000	2,000	13,000

Table 3 provides data for the shaft at El. 40. The steel lining from El. 40 and upwards is designed for a "standard" transfer of the loads to the rock (about 50%). Below this elevation, the load transfer ability is educed by the excavated rock at the power station. The steel thickness increases from 39 mm at E. 40 to 68 mm at the bottom El.0).

As shown in Table 3, the costs for the steel-lined part of the Sy-Sima supply system amount to about eight times the cost per metre length of the unlined tunnel.

The ratio between the value of the energy loss is even larger. The cost differences in Lang-Sima follow the same trends. Even if the cross-sections in Lang-Sima are somewhat smaller over the whole length; the water press re necessitates a larger steel thickness. The combined steel weight of 7500 ton , is therefore distributed approximately evenly between the two shafts.

Inclusive of various complementary mechanical equipment at Kjeåsen (for instance valves), the steel-lined parts of the sup- ply systems cost about 250 million NOK, and amount to the same cost as the unlined sections: about 15 km of supply tunnels with accesses, shafts, air cushion chamber and supplementary mechanical equipment.

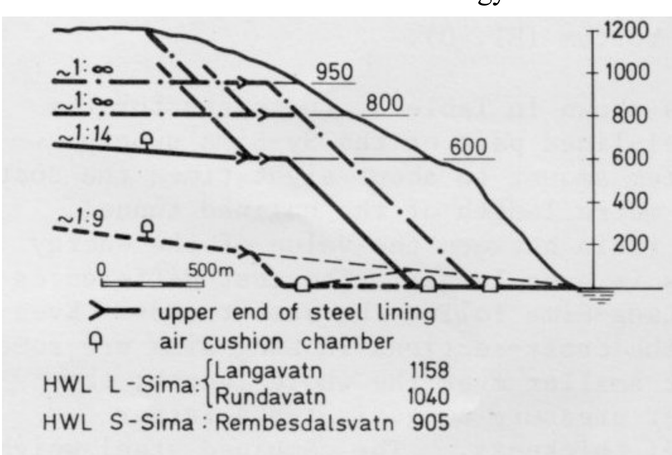


Fig. 4. Alternatives for building of Sima power station (intake – outlet)
Alt. 1: Carried out in the 1970s ————
Alt. 2: Typical for the 1950s ·--·--·--·--·
Alt. 3: Possible for the 1980s ------------

Since the economical advantages of using unlined tunnels are so important, persistent efforts have been made in the last 20 to 30 years to replace the lined tunnel concept with unlined constructions. This evolution is shown in Fig. 4 The figure gives the scheme used in the 1970s {alternative 1), the solution applicable to a design carried out in the 19508 (alternative 2), and the potential solution for a design made during the 1980s.
With alternative 2, the steel lining for Sy-Sima is taken up to El. 800 and to El. 950 for Lang-Sima. In addition to the access and work area at El. 600, the same must also be established at the higher elevations. The length of the supply tunnel remains unchanged, but the station could be moved 300 m outward, due to the steel lining system past Kjeåsen. It also gives a reduction in tunnel length at the power station level.
Table 4 gives costs and capitalized energy loss per metre length for the tunnels leading to the power station. The head loss in the tailrace tunnel is equalized to a change ill turbine level (Pelton turbine). An estimate of the cost for alternative 2 indicates an 80 million NOK additional cost relative to alternative 1. In addition, the capitalized energy loss would have increased.
The economical consequences of the different methods of carrying water from the intake to the outlet can be illustrated by a comparison of costs plus capitalized energy loss per metre along the horizontal alignment (using average energy loss for the shafts).

Table 4. Tunnels leading to power station. Illustration of costs and capitalized energy loss in kr/m lenght.

1 Tunnel	2 Technical data, Equipment, Dimentions	3 Cost kr/m	4 Value of energy loss	5 Total (3+4) kr/m
Access	Area = 48 m^2 with roadway, light system Monitoring cables, ventilation equipment.	17,000		17,000
Cable	Area = 22 m^2, 4x3 units 400 KV cables.	29,000	2,000	31,000
Tailrace	Area = 110 m^2	19,000	3,000	22,000
Total		65,000	5,000	70,000

For the Sima project, the following costs apply:
- unlined tunnels = 25,000 kr.
- steel-lined shafts = 300,000 kr/m .
- tunnels leading to the station = 70,000 kr/m.

If the Sima power development was planned today, one should seriously consider alternative 3, shown in Fig. 4. (A power complex in operation consists entirely of unlined systems with air cushion for water heads up to 780 m.)
The station is receded 600 m in the rock. Sy-Sima is also in this case designed with air cushion. With this alternative, the monetary savings relative to a surge shaft solution are larger than the value of lost energy from the River Åsåni (included in alternative 1, Fig. 4). The significant savings come from a reduction in length of the steel-lined section, -for example, the steel weight reduces from 7500 to 2500 tons -and from the elimination of the access and work area at Kjeåsen.
On the whole, the costs of alternative 3 are estimated as 180 million NOK less than the costs for alternative 1. The energy loss is also smaller, even where the 17 GWh inflow energy from Åsåni is not used. In addition, constraints -on the environment will be reduced and the regulation stability of the station improved.

Relative to alternative 1, alternative 3 presents some significant advantages. The reduction in costs corresponds however, to no more than 6 or 7% of. the entire power plant costs. The "savings" would be quickly lost if a mishap would shut down the power station operation for some time. Since the alternative 3 solution lies on the borders of today's experience and technology, advanced and thorough field investigations and tests should be conducted before finalization of the design.

4 SURGE SHAFT OR AIR CUSHION CHAMBER

4.1 General technical and economical factors

Compared to surge shafts, air cushion chambers will generally prove more economical the higher the water pressure, since the surge shafts must be taken up to the highest regulated water level and transport costs during construction and stabilization increase with shaft length.
The air cushions in use up to now require longer operation interruption than surge shafts during revision of the supply system. This question concerns essentially the choice of compressor capabilities. It is difficult to establish the reduction in quality of power complex as a function of length of interruptions in operation. One should investigate the expected need for revisions (inspections, emptying of sand trap, repair after a stone fall, etc), the stations operation time, the storage possibilities, etc.
With air cushion chambers, the station generally acquire a better regulation stability than technically and economically feasible with surge shafts, but it is again difficult to translate this advantage to monetary qualities.
The air cushion solution involves however, a risk factor: in case of mishap, it is possible that the air blows out through the supply system. The design therefore incorporates some safety measures that ensure that such an accident will probably not occur. Instrumentation and compressors for the air cushion chambers require supervision and maintenance, whereas experience with surge shafts has shown them as maintenance-free. The power consumption during filling and refilling of air is believed today to represent a relatively modest quantity.

4.2 Conditions at Sima

As shown in Fig. 5, Sy-Sima is designed with surge shafts, which also serve for water transport from the River Åsåni. The air cushion solution has therefore not been investigated at Sy-Sima. At Lang-Sima, design of an air cushion chamber was selected after the supply tunnel had been driven through its actual position. At this stage, possible stabilization with water curtains and semi-cylindrical steel lining was considered. Preparatory work was done to position water

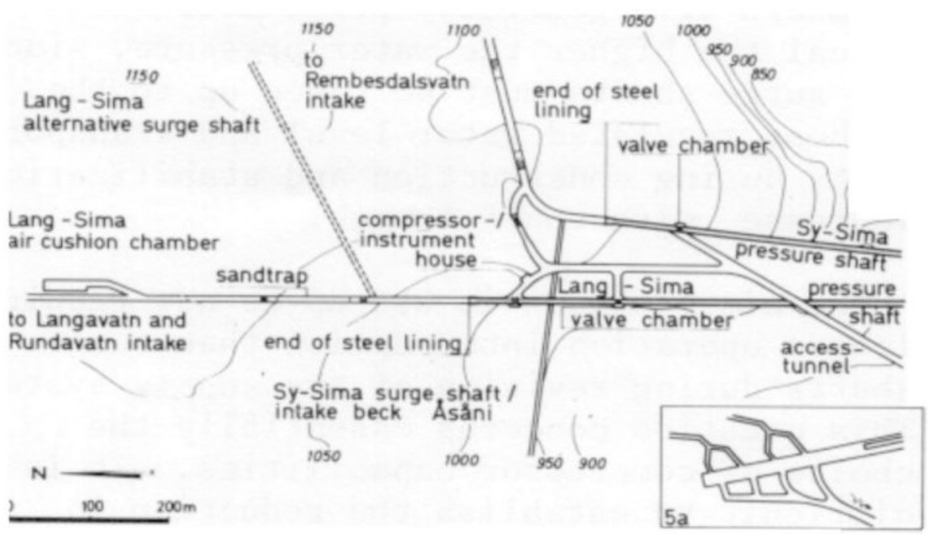


Fig. 5:
Plan of Kjeåsen surge chamner with valves, etc (ca El.600).
Fig. 5a:
Power station with branched pipes, tailrace, access and cable tunnels (ca El.0).

curtains in case air leakage would become unacceptably large.
The air cushion is dimensioned for 5000 cubic metres at a pressure of 48 atm (or 240,000 cubic metres at 1 atm), which correspond to the operating conditions for the highest regulated water level at Langavatn. For uniform operations under lowest regulated water level at Rundavatn (El. 1013) , the air cushion increases in volume to 7000 cubic metres. Because of the need for additional safety, a total volume of 10,000 cubic metres was provided, inclusive of the access tunnel. The rock in the chamber was stable and impervious, and no stabilization was necessary.

The compressor and the instrument housing were placed right outside the disassembly chamber, about 600 m from the air cushion. The air duct (for filling and emptying), the measurement duct, the measuring cables and the water duct (for placement of a water curtain eventually) were placed in a small concrete culvert in a corner of the tunnel.
The compressor for the main filling is mobile and is assumed to be us ed to fill the air cushion of two neighbouring power plants. It has a capacity of 20 l/min (about 29,000 cubic metres/day) up to a maximum pressure of 30 atm (which corresponds to the water level in the supply tunnel at El. 960). It follows that the chamber has sufficient volume for air filling at this pressure. For the refilling operation three compressors are installed, each 1.5 m^3/min capacity (corresponding to 6000 m^3/day up to a maximum pressure of 50 atm.
The time us ed for filling up the unlined supply system in the case of a surge shaft, is determined by the rate at which one wishes to transfer the load to the rock: about 100 m water pressure/day corresponds to 5 days necessary for filling. The air cushion alternative equipped with the previously mentioned compressor capacity, requires 10 days theoretically. This time can be reduced to 5 days by doubling the capacity of the main compressor.
The total costs involved in the air cushion chamber concept amount to 8 million NOK, of which 40% goes to construction costs. The total cost per unit metre for the connection between the air cushion and the compressor/instrument housing reaches approximately 5000 NOK of which 20% goes to construction works. The surge haft, 800-m long with cross-sectional area of 20 m2, costs 16 million NOK, inclusive of stabilization work (there is how- ever, some uncertainty involved with the extent of the stabilization work). The savings with an air cushion chamber solution are thus about 8 million NOK.
Air leakage has proved very small; the power consumption to the compressor is thus modest. It has not been necessary to use the water curtain. The installation of a semi-cylindrical steel lining would have cost 10 million additional NOK. In the event that the execution of the higher galleries reveals unsatisfactory rock quality for air cushion chamber, one would at this point change to a surge shaft solution.

5 ACCESS TO SUPPLY SYSTEM WITH HIGH WATER PRESSURE (> 150 m)

5.1 Choice of layout

a) Dismountable part of supply pipe or b) Access at upstream end of steel lining The need and requirements of the access to the tunnel depend on the conditions and characteristic of the supply system. A distinctive access is advantageous construction-wise when the construction sequence of the supply tunnel must follow a critical path. A layout with distinctive access facilitates entrance to the system while in operation. The choice of access layout must consider the geological conditions in the proposed area. A distinctive access in- creases the extent of unlined rock surface/ spaces exposed to maximum water pressures. Among the 50 unlined supply systems in the Norwegian power works with water pressure from 150 to 750 m, the large majority is designed with a distinct access (steel- lined).
In the Sima power complex, Lang-Sima has a 340-cm diameter dismounting pipe-part as access to the unlined tunnel, whereas Sy-Sima has a distinctive access to the unlined tunnel through a port

(see Fig. 6). The positioning of the transition between the steel- lined and unlined sections in the supply system should be based on geological and economical considerations and should not be considerably influenced by the access itself. The cost for the port layout at Sy-Sima (exclusive of the work for the separate access tunnel), amounts to 1.5 million NOK, 2/3 of which concern the steel construction.

Figure 6 presents for Lang-Sima a hypothetical access solution similar to the solution used in Sy-Sima. Because of the higher hydrostatic pressure at Lang-Sima the port layout would have cost in this case 2.5 million NOK. In addition, the access tunnel itself would have been about 100 m longer than in the case of a disassembly chamber, corresponding to an increase in cost of about one million NOK. The elimination of the disassembly chamber should lead to savings of one million NOK, mainly because of the reduction of steel weight by about 50 to 60 tons as a result of the increase in load transfer to the rock without the presence of the chamber. The increase in costs from the building of a distinct access will therefore amount to about 2,5 million norwegian kroner.

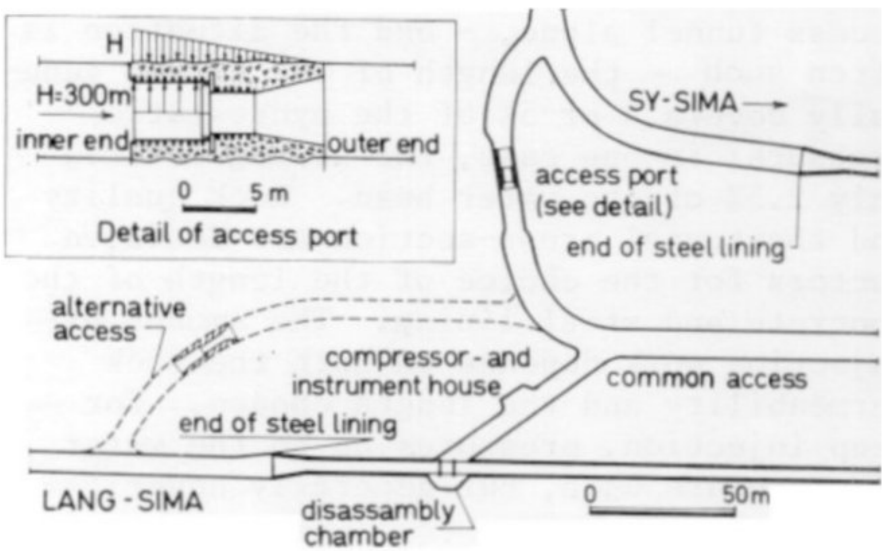


Fig. 6. Plan of Kjeåsen access system.

The sum must be viewed in light of various construction and operation advantages. Mounting, concreting, injection work, up-stream the disassembly chamber, ca. 50 m, took a total of about 12 weeks. The experience acquired at Sy-Sima indicates that a port layout at Lang-Sima would require about 8 weeks or a reduction in the construction time of one month. The Sy-Sima access will also facilitate transport during operation, for example in the case of emptying of sand traps, rock fall in tunnel, etc.

The Lang-Sima access design was preferred partly because the cost difference had been evaluated as larger, partly because of some uncertainties with respect to the rock conditions. The disassembly room could however, have been placed nearer the cone-shaped end of steel lining. (The costs would not change, but the construction time would have been reduced) .The distance from unlined tunnels exposed to water pressure is here significantly longer than used for access ports (see below), due to the disassembly chamber.

5.2 Realization of access layout placed upstream of the steel lining

As mentioned before, the Sy-Sima layout is the most us d access design in Norwegian water power projects. The localization of the access tunnel should consider the nature of t e rock, distance to other tunnels or caverns in the area, especially on the water side. When concrete lining is required based on the conditions along the access tunnel alone, -and the situation is often such -the length of the lining generally covers 4 or 5%; of the hydrostatic pressure; in one cast' , the lining covered only 2.5% of the water head. Rock quality and the tunnel cross-section are decisive factors for the choice of the length of the concrete and steel lining. The amount of injection work depends on both the rock permeability and the length chosen. For deep injection, pressures up to the water pressure are used, but generally never higher than 30 bars.

Leaks occur mostly at the concrete-rock boundary, but in locations with a relatively short steel-lining on the air side, the water comes through in the weaker concrete lining. Leakage has rarely been measured larger than 5 l/s. It is often necessary to inject several times until the leakage decreases to an acceptable level, dependent on the risk involved and the economy (for example value of leak water).

The selection of the port location along the concrete lining varies from the inner to outer end of the lining and it seems to depend on the owner or consultant. Most often, about half of the length of concrete lining is steel lined. A pipe is often preferred. The port opening is as large as 340 cm in diameter, or 270x 300 cm^2 in the case of rectangular openings.

Figure 6 shows the main dimensions of the access layout us ed at Sy-Sima. The port opening 260 x 260 cm2 was used. The port is constructed such that the load is transferred to the four sides. The hydrostatic pressure is approximately 300 m, with maximum fluctuation 20% higher. The port is placed about halfway in the concrete lining.

There exists some minor weakness zones in the rock, but with the concrete work and injection, the leakages were brought down to a modest value (< 1 l/s).

If the design had been carried out today, one would have moved the port with steel lining a little towards the outer end of the concrete lining and increased the opening at the inner end. One would then obtain a better seal between concrete and rock. This assures that the steel lining is watertight. One should pay special attention to the assembly weld right inside the port. Placement of the port at the inner end of the concrete lining would present a disadvantage with respect to leakage, as it could lead to a water trickle at this location, which results in a higher inward water pressure on the concrete construction.

ACKNOWLEDGEMENTS

The authors acknowledge Eng. Stein Sandvær from The Norwegian State Power System, for preparing the drawings, Dr. Suzanne Lacasse from Norwegian Geotechnical Institute (NGI) for translation and review of the paper and Mrs Kristin Eggestad from NGI for the layout and preparation of the manuscript.

REFERENCES

Broch, E. and R. Selmer-Olsen 1982, The development of unlined pressure shafts and tunnels in Norway. To be publ. in: International Society for Rock Mechanics. Symposium on Rock Mechanics related to Caverns and Pressure Shafts. Aachen 1982.

Johansen, P.M. and G. Vik 1982, Experience and measurements from air-cushion surge chambers ill Norway. To be publ. in: International Society for Rock Mechanics. Symposium on Rock Mechanics related to Caverns and Pressure Shafts. Aachen 1982.

Lien R. 1972, Oversikt over norske uforete tunneler og sjakter med vanntrykk over 100 meter samt enkelte andre med lavere trykk. (Review of Norwegian unlined tunnels and shafts with water pressure above 100 m). Norwegian Geotechnical Institute, Oslo. Internal report, 54402. 23p.

Myrset, Ø. 1980, Underground hydro-electric power stations in Norway. International Symposium (on) Subsurface Space. Rock- store '80. Stockholm 1980. Proceedings, Vol. 2, pp. 691-699.

Selmer-Olsen, R. 19174, Underground openings filled with high pressure water on air. International Association of Engineering Geology. Bulletin, No. 9, pp. 91-.9y.

Smith, P.T. 1974, The Eidfjord hydro development in western Norway. Water Power, Vol. 26, No. 7, pp. 239-245.

Terzaghi, K. 1962, Stability of steep slopes on hard unweathered rock. Géotechnique, Vol. 12, No. 4, p. 251-270. Also publ. as: Harvard soil mechanics series, 69. Norwegian Geotechnical Institute. Publication, No. 50.

Investigation for Norway's longest unlined pressure shaft.

By J. Bergh-Christensen and R. Kjølberg
Geophysical Division; Geological Engineering Division, A/S Geoteam.

The pressure shaft described here is unlined, and will sustain the highest water pressure in Norway, and probably the world. The geology of the site is described, along with the investigations undertaken to determine the design criteria.

A HYDROPOWER project planned by the aluminium producers, Årdal & Sunndal Verk in western Norway includes 1130 m long unlined pressure shaft with a static head of approximately l000 m. The topography of the site is complex and the main part of the scheme is at an elevation varying between 1000 and 1200 m, where the terrain is mountainous and rugged.

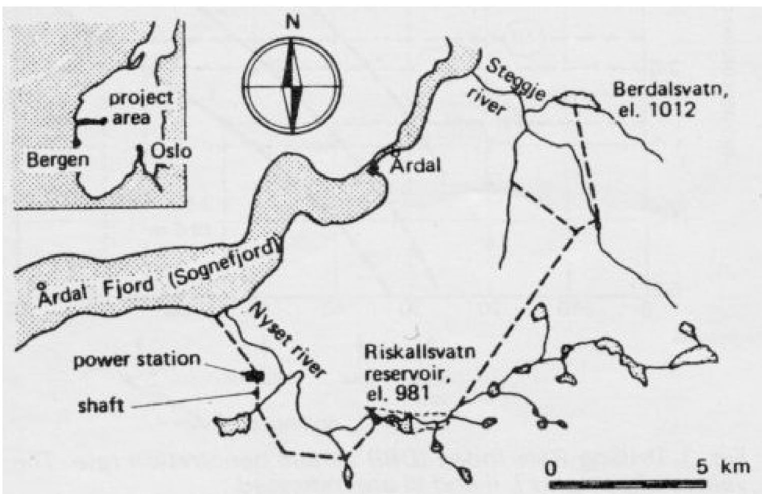


Fig. 1. The Nyset-Steggje Hydropower project.

Fig. 1 shows how the Nyset and Steggje rivers are regulated. Water from Berdalsvatnet lake in the northern part of the area will be taken through a 12 km-long tunnel to the reservoir. The Steggje river and a tributary are also directed into the tunnel on the way to the reservoir. In the reservoir area a 43 m-high earthfill dam with a bituminous membrane will be constructed.

The maximum water level will be at el. 980.3 m.

From the reservoir the water is fed through a headrace tunnel to the top of the pressure shaft at an altitude of 800m. The pressure shaft itself will have an inclination of 45°, with its lower end (at sea level) just inside the power plant. The shaft is planned to be unlined. From the power station a 2.5 km-long tailrace tunnel will lead to an outlet in the fjord.

The terrain in the shaft area consists of a high altitude plane at el. 1100-1200 m, with steep irregular rocky slopes down towards the fjord and the valley along the Nyset river, as shown on the map in Fig. 2.

Originally a steel penstock was planned from the headrace tunnel located at ground level and down the steep slope to the power plant. also located at ground level. The investigations along the pipe alignment showed. however. that there was a high risk of rock falls and slides and therefore the possibility of damage to the pipe was great. Because of this an alternative site was chosen with a shaft and power plant underground.

Geological setting

The sequence of rocks found in this part of Norway is:

Upper layers: The Jotun Nape; Mylonite and Cambro-Ordovician sediments.

Lower layers: Pre-Cambrian basement.

The tunnels in this area. as well as the pressure shaft and power station. will mainly be in the upper formation, the Jotun Nape. Only a very short section of the tunnel, between Berdalsvatn lake and the reservoir, will penetrate the thrust plane and pass into the Mylonite zone.

The Jotun Nape has a very complex structure with a varying mineralogical composition and degree of metamorphism. The rocks are mainly gneisses and granites, but diorites and gabbros are also encountered.

The tectonic structure of the area is dominated by the over thrust of the Jotun Nape. The thrust plane itself is within the Mylonite zone but differential shear zones can be found above the thrust plane.

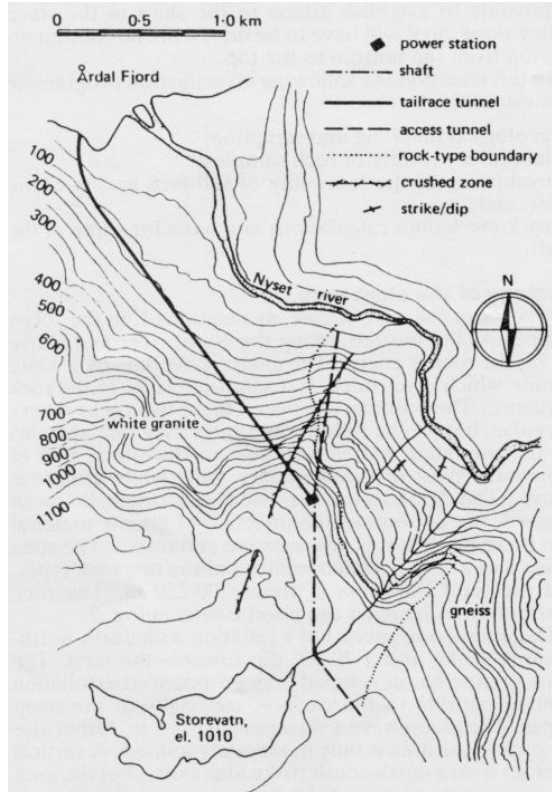


Fig. 2. map of the powerplant.

Investigation programme

As mentioned. the pressure shaft is planned to be unlined. With a static head of 980 m, this will be an unlined shaft with the highest water pressure in Norway (and probably in the world).

Because of the difficult topographical conditions, it is impossible to establish access to the shaft in the steep valley slope, so it will have to be driven in one continuous section from the bottom to the top.

In this situation the following investigation programme was established:

- geological mapping and sampling
- laboratory testing of rock samples
- evaluation of the feasibility of full-face boring of the shaft
- rock mechanics calculations as a basis for siting of the shaft

Geology of the shaft area

The rocks in the area belong, as mentioned, to the Jotun Nape. At the bottom, along the Nyset river and above el. 75, the rock is gneiss. The gneiss is overlain by a white granite which is intruded as a sandwich layer in the rock sequence. The boundary between these two rocks is very irregular, but can be followed up the valley side and also on the mountain plateau on the north-western side of Storevatn. West of the boundary, only white granite is found. The boundary zone itself has originally been crushed, but because of the injection of granite material into these masses, it is now massive and sound. The zone consists of a very irregular mixture of the two rock types. and the total thickness is about 200-250 m. The rock distribution as mapped can also be seen in Fig. 2.

The underlying gneiss has a foliation with north-north-westerly strike and a 70-90° dip towards the west. The white granite has developed very pronounced exfoliation jointing parallel to the surface, especially in the steep slopes. The plates have a thickness of 0.1-1 m. Otherwise the rock in the area is only moderately jointed. A vertical joint set with a north-south strike also exists, but the joint spacing is rather large. A fracture zone cuts the area, as shown in Fig. 2. It is very important that the shaft does not intersect this zone which has an easterly dip of 80°.

Drillability predictions

Typical samples of the rocks have been collected and laboratory-tested for drillability evaluations. The methods used have been presented earlier.

The indexes used are the DRI (Drillability Rate Index) and the BWI (Bit Wear Index). Besides these, the degree of jointing of the rock mass is also taken into account.
Table 1 shows the measured values of the DRI and BWI indexes and in Figs. 3 and 4 the values of DRI and BWI are plotted.

Table 1 – Measured values of the DRI and BWI indexes

Drilling Rate Index DRI Bit Wear Index BWI	Sample 1 Granitic gneiss	Sample 2 Gneiss	Sample 3 White granite
DRI	64 high	54 medium	60 high
BWI	20 low	21 low	25 low

The diagram in Fig. 3 shows the expected net penetration rate for a 3.5 m TBM (Tunnel Boring Machine) as a function of the DRI and the average joint spacing in the rock mass. Fig. 4 shows the expected cutter costs for a 3.5 m TBM (with single disc cutters) as a function of the BWI-index and of the joint spacing. As mentioned, the rocks are very moderately jointed, hence the curve designating distance between joints and partings longer than 20 cm is used.
Fig. 4. Bit Wear Index (BWI) versus cutter costs (sample I, II and III).
The diagrams show that in our case the net penetration rate is expected to be of the order of 1.7-2.1 m/h and the cutter costs are expected to be 50-80 Norwegian NOK ($8-13)/m3 of solid rock. This may be characterized as medium to good drillability and medium to low wear. The investigations thus indicate that the use of TBM will be feasible.

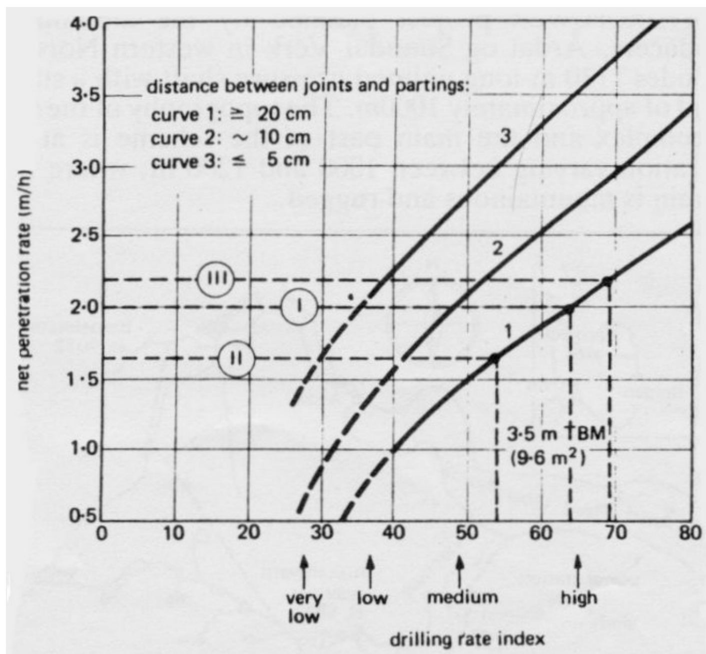


Fig. 3. Drilling Rate Index (DRI) versus penetration rate. The values of sample 1,2 and 3 are indicated.

Rock mechanics calculations
The method used for evaluation of the shaft placing is based on a calculation of total stresses by finite element plain strain analysis. This method was developed at the Geological Institute, Technical University of Norway, and has been described by Broch and Selmer-Olsen. The main condition imposed is that the minimum total stress should be higher than the water pressure at every point along the shaft, to avoid the risk of failure by hydraulic fracturing.
The computer program used is developed by SINTEF (The Foundation of Scientific and Industrial Research at the Norwegian Institute of Technology) for solving plain, linear elastic problems.

In this case the terrain is very irregular, and to make a plain element analysis possible it was necessary to establish a simplified topographic modelling which all the protruding noses were cut away. The simplified topographic model is shown in Fig. 5. Based on a simple calculation, a preliminary location for the shaft was chosen. This location is also shown on the terrain model.
The computer program used gives, for this type of finite-element analysis, the stress distribution along a vertical plane normal to the contour lines of the terrain model. Pressure shafts are usually located in the direction normal to the contour lines, and the program then gives a complete picture of the stresses along the shaft. But in this particular case, the shaft has to be located obliquely in relation to the valley side, and consequently the standard program does not give the complete stress distribution along the shaft in a single run.
A three-dimensional model was found to be unrealistic and it was therefore decided to perform the calculations with the use of three two-dimensional models representing profiles cutting the lower end of the shaft and at 225 m and 425 m above the bottom. Earlier calculations show these points to be critical.

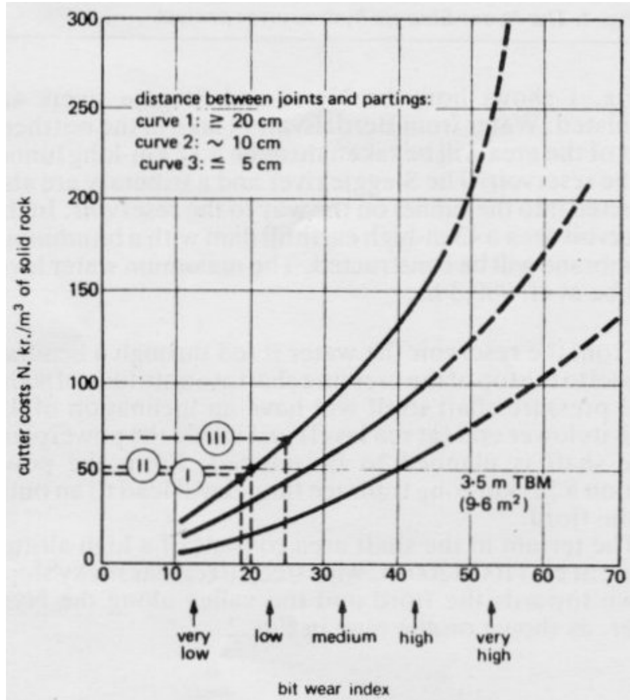
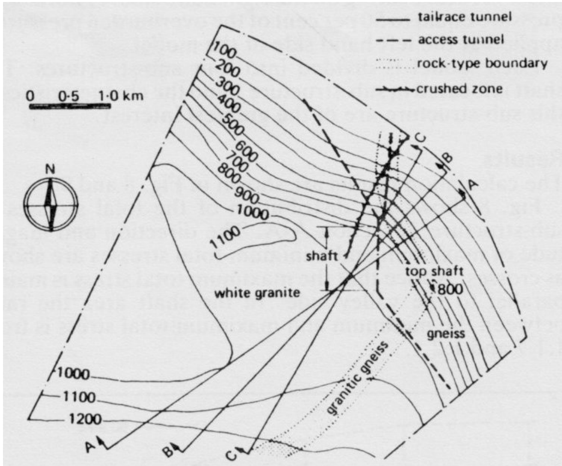


Fig. 4. Bit Wear Index (BWI) versus cutter costs. The values of sample 1, 2 and 3 are indicated.

Table 2 – Characteristics of the various rock types.

Sample	Bulk density	Poissons ratio	Young's mod. E.
(Granitic gneiss)	2,78		
Foliation		0,05	$0{,}026 \cdot 10^6$MPa
Foliation		0,14	$0{,}035 \cdot 10^6$MPa
Gneiss	2,93		
Foliation		0,11	$0{,}025 \cdot 10^6$MPa
Foliation		0,14	$0{,}025 \cdot 10^6$MPa
White granite (homogenous, without foliation)	2,62	0,07	$0{,}027 \cdot 10^6$MPa

When evaluating hydraulic fracturing, the total stresses are of interest for comparison with the water pressure, and therefore transformation of the stresses to the shaft direction is unnecessary. The rock material is supposed to be linearly elastic and orthotropic (with different values in the x- and Y-direction). The bulk density, Poisson's ratio and E-modulus for the different rock types are given in Table II.
The locations of the three sections A-A, 8-8 and C-C, designating the three two-dimensional models are shown in Fig. 5. The geometry of one of the models is shown in Fig. 6. All analyses have been done with models of the same dimensions, that is, the same number of elements and a length of 5250 m. The models (shown in Fig. 7) are designed with a horizontal roller bearing at the



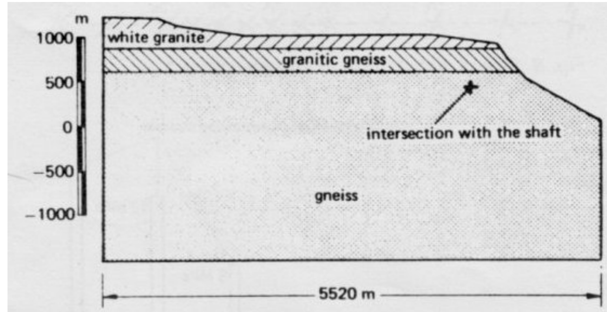
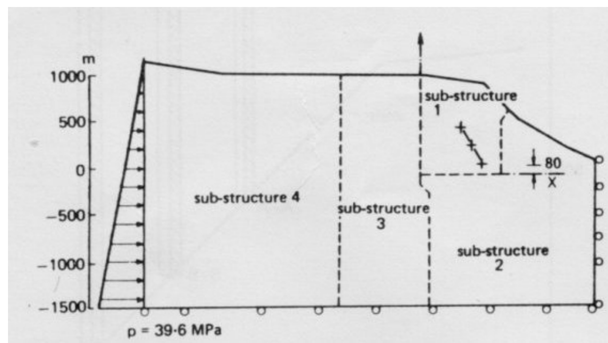


Fig. 6. Geometry of the finite-element model.

Fig.5 Simplified topograpical model.

Fig. 7. The model divided into sub-structures



bottom and a vertical hearing on the right hand side. A horizontal pressure equal to 50 per cent of the overburden pressure is applied at the left hand side of the model.
Each model is divided into four sub-structures. The shaft is located in sub-structure 1 and the characteristics of this sub-structure are of the greatest interest.

Results

The calculation results are shown in Fig. 8 and 9. Fig. 8 shows the distribution of the total stresses in sub-structure 1. section A-A. The direction and magnitude of maximum and minimum total stresses are shown as crosses. Notice that the maximum total stress is mainly parallel to the valley side. In the shaft area the ratio between the minimum and maximum total stress is from 1:1.7 and 1:2.1.
Fig. 9 shows the calculated minimum total stress at the intersection points between the shaft and three sections A-A, 8-8 and C-C.

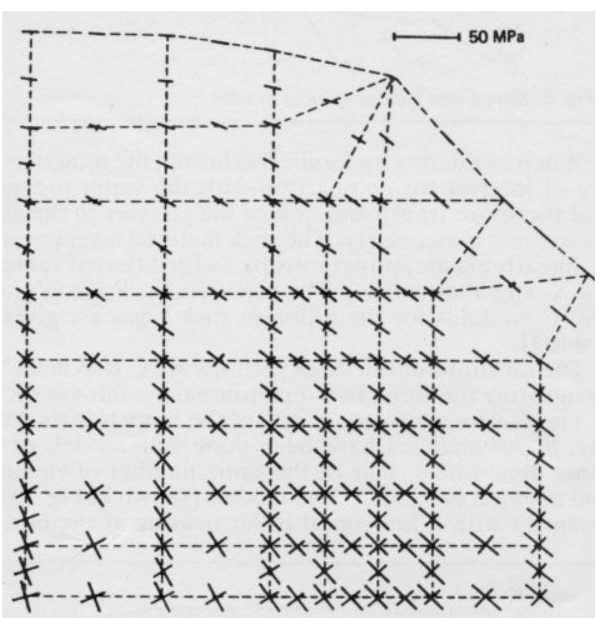


Fig. 8. Stress distribution in sub-structures 1.

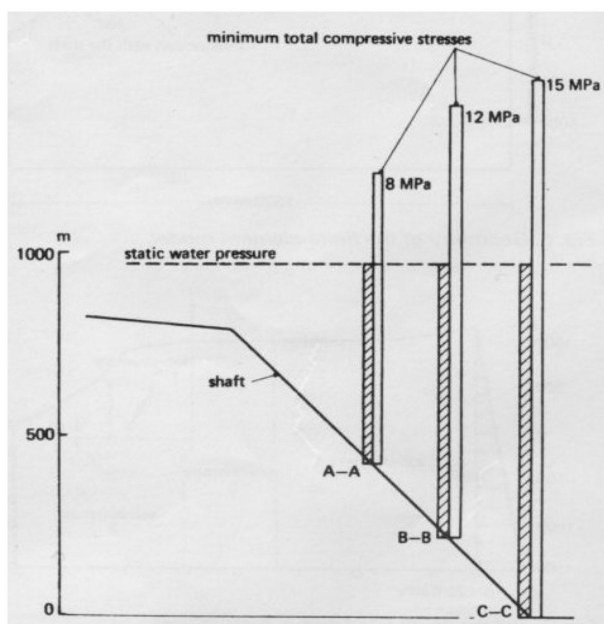


Fig. 9 Minimum total stress versus water pressure

The corresponding water pressure is also shown. The following values for the ratio between minimum total stress and water pressure are found:
Section A-A: 2.1
Section 8-8: 1.7
Section C-C: 1.5
According both to earlier experience and the above calculations, it was concluded that the preliminary location of the shaft should be sufficiently safe, and this location was chosen for the final design of the power station.

Further investigations
It is important for the final decision on shaft design that the calculated stresses are fairly correct, in other words, similar to the stress conditions found on site. W hen the tunnelling works begin, the access tunnel to the power- plant and a pilot adit towards the bottom of the shaft will have the highest priority. As soon as access to the bottom of the shaft is possible, stress measurements will be carried out on site. Hydraulic fracturing tests are also planned to be done on site. If the rock stress measurements indicate more adverse stress conditions than calculated, and hence lining of the shaft is considered necessary, the time schedule will allow for this.
If full steel lining should be necessary, a very ex pensive ropeway has to be built up to the top of the slope. However, another possibility would be to line only the lower part of the shaft. By driving a tunnel with a rise of I: 10 from the access area up to an elevation of about 250 m. in the shaft, the lower part of the shaft only can be lined.
These options are less desirable economically than the unlined shaft, so good agreement between measurements on site and initial calculation~ are hoped for. D
References
I. BI.INDHEIM. O. T. .DAHL JOHANSEN. E. AND JOHANNESSEN. O. “Criteria for the selection of full face tunnel Boring or conventional tunnelling”. Proceedings. 4th Congress, International Society for Rock Mechanics, Montreux. Switzerland; 1979.
2. BROCH. E. '.The development of unlined pressure shafts and tunnels in Norway". International Symposium on Rock Mechanics related to Caverns and Pressure Shafts,.. Aachen. Germany; 191!2.
3. SELMER-OLSEN. R., "Underground openings -filled with high- pressure water or air. " Bulletin of the International Association of Engineering Geology; No. 9, 1974.

DESIGN AND SUPERVISION OF UNLINED HYDRO POWER SHAFTS AND TUNNELS WITH HEAD UP TO 590 METERS

B. BUEN & A. PALMSTRØM
Ing. A.B. Berdal A/S, Norway

SUMMARY: Experience has proved careful design, supervision and control during filling of pressure shafts and tunnels necessary to prevent costly and unforeseen repairs and production losses.
To prevent hydraulic splitting in unlined pressure tunnels and shafts, the internal water pressure must be less than the minimum principal stress. Today, the theoretical design is usually done by using finite element methods. Vital for a successful design is the use of engineering judgement. In every case the theoretical models must be modified according to the geological and topographical features.
During construction of the plant, registration of water inflows and mapping of relevant geologic features is done in the tunnels and shafts. This makes identification of permeable zones possible. In critical areas more pressure measurements are used to determine the necessity of sealing.
In the process of filling the shafts and tunnels, leakage control is done. The tunnel system is filled in steps of 100-150 m in order to carry out the leakage measurements.
Summary of design, supervision and leakage measurements is given for 6 hydropower plants with water pressure on unlined rock in the range of 25-59 bar. The main rock types, represented are granite, gneiss, michaschist and phyllite. The coefficient of rock mass permeability calculated from the measurements varies from 10^{-8} to 10^{-9} m/s

l. INTRODUCTION
Characteristic for large parts of the Norwegian topography are the steep valleys close to high mountain plateaus containing lakes and rivers. With such features a great amount of hydro-electrical power plants are constructed as high pressure plants. These plants are characterized by the power station being situated in the valley close to the slope and with a pressure shaft and a headrace tunnel to the reservoir.
The conventional high pressure power plants were constructed with steel lined pressure shafts or steel pipes on the surface. From the middle of the 50's, however, a new design with unlined pressure shafts or unlined pressure tunnels was introduced as shown on Fig. 1 and 2, which gave great reductions in the construction costs and time. The solution involves that high water pressure is introduced directly on the rock mass. This new stress situation around the tunnel results in possibilities for deformations and great leakages to the surface.

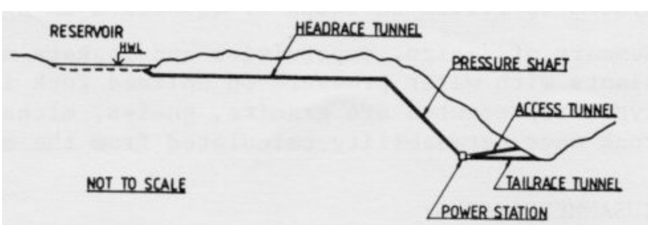


Fig. l. Layout for a power plant with headrace tunnel and pressure shaft in principle.

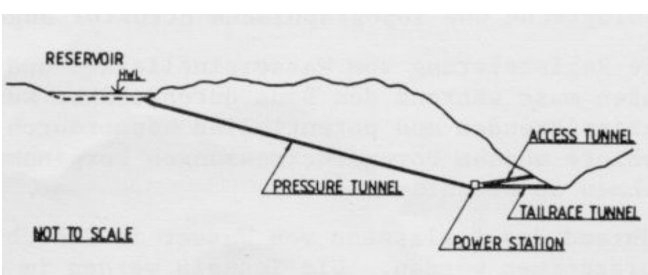


Fig. 2. Layout for a power plant with pressure tunnel in principle.

Unforeseen deformations and/or great uncontrollable water leakages have occurred at Norwegian power plants as late as 1968, 1970 and 1971 during filling up of the water system. Most of the leakages occurred 1-3 days after the tunnels were filled, with insufficient rock cover and/or unfavourable fractures being the main reasons, Ref. (2).
A safe construction of unlined high pressure shafts and tunnels is therefore highly dependant upon a design based on sufficient rock cover with regard to the rock mass quality, together with a planned supervision during construction and the first filling up of the pressure system.

2. DESIGN

At first when the unlined high pressure shafts were introduced an evaluation based on a simple equilibrium state of stress was used. The principle for this was that the weight of the rock mass overburden should exceed the water pressure in the shaft at any point. In Fig. 3 experience gained from 45 unlined Norwegian power plants is compared with the calculation criterion mentioned.

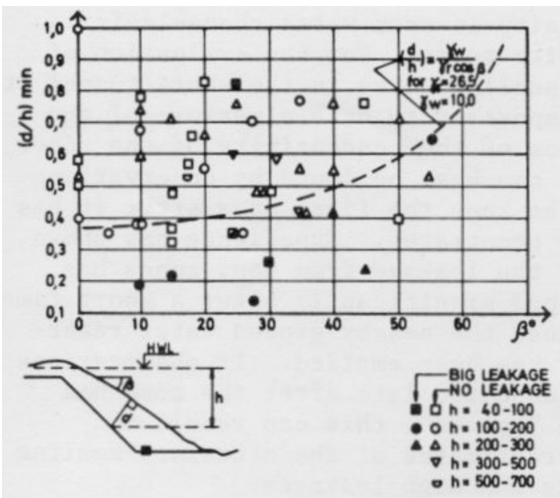


Fig. 3. Unlined tunnels and shafts compared to the "overburden" criteria of design, Ref. (2).

A better basis for design of the unlined high pressure shafts/tunnels became available in 1972 w hen a method based on finite element analysis of two-dimensional models was introduced, Ref. (4). The main criterion for this model is that the inner water pressure in the shaft/tunnel shall no~ exceed the minimum principal rock stress. An example of one of these models is shown on Fig. 4 where the design of the pressure shaft of Leirdøla power plant is shown. The critical line H_0/H is for Leirdøla 0,7. The unlined pressure !shaft is placed inside this critical line with a safety factor of F = 1.4.
An understanding of the influence of both the geological and topographical features combined with the calculation criteria is a condition for the engineers evaluation of the safe design of the unlined pressure shaft/tunnel.

3. INFLUENCE FROM A PRESSURE SHAFT UPON GROUNDWATER FLOW AND LEAKAGE

Before excavation of the power plant the ground water in a valley side will flow almost parallel to surface on its way down to the bottom of the valley, Fig. SA. In moderately jointed rock masses our measurements indicate-g mean coefficient of permeability $K=10^{-8}$ to 10^{-9} m/s which equals 0,003-0,3 m/year. In jointed permeable zones the permeability can be considerably larger.

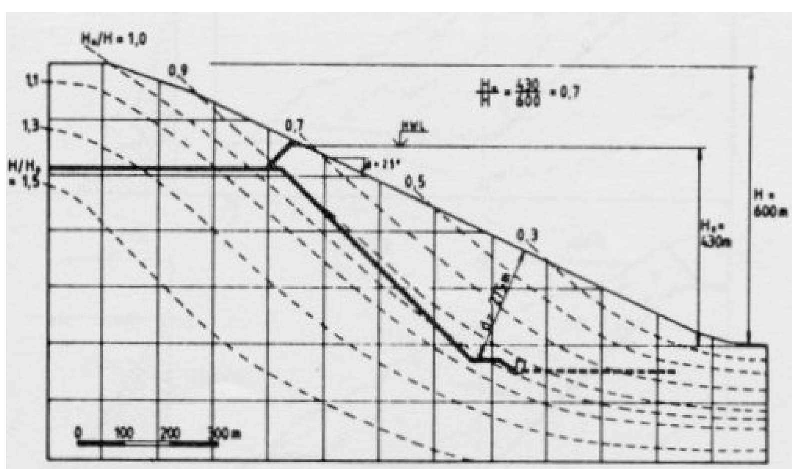


Fig. 4. Model based on finite element methods applied on Leirdøla power plant, Ref. (4).

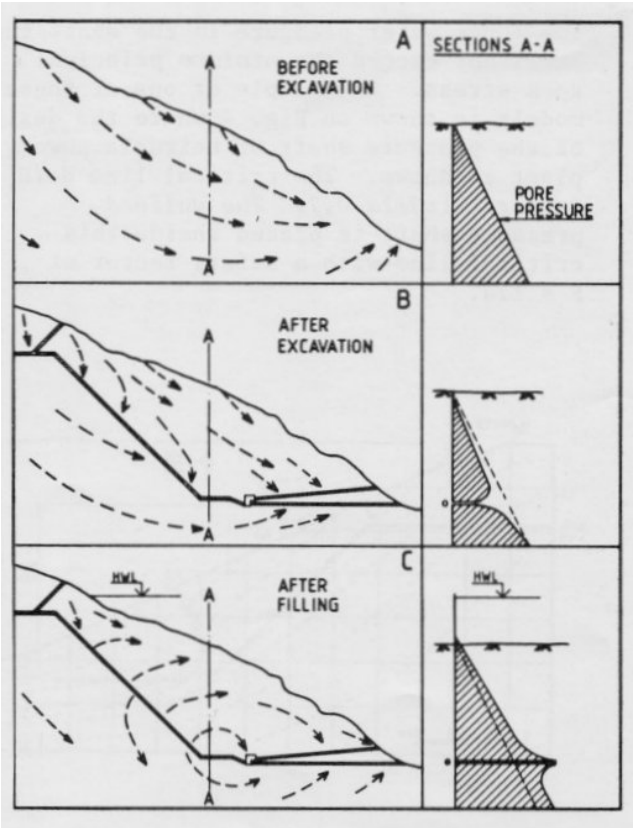


Fig. 5A. B and C. Idealized flow in a homogeneous and isotropic. rock massif before and during excavation and after filling.

During and after excavation of the shaft/tunnel pores and joints around it are being drained, and water flows into the shaft/tunnel. A simplified flow sketch for a homogeneous and isotropic condition is shown on Fig. 5B, where it can be seen that most of the leakage water comes from the upper part of the valley side. A considerable reduction
of the pore pressures will occur in the rock masses surrounding the shaft/- tunnel. In more permeable rock masses this can result in a lowered water table as indicated on Fig. SB. This will, however, most of ten take much longer time than the construction of the power plant.
After the shaft/tunnel has been filled up, Fig. 5C, the water will flow out from it and towards the valley side. Close to the shaft/tunnel the flow will be almost perpendicular to its surface, but further out the flow will be more influenced by the topography. The situation shown in Fig. 5C will cause a small leakage into the shaft at its upper part. When moving down the shaft the leakage out from the shaft will increase with the head. As shown, the water pressure drops significantly within a short distance from the shaft. This has been verified at Skjomen power plant by pore pressure measurements,
Ref. (5). The elevation of the ground water level is highly dependant upon the rock mass permeability. As described in the following, any permeable zones will strongly influence on the idealized situation and will result in a more complicated flow net.
Important features of a permeable zone are:

1. The composition and structure of it, mainly its conductivity.
2. The orientation of the zone with regard to the topography. i.e. the valley side.
3. The head in the shaft/tunnel where the zone is found."

The conductivity of a zone can vary from almost zero in clay seams to almost infinity in open water channels in calcite zones. For the evaluation of
the sealing works in the shaft/tunnel it is important to have a measure of the degree of the conductivity of the zone. This can best be found by observations
of the zone the first days after it has been penetrated. Experience has shown that the leakage from many zones has dropped significantly after a short time because the nearby ground water reservoir has been emptied. It observations are done too late after the zone has
been "opened", this can result in underestimates of the necessary sealing works and high leakages. The orientation and position of the zone in the shaft/tunnel can be even more important than its structure. To illustrate this four different cases of permeable zones are shown in Fig. 6
where the gradient and hence the leakage can vary a lot.

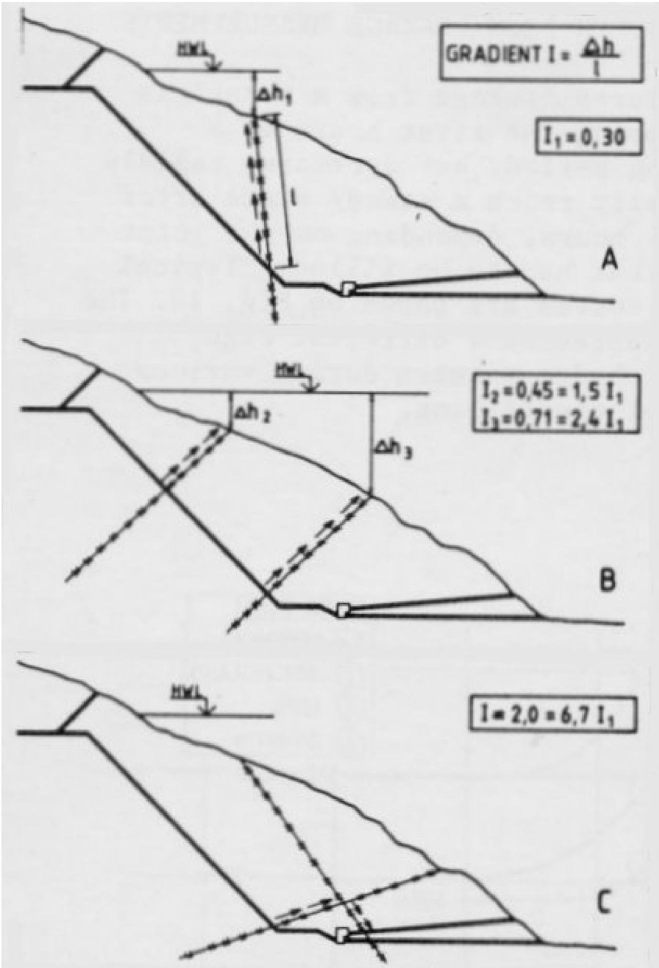


Fig. 6A, B and C. The influence of orientation and position on the leakage potential of permeable zones.

A most unfavourable situation arises when two zones happen to intersect as shown in Fig. 6C, giving short flow distance down to access- and tailrace tunnel. The gradient can then be 6-7 times larger than a single zone cutting the shaft in example 6A. These examples stress the importance of well planned investigations and evaluations of unlined pressure shafts and tunnels before filling and the need for a careful control during the first filling of the system.

4. SEALING WORKS IN UNLINED PRESSURE TUNNELS AND SHAFTS

Potential leakage zones in tunnels and shafts roust always be sealed. Sealing is usually done by means of grouting, using a pressure slightly higher than the static head in the tunnel. Shotcrete lining or cast in place concrete lining is as a rule used only for stability purposes. A special sealing method by using rock bolts, shotcrete and reinforcement has been used together with grouting of open, permeable zones.

Figure 7 shows theoretical flow lines and equipotentials for flow from a tunnel situated below a horizontal ground surface. It is evident that a rapid pressure drop takes place over a short distance out from the tunnel wall This points to the necessity of concentrating the sealing effort to where it is most effective: close to the tunnel surface.

A simplified calculation of water flow through an idealized channel, Fig. 8 illustrates the effect of sealing, here assumed achieved by grouting.

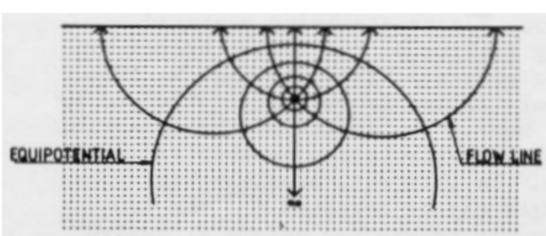


Fig. 7 Theoretical flow from an opening below a horizontal surface

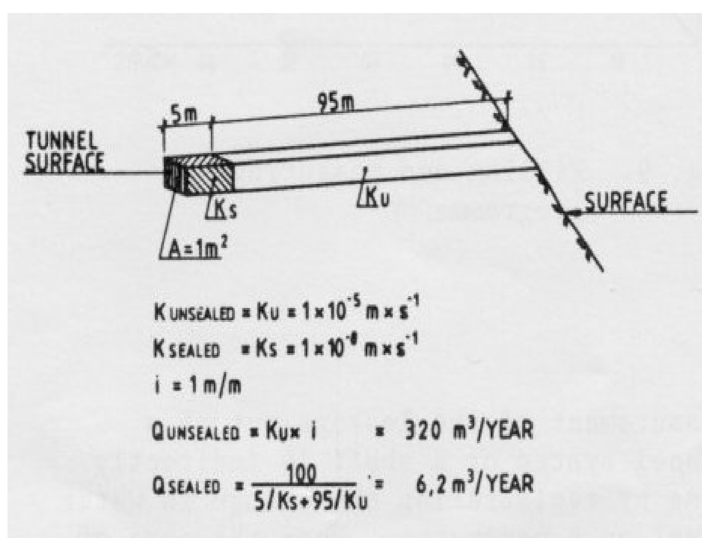


Fig. 8. Simplified channel flow from tunnel to surface

5. MEASUREMENTS DURING CONTROLLED FILLING OF THE PRESSURE SYSTEM

A large and rapid change in the pore pressure and water flow is introduced within a short time during filling of an unlined tunnel/shaft. The big pressure gradients being set up may result in possibilities for transport of the material in faults and fractures. If local stress anomalies occur deformation also may take place. A well planned control and a low filling-rate will reduce such abrupt change in the stress situation around the shaft/- tunnel. With the term controlled filling is meant that the water leakages out into the rock masses are being measured during filling of the tunnel system by making pauses as shown on Fig. 9. The rate of filling is normally limited to 10 m per hour. The length of the measuring periods usually varies between 12 and 24 hours. In this way. possible big leakages may be detected and the tunnel system can be emptied before extensive damages are created.

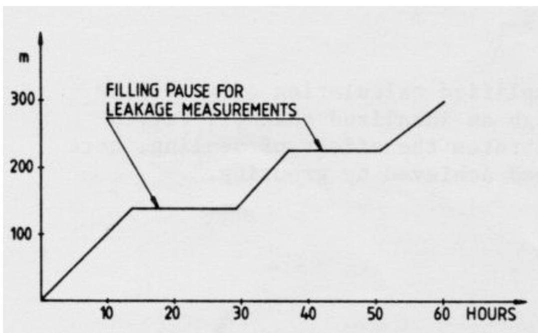


Fig. 9 Filling and measuring programme.

Measurement of the leakage out of a tunnel system or a shaft is indirectly done by registration of the change in water level on a manometer. When the area of the water surface and the leakage through bulkheads and gates are known, the net water leakage going into the rock can be calculated. .

6. RESULTS FROM LEAKAGE MEASUREMENTS

The measured leakage from a tunnel is large during the first hours of a measuring period, but decreases rapidly and usually reach a steady state after 12 to 24 hours, depending on the joint volume that has to be filled. Typical leakage curves are shown on Fig. 10. The curves represent 4 different high pressure hydro schemes during various stages of the filling.

The leakages are all small as evident from Fig. 10 and also table l. This is due mainly to the low mass permeability of the rock itself, but also to the careful sealing of pervious zones.

Predictions on mass permeability of the rock and leakage from the tunnel systems were done for the last two hydro power plants listed in table l. The predictions were based on measurement of water leaking into the tunnel system before filling. and on assumed or measured pore pressures. As evident in table 1 the predictions were fairly accurate.

The mass permeabilities given in the table 1 are based on measured water leakage from the tunnel and on measured

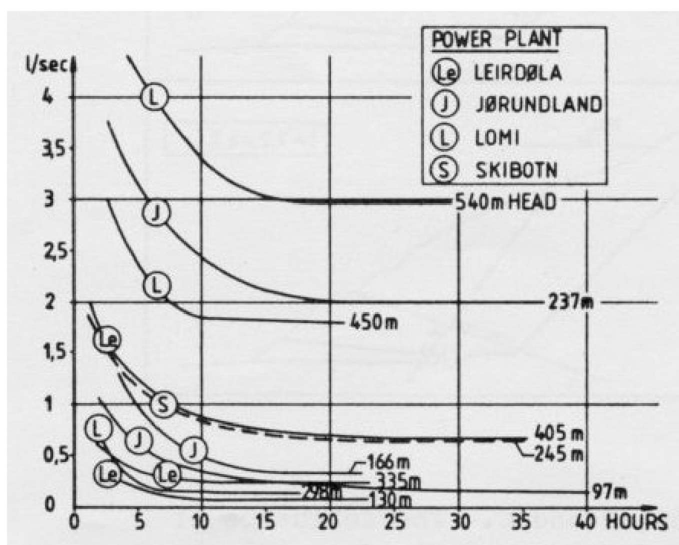


Figure 10. Flow rate curves for 4 different tunnels and shafts at different heads.

or assumed pore- pressures in the rock. The calculations are based on equation l, Ref. (l) which give the volumetric leakage rate from an underground opening of length L.

$$Q = (2\pi \cdot km \cdot L \cdot g \cdot Pr) / (\mu \cdot Ln \cdot (2D/r))$$

Q =	volumetric flow rate
k_m =	equivalent mass permeability m
g =	acceleration due to gravity
p_r =	excess pressure in opening of radius r
μ =	viscosity of liquid
D =	depth of centre of opening below surface

Table 1. Summary of layout, geology and leakage control results from six hydropower plants.

Power plant	Max head on unlined rock (bar).	Layout	Geology	Predicted leakage l/s	Measured leakage l/s	Calculated mass permeability m/s
Jørundland (1971)	28	2,0 km pressure tunnel	precambrian granite & gneiss	--------	1	$1 \cdot 10^{-9}$
Skjomen (1973)	36	2,6 km pressure tunnel	precambrian granite	--------	1 – 2	$3 \cdot 10^{-9}$
Borgund (1974)	25	2,9 km pressure tunnel	precambrian gneiss	--------	3 – 4	$1 \cdot 10^{-8}$
Leirdøla (1978)	45	0,6 km pressure shaft	precambrian gneiss	--------	0,9	$1 \cdot 10^{-8}$
Lomi (1979)	59	0,7 km pressure shaft	ordovician mica schist and phyllite	1 – 5	3 – 6	$5 \cdot 10^{-8}$
Skibotn (1980)	44	4,0 km pressure shaft	ordovician mica schist	2 – 10	10 – 18	$3 \cdot 10^{-8}$

7. Conclusion

The solution with unlined pressure shafts and tunnels is both cost- and time saving. Unexpected adverse geological conditions have, however, in the past been the cause of extensive repairs and loss in production in several hydro power schemes. This proves the necessity of thorough planning and control during all phases of construction. The main point in this process are listed below:

- Collecting of existing geological information and supplementary field mapping.
- Core drilling, permeability testing and observations of groundwater level.

- Assessment of stability for the pressure shaft or -tunnel with respect to geology and topography.
- Mapping of geology, potential leakage zones and registration of water leaking into the shaft or tunnel during construction.
- Testing and control of permeability
- and pore pressure in the most critical leakage zones.
- Assessment of necessary sealing works and estimate of water losses from the tunnel system in operation.
- Controlled stepwise filling of the tunnel system and calculation of the real water losses.

To sum up we would like to point out at the profitability of investments in sealing works is good. With today's energy prices it is astounding what a leakage as small as 1 lugeon/second can amount to in lost production over the years in a high pressure hydro power plant.

REFERENCES

BARTON, N. 1972: Estimation of leakage rate and transport time for fluid flow from underground openings in jointed rock. Report 54203, Norwegian Geo- technical Institute, Oslo.

NGI, 1972: Oversikt over norske
uforede tunneler og sjakter med vanntrykk over 100 m samt enkelte
andre med lavere trykk. (In Norwegian). Report 54402, Norwegian Geotechnical Institute, Oslo.

NGI, 1974: Uforede tunneler. Spesielle problem ved store tverrsnitt (In Norwegian). Report 54202-2, Norwegian Geotechnical Institute.

BJØRLYKKE, S, SELMER-OLSEN, R, 1972: Nødvendig overdekning i dalsider ved fjellrom med høyt innvendig vann- eller lufttrykk. (In Norwegian). Report no. 6, Department of Geology, Technical University of Norway.

BUEN, B. 1973: Ingeniørgeologiske forundersøkelser ved Skjomen Kraftverk. {In Norwegian). Fjellsprengningsteknikk- Bergmekanikk, Tapir, Trondheim.

SELMER-OLSEN, R. 1969: Experience with unlined pressure shafts in
Norway. International symposium on large underground openings, Oslo 1969.

FLOW RATES OF AIR AND WATER FROM CAVERNS IN SOIL AND ROCK

O. TOKHEIM ,
City of Oslo Geotechnical Division, Norway
N. JANBU
Norwegian Institute of Technology, Trondheim, Norway

SUMMARY
Straightforward approximate formulas are presented for the estimation of flow rates of air (gas) and water (incompressible liquid) from a cavern in a permeable mass; the cavern acting as a source or a sink. The mass may be soil or fractured rock and the cavern may be a drill hole, a tunnel or compressed air surge chamber among others.
The idealised geometry of the cavern and the distance to 4 boundary equipotential, which may be the groundwater table, determines the value of a geometry factor appearing in the flow rate formulas. One formula applies to water flow in !completely water saturated media and a slightly different formula to air flow in dry media. Special emphasize is given to the application of the formulas in questions regarding design of unlined compressed in surge chambers in jointed rock.

1. INTRODUCTION

The planning of closed air surge chambers in Norway in the early seventies called for methods to predict air loss from caverns located in fractured rock. The work undertaken by the present authors (1973) was directed towards the development of appropriate formulas for estimating rock mass permeability from borehole tests(Lugeon tests) and measurements of water flow into tunnels and caverns. Furthermore to develop formulas for estimating water and air flow from a cavern knowing the effective permeability of the rock mass.

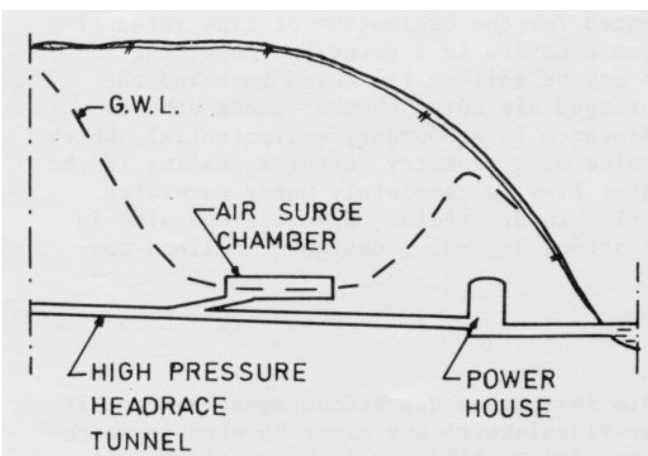


Fig. 1. Closed air surge chamber in a fractured rock mass (not in scale).

Fig. 1 shows the typical problem; an air surge chamber partially filled with compressed air. Under air pressure higher than, or even slightly below the original water pressure at the chamber location, compressed air will migrate into the rock joints intersecting the chamber. The air loss must be compensated for by compressors. One crucial question is the compressor capacity needed.
The analysis of flow problems in fractured rock is difficult for several reasons. The flow taking place in hard rock suitable for the location of air surge chambers is exclusively limited to fissures and joints. The spacing, distribution and openings of such are likely to be most irregular. Considering even a single joint parallel plate theory may not be correct as the flow may take place in channels. In the theory to be presented the medium in which flow takes place is considered homogenous. Thus the theory itself is not developed particularly for flow problems in fissured rock. The flow equations to be presented applies to soils and other permeable media as well. In the present paper emphasize is, however, given to the application of the flow equations to in-homogenous rock.

The combined flow of air and water (two fluid system) is hard to analyse. Much easier is the analysis of water (liquid) flow in a completely water saturated porous medium and the analysis of air (gas) flow in a dry porous medium. When able to handle the latter problem at least an upper limit of the air flow in a mixed air-water flow problem is given. The theory to be present ed is limited to single fluid flow, but some emphasis is given to the application of the results to the mixed flow problem of an air surge chamber.
During the filling of air into a surge chamber a transient period will take place under which air displaces water in some of the rock joints. The rate of air loss during the transient period is smaller than during the steady state reached after some time. Steady state conditions only will be considered in this paper.
Various analytical solutions to flow problems are available in the literature. Muskat (1937) presented solutions to fluid flow problems involving spherical and cylindrical flow, including also two-dimensional flow from a line source to a plane equipotential (isobar).
Zilngar et. al.(1953),among other problems, analysed water flow from a short section of a test-hole below the groundwater table.
The analytical solutions available are s a general not readily applicable to the geometry of an air surge chamber without simplifications and approximations. One goal of this paper has been to rationalize the analytical solutions so as to present straight forward formulas applicable to a variety of flow problems as illustrated by the geometry in Fig. 2.

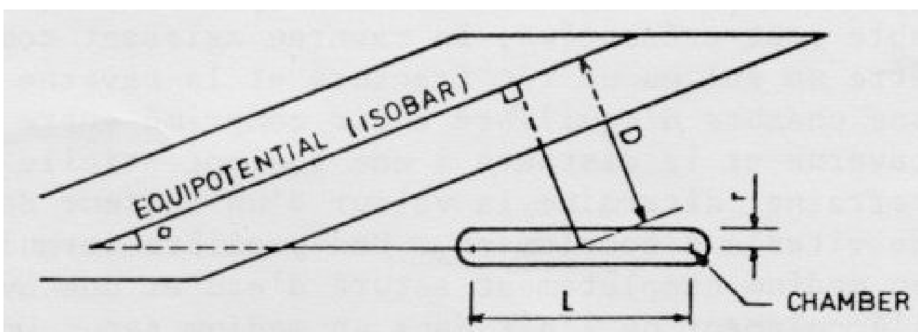


Fig. 2. Geometry in an idealized flow problem.

The boundary of a cavern acting as a source or a sink shall be represented by a cylindrical surface with ball shaped ends. A plane equipotential or isobar is the other boundary; the plane normally being associated with the groundwater table. The centre of the cavern is located at a distance D from the outer boundary while the axis of the cavern intersects the boundary at an arbitrary angle α.

2. FLOW EQUATIONS

2.1 Fluid flow theory.

In the flow analysis below the more simple problem of fluid from a source in an infinite homogenous porous medium is considered first. The source is cylindrical with ball shaped ends as shown in Fig. 3. The flow is governed by the generalized Darcy's law which, when ignoring gravity, may be written:

$$v = -K/\mu \ \ dp/dR \qquad (1)$$

where v is the average velocity and -dp/dR is the radial pressure gradient.

Furthermore K[1]) is the intrinsic permeability and μ is the dynamic viscosity.
The key to the simplified solution is to assume isobars and equipotentials having the same shape as the source. Comparison of the resulting flow rate equations with more accurate solutions developed for special cases indicate this being a fair approximation. If the length L of the cylinder is infinite (two-dimensional flow) or zero (strictly radial flow) no approximation is involved at all.

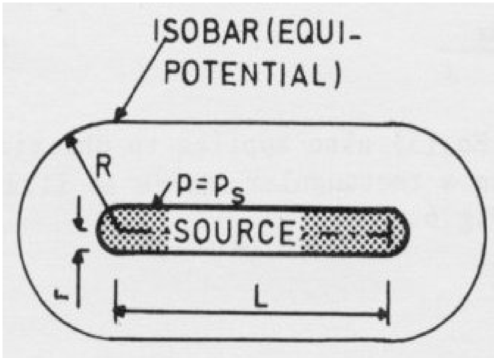


Fig. 3. Section through source and equipotential {isobar) in an infinite porous medium

1)
The intrinsic permeability K is related to the hydraulic conductivity (permeability coefficient) through $K=\mu k/\rho g$ in which ρg is the unit weight of the fluid.

The assumption above implies constant mean flow velocity v at distance R from
the centre of the source. The volume flux Q follows as:

$$Q = \rho/\rho_0 \ vA \qquad (2)$$

where p and ρ_0 are the fluid density under the actual pressure p and the reference pressure Pa respectively. "A" is the area of the equipotential surface; being readily expressed as a function of R and L.
Density and pressure are related to each other through the state equation

$$\rho/\rho_0 = (\rho/\rho_0)^m \qquad (3)$$

in which m is a thermal constant. Combining Equations. (1) -(3) leads to the following pressure field:

$$(\rho/\rho_0)^{m+1} = (\rho_s/\rho_0)^{m+1} - (m+1)Q\mu \,/\, 2\pi KL\rho_0 \cdot \zeta n \cdot (L+2r)R \,/\, r(L+2R) \qquad (4)$$

in which ρ_s is the pressure in the source (R=r). Given an equipotential with pressure $\rho=\rho_e$ at a distance R=R1 from the
source E~.(4) may conveniently be solved with respect to the flow rate Q. This solution shall be included in the more general solution to be presented shortly.
As we are particularly interested in the case in which a plane equipotential or an isobar is a boundary, some further progress must be made.
Imagine a sink with capacity -Q located at a distance 2D from the source; D being the distance between the source and a plane equipotential as shown in Fig. 4. The separate pressure distributions for a source and a sink are given in expressions similar to Equation (4). In the joint case of a

source and a sink the pressure at a given point is found by adding two pressure fields of the form given in Equation(4); one field associated with the source and one with the sink. Given the pressure p at the plane equipotential the resulting pressure distribution may be solved with respect to the flow rate Q, yielding

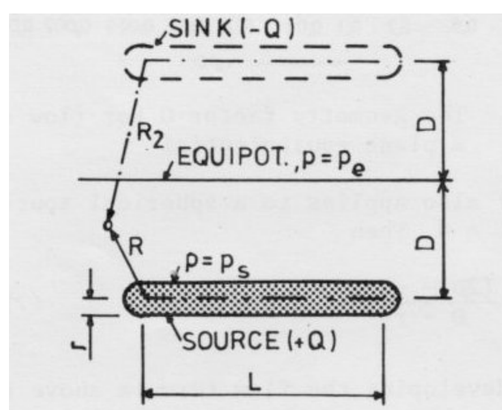


Fig. 4. Section through source and imaginary sink

$$Q = 2\pi KL\rho_0/(m+1)\mu G\ [\ (\rho_s/\rho_0)^{m+1} - (\rho_e/\rho_0)^{m+1}] \qquad (5)$$

In the equation above G is a geometry factor depending on r, L and D. In the general case involving a source or a sink and a plane equipotential G is given by:

$$G = \zeta n\ [2D\text{-}r)\cdot(L+2r)]\ /\ |L+2\ (2D\text{-}r)|r] \qquad (6)$$

which is represented graphically in Fig. 5

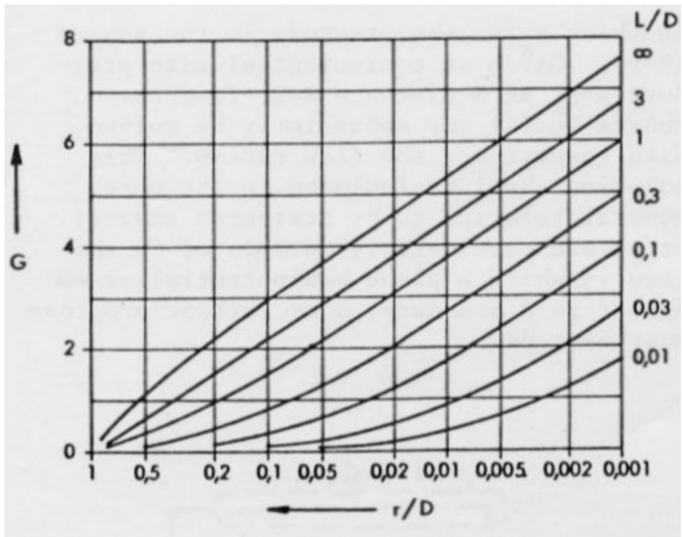


Fig. 5. The geometry factor G for flow to a plane equipotential

Equation(5) also applies to a spherical source, i.e. L = O Then:

$$L/G = (2D\ \text{-}r)r\ /\ D\text{-}\ r \qquad (7)$$

When developing the flow formula above the axis of the source has been parallel to the plane equipotential. Equation(5) may, how- ever, be applied with some care to provide approximate results also when this is not the case, i.e. $\alpha \neq 0$ as illustrated in Fig. 2. It is required that the minimum distance between the source and the equipotential is not small as compared with the length L of the source.

In the extreme case when D becomes much greater than L and r the geometry factor G tends to becomes independent of D. Evidently the direction of source axis is of no concern in this case.

As already indicated Eq.(5) also applies to the case in which a boundary equipotential with pressure ρ_e as the source (or sink) with pressure ρ_s, see. Fig. 3. Then:

$$G = \zeta n\ R(L+2r)\ /\ (L+2R)r \qquad (8)$$

and for spherical flow (L=O)

$$L/G = 2Rr/(R\text{-}r) \qquad (9)$$

Finally Eq.(5) also applies to uni-axial flow from a rectangular source as illustrated in Fig. 6.

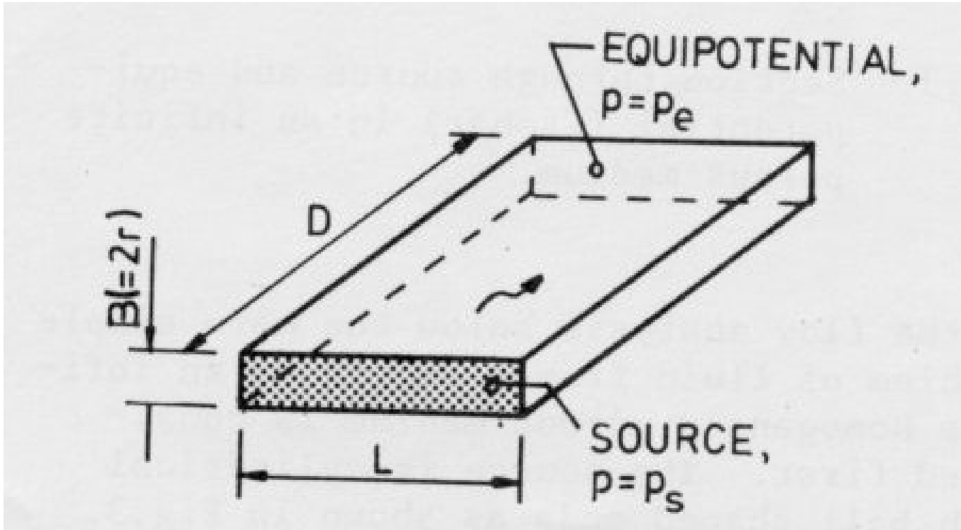


Fig. 6. Linear flow from a rectangular source

The geometry factor for linear flow is simply:

$$G = 2\pi\ D/B \qquad (10)$$

2.2 Air (gas) flow
In the case of isothermal gas flow the thermal constant m = 1, reducing Eq.(5) to:

$$Q_a\ = \pi KL\ \rho_0\ /\ \mu_a G\ [\ (\rho_s/\rho_0)^2 - (\rho_e/\rho_0)^2] \qquad (\ 11\ )$$

For the overview all symbols are repeated below, giving also preferred dimensions;

Q_a= air (gas) flow rate through a dry porous medium in m^3/s under reference pressure ρ_0 normally 105 Pa (1 bar)
K= intrinsic permeability, m^2
L= length of source, m
μ_a= dynamic viscosity of air (gas); at 10ºC μ_w= 1.8 .10^{-5} kg/m·s
ρ_0= reference pressure, normally 105 Pa
ρ_s= absolute pressure in source or sink, Pa
ρ_e= absolute pressure at a boundary isobar, normally atmospheric pressure (~105 Pa)
G= geometry factor as represented in Fig. 5 and Eqs.(6)-(10)

2.3 Water (incompressible liquid) flow
Eq.(5) is valid for water flow when inserting m = O. However, gravity has so far been ignored. This discrepancy may be overcome by substituting ρ_s - ρ_e with ρ_r which is the pressure heads in the source or sink. Eq.(5) thus yields:

$$Q_w = 2\pi KL\rho_r/\ \mu_w\cdot\ G \qquad (12)$$

The symbols are:

Q_w = water flow rate in m3/s through a completely water saturated medium
K, L, G as in section 2.2
μ_w = dynamic viscosity of water; at 10ºC $\mu_w = 1.3 \cdot 10^{-3}$ kg/m ·s
p = pressure head in source or sink, Pa

3 ROCK MASS PERMEABILITY ESTIMATES

The flow formulas, Eqs.(11) and (12) are developed for steady state flow in a homogenous permeable mass. The intrinsic permeability K is a constant for the particular medium in which flow takes place, regardless of whether the fluid is air or water.
It is recognized that rock is indeed no homogenous permeable mass. Considering hard rock no intergranular flow takes place, the flow being located in joints.
In some cases the joint system may be fairly regular, but more of ten the distribution of water conducting joints is most irregular. As regards for the air surge chambers constructed so far in Norway, these are located where the rock conditions are favourable; the chambers being intersected only by one or two water con- ducting joints. It is also an experience of many geologists that water flow is by no means uniform over a joint, rather the flow is taking place in channels.
In conclusion hereof it is doubtful to consider hard rock as a homogenous permeable mass. However, an exhaustive exploration of the true system of water conducting joints and channels may become insurmountable. In many cases a rough estimate of the rock mass permeability will do. So is the case with a compressed air surge chamber where the key questions are compressor capacity, need for grouting or lining etc.
The applications of the flow equations presented above may be many due to the variety of the geometry encountered.
Below we shall, however, be concerned with rock mass permeability estimates only and the application of the results to the estimation f the air leakage rate from an unlined air surge chamber. In consideration of the non-homogeneity of jointed rock one major concern is to take into account the jointing system. In this respect a key word is scale, i.e. the dimension of a subsurface opening as compared with the joint spacing. First we shall apply Eq.(12) to the determination of the equivalent rock mass permeability.

3.1 Borehole pumping tests (Lugeon tests)

Borehole pumping tests may be performed with different packer arrangements as illustrated in Fig. 7. When using two packers (Fig. 7 a) or when moving one packer stepwise along the borehole (Fig. 7c) the distribution of the water conducting joints may be estimated as suggested by Snow (1968) One packer only in the outer end of the borehole (Fig. 7b) is a less time consuming set up, but the interpretation of the result is more severe.

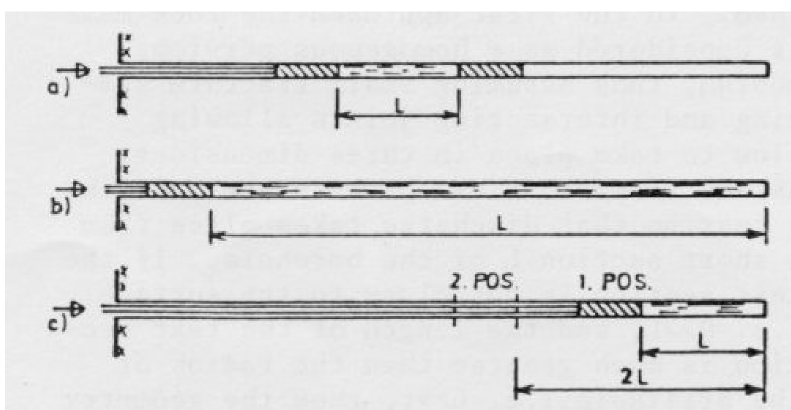


Fig. 7. Alternate packer arrangements.

It is important to get a rough

view of the jointing system and the joint spacing. Depending on the scale of the joint system the flow will be considered two- or three-dimensional as will be explained below.
As the diameter of a borehole is typically a few centimetres only it is required that flow takes place in joints rather than in a few channels. Otherwise the chance of hitting a channel with a borehole is marginal and it must be expected that borehole pumping test by no means give a true picture of the water conducting joint system. If the requirement above is not satisfied, permeability estimates must be based on the recording of water flow into a cavern or a tunnel as shall be discussed in a section to follow.
Eq.(12) is in most cases accurate enough for the interpretation of borehole pumping tests; the solving with respect to the intrinsic permeability gives;

$$K = (\mu_w \cdot Q_w \cdot G)/(2\pi L \rho_r) \qquad (13)$$

In borehole pumping tests it is customary to record the leakage in terms of the unit "Lugeon"; 1 Lugeon denoting 1 litre per minute and metre borehole at 10 kp/cm^2 (1000 Kpa) pressure head. As shall be seen below a rough proportionality exists between the Lugeon value and the intrinsic permeability.
Two approaches to the interpretation of borehole pumping tests shall be looked into. In the first approach the rock mass is considered as a homogenous pervious medium, thus assuming small fracture spacing and intersecting joints allowing flow to take place in three dimensions. Assume first that two packers are used in a test so that discharge takes place from a short section L of the borehole. If the test section is not close to the surface, i.e. D»L, and the length of the test section is much greater than the radius of the drill hole, i.e. L»r, then the geometry factor given in Eq.(6) reduces to:

$$G = \zeta n \cdot (L/2r) \qquad (14)$$

Note that G is independent of the distance D. from the ground surface. Eq.(14) in fact indicate that the pressure head is eliminated at a distance L/2 from the bore hole.

1) One may have noticed that Eq.(14) differ somewhat from the corresponding geometry factor included in Glovers formula published by Zacgar (1953), wherein G = ζn L/r. As both formulas require L»r the discrepancy is no great concern.)

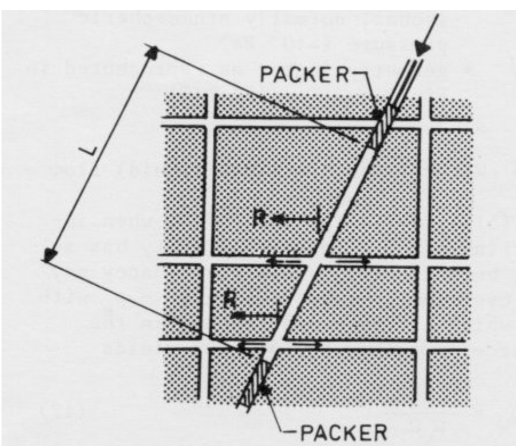


Fig. 8 Drill hole with packers in fractured rock under idealized conditions .

Assuming a borehole diameter in the order of 5 cm and test sections between 1 and 5 metres,
Eqs.(13) and (14) indicate that 1 Lugeon corresponds to K_b in the order of $1.0 \cdot 10^{-14}$ m^2 to $1.6 \cdot 10^{-14}$ m^2 at 10ºC water temperature.
In the approach above three-dimensional flow has been assumed. However, even in strongly fractured rock the joint spacing is likely to be great as compared with the diameter of the drill hole. Fig. 8 illustrates two-dimensional flow likely to take place in random joints intersecting a bore hole. The geometry factor, as given in Eq.(8), reduces to:

$$G = G = \zeta n\, R/r \qquad (15)$$

in which R is the radius from the borehole for which the pressure head is eliminated. If two or three intersecting joint sets exist, the water conducting joints intersecting the borehole will also intersect other joint sets at some distance from the borehole. As joint sets intersect the effective area subjected to flow increases substantially, thus causing the pressure head to drop rapidly. If the average joint spacing is in the order of say maximum 10 metres, the expected average distance between the borehole and the closest joint intersection is in the order of a few metres. Thus the pressure head is likely to be eliminated at a few metres distance from the borehole. The value of the geometry factor according to Eq.(15) is there- fore likely to be slightly higher than, or in the same order as suggested by Eq.(14).

In the case of three-dimensional flow first considered a two-packer test arrangement was assumed. In the latter case two-dimensional flow has been assumed close to the borehole regardless of the distance between the packers. Thus the latter interpretation goes for all test arrangements illustrated in Fig. 7.

Keeping in mind the significant uncertainties involved in the determination of K from borehole pumping tests the following rough interpretation formula is suggested for all tests:

$$K_b = Q_{w1} \cdot 1{,}5 \cdot 10^{-14}\ m^2 \qquad (16)$$

where Q_{w1} is the flow rate in terms of Lugeon units. The sub b indicates K being determined from a borehole pumping test.

As pointed out already the flow in the proximity of a borehole is basically two- dimensional due to the small diameter of the hole as compared with the joint spacing. On the other hand w hen analysing problems involving flow around a tunnel or cavern for which the dimensions are larger- or in the same order as the average joint spacing, it is fair to consider the flow three-dimensional in as far as flow takes place in all joint systems simultaneously. Due to scale effects it is fair to correct the intrinsic permeability K_b estimated from borehole pumping tests.

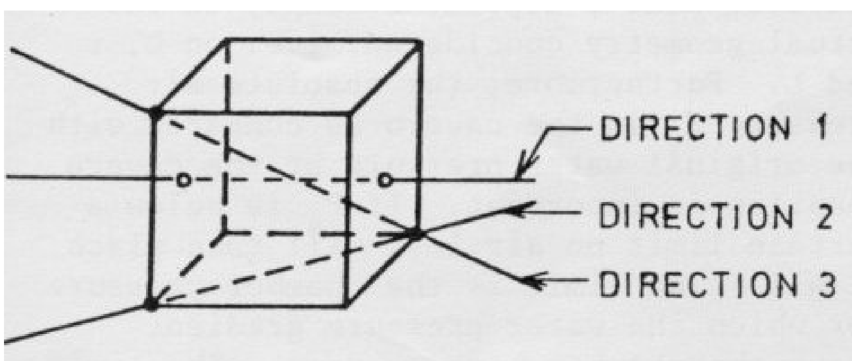


Fig. 9. Various drill hole orientations in a cubic joint system.

A correction factor c ideally depends on the drill hole direction in comparison with the orientation of the joint system. In a cubic joint system the three directions illustrated in Fig. 9 intersect one, two and three joint systems, respectively. When writing the correction due to scale:

$$K = c \cdot K_b \qquad (17)$$

the correction factor c equals 2.0, 1.4 and 1.2 for directions 1, 2 and 3 respectively. When no detail analysis is made; c = 1.5 is recommended.

As a joint system is likely to be irregular drill holes for pumping tests should if possible be made in 3 orthogonal directions. When! averaging the results from borings in different directions most weight should be put on the borings in the direction exhibiting the highest permeability.

3.2 Water flow into a cavern or a tunnel.
In the previous section some of the problems in borehole pumping tests were pointed out, such as a scale effect, as to whether several intersecting joint sets exists, the joint spacing, and to whether flow takes place in channels or joints of the parallel plate type. These problems are to some extent overcome when the permeability estimate is based on the recording of water flow into an excavated tunnel or a cavern. Thus if the problem is to estimate air loss from a compressed air surge chamber borehole pumping tests may be made at an early stage to select a suitable location for the chamber and to get a rough idea of the future air loss. However, the estimate of the compressor capacity needed for compensation of the air loss should be based on permeability estimates after the chamber is excavated. If the rock mass is permeability-wise homogenous at a large scale, reliable results may also be obtained based on water flow into the nearby tunnel.
The interpretation formula in Eq.(13) is suitable for estimating the equivalent intrinsic permeability based on water flow into a cavern or tunnel. The actual K- value calculated depend on whether the flow is two-dimensional or three-dimensional; the former being the case when only one major set of parallel joints intersect the tunnel or the chamber. However, it is not so important to get a true picture of the joint system w hen the permeability estimate is based on water flow into a cavern. Errors due to incorrect modelling are likely to cancel out when the same formulas subsequently are used for the estimation of air loss (reversed flow directions).

3.3 Special problems; only one or two major joints
So far we hate assumed a basically regular joint system, justifying the notion of homogeneity at a large scale. More typical is perhaps the case in which one or two major water conducting joints inter- sect a cavern as illustrated in Fig. 10. Then special consideration must be made.
In borehole pumping tests either significant flow or virtually no flow at all will be recorded depending on whether or not the drill hole intersects one of the water conducting joints. Evidently the average result of a number of borehole pumping tests may not give a true picture of the permeability when no homogeneity exists even at a large scale. Single joints must be treated individually.

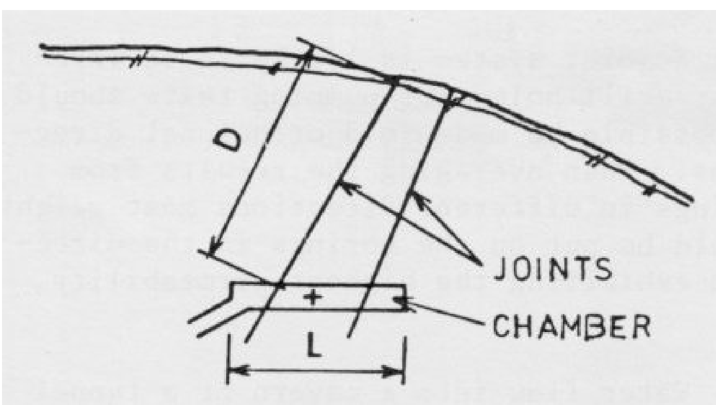


Fig. 10. Two major joints intersecting a cavern.

Still borehole pumping test and recording of water discharge into a cavernmay be used for permeability estimates. However, one may not determine a representative K-value for the rock mass at large. Rather a K-value applying to a specific tunnel section or cavern is estimated.
Regardless of whether permeability estimates are based on borehole pumping tests or water discharge into a cavern, the flow must be considered two-dimensional, i.e. $L/D = \infty$ must always be entered in to the geometry factor formula.
When borehole pumping tests are made the drill holes must be set in order to detect all major joints intersecting the future chamber. The average discharge rate at a given pressure head must be recorded for each joint. When adding together the average flow rates for all joints intersecting the cavern, an equivalent rate over the cavern length is obtained. Accordingly a representative Lugeon value for the cavern may be calculated, or the intrinsic permeability may be calculated directly

from Eq.(13). When using borehole pumping tests for single joints no correction due to scale is necessary as two-dimensional flow is considered all the time.

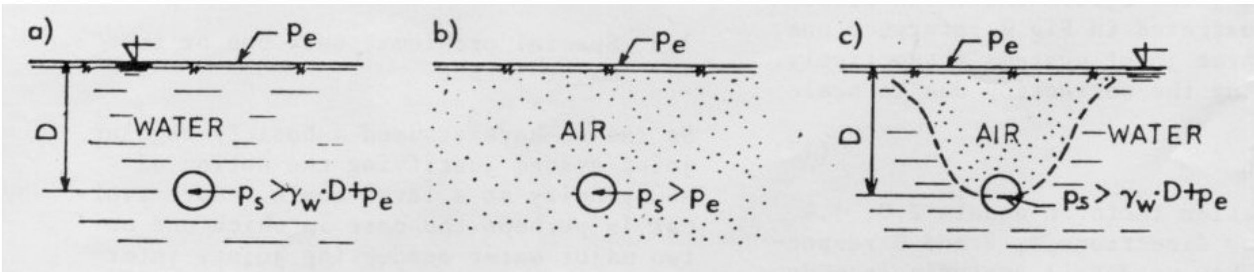


Fig. 11. Water, air and mixed water-air flow from a cavern.

4 AIR LOSS FROM A CHAMBER

In the previous chapter guidelines are given as to how the intrinsic permeability K of a rock mass may be calculated. As a rule the joint system must be fairly regular justifying the assumption of homogeneity at a large scale. However, it was also seen that even if this is not the case, an equivalent K-value applicable to a specific problem may be estimated. Now the problem is the estimation of air loss from a compressed air surge chamber, given the equivalent K-value.
The flow rates ~ of water from a cavern in completely water saturated, fractured rock and Q_w of air in a dry rock mass are calculated from Eqs.(12) and (11) respectively. In the case of mixed air and water flow the air flow rate Q_{aw} evidently must be a portion of the corresponding flow rate Q_a in dry rock as some of the joints are water filled and may not conduct air. Hence:

$$Qa_w = \psi\ Q_a \qquad (18)$$

where ψ is a coefficient less the unity.
No detail study is made on ψ . However, some comments shall be made. First, it is believed that ψ depends strongly on the actual geometry considered, i.e. on D, r and L. Furthermore, the absolute air pressure p in the cavern as compared with the original water pressure at the cavern location is important. If p is below a certain limit no air loss will take place at all. The limit is the chamber pressure for which the water pressure gradient above the chamber becomes zero. The limit pressure is slightly smaller than the hydrostatic water pressure. As the limit pressure is exceeded an air finger may escape from the chamber. Gradually, as the chamber pressure rises air will displace water in the joints above the chamber. The pressure in a chamber may be characterized in terms of the mean water gradient:

$$i_m = (p_r / \gamma_w D) \qquad (19)$$

in which p_r is the pressure head in the chamber and $\gamma_w \cdot D$ is the hydrostatic pressure at the chamber location.
As regards for compressed air surge chambers the value of i_m will often be in the order of 0.5, and never exceeding unity for flow to the ground surface. In the absence of detail analysis and to some extent supported by model tests by Barton (1972) it is believed that ψ is in the order of 0.2-0.5 for i_m in the order of 0.1-0.5.
The method described above has been used by Johansen and Vik (1982) for the prediction of air loss from compressed air surge chambers at three different sites in Nor- way. The intrinsic

permeability K was estimated from Lugeon tests, water discharge into boreholes and water flow into the chamber. The predicted air loss is in reasonable agreement with the field observations. Best results are obtained from the measurement of water flow (including evaporation) into the chamber.

REFERENCES

Barton, N. 1972,
A parallel plate model study of air leakage from underground openings situated beneath ground water level. Norwegian Geotechnical Institute, Oslo, internal report 54203-2, 66 p.

Johansen, P.M. and G. Vik 1982,
Prediction of air leakages from air -cushion surge chambers. Paper to be published in International Symposium on Rock Mechanics related to Caverns and Pressure Shafts, Aachen, 5 p.

Muskat, M. 1937,
The flow of homogenous fluids through porous media. McGraw- Mill, New York, reprinted 1946 by Edwards Inc., Ann Arbor, Michigan.

Snow, D.T. 1968,
Rock fracture spacings, openings, and porosities. Proc. ASCE, 94 (SM1), pp. 73-91.

Tokheim, O. and N. Janbu 1973,
Overslag over luftlekkasjer fra lukkede fordelingsbasseng i fjell (in Norwegian). Publ. in: Lukket fordelingsbasseng med luftpute, samlerapport. Ed. by River and Harbour Research Laboratory, NTH, Trondheim, pp. 75-113.

Zangar,C.N. 1953,
Theory and problems of water percolation. U.S. Dept. of Interior, Bur. of Reclam., Denver, Colorado, Eng. Monograph No 8, 76 p.

PREDICTION OF AIR LEAKAGES FROM AIR CUSHION SURGE CHAMBERS

P.M. JOHANSEN & G. VIK
Norwegian Geotechnical Institute, Oslo

SUMMARY
The air leaking out of air-cushion surge chambers has to be compensated for by stationary compressors. To predict the air loss, the Norwegian Geotechnical Institute has utilized a method developed by Tokheim and Janbu (1973). The paper presents calculations and measurements made at three sites. The correlation between predictions and observations is reasonable. The observed air losses at the three sit is ranged from about $8.3 \cdot 10^{-4}$ m^3/sec (at 10°C and 1 bar) to $3.3 \cdot 10^{-1}$ m^3/sec (at 10°C and 1 bar).
The paper also describes briefly the 120000m3 air-cushion surge chamber at Kvilldal, West Norway, operating since December 1981.

1 INTRODUCTION

Developments in tunnelling and power plant design have led to the use of deep high- pressure headrace tunnels. In this new design, the traditional surge shaft is re- placed by an unlined rock cavern filled partly by water and partly by air. The air-cushion is meant to dampen the pressure oscillations occurring with decreases or in- creases of the generator input, see fig. 1.
The new design required a method to predict air leakage rates as a basis for selection of compressor capacity. The compressor size depends on the maximum time allowable for chamber filling, and the capacity necessary to compensate for air loss through the rock mass. I Norway, the State Power System, uses mainly mobile compressors for filling up, and stationary compressors to compensate for lost air. To predict air loss rates, the Norwegian Geotechnical Institute has used a method developed by Tokheim and Janbu (1973) at the department of Soil Mechanics at the Norwegian Institute of Technology. The method is described in detail by Dr. Tokheim and Prof. Janbu elsewhere in this symposium.

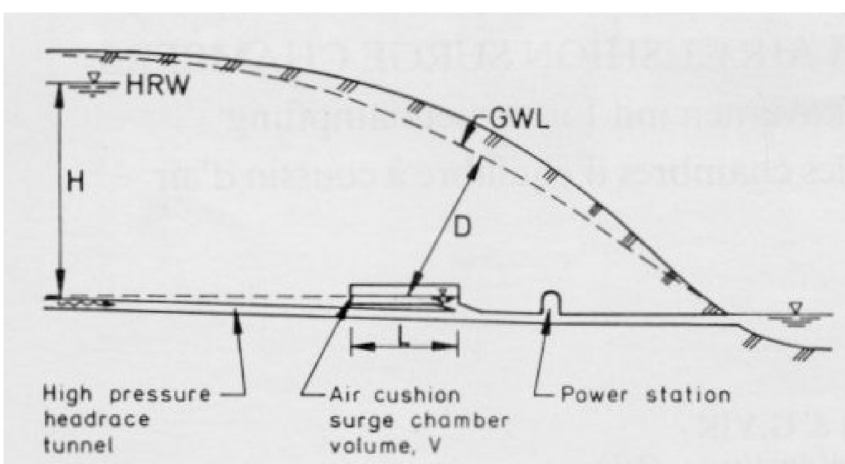


Fig. 1. General Layout of a power plant with air cushion surge chamber.
D = Distance to groundwater level (equipotential surface)
H = Hydraulic head
L = Lenght of chamber.

At three power plant sites The Norwegian Geotechnical Institute (NGI) conducted the geological investigations necessary to determine the best site possible for the location of air-cushion surge chambers, and predicted the air leakage according to the method of Tokheim and Janbu. The three power plants owned by the Norwegian State Power System are Lang-Sima, Oksla and Kvilldal. At a fourth location, the Nye Osa power plant, relatively large air losses were recorded. The owner, the Energy Authority in the County of Hedmark, provided NGI with permeability measurements which made possible a prediction of the rate of air loss.

2 THEORETICAL BACKGROUND

The Tokheim and Janbu theory formulates the rate of air leakage through water saturated rock mass, Q_{aw} as follows:

$$Q_{aw} = \psi \cdot Q_a = \psi \cdot (\pi \cdot K \cdot L \cdot \rho_0) / (\mu_a \cdot G) \cdot [(\rho_s/\rho_0)^2 - (\rho_e/\rho_0)^2]$$

List of symbols:

Q_{aw} = Air leakage rate through water saturated rock mass in m^3/s at 10ºC and a pressure of $1.0 \cdot 10^5$ Pa.
Q_a = Air leakage rate through dry rock mass, m^3/s at 10ºC and a pressure of $1.0 \cdot 10^5$ Pa.
ψ = Constant, $0 < \psi < 1$.
K = Intrinsic permeability, m^2.
L = Length of chamber, m.
μ_a = Dynamic viscosity of air 1.76 .10-5 kg/m .s .
G = Geometry factor, depending on the dimensionless parameters r/D and L/D.
r = Chamber radius (idealized geometry).
D = Distance from cavern centre to equipotential surface (= ground water level).
ρ_0 = Reference pressure, 1.0.105 Pa.
ρ_s= Maximum absolute air pressure in the cavern, Pa.
ρ_e= Absolute air pressure on the dimensioning equipotential surface. This is usually equal to the ground water level and $\rho_e = \rho_0 = 10^5$ Pa.

As stated by equation (1), the air flow through the saturated rock mass is reduced with a factor ψ compared to flow through the dry rock mass.

2.1 The value of ψ

Air leakage rate in water saturated rock mass is a function of the dimensions D, r, L and the absolute air pressures ρ_s and ρ_e. The mean water gradient, i_m, in the outward direction from the water filled part of the chamber after the surge chamber has been put into operation, is defined according to Tokheim and Janbu:

$$i_m = \rho_r/(\gamma_w \cdot D)$$

ρ_r= excess pressure of water in the chamber compared to hydrostatic distribution of pore water pressure, Pa.
γ_w = unit weight of water, 10 kN/m^3.

In the case of air-cushion surge chambers, the gradient i_m, is generally about 0.5 and never more than 1.0 for free drainage to the surface. This implies that ψ rarely exceeds 0.5 (Tokheim and

Janbu, 1973). For gradients i_m not too far below 0.5, ψ will presumably lie within the interval 0.1-0.5 for usual chamber geometries.

2.2 The value of G

To calculate the value of the geometry factor G, one needs to estimate the geometry of the flow through the rock mass. If the dimension of the excavations is of the same order as the spacing between water filled fissures, three-dimensional flow can be assumed. In the case of anisotropic flow, a two-dimensional model must be considered. In the three-dimensional case the chamber geometry is modelled as a cylinder with spherical ends, and the length, L, is in this case the distance between the sphere centres. For two-dimensional flow the real length is used. Common values of G for three-dimensional flow range between 1.2 -1.8 and for two-dimensional flow between 3.5-5.0.

2.3 The permeability K

The influence of the factors ψ and G on the air leakage rate is of the order of 10.
The intrinsic permeability, K, is the parameter with the largest influence on the air-leakage rate. The geological investigations aim therefore at obtaining a reasonable value of the average permeability.

3 SELECTION OF CONSTRUCTION SITE

In Norway, six air-cushion surge chambers have been constructed to date (1982). All the chambers are located in gneissic and granitic rock mass of precambrian age.
The quality of this rock is generally very good. Occasionally the rock has discontinuities containing swelling clay or other clay minerals. The hydraulic function of the surge chamber limits the length of the head- race tunnel available to about one km.
Within this section, the best site possible is selected after investigations by core drillings and measurements of hydraulic conductivities.

4 DETERMINATION OF K

To increase the confidence in the measured permeability, NGI measures the permeability in three ways

- Constant head test (Lugeon tests) with stepwise increasing hydraulic heads until the maximum head equals maximum absolute air pressure in the surge chamber.
- Measurement of water flow out of free draining boreholes (if any).
- Measurement of water flow into the! excavated cavern, including theoretical evaporation from the cavern walls. The discharge is also measured during excavation.

5 MEASUREMENTS FROM THREE SITES

Table 1 lists the permeabilities measured by the above three methods at the power plants under study and compares measured values of the intrinsic permeability used in prediction of air leakage.

Table 1. Measured intrinsic permeabilities of three sites, $K(m^2)$

Site: Measuring method	Lang-Sima Sima power plant Eidfjord, Western Norway.	Oksla power plant Odda, Western Norway	Nya Ona power plant Rena Eastern Norway
Lugeon test, mean	$5,1 \cdot 10^{-18}$	$3.0 \cdot 10^{-17}$	$9,8 \cdot 10^{-15}$
Lugeon test, median	$4,7 \cdot 10^{-18}$	$3.7 \cdot 10^{-17}$	$6.0 \cdot 10^{-15}$

Table 1. Cont.

Site: Measuring method	Lang-Sima Sima power plant Eidfjord, Western Norway.	Oksla power plant Odda, Western Norway	Nya Ona power plant Rena Eastern Norway
Water discharge from boreholes	$8,8 \cdot 10^{-19}$	$2.4 \cdot 10^{-18}$	$9.9 \cdot 10^{-15}$
Water flow (+ evaporation) into the chamber	$2.9 \cdot 10^{-19}$	$3.4 \cdot 10^{-18}$	$1.7 \cdot 10^{-15}$
Selected K-value for air leakage calculations	$5.1 \cdot 10^{-18}$	$3.7 \cdot 10^{-17}$	$9.9 \cdot 10^{-15}$

Table 2. Parameters used in leakage calculations

Site: Parameter unit	Lang-Sima Sima power plant Eidfjord, Western Norway.	Oksla power plant Odda, Western Norway	Nya Ona power plant Rena Eastern Norway
D,m	350	450	125
r, m	8.0	8,5	7,0
L_2, m 1)	---	80	70
L_3, m 1)	36	63	56
i_m	0,58	0,11	0,60
ψ	0,50	0,15	0,5
P_s, 10^5 Pa	49	46	20
G_2 1)	---	4,6	3,7
G_3 1)	1,3	1,5	1,7
V, m^3	6200	17000	12500

6. PREDICTION OF AIR LEAKAGE.

The values listed in table 1 and 2 were used in equation (1) for the leakage calculation for three-dimensional flow, the following rates of air leakage were predicted: (all leakages are measured at 10 C and 1 bar pressure):

Lang-Sima: $Q_{aw} = 3.0 \cdot 10^{-3}$ m^3/s (180 l/min)
Oksla: $Q_{aw} = 8.8 \cdot 10^{-3}$ m^3/s (530 l/min)
Ny Osa: $Q_{aw} = 1.2 \cdot 10^{-3}$ m^3/s (70 l/min)

In the case of two dimensional flow, the predicted rates were:

Oksla: $Q_{aw} = 3.6 \cdot 10^{-3}$ m^3/s (220 l/min)
Ny Osa: $Q_{aw} = 6.7 \cdot 10^{-1}$ m^3/s (40 m^3/min)

7 MEASUREMENTS OF AIR LOSSES

The air volume in the chamber is measured by the water level, (and thus the water volume) and the air pressure P. The product $P \cdot V$ is obtained. Even with good instruments, it is difficult to measure

water levels with accuracy better that +/- 1 cm. With an area of about 800 to 1000 m^2, a one-cm increase in water level corresponds to a compressed air loss of 10 m^3.
The reported air losses at the three sites are (at 10°C and 1 bar):

Lang-Sima:	$Q_{aw} = 7.5 \cdot 10^{-4}$ m^3/s (45 l/min), the pressure ρ_s averaged only 36.3 bars during the measuring period.
Oskla:	"Very modest leakage, almost impervious."
Nye Osa:	$Q_{aw} = 0.1$ -0.37 m^3/s (6 -22 m^3/min).

The air loss varied considerably with water level in the chamber. Based on compressor running time only, the rate of air leakage was increasing. The compressor capacity was however not checked.
The air loss at Lang-Sima, if calculated at actual ρ_s-value during the measuring period was:

Q_{aw} (3D)= $1.3 \cdot 10^{-3}$ m^3/s (80 l/min) with $i_m = \psi = 0.40$.

Table 3 compares the predicted and measured rates of air loss.

Table 3. Predicted and measured rates of air loss, Q_{aw} {m^3/s at 10ºC and 1 bar)

Site	Lang-Sima (p_s = 36,3 bars)	Oksla	Nye Osa
Calc. 3-dimentional	$1,3 \cdot 10^{-3}$	$8,8 \cdot 10^{-3}$	1,20
Calc. 2-dimentional	---	$3,6 \cdot 10^{-3}$	0,67
Measured	$7,5 \cdot 10^{-4}$	minimal leakage almost impervious	0,1 – 0,37

In view of the accuracy of the water level measurements and since the dimensioning permeability is taken as either the maximum mean or maximum median, the Tokheim and Janbu model is believed suitable for prediction of air leakages.

8. SOME DATA ON KVILLDAL, ULLA-FØRRE, WEST-NORWAY, THE LARGEST POWER STATION AND AIR CUSHION CHAMBER IN THE COUNTRY.

At Kvilldal, the fist of four generators each capable of producing 300 MWh, was put into operation at the end of 1981 The power plant comprises an air-cushion surge chamber with total volume of about 120 000 m^3 including access and transportation tunnels.
With all generators running, the air volume in the chamber will be about 80- 95000 m^3 at an absolute pressure of 42 bars.
The chamber is excavated around a rectangular central pillar measuring 84 x 46 m. The cavern has a span of 16 m and the height varies between 17 and 24 m.
The maximum operating pressure in the chamber will be 42 bars. The Norwegian Geotechnical Institute has calculated the rate of air leakage. The predictions are however hampered by the unusual chamber geometry and uncertainties in the pore pressure value. Piezometer installations indicated high pore pressure at chamber depths, thus low mean water gradient i_m and low air losses. Measurements near the chamber give low pore pressures and therefore higher calculated air losses. The Tokheim and Janbu theory yields a rate of air f low of 1 .7 .10-2 m^3/s (1 m^3/min}. Until chamber filling is complete, it is difficult to measure air losses, since the water surface is very large (ca. 5000 m^2} but measurements during the first months of filling indicate a total air loss of the order of 1.7- 3.3. 10^{-2} m^3/s (1- 2 m^3/min} .

9 REFERENCES

Tokheim, O. and N. Janbu 1973
Overslag over luftlekkasjer fra lukkede fordelingsbasseng i fjell (Estimates of air leakages from air-cushion surge chambers). Publ. in: Lukket fordelingsbasseng med luftpute. Samlerapport. Ed. by Norwegian Institute of Technology. Dept. of Soil Mechanics, Trondheim, a.o. pp 75 -113.

Tokheim, O. and N. Janbu 1982
Flow rate from a source or sink located in a semi infinite permeable mass. Paper to be published in International Symposium on Rock Mechanics related to Caverns and Pressure Shafts, Aachen 1982.

Surge Chamber design for Jukla

By J. Bergh- Christensen.
A/S Geoteam, Oslo, Norway.

The design of an unlined compressed air surge chamber for the Jukla pumped-storage plant in Norway is described. operational air pressures in the 6200 m^3 chamber vary between 6 and 24 kg/cm^2.

The use of unlined, pressurized rock cavities has a long tradition in Norway. More than 50 hydroelectric power plants with unlined pressure shafts and tunnels have been constructed, involving water pressures ranging from 100 to 500 m static head.
The traditional high pressure plant comprises a head-race tunnel at near reservoir level, with a surge shaft at the downstream end, and a pressure shaft down to the power station.
Recent developments in power plant design have called for the use of deep lying pressure tunnels. The conventional surge shaft is replaced by a deeply bedded chamber containing a large bubble of compressed air. The air bubble or cushion functions as a shock absorber to ensure hydraulic stability of the system, see Fig. 1.

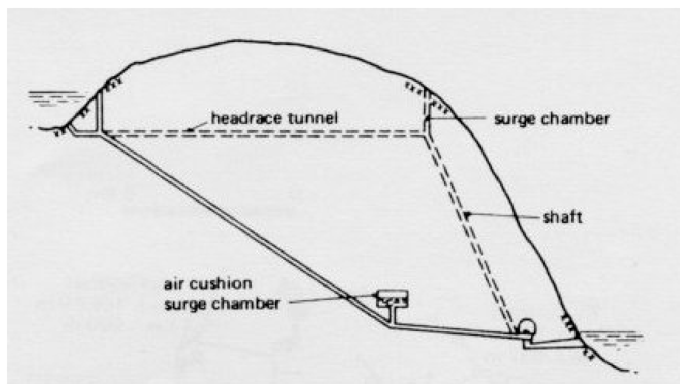


Fig. 1. Use of compressed air-cushion surge chamber as compared with a conventional design.

In 1973 and 1974, the world's two first high-pressure air cushion surge chambers were successfully put into operation, at the Driva and Jukla power plants in Norway, thus providing practical experience of large, unlined rock chambers for gas containment.
The Jukla pumped-storage project, (Fig. 2.) developed and owned by Norway's State Power Board, uses runoff from the great Flogefonni glacier in western Norway. This project includes a 6200 m^3-compressed air surge chamber, with operational air pressures varying between 6 kg/cm^2 and 24 kg/cm^2 (Fig. 3.).
Acting as engineering geological consultant to the State Power Board, A/S Geoteam was responsible for site investigations and rock mechanics design evaluations performed before and during construction of his chamber.
The investigations were performed in three consecutive phases:

- geological mapping of tunnels
- diamond core drilling, pore pressure measurements and air and water leakage tests
- inspection and control during excavation

Initially a detailed engineering geological survey was carried out in the already completed headrace pressure tunnel. The rock here is Precambrian gneiss showing near horizontal foliation and moderate schistosity. The head-race tunnel crosses a few major swelling-clay gouges calling for local on-face concrete lining support. Except for these zones, the rock is moderately jointed.
Based on the geological survey, a 100 m section of the headrace tunnel, 600 m upstream of the power station and with a rock overburden of 340 m, was provisionally selected for detailed

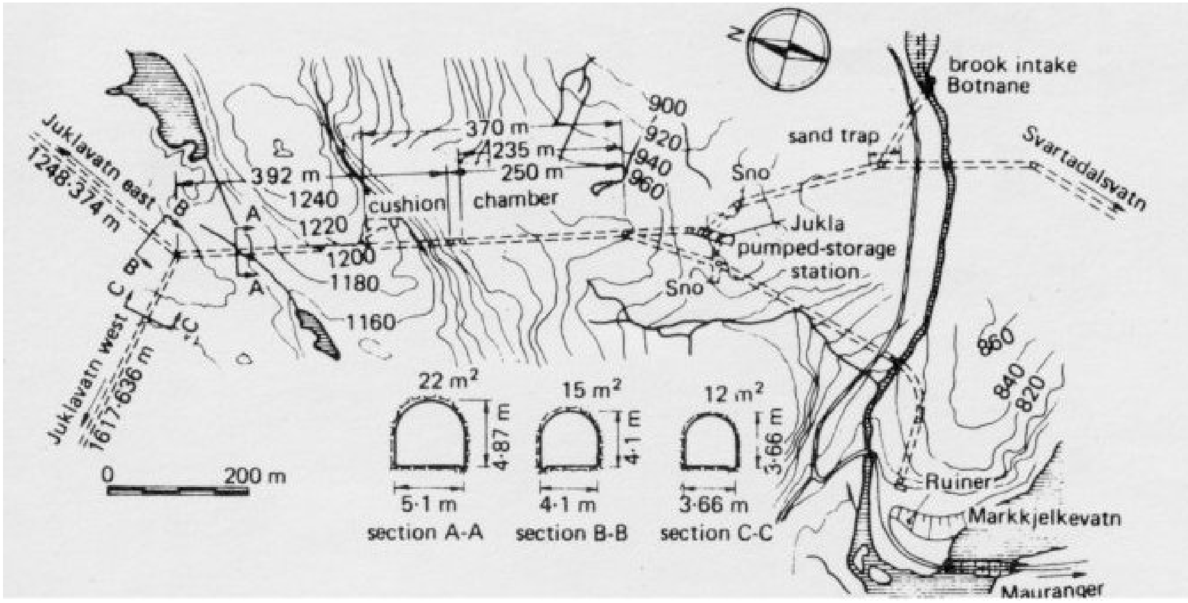


Fig.2. Plan view of the Jukla power plant showing the ection and areas of the tunnel.

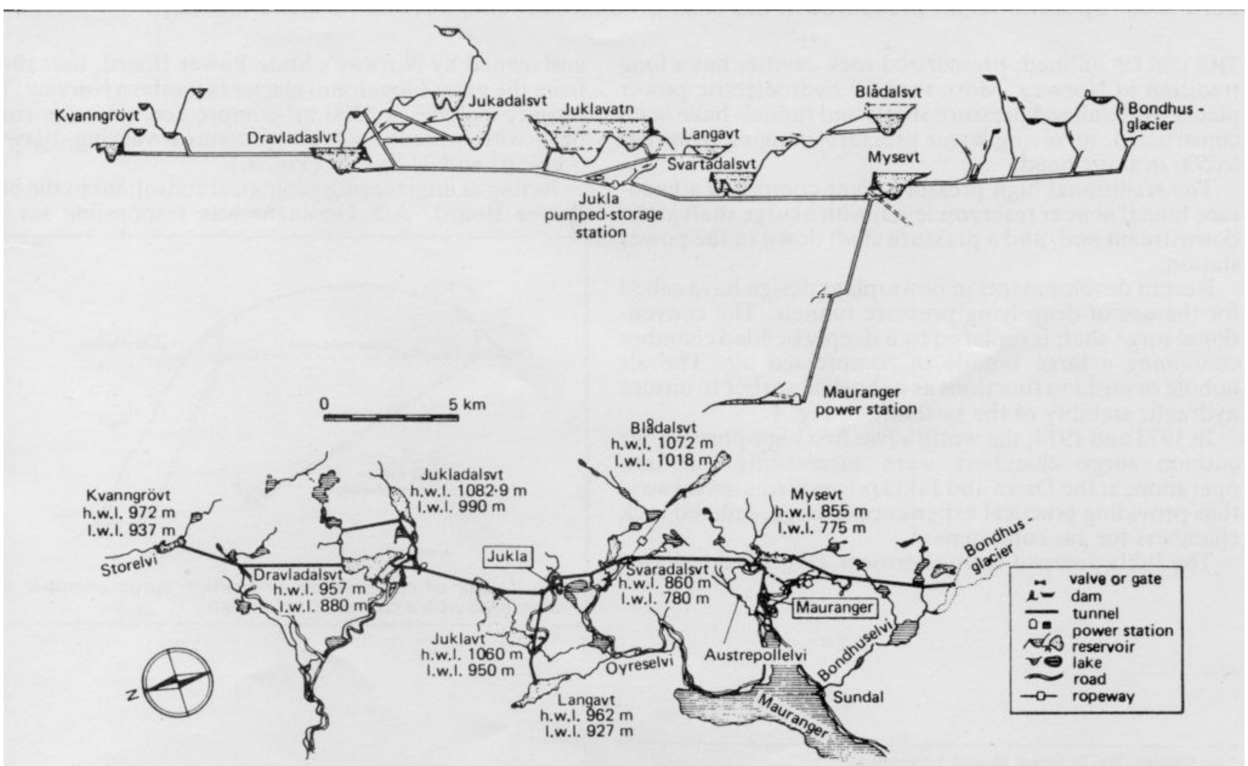


Fig. 3. The Folgefonni project showing the position of the Jukla plant in relation to the other parts of the scheme.

investigations. Three 40-80 m deep diamond core drill holes, and two 20 m-percussive drill holes were drilled here to investigate the rock quality. They revealed massive rock with ROD values of better than 90 at the proposed chamber location (Fig. 4.).

The rate of air loss to be expected as a result of leakage through the rock is decisive for a feasibility evaluation of an unlined air cushion surge chamber. Theoretical studies indicate that such losses are governed by two main factors, the permeability of the rock mass, and the ratio of chamber gas pressure to pore water pressure in the surrounding rock.

Pore water pressure in the surrounding rock was recorded continuously during site investigations and the subsequent construction period, and showed a natural pore pressure in the surrounding rock of 17 kg/cm^2 (Fig. 5.).

For assessment of rock mass permeability, air and water pumping tests were performed on site in two of the diamond core drill holes, using special test equipment designed and operated by personnel from the Norwegian Geotechnical Institute.

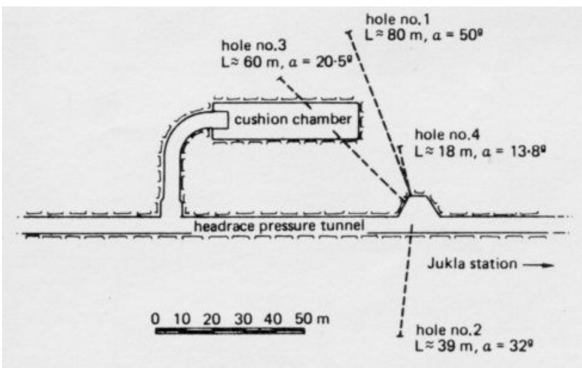


Fig. 4. Exploratory drillings of the chamber site.

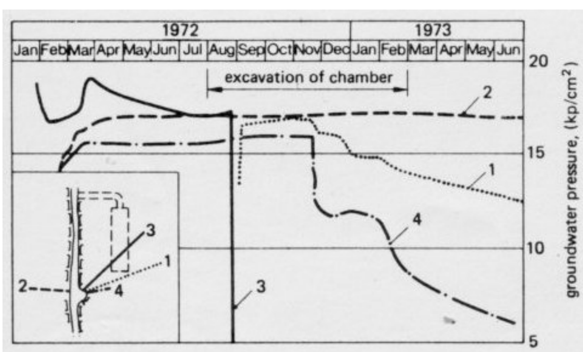


Fig. 5. Pore pressures recorded at the chamber over 18 months.

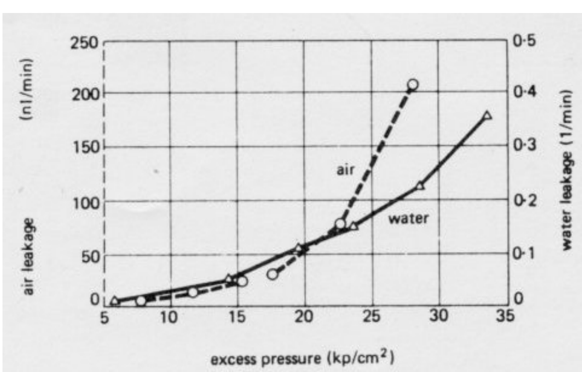


Fig. 6. Borehole No. 1; water and air leakage tests (nl is a normal liter)

Water pressure tests were carried out at pressures of up to 50 kg/cm^2, using borehole test sections which were 67 m and 46 m long. At peak pressure, water losses of 360 and 30 cm^3/min respectively were recorded for the two test sections, corresponding to Lugeon values of 0.0015 and 0.0002. This extremely low rock mass permeability clearly indicated favourable conditions for surge chamber placement.

Air leakage tests were performed in the same borehole test sections as mentioned above. At peak pressures of 40 kg/cm^2 the recorded air leakage , reduced to equivalent volume at atmospheric pressure and given in normal litres per minute (nl/min), was 215 and 6 nl/min respectively for the 67 and 46 m/long borehole sections (Figs. 6. and 7.).

Both theoretical evaluations and practical experience indicate that water pressure tests give the most reliable basis for rock mass permeability assessment of potential air leakage problems. Taking into consideration the difference in viscosity of water and air, water permeability test values may be used for the computation of possible air leakage rates from an underground chamber .

Based on an evaluation of all permeability data, possible air leakages from the surge chamber were calculated to be of the order of 100-500 nl/min, and hence well within acceptable limits.

The surge chamber, with a cross-section of 128 m^2, length 48 m and a total volume of 6200 m^3, was excavated between August 1972 and February 1973. Because of the near horizontal rock schistosity, the roof was permanently secured with grouted rock bolts. No further reinforcement or seating measures were found to be necessary.

In May 1974 the compressed air surge chamber

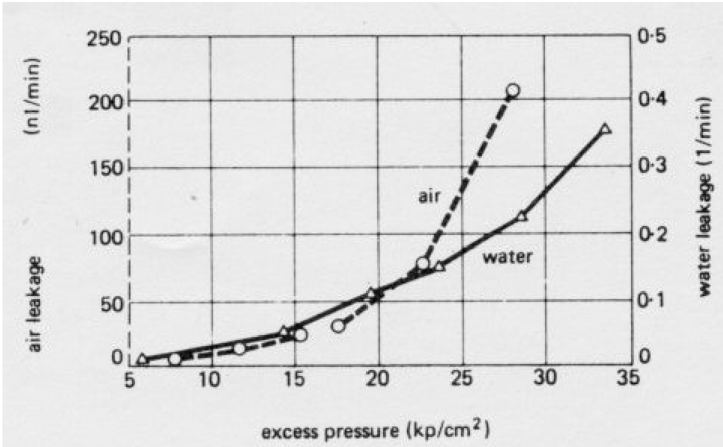


Fig. 7. Borehole No. 3; water and air leakage tests.

was put into operation. The experience so far shows that the site investigations provided a satisfactory basis for leakage evaluation (Fig .8.).

During operation of the surge chamber. total air losses of the order of 20-200 nl/min were recorded at working pressures beyond 17 kg/cm^2. With pressures lower than the recorded ground water head of 17 kg/cm^2, no appreciable air losses were recorded.

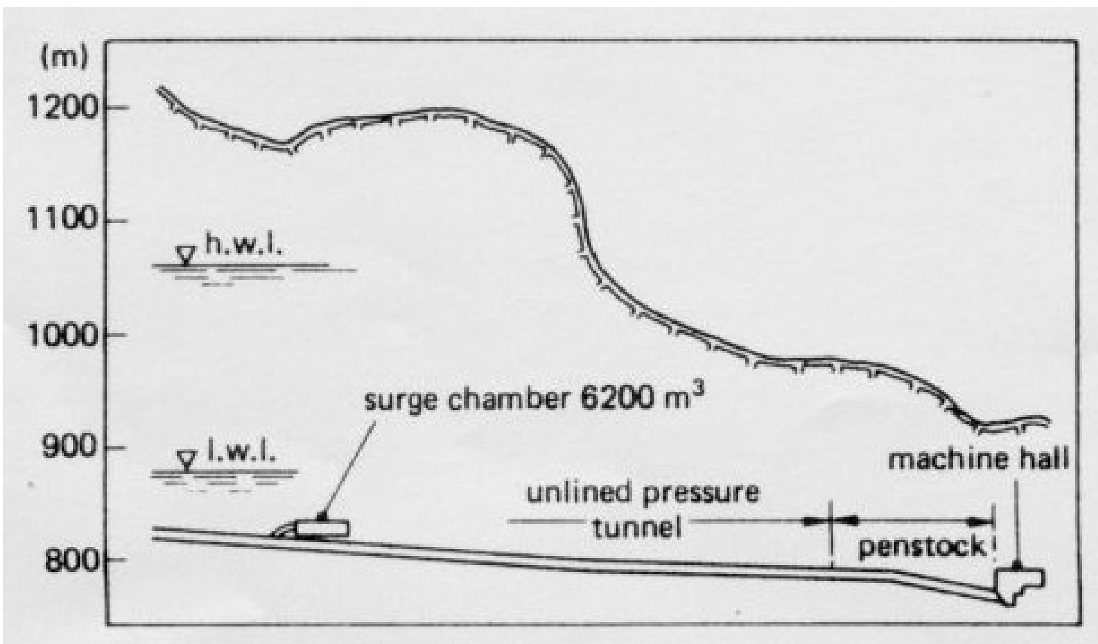


Fig. 8. Position of the air cushion chamber at Jukla.

EXCAVATION AND LEAKAGE CONTROL OF UNLINED AIR CUSHION SURGE CHAMBER AT THE OSA HYDROELECTRIC PROJECT

Sverre Edvardsson
Consulting engineer, Sivilingeniør Elliot Strømme A/S (Norconsult) Hamar, Norway
Birger Sætren
Chief engineer, Hedmark Energiverk (Hedmark Energy Board), Hamar, Norway

SUMMARY
The Osa Hydroelectric Project is an interesting case in the use of unlined air cushion surge chambers in Norway The chamber is situated 140 m below ground and has an air volume of 10000 m^3. The average working pressure is 18 bars. When the power station was commissioned in 1981 unacceptable air loss was experienced from the air cushion. After nine months the power station was shut down and the air leakage was reduced by 95% by grouting carried out from inside the emptied chamber. Since then the station has been in continuous operation without any problem caused by air leakage.
This paper describes the project, operating experience, grouting performance and cost benefits.

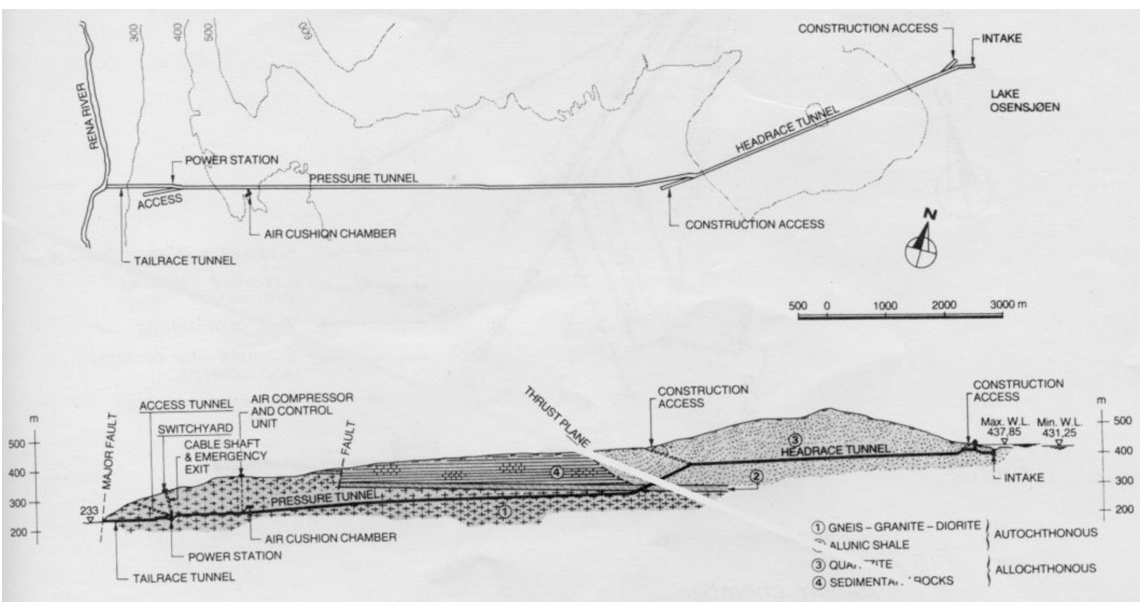


Fig. 1. Osa Hydropower Project. Layout and longitudal section

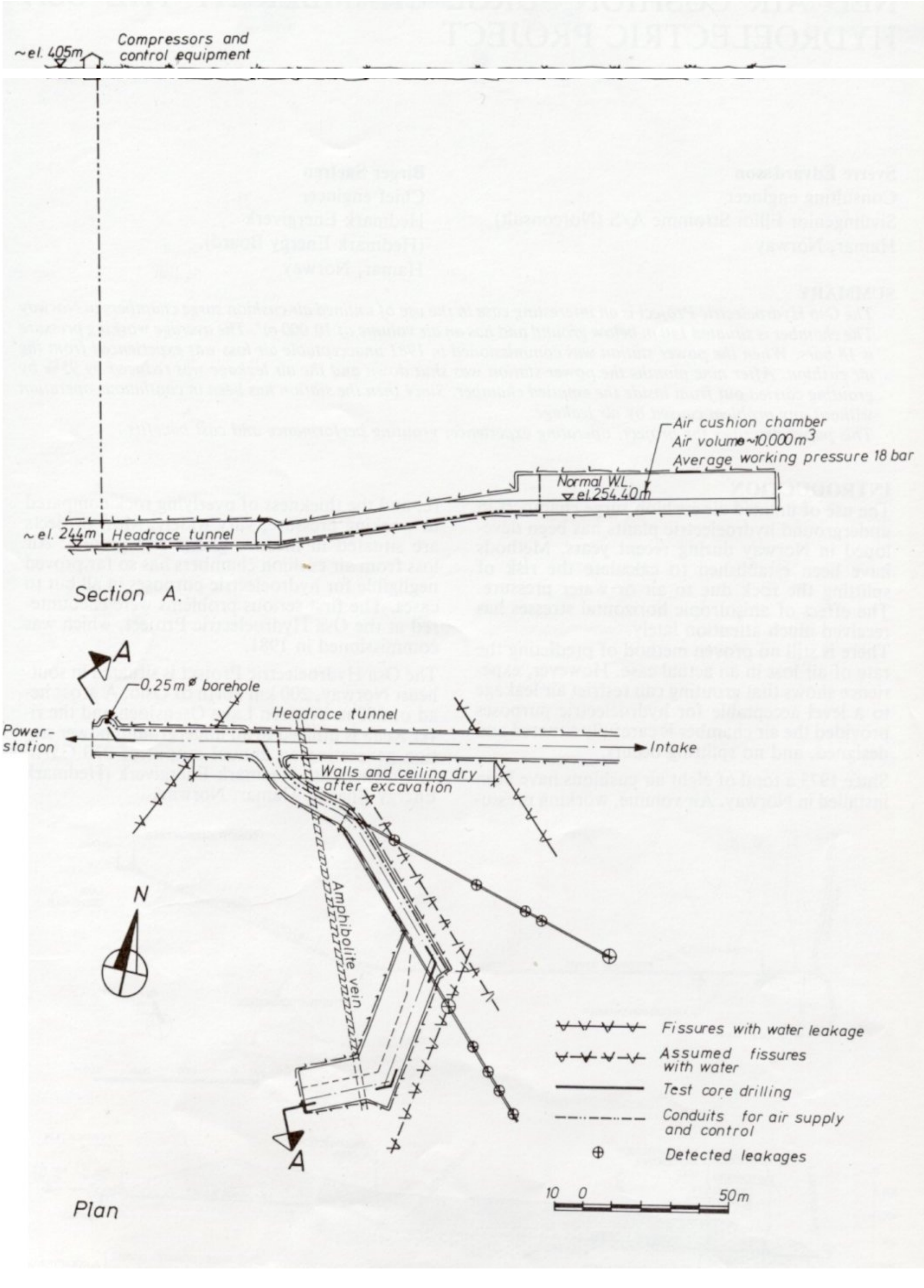
INTRODUCTION
The use of unlined air cushion surge chambers in underground hydroelectric plants has been developed in Norway during recent years. Methods have been established to calculate the risk of splitting the rock due to air or water pressure. The effect of anisotropic horizontal stresses has received much attention lately.
There is still no proven method of predicting the rate of air loss in an actual case. However, experience shows that grouting can restrict air leakage to a level acceptable for hydroelectric purposes provided the air chamber is carefully situated and designed, and no splitting occurs.

Since 1973 a total of eight air cushions have been installed in Norway. Air volume, working pressure, and the thickness of overlying rock compared to working "pressure, vary widely. Most projects are situated in massive gneiss and granite. Air loss from air cushion chambers has so far proved negligible for hydroelectric purposes in all but to cases. The first serious problems were encountered at the Osa Hydroelectric Project, which was commissioned in 1981.
The Osa Hydroelectric Project is situated in southeast Norway, 200 km north of Oslo. A gross head of 200 m between Lake Osensjøen and the river Rena is utilized in an underground power station generating an annual output of 270 GWh. The owners are Hedmark Energiverk (Hedmark Energy Board), Hamar, Norway.

DESCRIPTION OF THE PROJECT
The layout and longitudinal section are shown in Fig. 1. From the reservoir a 14.5 km pressured headrace tunnel leads to the underground power station. The tailrace tunnel is 900 m long. Geological constraints have greatly influenced the design of the project. A major fault followed by the Rena valley produced a number of associated fracture zones in the gneiss-granite east of the river. These created some construction difficulties, the main one being related to the influx of water, and as the access tunnel slopes downwards at 1 in 8, the water had to be continuously pumped out until the tailrace tunnel was completed.
The air cushion chamber is situated in gneiss- granite 140 m below ground. The air volume is 10,000 m^3 and the average working pressure is 18 bars. The alternative to an air cushion surge chamber was a shaft to the surface and a 40 m high water tower which was found to be more expensive and also undesirable for environmental and operational reasons.
The decision to choose an unlined air cushion chamber was taken when a suitable location was found adjacent to the headrace tunnel, about 1 km from the power station. This is almost the critical distance with respect to the station's operational stability. Test drilling was carried out so that major fissures could be avoided and led to the design shown in Fig. 2.
Air compressors and control equipment are located above ground. The connecting conduits are installed and grouted in a vertical 250 Ø mm, 150 m long borehole. The installed compressor capacity is $2 \cdot 19.4$ Nm^3/min. (1 Nm^3 = 1 m^3 under atmospheric pressure)..

EXCAVATION AND GROUTING BEFORE COMMISSION
Test drilling before and during excavation had a significant influence on the design of the chamber. The excavation of the chamber itself followed a procedure of drilling 24 m long test holes ahead of the tunnel face, conducting water loss tests, pregrouting and blasting. Grouting was done with cement suspension or Stabgel FR 30/70 according to the results of the water loss tests. The static ground water pressure near the chamber was 15 bars. The pressure for water loss tests and grouting was prescribed not to exceed 25 bars. Most test holes were considered tight and pregrouting was only carried out in a few places. The excavation was carried through without intersecting any major water-bearing fissure. Nevertheless, the total water leakage into the chamber after excavation was estimated to be about 15 l/min.
To predict the amount of air loss from an unlined air cushion in rock is not an easy task. One approach which seemed reasonable was to consider the amount and distribution of water leaking into the chamber. So far no proven method based on this approach has been established in engineering practice.
In previous cases in Norway water leakage into the chambers has been less than that experienced in the Osa case, and air losses have been undetectable or negligible for the purposes of a surge chamber. After completion of the chamber excavation, grouting proceeded in steps, mainly using a cement suspension (Grouting pressure up to 25 bars). Most of this work concentrated on stopping water leakage from existing cracks above operational water level, as these cracks were expected to account for most of the leaks from the chamber after its commission.

Fig. 2. Air cushion chamber.



When the grouting work was concluded the total water leakage into the chamber had been reduced to 3 l/min, observed as local dripping in certain parts of the chamber, especially in the north-eastern corner and along the upper part of the east wall.
Based on the distribution and rate of water leakage, a total air loss of about 0.15 Nm^3/min. out of the chamber was anticipated, which would be negligible with regard to the operation and cost effectiveness of the project.

PERFORMANCE AFTER COMMISSION

Shortly after the power station was commissioned in July 1981, air loss from the air cushion became a problem. At the end of the month the leakage was accurately measured to be about 15 Nm^3/min., which represented 38% of installed compressor capacity.
This was found to be unacceptable as a permanent state, with respect both to operational safety and energy consumption.
Through the first winter the surge chamber was operated with an air volume of 8800-7800 m^3, which reduced the air loss to about 5 Nm^3/min. However, this also involved restricting the power station' s operational capacity. Although the air loss varied during this first winter, the trend was a slow but alarming increase, Fig. 3. After a couple of sudden outages of the turbines, the rate of air loss increased significantly. In some periods a confusing decrease occur- red for which the only explanation seemed to be a temporary build-up of water pressure within the paths of leakage.
A net water loss of about 50 l/sec out of the headrace tunnel system had been measured which represented an annual loss of power output of about 700,000 kWh. Most of this leakage probably occurred between the power station and the air cushion chamber, where the static water head corresponds to a level higher than the overlying rock. In this area an increase of surface discharge had been observed after the power station was commissioned.
By the end of 1981 it was decided to carry out a sealing programme during the summer 1982 to reduce the air loss.

PLANNING THE SEALING PROGRAMME

The distribution of air leakage with respect to a vertical axis was determined by letting the air leak out without replacing it. The water level rise in the chamber was then plotted against time. By comparing this curve with a volume level curve of the chamber, the distribution of leakage at different levels was established. The leakage was concentrated at two levels. About 70% occurred near the bottom of the air cushion and could not be explained by any observation made during the period of construction. A further considerable air loss was located at the level of intersection between walls and ceiling, which corresponded to the level of concentrated leakages reported be- fore commission. About 5% of the leakage apparently occurred near the top of the ceiling. During the winter the snow covered ground surface in the area was examined thoroughly to detect the escape of air. The inspection was done on foot and by helicopter equipped with a thermo sensitive camera. W hat was considered to be the main outflow of air was found as far as 600 m away from the chamber even though the chamber is only 140 m below ground.

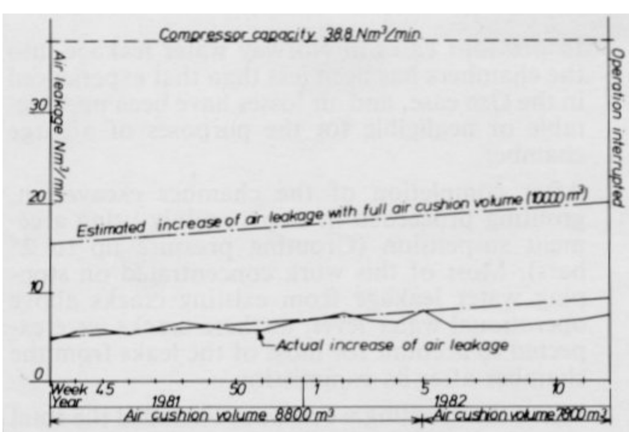


Fig. 3. Osa Hydroelectric project. Air cushion surge chamber. Air leakage after commission.

Different sealing methods were studied.

To continue the grouting from inside the chamber was considered the most reasonable approach. In order to avoid the large leakage near the bottom of the chamber, enlargement of the chamber above this level was also considered to enable it to be operated with a higher water level without restricting the power station operation. A possible extension of the chamber is shown in Fig. 4. Different kinds of air tight membranes were also considered but on account of the installation costs, a membrane was regarded as a last resort, only to be used if vertical splitting appeared to be the cause of the leakage near the ceiling, or if grouting proved unsuccessful.

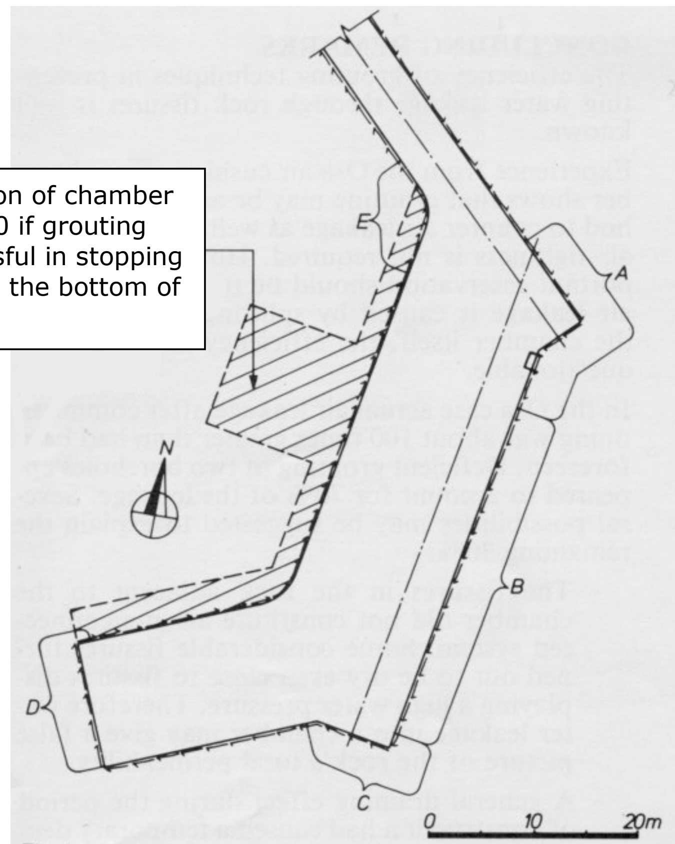


Fig. 4. Air cushion chamber. Plan. Grouting programme included parts of walls marked A – E.

EXECUTION OF THE SEALING WORK

The Osensjøen reservoir reached its lo west level on March 20th 1982 and operation of the power station was then interrupted.

The air cushion was released through the air supply conduits and the whole surge chamber was subjected to full water pressure for about two weeks. The headrace tunnel was emptied and on April 14th the chamber was inspected.

A number of concentrated spouts of water were found along the headrace tunnel between the power station and the air cushion chamber. These corresponded to water discharges found on the ground surface and it was decided to carry out grouting to reduce water leakage from the head- race tunnel, in addition to the sealing work in the surge chamber.

The cause of the major air leak near the bottom of the cushion was easily detected as air was now blowing back into the chamber through two boreholes at this level in the north-eastern corner of the

chamber, where veins of water had been traced close to the surface of the chamber wall. Apparently the greatest part of the air leakage was due to deficient grouting of these boreholes.
The water leakage otherwise showed the same pattern as before the power station had been commissioned, although the total amount of water leakage into the chamber was larger than at the end of the period of construction. However , no sign of splitting caused by air pressure could be detected on the chamber surface.
The conclusion from this inspection was that the sealing programme in the chamber should be based exclusively on grouting.
The chamber walls were divided into sections represented in Fig. 4 by the letters A, B, C, D and E. The sections were given priority according to their estimated share of the air leakage. Section A and the upper part of section B were given first priority. The rest of section B, section C and section E were given second priority, while section D and the ceiling were given lowest priority.
Farts of walls not represented by a letter showed no detectable water leakage and grouting was considered unnecessary there.
Tests showed that the intrusion of cement suspension increased significantly when the grouting pressure exceeded 35 bars. Therefore it was decided to set the upper limit for grouting pressure at 40 bars. It was also decided that the cement grouting barrier created during the construction period should be extended to a greater depth. 15 m long boreholes were drilled in a 1.5 m x 1.5 m pattern. In all areas with cement intrusion, additional holes were drilled in between and grouting continued until the new control holes proved tight.
This was carried out in sections A, B, C and D. Section E showed negligible water loss and the grouting in this area was quickly concluded. In areas which still showed water drainage from fine cracks, shallow grouting was carried out using the chemical solution Siprogel. This was the case in sections A and D.
During the grouting of the walls, water leakage from the ceiling increased. The work concluded by grouting points of leakage in the ceiling with *Siprogel*.
All the boreholes, water loss tests and grouting were carefully recorded through the whole pro- cess to enable a complete check to be made at the end.
When the work was completed in the chamber on June 30th, 6800 m of borehole had been drilled and 36 t cement and 5500 l *Siprogel* had been grouted.
Work was carried out during the same period to reduce water loss from the headrace tunnel, where a different approach had to be employed. Before a cement grouting could be done, each concentrated spout of water had to be plugged to prevent the cement suspension from being washed back into the tunnel, The rapidly, expanding chemical agent *Taccs 020* and the grouting compound *Mauring* were used with success. The latter proved conspicuously efficient in stopping even the most powerful spouts.

PERFORMANCE AFTER GROUTING

Refilling the headrace tunnel started on July 7th. For about three weeks the chamber was subjected to water pressure only, before the air cushion was established.
Shortly after the power station was back in operation the measured air loss was 1.7 Nm^3/min.
During the following year a slow but steady and encouraging decrease in air leakage was observed, which indicates that water pressure is building up in the paths of leakage. A permanent leakage rate of 1.2 Nm^3/min or 3% of installed compressor capacity seems to be established. This is regarded as satisfactory with respect to operational demands and energy consumption.
The total water loss out of the headrace tunnel has been reduced from about 50 l/sec. to approximately 17 l/sec. The corresponding increase in annual power output is nearly 500,000 kWh.

COST BENEFIT
The air loss from the surge chamber involved the following aspects of financial interest:

- Cost of reducing air loss by sealing work
- Energy consumption of compressors
- Maintenance cost (wear and tear) of the compressors
- Operational safety

Cost of seating work
During the period of construction NOK 500,000 were spent on grouting the surge chamber .
The total cost of sealing work within the surge chamber during the summer 1982 amounted to NOK 4,700,000; (Rate of exchange July 1982, 1 $ = 6,25 NOK)

Energy consumption
The compressor installation consists of two units, each with a capacity of 19.4 Nm^3/min, powered by 180 kw electric motors.
When the operation of the power station was interrupted in March 1982, the air loss rate from a full air cushion (10,000 m^3) was estimated to be about 20 Nm^3/min. The corresponding annual energy consumption of the compressors maintaining a full cushion chamber would have been about 1,620,000 kWh. The sealing work has reduced the air loss to 1.2 Nm^3/min. which means an annual energy saving of 1,530,000 kWh.
The value of this annual energy saving in 1982 was NOK 239000 (NOK 0.157 pr. kWh).
Neglecting the general rise in energy prices, and regarding a period of 40 years and 7% discount rate which is often used in hydroelectric development in Norway, the net present value of this energy saving will be about NOK 3 300 000.

Maintenance cost (wear and tear) of compressors.
At the Osa project it was decided to have a permanently installed compressor capacity capable of filling up the air cushion in a reasonable time after inspections of the headrace tunnel. That meant the installed capacity had to be far greater than that needed merely to compensate an acceptable air loss during operation of the plant.
Except for the wear and tear of a few spare parts, maintenance cost and life expectancy of the equipment are not much influenced by a decrease in running hours.
Accordingly a reduction in air loss is considered to be of little consequence for depreciation, interest and maintenance costs of the compressors, and almost negligible compared to the value of energy saving.

Operational safety
The value of operational safety might be calculated by an analysis based on statistics involving the probability and cost of different kinds of disruptions and accidents in the power production and transmission systems. In the present case, however, there was no need for a complicated and extensive analysis to arrive at the conclusion that the operational safety was unsatisfactory. The amount of air loss was too great compared with the installed compressor capacity, and the leakage was increasing. The potential for energy savings appeared so convincing that it was decided to

carry out as extensive a work programme as possible to reduce air leakage during a shut-down of acceptable duration. The actual cost of the increased operational safety can be obtained as follows:

Total cost of sealing the surge chamber	NOK 4 700 000
Net present value of energy saving for air compressors	NOK 3 300 000
Cost of increased operational safety	NOK 1 400 000

Reduced water loss from the headrace tunnel

The estimated increase in energy output due to reduced water loss from the headrace tunnel is 500,000 kWh annually.

Based on the same assumptions as mentioned before, the net present value of this energy saving will be NOK 1 100 000.

The total cost of the grouting work in the headrace tunnel was NOK 900 000.

CONCLUDING REMARKS

The efficiency of grouting techniques in preventing water leakage through rock fissures is well known.

Experience from the Osa air cushion surge chamber shows that grouting may be an effective method to counter air leakage as well, provided total air tightness is not required. However, one important reservation should be recognized. If the air leakage is caused by splitting originating in the chamber itself, the efficiency of grouting is questionable.

In the Osa case actual air leakage after commissioning was about 100 times greater than had been foreseen. Deficient grouting of two boreholes appeared to account for 70% of the leakage. Several possibilities may be suggested to explain the remaining 30% :

- The fissures in the rock adjacent to the chamber did not constitute an interconnected system. Some considerable fissures turned out to be dry even close to fissures displaying a high water pressure. Therefore water leakage into a chamber may give a false picture of the rock's total permeability.
- A general draining effect during the period of construction had caused a temporary decrease in water seepage.
- Splitting may have occurred at levels above the chamber without being traceable within the chamber itself .
- Uncertainties in assumptions concerning the width and nature of fissures.

The permeability of air in fissures is anticipated to be 30-1000 times higher than the permeability of water, depending on the width of the crack. Accurate performance and control of grouting is therefore vital. An omission or deficiency of little consequence to water leakage may considerably effect the air leakage.

In the Osa case we believe that pregrouting with a grouting pressure of 40 bars instead of 25 bars used during construction might have prevented the problem.

The accurate observation and description of all cracks and water drainage after every round of blasting needs to be an established routine during excavation. Additional grouting should be carried out as soon as possible after the excavation is completed, to maintain the ground water level above the chamber and thus minimise air leakage.

OPERATIONAL EXPERIENCE WITH THE AIR CUSHION SURGE CHAMBER AT KVILLDAL POWER STATION

AIR LEAKAGE CONTROL BY A PRESSURIZED "WATER UMBRELLA"

A. Pleym and O. Stokkebø
Senior Engineers Norwegian Water Resources and Electricity Board
Oslo, Norway.

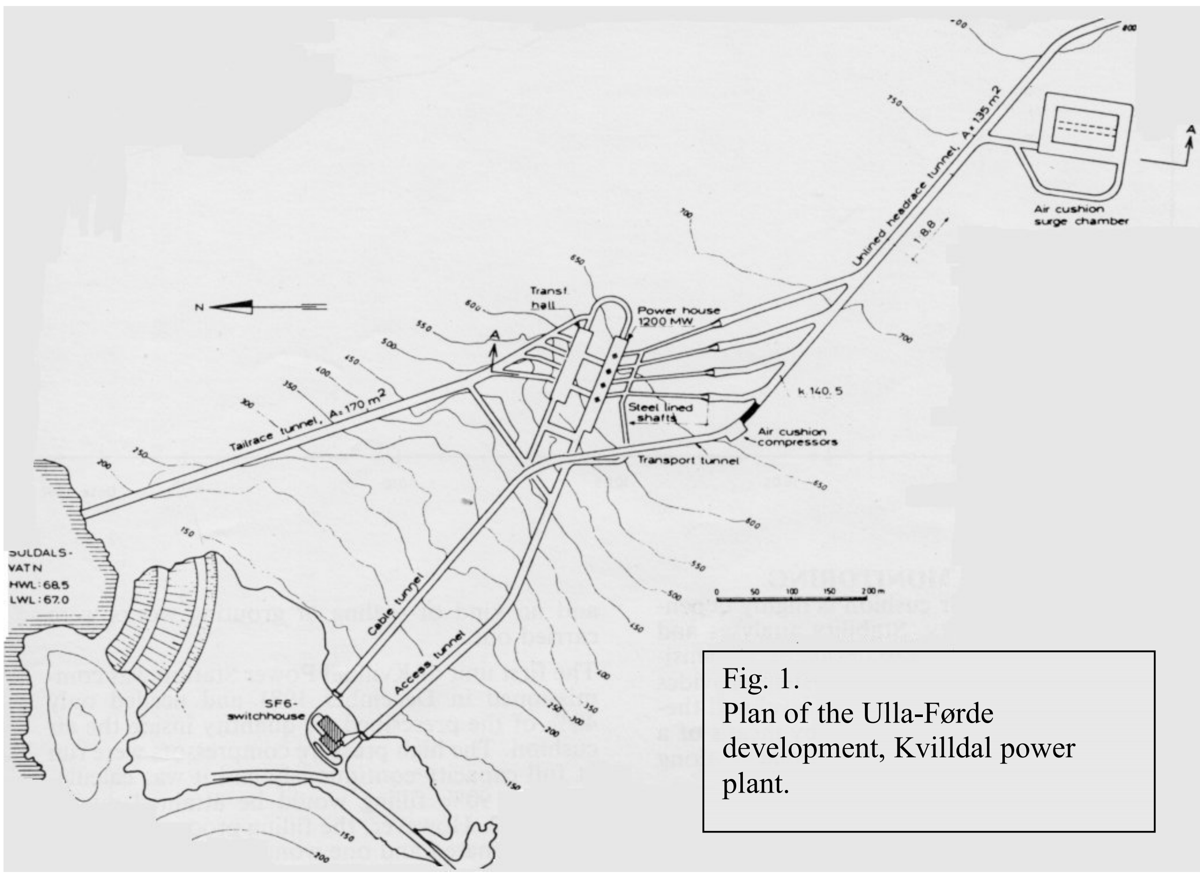


Fig. 1.
Plan of the Ulla-Førde development, Kvilldal power plant.

1. INTRODUCTION

Kvilldal Power Station and the air cushion surge chamber have already been described. References (2), (3) and (6). (see Fig. l). This article will report on the operation experiences up to the present date and on some works executed to reduce air leakage.

2 FILLING AIR INTO THE AIR-CUSHION

The air cushion chamber has a total volume of 120 000 m^3 but only 75 000 -95 000 m^3 is occupied by air during operation. Air pressure varies therefore between approx. 4,2 and 3,4 Mpa absolute pressure and thus approx. 3, 1 mill. m^3 free air must be pumped into the chamber in addition to the air already enclosed. For this purpose two mobile 3 Mpa and three stationary 6 Mpa high- pressure compressors are used. The latter are for compensating air leakage during regular operation of the air chamber. Total capacity is 2400 and 500 m^3 free air, respectively. By keeping water level in the headrace tunnel sufficiently low during filling, the whole air quantity can be filled by using all the compressors. In spite of their considerable capacity it takes approximately 45 days to complete a filling operation. Some unforeseen problems have occurred in the headrace tunnel and adjacent installation and the air cushion has therefore been filled three times, thereby demonstrating that it is very important for an economical running of the power station to reduce the filling time. The operation staff accepted some restrictions on the governing capability of the plant and put the air cushion into operation at only 40% of the design air volume. The remaining 60% is filled in during operation by the high-pressure compressors. Users of air cushion chambers should ensure that compressor capacity is sufficient for this task.

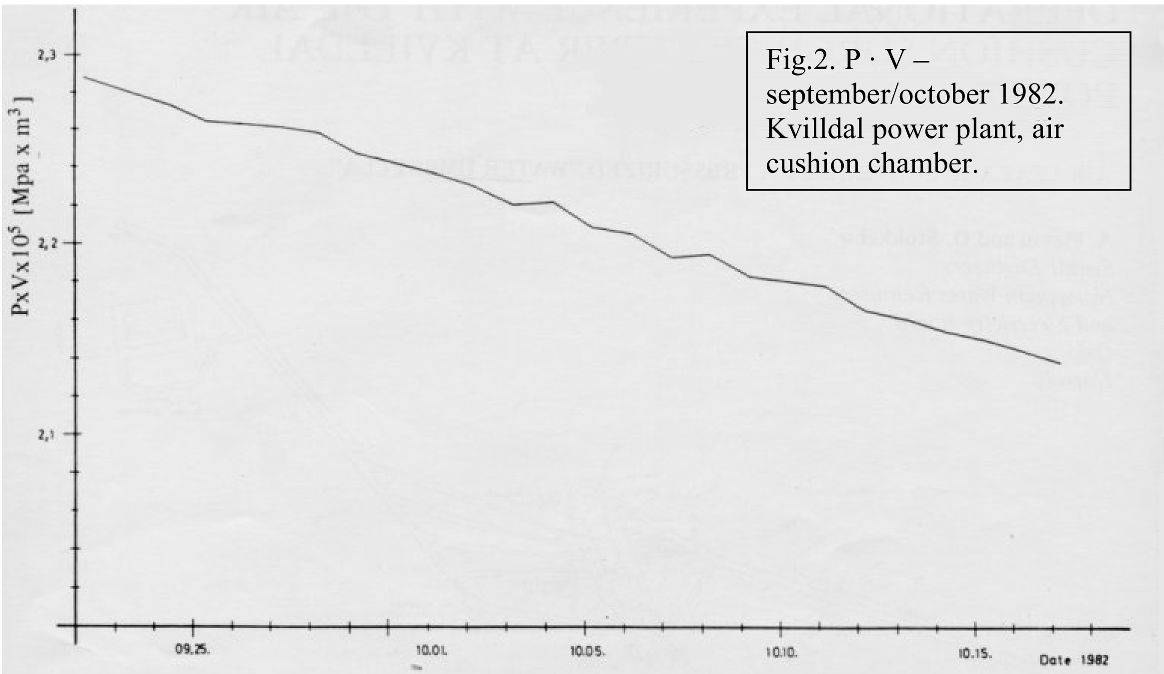


Fig.2. P · V – september/october 1982. Kvilldal power plant, air cushion chamber.

3 CONTROL AND MONITORING

The efficiency of an air cushion is highly dependant on the air quantity. Stability analyses and surge calculations are used to decide the dimensions and an extensive monitoring system provides a continuous check (6). The water level and thereby the air volume are measured by means of a metal column with electric elements placed along it at 5 cm intervals.
In addition the water level and the air pressure are registered by means of two pressure meters using oscillating crystals.
Experience has shown that it is essential to have two independent control systems, especially in the running-in period, because of various instrument problems.

4. AIR LEAKAGE

With a high pressure air cushion chamber like the one put into use at Kvilldal Power Plant, it is essential to investigate the surrounding rock with regard to air leakage.
If leakage is in excess of the compressor capacity, the air cushion will not function and the power station must be shut down or run with very limited governing capability until air leakage is sufficiently reduced. At Kvilldal the air cushion chamber is situated in a moderately fissured gneiss

of Precambrian age and was sited after careful geological investigations combined with extensive exploratory core drillings from the head-race tunnel. The predicted air leakage of approximately 60 m^3 per hour was considered tolerable and no kind of sealing or grouting works were carried out.
The first unit of Kvilldal Power Station was commissioned in December 1981 and needed only 40% of the prescribed air quantity inside the air cushion. The high pressure compressors were run at full capacity continuously and it was calculated that 90% filling would be attained during May 1982. However, the filling proceeded slower than estimated and one wondered why. A rough estimate based on the gross capacity of the compressors indicated a leakage of 200-250 m^3 free air per hour, that is 3 to 4 times more than calculated. In spite of this the operation of the power plant was not impeded. At that time it had been decided to take the power station out of operation and empty the tunnel anyhow, during January 1983 after the reservoirs had been drawn down. W hen the compressors were stopped in September 1982, it was possible to improve the rough estimate of the air leakage. As indicated in Fig. 2 the reduction of the product P · V was very stable for a period of 25 days, showing an air leakage of 250 m^3 free air per hour .
At the same time the matter was under careful consideration and the important question was whether any work should be carried out to reduce the air leakage. The leakage was certainly not serious but one felt uneasy about the possibility of the leakage increasing in the future and realised that a shut-down of the power station once all units were in operation and the whole scheme completed would cause a considerable loss of water and output
One also estimated that if the power consumption of the compressors were reduced to zero, that alone would justify an investment of 1,5 million NOK in sealing work. It was therefore decided to proceed with plans to minimise the leakage. Three alternatives were considered:

1. Chemical grouting
2. Steel fibre shotcrete
3. Inject water with high pressure above

and around the chamber, like an «umbrella» After some consideration and discussion one decided to choose the «water umbrella», see Fig. 3, and to keep water pressure 1 Mpa above the air pressure in the cushion.
With the air chamber still at full pressure the mountainside above the surge chamber was examined minutely, but no concentrated leakages could be discovered. One also considered to take photographs of the terrain using a very sensitive infrared camera, but the temperature was too high during the available period.

In January 1983 after the air chamber and the tunnel had been emptied, some interesting observations were made:
Air enclosed in the rock was leaking out of fissures and joints which pass through the chamber and the access tunnel, Fig. 1. Now that the cause of the leakage was clear, the borehole pattern could be decided. Planning and construction work had to follow a tight schedule.
The power station had to be on line before the thaw in May to avoid considerable wastage of water. The job included more than 2500 m of bore holes, approximately 500 m of steel pipe fastened at 7,5 m's length in the bore holes by cement mortar. The whole system of pipelines is connected to a feed pipe embedded in concrete from the compressor chamber where the high pressure pump is placed. This pump has a maximum gross capacity of 120 l per minute and is equipped with a gear to make it possible to adjust the rate of flow and hence the pressure to the real consumption of water in the «umbrella». Later a back up pump with the same capacity was installed.
The whole job was completed on schedule and the «water umbrella» was put under pressure before the air-filling started in order to fill all fissures and cracks with water under pressure to prevent air leakage.

The power plant was once again put into operation in May 1983 and very soon the solution was seen to be successful. With a mean consumption of approximately 32 litres per minute in the «water umbrella» the leakage of air is reduced to almost zero.

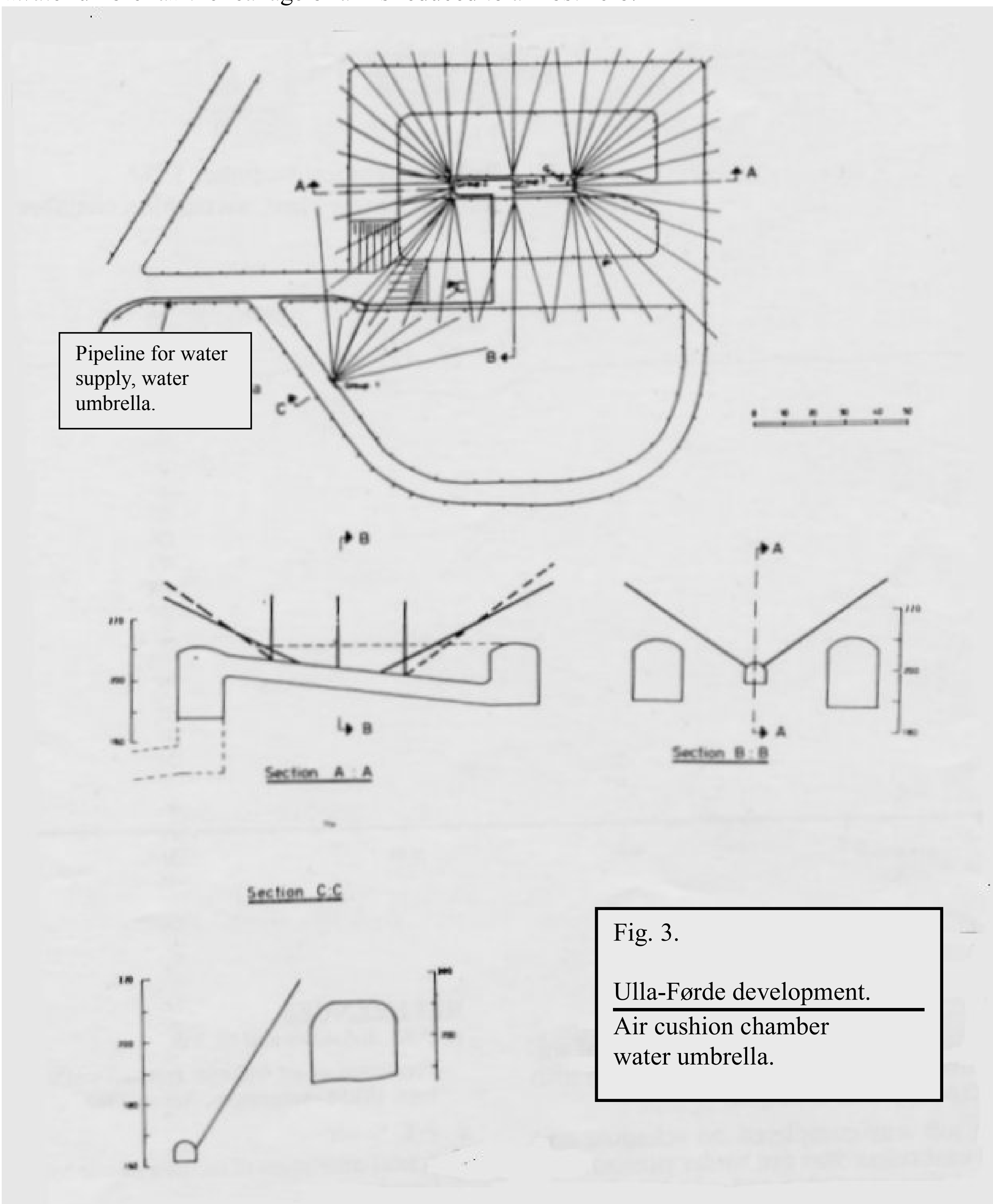


Fig. 3.

Ulla-Førde development.

Air cushion chamber water umbrella.

Figure 4 shows how the P · V-product changed during a period when the compressors were stopped.

The pumps have to be run continuously to keep the «water umbrella» in operation. W hen the tunnel and the air cushion have to be emptied next time, some experiments are planted to find out how long the pump can be stopped without reducing the effect of the «water umbrella»,

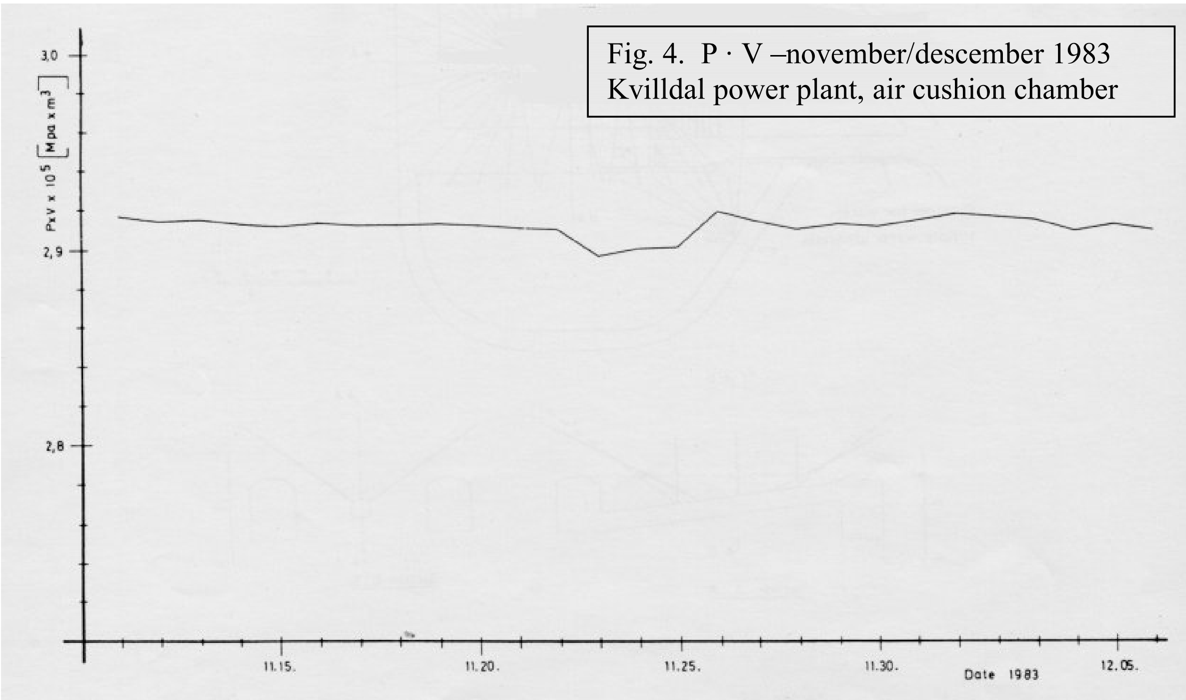


Fig. 4. P · V –november/descember 1983
Kvilldal power plant, air cushion chamber

REFERENCES

1. P.M. Johansen and G. Vik:
Prediction of air leakages from air-cushion surge chambers. lSRM-symposium, Aachen 1982.

2 .P.T. Smith:
A brief description of the Ulla-Førre hydropower development.
Norwegian hard rock tunnelling, publication no. 1, 1982. 3 .O. Stokkebø and E. Tøndevold: Designed for year-round efficiency at Ulla-Førre. Water Power & Dam Construction, August 1978.

4 .0. Stokkebø:
Jukla pumped-storage project in Norway.
Symposium on Hydro-electric pump storage schemes, Athens, November 1972.

5 .R. Swee:
Surge chamber with an enclosed, compressed air- cushion. International conferences on pressure surges, Sept. 1972.

6 .S. Walbø:
Air-cushion surge chamber at Kvilldal Power Plant. Norwegian hard rock tunnelling, publication No. 1, 1982.

FUNCTIONING OF THE AIR CUSHION AT OKSLA POWER STATION

Marvin Aarvoll
Ingeniør Chr. F. Grøner A.S. Sandvika (Oslo)

Oksla Hydroelectric Power Plant
The Oksla scheme, exploiting the lower portion of the river Tysso near Tyssedal in Hardanger , incorporates a gross head of 464 m between Ringedalsvatn reservoir and the sea at Sørfjorden. The project was developed in the period 1975- 1980 by the State Power Board of Norway, with consultants Ingeniør Chr. P. Grøner A.S., Sandvika (Oslo), and Norwegian Geotechnical Institute, Oslo, on civil and geotechnical works, respectively.*)
The 34.5/37.7 m^2 headrace pressure tunnel, 4,125 m long and sloping at 1 :8.7, is unlined, except for 275 m of steel lined conduit, 3.4/2.9 m in diameter, at the approach to the underground power station. An 18,000 m^3 unlined compressed air surge chamber is located in conjunction with the sand trap, immediately upstream of the steel lined tunnel section. The 36 m^2 tailrace tunnel, 400 m long, discharges into Sørfjorden 8 m below sea level. The machine hall houses one 206 MW vertical
Francis turbine, generating 710 GWh annually at a maximum flow rate of 53.5 m^3/s.
The headrace, tailrace and surge chamber are de- signed to incorporate a future 100 MW increase in generating capacity, entailing a potential flow rate increase to 80 m^3/s.

Surge Chamber
Chamber location and configuration.
The unlined compressed air surge chamber, 76,5 m long, with average cross-section and total volume of 235 m^2 and 18,000 m^3, respectively, is located near the sand trap, some 350 m from the power station.
The chamber is connected with the headrace through a 60 m long tunnel, cross-section 18.3 m2, sloping towards the elevated chamber at a 1 :8 gradient. The chamber-invert slopes longitudinally at 1 :25 (uphill) in a length of 20 m, changing to 1 :50 for the remaining 56.5 m. The roof portion of the rock cavern was excavated horizontally, with a cross-sectional radius of 13 m.

Geological Conditions
The bedrock is a massive granitic gneiss with a typical joint spacing of 1-3 m. There are two main joint sets with strikes aligned at right angles to each other, N-S and E- W, respectively.
The most pronounced weakness zones are oriented N-S or a little towards NNW-SSE, usually dipping westerly at $40\text{-}100^g$, Some of these zones contain clay, but only rarely do they have a width of 0.5 m or more. Occasional joint planes occur parallel to the valley side surface. The E- W oriented joints are considered to be the oldest, and they are generally free of clay.
The air cushion chamber is located in an area of good rock quality without any major joints. However, both downstream and upstream from the chamber, there are conspicuous joint zones, as shown in Fig. 1 and 2.
Downstream there are two joint zones in particular which are significant, striking N-S and sipping westerly at $40\text{-}60^g$, Both are clay-bearing and about 0.5 m wide. One of them may correspond to a

(* Refer to article: «Tunnel break through beneath 85 m water at Ringedalsvatn), by Civil Engineer Øystein H. Stormo, Ingeniør Chr . F. Grøner A.S, «Norwegian Tunnelling Technology, Publication No. 2», Tapir Publishers 1983.)

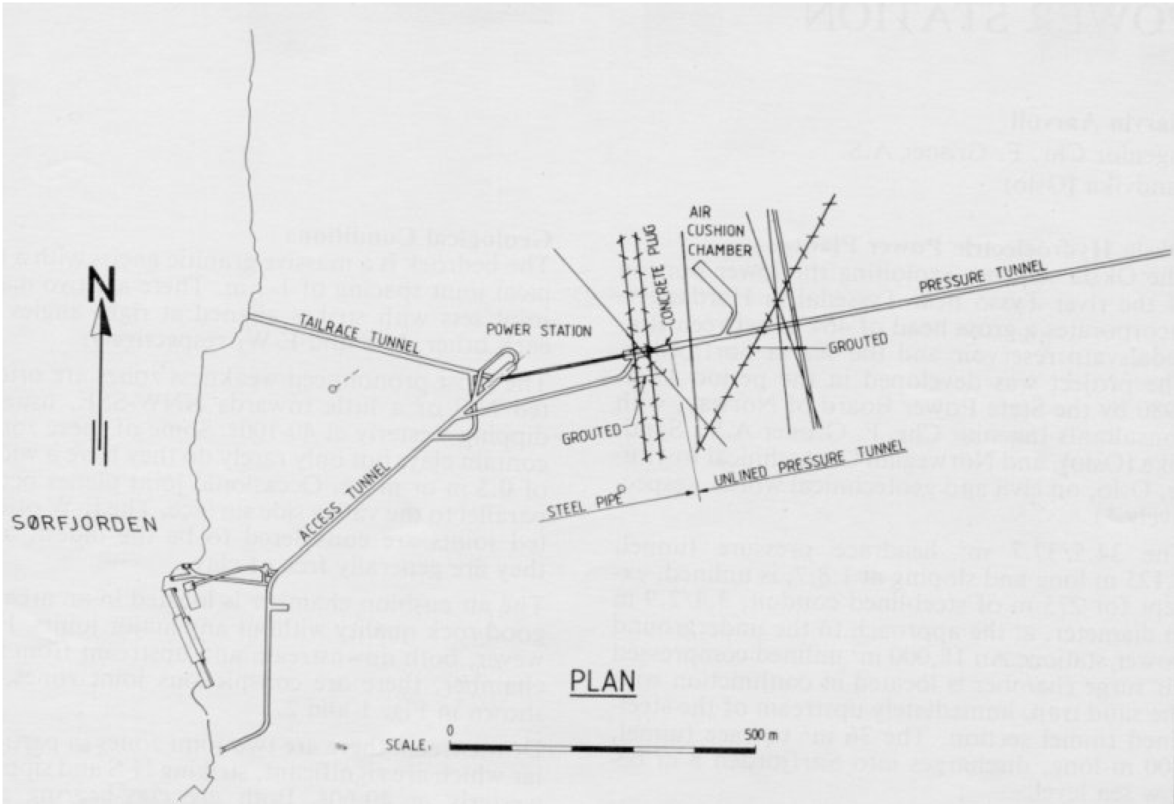


Figure 1.

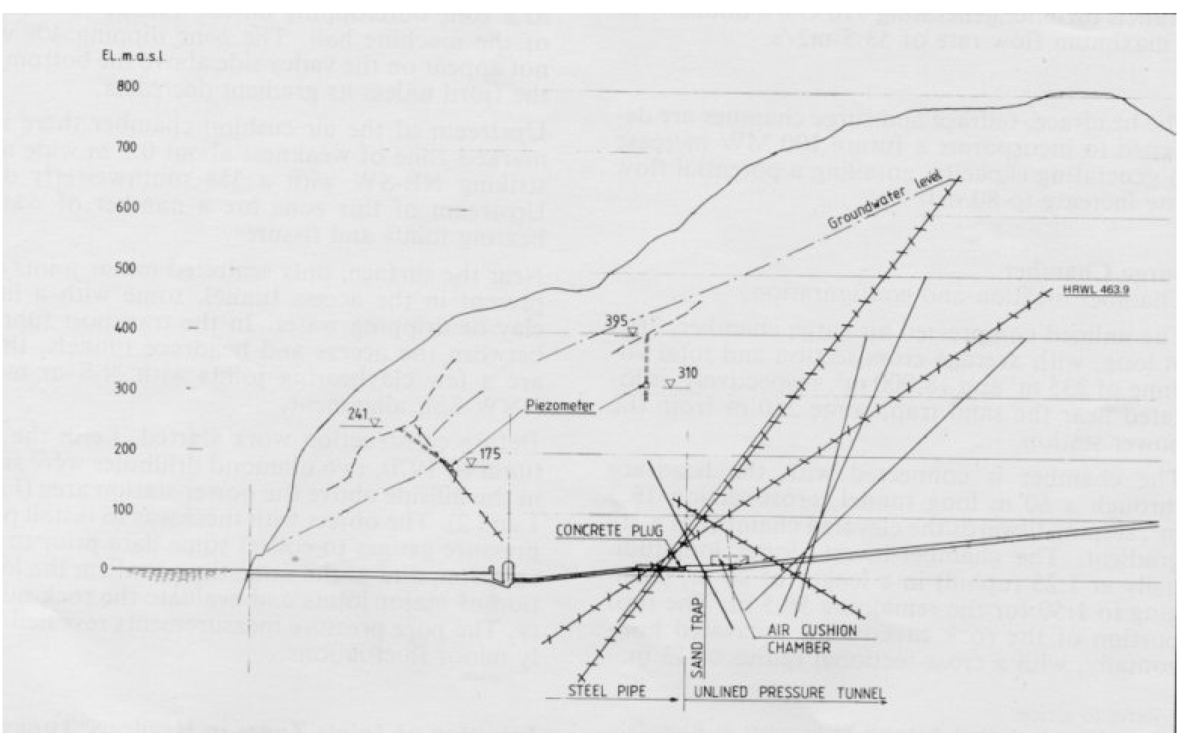


Figure 2.

zone outcropping on the hillside 700 m NE of the machine hall. The zone dipping 40^g will not appear on the valley side above the bottom of the fjord unless its gradient decreases. Upstream of the air cushion chamber there is a marked zone of weakness about 0.5 m wide and striking NE-SW with a 35^g south-westerly dip. Upstream of this zone are a number of water-bearing joints and fissures. Near the surface, only scattered minor joints are present in the access tunnel, some with a little clay or dripping water. In the transport tunnel, between the access and headrace tunnels, there are a few clay bearing joints with N-S or more NNW -SSE alignment.
Before construction work started, i.e. in the autumn of 1970, two diamond drill holes were sunk in the hillside above the power station area (Figs. 1 and 2). The object with these was to install pore pressure gauges to collect some data prior to excavation, and at the same time confirm the location of major joints and evaluate the rock quality. The pore pressure measurements revealed only minor fluctuations.

Injection of Joints Zones in Headrace Tunnel
The two reaches with joint zones downstream and upstream of the air cushion chamber were injected using cement.
The zones downstream of the air cushion chamber cross the headrace pipe tunnel downstream of the concrete plug. The area surrounding the plug was injected in the usual manner and the weakness zones stabilised by means of drained shotcrete.
The joint zones upstream were injected in order to maintain/increase the pore water pressure in the area surrounding the air cushion chamber .

Assessment of Rock Stability and Permeability
Investigation and testing to detect potential stability and air leakage problems were conducted prior to, and during construction of the chamber .
In assessing the cavern stability, it was decided early on that systematic rock support would not be required. A decision regarding grouting, on the other hand, called for detailed evaluation of economical and operational considerations.
The anticipated rate of air loss as a result of leakage through the rock mass was based upon:

1. accurate recording of pore water seepage through the excavated roof section of the cavern,
2. permeability tests conducted in four boreholes drilled at the chamber site, one of these was diamond core borehole.

The diamond core borehole was drilled parallel the chamber axis prior to excavation, whereas the next two, and the remaining one, were drilled from within the chamber, and from the tunnel into the chamber area, respectively.
Based upon evaluation of recorded data, potential air leakage from the surge chamber according to methods 1 and 2 above was computed as follows:

1. 100-200 normal litres per minute (nl/min)
2. 150-1500 nl/min

Method 1 was judged to provide the better basis for evaluating the total rate of air loss from the chamber during operation, and consequently a leakage value near the lower end of the range 100-1500 nl/min was selected for use in assessing the overall feasibility of grout-sealing the chamber.
A grouting programme (epoxy) covering there faults, each 50 m in length, and one quartz zone extending for about 40 m, was devised as a basis for assessing the costs and benefits of improving the natural ability of the chamber to contain air predetermined pressures. Calculations indicated that a 50 % reduction in air leakage rates (500 to 250 nl/min) could be achieved by carrying out the

grouting programme. By comparing total programme costs (NOK 240 000) with the net present value of annual operational savings (NOK 75 000), however, it was apparent that the grouting programme could only be labelled uneconomical.
Although the estimated reduction in operating time of pressurizing equipment from 20 to 10 min/hr was considered significant, the decision was made to hold off on any grouting until the available design data could be verified through full scale operation of the power plant. Significant storage capacity and plant operation at full capacity limited to 3 450 hours annually, facilitating execution of subsequent grouting works if required, were additional factors in support of this decision.

Operational Data
The Oksla Hydropower Project was put into operation during the winter of 1980.
In roughly five years of successfully operating the unlined compressed air surge chamber, total air losses of less than 100 nl/min have typically been recorded. The experience so far shows that the results of the preliminary investigations provided a reliable basis for evaluating actual leakage of compressed air from the chamber, and for asses- sing the need for grout-sealing the chamber to reduce air losses.
The compressor plant, consisting of three 37 kW units, each with a capacity of 98 m^3/hr, was at first operating automatically, with water stage sensors located inside the chamber providing the required signals. As a result of low leakage rates and infrequent operation (4 to 5 times per year) to maintain the required air cushion, however , the compressor plant is currently manually operated.

MODEL TESTING OF UNDERWATER PIERCING OF A TUNNEL

J .A. Sandvik
The Norwegian Hydrodynamic Laboratories, Norway

Summary
Underwater tunnel piercing is frequently necessary during the final phase of a hydro power project. There are two main alternatives for the piercing of underwater tunnels in lakes.

Alt. 1 The plug which is to be removed connects to an open tunnel where atmospheric pressure in maintained. for instance via a vertical shaft as shown in Fig. l.
Alt. 2 The tunnel behind the blasting plug is closed. for example with a closed gate as shown in Fig. 2.

The situation in alternative 1 can in some simple cases be analysed theoretically or tested in a physical model based on well known techniques. On the other hand. underwater tunnel breakthrough against a closed gate represents a more complex problem. A research project at the River and Harbour Laboratory has helped to develop a special technique to test such a situation in the laboratory.
Models of this type have been used in connection with the planning of some projects in Norway. The tests have solved the following two main hydraulic problems:

- finding the pressure and the load gate constructions.
- predicting the location of the rock which will be jarred loose as a result of the blasting.

In the following I will describe the problems and the special model technique we have developed.

1. INTRODUCTION

Underwater tunnel breakthrough is frequently necessary during the building of hydro power stations. The access tunnel is blasted towards the intake basin, where the last explosion makes a breakthrough. Such underwater tunnel piercing can be done either:

1. By blasting with a water-filled tunnel and a open surge chamber .
2 . By blasting with a dry or partly water filled, closed tunnel.

Some hazard is generally connected with such tunnel piercing. Consequently it is of great importance that efforts be made to prevent unfortunate tunnel breakthroughs. Planning of a tunnel piercing with an open gate, as shown in Fig. I, is normally rather straight forward. Planning of a breakthrough with a closed tunnel, as seen in Fig. 2, is far more complex; the problems of which I will concentrate on in this paper .
The hydraulic problems concerning the closed breakthrough are to:

- find the pressure and the loads on the gate constructions due to the water flushing into the tunnel from the intake basin,
- predict how far into the tunnel the rock and the gravel from the final blasting will be transported by the in-flowing water masses.

Breakthroughs have occasionally led to extreme pressures, from which gate construction damage has resulted. On other occasions there have been problems due to the masses being concentrated close to the gate constructions. Evidently, such unfortunate breakthroughs have considerable economical consequences, both concerning repair costs as well as loss of energy production.

For use in planning of closed tunnel breakthrough, a mathematical model has been used to some extent, estimating the pressure of the non-stationary water and rock mass inflow. In this model we have to estimate the coefficients for both concentrated and frictional losses, which have to be considered separately for each breakthrough. The lack of detailed knowledge about these coefficients gives the mathematical model a considerable uncertainty.
Consequently, we have chosen to make use of laboratory model tests during the planning of more complex, closed tunnel piercing. These tests also give the opportunity to study the problems connected with the depositions of the blasted masses in the tunnel.

2. MODEL LAWS

Underwater tunnel piercing is from a hydraulic point of view, a balance of the gravity and inertia forces. Scaled tunnel piercing models have to be modelled in accordance with the Froude model law, stating:

- The Froude number is to be equal in model and prototype.
  The Froude number is defined as

$$F = u / \sqrt{gL} ,$$

  where
  u = the current velocity (m/s)
  g = the gravity acceleration (m/s^2)
  L = characteristic length (m)
- Scaled velocity will be equal to the square root of the length scale.
- Scaled pressure will be the scaled pressure height (meter water column, absolute).
- Scaled volume will be the third power of the length scale.
- Scaled time will be the square root of the scaled length.

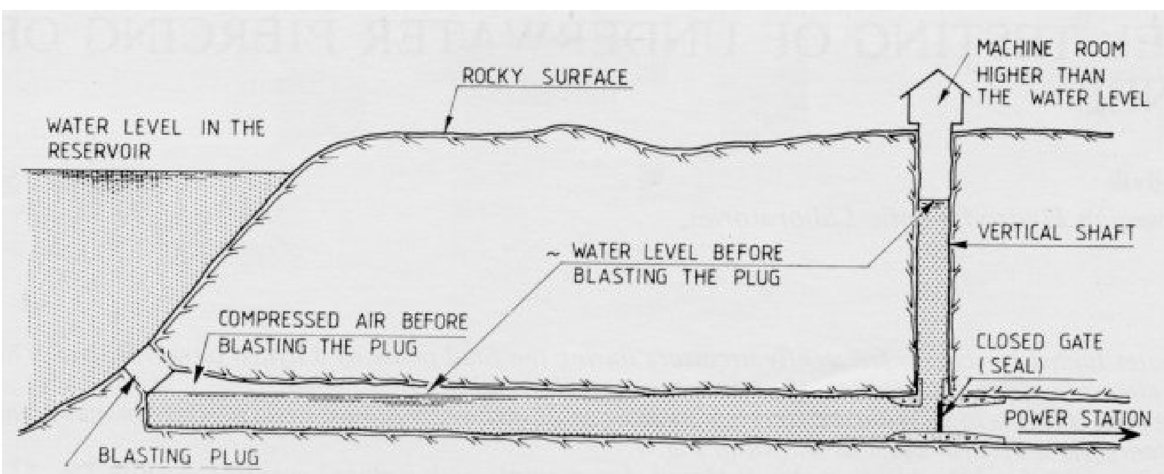


Fig 1. Underwater piercing of a tunnel with an open surge chamber.

3. THE PROBLEMS OF MODELLING CLOSED TUNNEL PIERCING

The enclosed air in the model has to be scaled so that the adiabatic pressure/volume relation is equal in model and prototype. To comply with this demand the absolute pressure should be scaled according to the length scales derived from Froude's model law. Both the air pressure in the tunnel

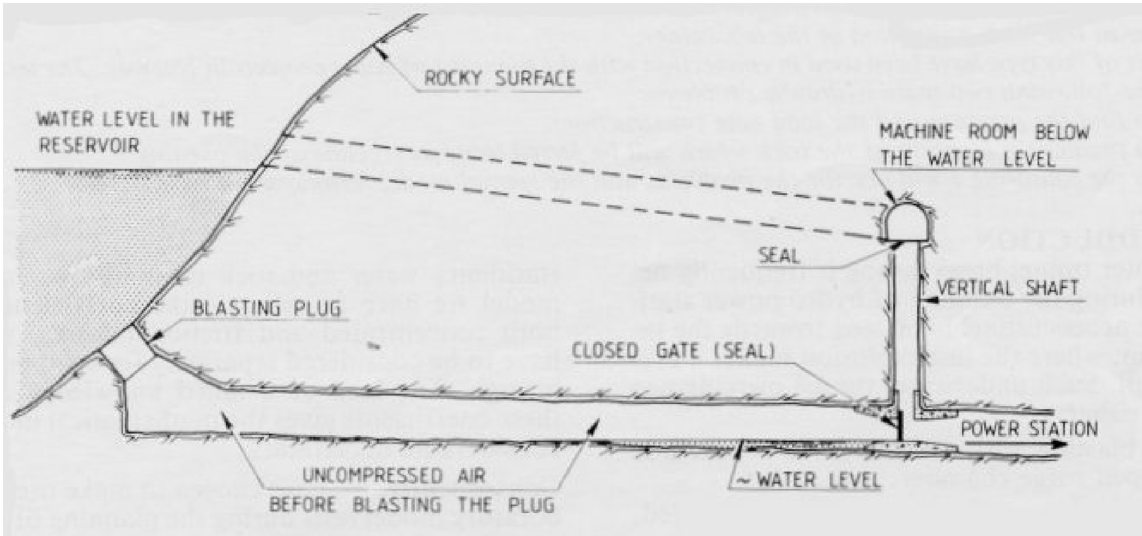


Fig. 2. Tunnel breakthrough with a closed tunnel.

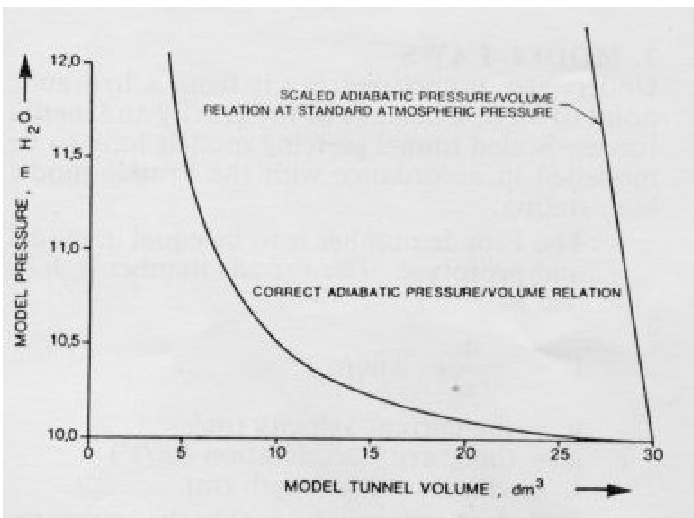


Fig. 3.
Pressure / volume relationship by direct modelling of a closed tunnel-piercing at atmospheric pressure.

and at the intake, i.e. atmospheric pressure, must be scaled. Without scaling the atmospheric pressure according to the length scale, the enclosed tunnel air would be too stiff in the model (Fig. 3). At a suitable model scale of, say 1 :70, a reduction of the atmospheric pressure by 70 is required, that is near to zero. Such model demands are associated with certain practical problems. Firstly, there will be boiling and steam pressure if the water temperature is not sufficiently low. Secondly, some difficulty is connected with finding a suitable room to accomplish the tests under these low pressures. Consequently we had to find other solutions which, without scaling the atmospheric pressure, would give the enclosed air the correct elasticity. Several ad hoc arrangements have been evaluated in this connection. In the following, three alternative solutions are described.

4. PRINCIPLE METHODS OF SOLUTION

The main problem concerning model tests of a closed piercing is, as mentioned earlier, to gain the correct elasticity of the tunnel air without scaling the atmospheric pressure. Several alternative methods of solution have been evaluated, three of which will be briefly explained here.

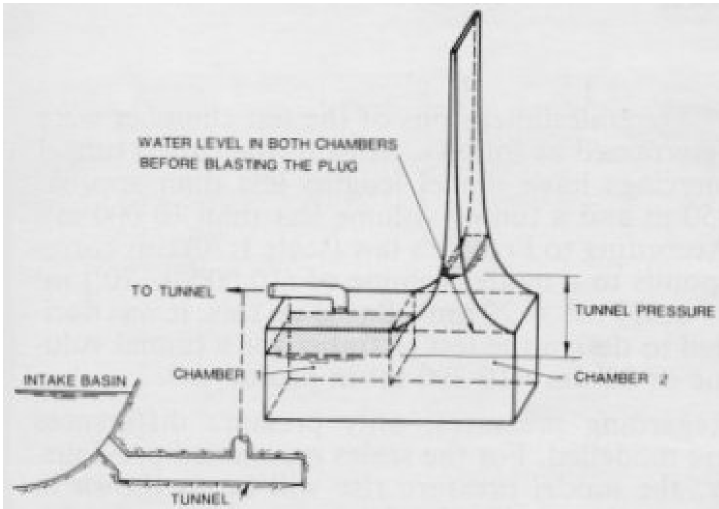


Fig. 4. At atmospheric pressure, the pressure / volume realtionship is correctly modelled by a non-linear water spring.

In Fig. 4 the method of solution makes use of a non-linear water spring to gain the correct scaling of the pressure/volume relation. W hen the water flows into the tunnel and compresses the enclosed air, the water surface will fall in chamber No. I and rise in chamber No. 2. The pressure in the tunnel will be determined by and large by the water column between levels 1 and 2. In this way, a rather accurate pressure/volume relation in the tunnel can be scaled. This solution is, however , not flexible. Any change in the tunnel air volume will result in corresponding changes of the water spring dimensions.

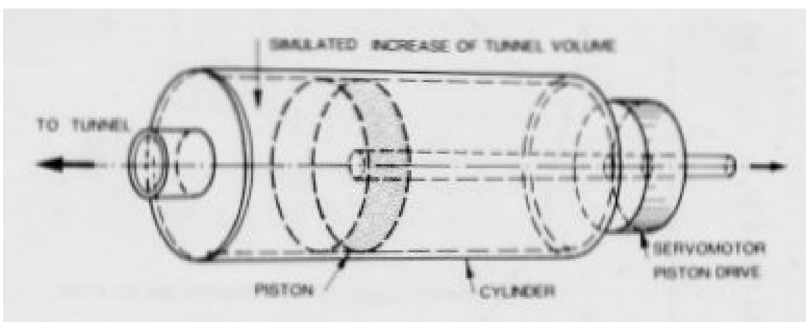


Fig. 5. Shock-absorbing piston giving a correct pressure / volume relationship in the tunnel.

The air elasticity can also be modelled by using a cylinder and as shown in Fig. 5. During the compression, the piston is drawn back and additional volume is supplied. If this additional volume is controlled by the increasing pressure, the piston movements can be programmed, gaining the desired pressure / volume relation. Fig. 5 gives a schematic view of the technique. This method requires rather complicated and ex pensive control units due to an instability caused by inertia.

In the solution of Fig. 6 the pressure/volume relation is tailored by an artificial additional volume which can be reduced during the pressure rise. W hen the water flows into the tunnel and the pressure starts to increase, an extensive air volume connected to the tunnel is compressed. As the pressure increases and there is a need

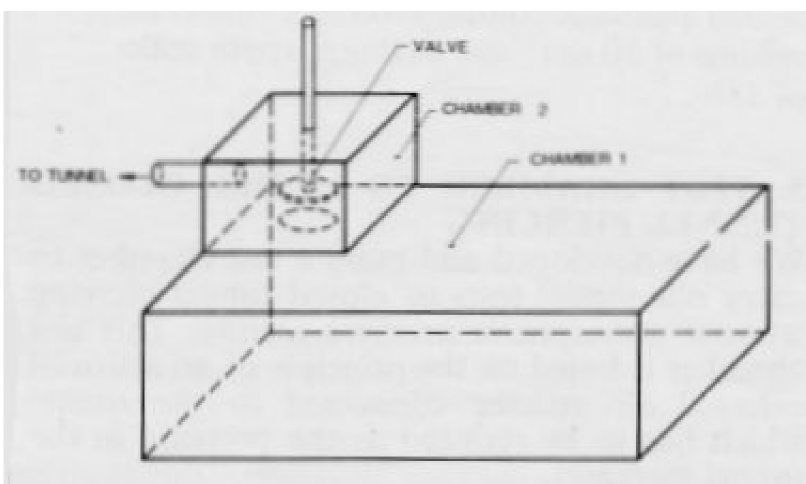


Fig. 6. "Artificial additional volume giving a correct pressure / volume relationship.

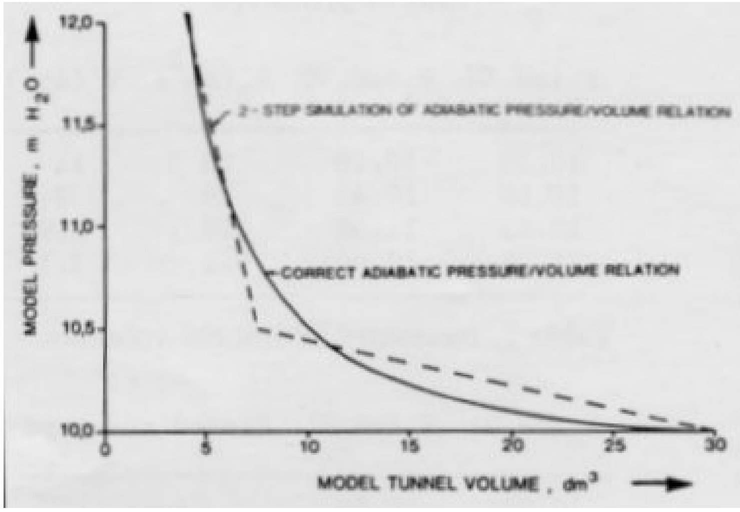


Fig. 7. A two-step reduction of the additional volume giving an approximately correct pressure / volume relationship.

for an increasing elasticity the valves close the chambers 1, 2 and 3. In this way, an approximate adjustment of the pressure/volume is possible. In Fig. 7 a two-step adjustment of pressure/volume relation is shown. A closed tunnel piercing based on this method would be rather practical. Changes in tunnel volume could easily be corrected. For example. reduction of the tunnel air volume by 10 %, would reduce the additional volumes accordingly. We have chosen this solution.

5. TEST CHAMBER TO MODEL CLOSED TUNNEL PIERCING

We have developed and build a test chamber to carry out model tests of closed tunnel piercing without atmospheric pressure scaling. This test chamber is based on the principle of an artificial external air volume connected to the tunnel which has to be reduced as the pressure in the tunnel increases.
A model scale of 1 :70 is chosen as a basis for the test chamber dimensions. The choice of the model scale is made for convenience of reproducing film shots in real time. According to Froude's model laws, the following time relation yields:

$$T_m \cdot T_p = 1{:}836m.$$

where

T_m= time in model
T_p= time in prototype

The overall dimensions of the test chamber were determined as follows. Most of the closed tunnel piercings have tunnel lengths less than approx. 350 m and a tunnel volume less than 10000 m^3. According to Froude's law (scale 1 :70) this corresponds to a model volume of (10000 + 703) m^3 = 0.029 m^3 := 29 dm^3. Based on this, it was decided to design the test chamber for a tunnel volume of 30 dm^3 (10 290 m^3 in prototype). Regarding pressures, only pressure differences are modelled. For the scales mentioned previously, the model pressure rise will be as shown in Fig. 8.

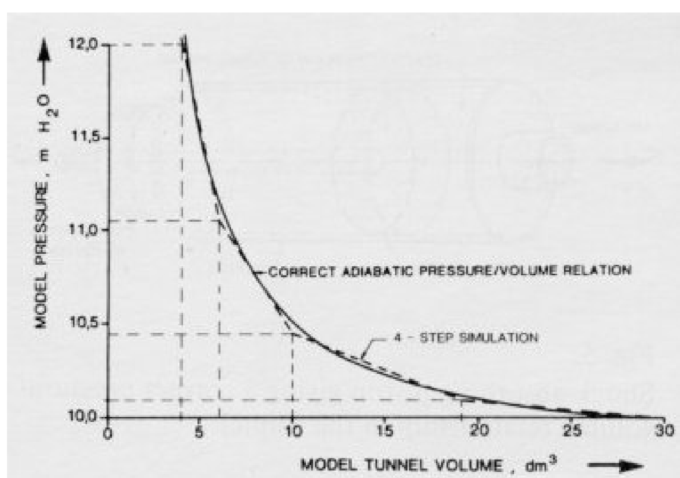


Fig. 8. Scaled adiabatic compression of a tunnel air volume of 30 dm^3 and having ascale lenght of 1:70.

A test chamber having 4 additional volumes is adequate for the purpose. As the pressure in the tunnel increases, 3 of 4 chambers will close in sequence, and hence simulate the adiabatic compression as shown in Fig. 8.
The additional volume of the test chamber can be determined by
$P_0 \cdot V_0^K = P_1 \cdot V_1^K$

Substituting $V_1 = V_0$ -V' where
V_0 = air volume at P_0
V' = volume reduction in the tunnel V_1 = compressed air volume at P_1 we obtain

$$P_0 \cdot V_0^K = P_1 \cdot (V_0 - V')^K$$

$$V_0 = V' / [\,1 - (P_0/P_1)^{(1/K)}]$$

According to Fig. 8 the additional volumes of the model can be tabulated as in table 2:

Table 1. Necessary aditional volumes.

P_0 (mH_2O)	P_1 (mH_2O)	V_T (dm^3)	V' (dm^3)	V_0 (dm^3)	V_0- V_T (dm^3)	Remarks
10,00	10,10	30	11	1553,2	1523,2	V_T = air tunnel
10,11	10,45	19	9	374,4	355,4	vol. Standard
10,45	11,05	10	3,9	92	82,0	atm. pressure:
11,05	12,00	6,1	2,1	36,7	30,6	10 mH_2O

Table 2. Table showing chamber closure, see Fig 8.

P_0 (mH_2O)	P_1 (mH_2O)	Closed chambers	V_0- V_T (dm^3)	Remarks
10,00	10,10	none	1523,2	10,00 mH_2O =
10,11	10,45	1	355,4	Standard
10,45	11,05	1 + 2	82,0	atmospheric
11,05	12,00	1 + 2 + 3	30,6	pressure

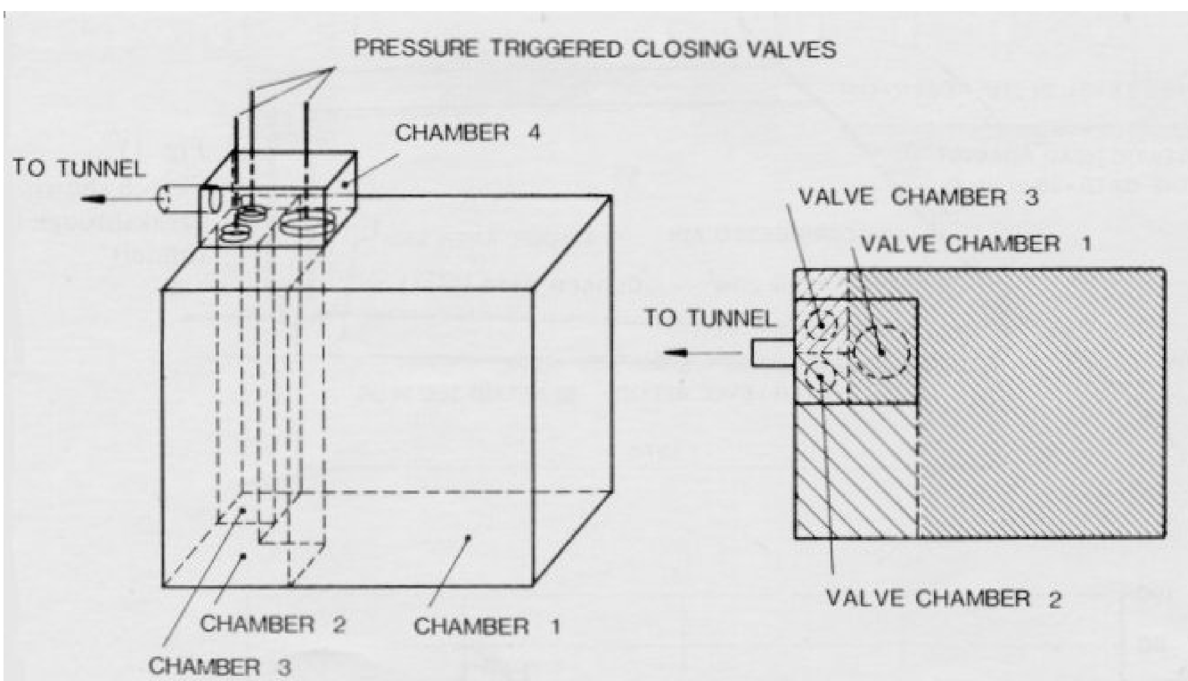


Fig. 9. Sketch of the test chamber.

The test chamber was built having 4 volume chambers as shown in Fig. 9. Three of the chambers were furnished with closing valves. The valves were fitted with springs and were kept open by electromagnets. The electric current supply to each of the valves was terminated at preset pressure values, hence reducing the additional volume as needed.
As the valves are not automatically re-opened by reductions in the tunnel pressure, the pressure / volume relationship may be erroneously modelled as soon as the first maximum pressure has been obtained. The peak pressure of the first cycle is the overall maximum pressure; -the peak pressures of the subsequent cycles are hence of minor interest as these have been damped by friction and other losses. The lack of valve reopening previously mentioned has no effect on the overall maximum pressure. Furthermore, the mass transport will primarily be determined by the conditions during the first cycle, and the subsequent cycles are of minor interest also in this respect.

Fig. 10.
Photograph showing a la year old tunnel breakthrough where the rock masses were transported 135 m and blocking the gate.

6. MODEL TESTS OF CLOSED TUNNEL PIERCING

The test chamber has been used to model several rather complicated tunnel ruptures. The main aims of these tests have been to

- minimize the maximum pressure at the gate constructions
- minimize the transport length of blasted rock masses.

In Fig. 10, a 10 year old photograph shows the damages that may arise when the blasted rock masses are transported all the way to the gate. Today's knowledge and model techniques should make it possible to avoid such situations.
All the tests executed in the laboratory will not be described in this paper. The results from one experiment shall, however, be compared with prototype measurements.

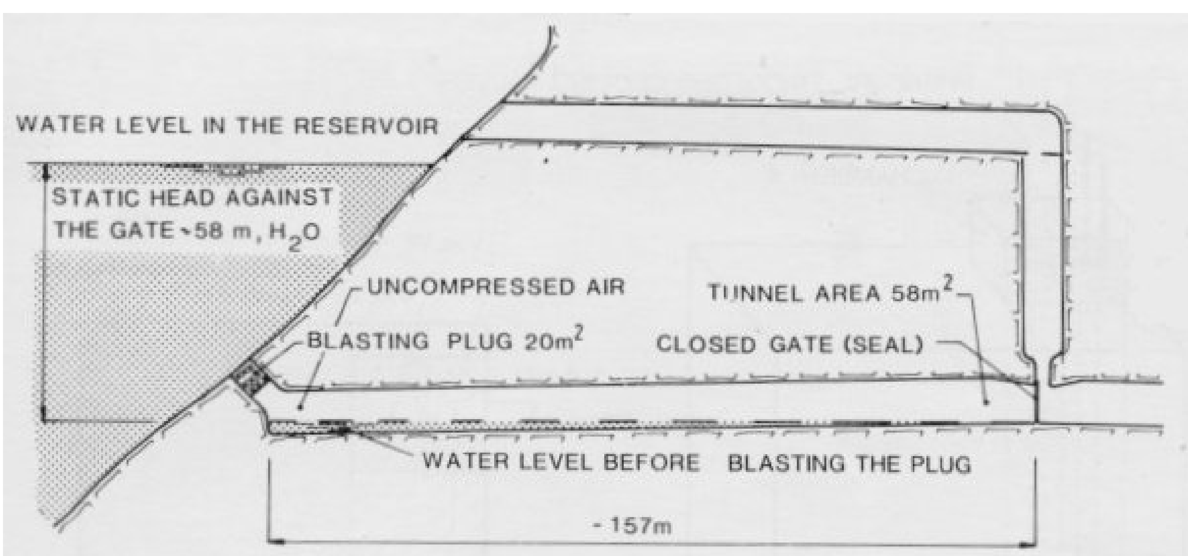


Fig. 11. Sketch showing a specific breakthrough (closed tunnel).

7. COMPARISON OF MEASUREMENTS IN MODEL AND PROTOTYPE

The correlation between the model results and the prototype measurements is given in this section. The main aims in this model test were to:

- avoid higher pressures than the gate construction can withstand,
- prevent the blasted rock masses from being transported all the way to the gates.

The tunnel rupture is sketched in Fig. 11. The model yielded the following results:

- An initial water volume equal to 20% of the tunnel volume was sufficient to stop the blasted rock masses at a safe distance from the gates.
- This initial water volume reduces the inflow volume after the rupture. This in turn, leads normally to a reduced head loss and hence a higher maximum pressure.
- A safe and acceptable maximum pressure was obtained by «choking» the rupture area to 20 m^2 while the tunnel area was 58 m^2.

The pressure measurements from both the model and prototype are shown in Fig. 12. There is very good agreement between model and prototype. Regarding rock mass transport, there are no corresponding prototype measurements. However, all the rock mass stopped in the prototype before reaching the gates. It is therefore fair to believe that the masses have settled out along the floor of the tunnel in the prototype in a pattern similar to that observed in the model.

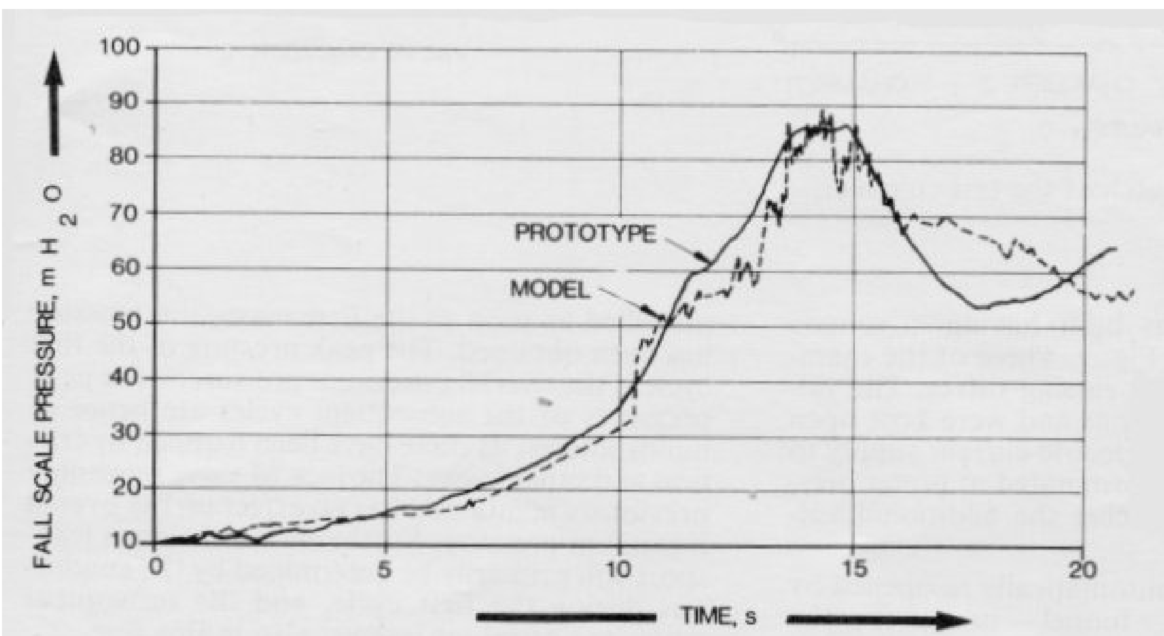


Fig. 12. Pressure measurements in model and prototype for the case shown in Fig. 11.

Underwater piercing of a tunnel

By Ø. Solvik
Senior Research Engineer Norwegian Hydrodynamic Laboratories, River & Harbour Division

Summary
There are two main ways of piercing a tunnel under a lake. In one the tunnel is completely or partially filled with water prior to final penetration. In the other the tunnel is generally left dry and blocked to atmosphere.
The author describes the advantages and disadvantages of each method.

Introduction
Underwater piercing of a tunnel has been and still is considered a Norwegian speciality.
Ever since the beginning of this century when the use of Norwegian hydroelectric power was first developed seriously, Norwegian consultants and contractors have been confronted with this task of blasting a final penetrating passage that will open the way for the water in the reservoir to flow through to the hydropower turbines.
The number of these penetrations is unknown but there must presumably have been several hundred of such blasts of varying difficulty.
Norway has almost certainly led in this area because of its special topographical and geological conditions. The glacial activities have created a number of natural and very deep lakes forming cheap reservoirs.
These can be used to drain the lake down below the natural water level with the aid of a tunnel that is underneath and with an inlet at the bottom of the lake, see Fig. 1.
Piercings at depths up to about 100 m have been performed. As a rule, the problems tend to increase with depth but unsuccessful penetration can occur at any depth. The need for the best possible understanding of the complexities associated with the different aspects of tunnel piercing are therefore obvious. An unsuccessful piercing can often result in serious economic consequences as well as danger to life. There is therefore an obvious need for systematic collection of data and an understanding of the physical processes that occur.
Ever since the beginning of the 1960s, the Norwegian Hydrodynamic Laboratories (VHL) have been involved with this problem through model tests, theoretical evaluations, field measurements and in an advisory capacity. The following shows the main aspects of this activity.

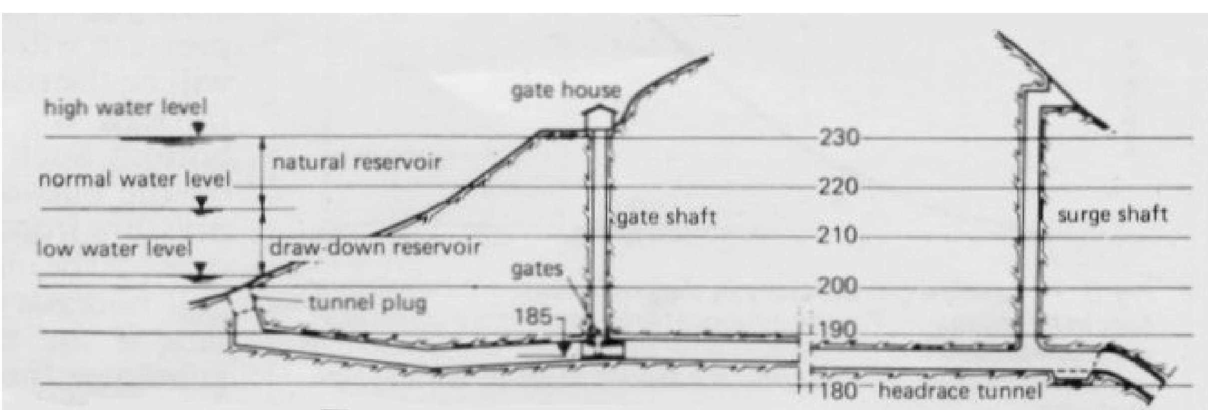


Fig. 1. Typical situasion for underwater tunnel piercing.

Planning underwater tunnel piercing

When planning an underwater piercing there should be an estimated reservoir requirement based on the hydrological conditions and the profitability of the natural reservoir. These evaluations are usually simple and conclude that the water level can be lowered as far as possible with respect to certain conditions. The most important being the bottom

topography of the lake which determines the size of the natural reservoir. In addition there are the secondary effects that include, for example, danger of slides when the water level is lowered, wave erosion along the lowered new shoreline, erosion at all streams and rivers flowing into the lake, ground water erosion in the newly exposed dry shoreline and so on. These factors are not directly connected with tunnel piercing itself but they must be considered in the evaluation before it is decided how far the level may be lowered.

When these secondary conditions are evaluated and the required depth of penetration is known, then it is time to plan which procedure to choose and finally, the location of the best position for penetration.

For this the following is required:

1. good, large-scale detailed maps of the lake bottom and nearby shore;
2. seismic profiles that show rock quality with possible crushed zones and cracks, and so on, as well as loose sediment zones;
3. tests that show the loose sediment structure and density; and,
4. exploratory drilling that shows accurately where the rock lies around the penetration area.

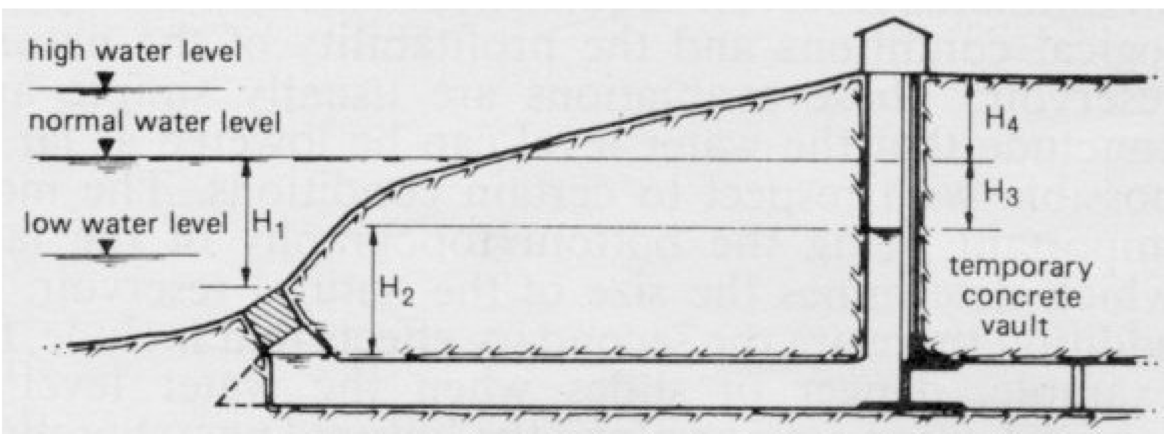


Fig. 2 Piercing with the tunnel open to the atmosphere.

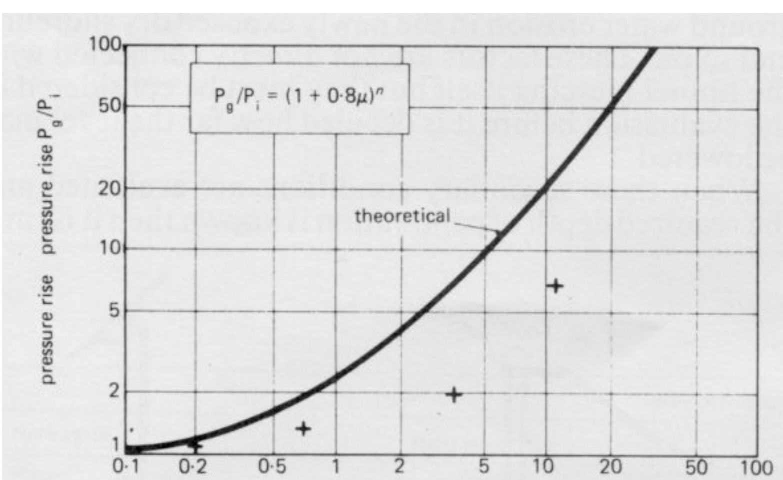


Fig. 3. Theoretical gas pressure in the pocket just after deterioration. In the equation;
P_i = the airpocket pressure before blasting.
P_g = the gas pressure in the pocket just after detonation
μ = (amount of explosive/trapped air volume), measured in m of atmospheric pressure.

Map and seismic profiles are used to determine the tunnel routing on the last stretch from the land and underneath the lake bottom. The exact location of the point of penetration is determined based on additional information obtained by exploratory drilling which gives the possible size and composition of the loose sediments. The requirement should be that the point of penetration should be where the rock quality is the best possible and where there is as little loose sediment as possible. Also it should be where the land has such a slope that possible seeping in of material from the surroundings can be avoided.
To ensure a good penetration, it has been considered necessary to overcharge in comparison with the normal blast. Also one must consider the fact that the charge must also break out through the outer shell which is not completely drilled through and also break up possible layers of loose sediments. Between 3 and 5 kg of explosives per m^3 of firm rock is normally used. The disadvantage with overcharging is that it increases the internal pressure against possible closing systems and that it can push out boulders that can block the opening either completely or partially. If the opening is bigger than planned, the closed penetration could experience a larger pressure increase than calculated. There are, therefore, many reasons why one should show moderation when determining the size of the charge. In many instances the limitation of the size of the charge is an essential part of the art of tunnel piercing.

Choice of method of penetration
The method of penetration should be decided upon at the planning stage. To ensure it is the best possible and most successful, it is often essential to adapt the tunnel routing and additional geometry to the method of penetration. All possible water levels at the moment of penetration should be evaluated in advance. We can divide the methods roughly into two main groups: .penetration against the open tunnel shaft (open system)-; and, .penetration against the closed tunnel shaft (closed system). These are discussed separately below.

Open tunnel shaft method
The usual method for this type of penetration is shown in Fig. 2. Here there is an open connection from the face of the tunnel, where the penetration takes place, and up through the gate shaft. The downstream side of the shaft is closed to prevent an uncontrolled flow of water and rock debris being carried through. This method requires the tunnel to be partially filled with water to reduce the transport of loose sediment from the explosion and towards the gate. Even so, debris may be carried along several hundred metres with a possibility of damaging the gate construction. Moreover the inflowing water mixed with stone may generate a powerful upsurge in the gate shaft with resultant damage especially to the gatehouse. Therefore the partial filling is also important for limiting the surge in the shaft after the penetration. This will occur if there is a positive difference between the level in the reservoir and in the shaft. Depending on the losses because of the friction and turbulence the upsurge above the reservoir water level can be as much as 70-90 % of the difference between the reservoir and the filled level in the shaft.
The maximum allowable upsurge is usually limited to under the floor of the gatehouse, since the gates and accessories are normally already installed at the time of the detonation. To counter the upsurge alone, the shaft has to be filled as much as possible, but there is a limiting factor. This is to do with the pressure conditions at the working face. The situation is such that the propagation of the blast pressure can mean an unacceptably high load on the gates. Therefore an air pocket has to be trapped at the working face and at the point where the charge is placed, that is at the plug. A pocket like this will prevent dangerous hydro elastic shock pressure being transmitted through the water against the gate.

But during the filing of the system, this air pocket will be compressed. The pressure within this pocket should not then become more than the outside water pressure on the plug. If the inside pressure is greater there is a risk that air will leak into the reservoir and if the worst comes to the worst, the pocket may empty.
In good, solid rock it is unlikely that a small over-pressure will create a large air leakage. However, there will be the risk of leakage through the boreholes drilled through the plug into loose sediments on the lake bottom. Such holes do not leak because of water pressure on the outside, but may be blown open w hen the air pressure from the inside increases during the filling and exceeds the outside pressure.
If necessary air should be provided for the working face if the natural en closed air is not sufficient to guarantee the separation of the penetration charge and water in the tunnel.

Pressure increase caused by explosion gases
We have emphasized the importance of the air pockets in preventing water hammer. Gas released during the explosion increases pressure in the pocket. It is reckoned that 1 kg of explosives forms about 0.8 m^3 of gas at atmospheric pressure.
The largest possible increase in pressure at the working face will occur if the penetration is unsuccessful but where all the explosive charges are detonated. In this case calculate the pressure increase assuming no energy is lost and that all the explosive energy goes into the pressure increase, see Fig. 3.
Both the en closed air and transported gas from the explosive can cause great damage where the conditions are such that the compressed gas is forced out, for example, through the gate house.

Collection of debris
With this type of open piercing there will be low water velocity in the penetrating area because of the prior filling of the tunnel. The problem of collecting the debris is not difficult. Quite often collection under the plug in the penetrating area will be effective. It can be assumed that most of the debris will fall down because of its own weight and a cavity into which the debris will fall will be excavated accordingly. Spreading of debris caused by the explosion requires that the cavity covers an adequate area.

Other methods for open piercing
There are several methods of carrying out an open piercing. For instance, pre-filling may be eschewed. If the penetration is made with a dry tunnel, the inflow water velocity, v, will be very great ($v = \sqrt{2}gh$, where h = the pressure head and g = gravitational acceleration).
In this case it will be very difficult to collect debris after the detonation. Shallow cavities situated under the penetrating area have proved to be ineffectual. A deep and narrow cavity is effective but is difficult to make in practice.
If some of the cavity lies with an overhang under the plug itself(see Fig. 2), it could collect some of the debris. In a dry tunnel one should be prepared for the fact that the debris is carried right up to the gate shaft and can damage the gate. The method involving little or no filling should, therefore, be model tested where different measures to prevent spreading of the debris can be tried. This method has in fact be en tested with varying success and is certainly best suited to long tunnels, where friction plays an important part in reducing the incoming water velocities.
A partly water-filled system will reduce the water velocity and therefore will be favourable for the transport of debris. But pre-filling the cavity only seems to be able to work in the opposite way by the inflow being deflected from the surface of the cavity.
In all these cases where it is essential to be able to control the transport of debris, the model test is not only useful but necessary. One should be aware that only small variations in the geometry can

cause great variations in the transport of debris. General rules are difficult to formulate and should be used critically.
A solution that has been used is to build a temporary vault downstream of the gate to blast against this with the gate in a safe position out of the water flow. A similar effect can be achieved if a plug that can be blasted against is left in the tunnel. To avoid problems with the transport of debris and the risk of damaging or blocking the gate guides, this type of piercing should be done with a high level of water in the tunnel. Another choice is to do the piercing with a free opening downwards and the gate safely positioned in the shaft. This has been done in cases where the water can run out freely. But in these cases there is a risk of damage to the guides with possible blocking. This is not considered to be a good and reliable solution and is best reserved for situations where there are no closing systems in the water-ways.

Piercing against a closed shaft (closed system)
A tunnel penetration can also be carried out by the so-called closed piercing method. This means that the tunnel at one or another point downstream of the piercing is closed off with a gate or valve so that there is no contact with the astrosphere. An example of this is when the closed gate is secured on the upstream side of the gate shaft or bas a stuffing box, see Fig. 4.

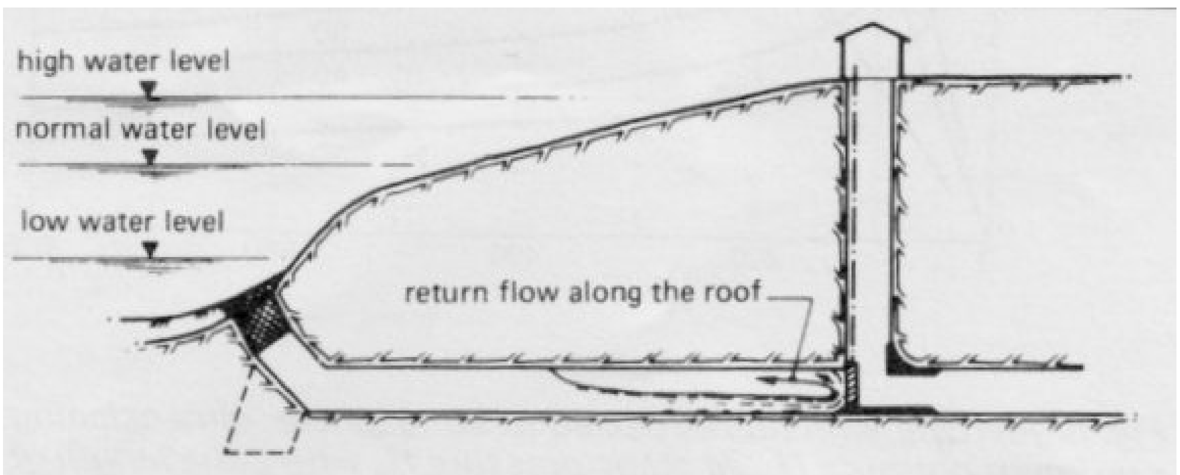


Fig. 4. Piercing with a tunnel closed to the atmosphere.

This method has been used on many occasions with good results. In these instances it is a rather long stretch of tunnel between the piercing and the closing system. It must be emphasized that tunnel friction is important and the longer the tunnel, the less pressure there is against the closing system.
This type of piercing bas also been done with a high water pressure and a sort tunnel, and consequently with somewhat bad results. The reason is that the increase in pressure was so great that the closing systems were damaged. This has meant a thorough evaluation of the whole problem. Primarily this applies to pressure increase and the factors that determine it. In addition the spreading of the debris is important in this case, because the water velocity and thereby the ability to transport the debris is great.

Increase of pressure after the piercing
With these closed piercings, the enclosed volume of air is often so large that the extra pressure created by the explosion, even with an unsuccessful penetration, is moderate. With the closed piercing method it is the energy of the inflow water, provided the penetration is successful, that

gives the largest pressure increases and thereby the greatest loads on the closing system, We will look closer at this situation.
W hen the penetration charge is detonated, and the tunnel has been pierced against the lake, the water will flow in and compress the air in the pocket. The easiest analogy is that of a piston, Where the power of the piston is constant, and we disregard the inertial forces of the water piston, the air in the pocket will be compressed until the internal pressure corresponds to the force of the piston, in other words the external water pressure. This is, however, very simplified. The inertial forces of the inflowing water are of paramount significance and will cause the pressure to increase well above the static water pressure. This is associated with the kinetic energy of the water in the tunnel which will increase until the internal pressure is equal to the external driving pressure, But from then on the water will decelerate and it is this retardation power that gives the large pressure peaks inside the pocket. We will consider in more detail at w hat happens in this type of closed system.

Frictionless inflow

Where the inflow takes place without loss of energy in the form of friction loss in the wetted area of the tunnel and flow losses in the opening bend, water front, and so on, the maximum pressure can be calculated exactly. We assume that the compression should be calculated adiabatically, that is, without loss of heat to the surroundings. The principle for the calculation is simple: the energy that the water gives off w hen it moves down into the tunnel appears again in the compression energy in the air pocket. The equation governing this process is:

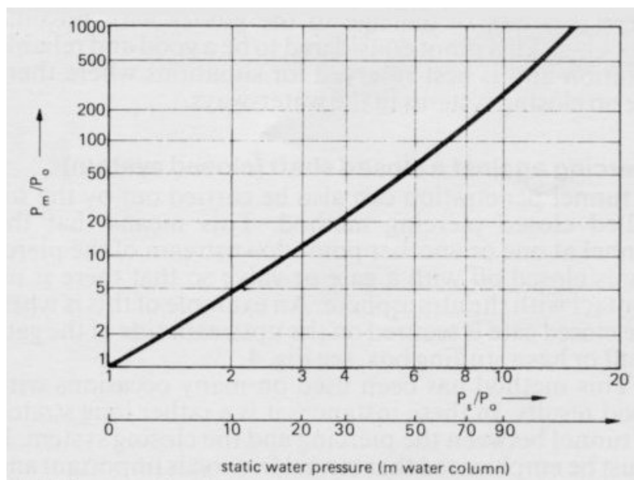


Fig. 5. Pressure rise with frictionless inflow.

$$P_s/P_0\,[1 - (P_m/P_0)^{-1/K}] = 1 / (K-1)\,[(P_m/P_0)^{(K-1)/K} - 1]$$

where:

P_0 = atmospheric pressure
P_s = total pressure on the plug
P_m = maximum pressure in the pocket
K = 1.4.

From this we see that the pressure increases greatly with increasing external water pressure. The combination of high external water pressure and short tunnel results in little friction and is particularly undesirable. In general one could say that the inflow losses are a necessary condition of the closed piercing, even with a small water depth.

Inflow with friction

As already mentioned, it is the losses created w hen the water flows into the tunnel after the penetration that are paramount in determining the usefulness of the method. The energy that the water loses during the inflow is made up of: losses in the inflow opening; changes of direction; losses at the water face; friction losses along the wetted area; and kinetic energy for the water including possible waste materials carried along in addition to the compression work. With some uncertainty these losses can be calculated and with today's computer technology the calculations are relatively easy. Uncertainty exists in a sensible assumption of the hydraulic loss. This non-stationary inflow situation obviously gives losses that are considerably greater than for the known

stationary situation. Probably many of the substantial losses lie in the water face and these losses can be included using low Manning's numbers.
VHL has developed its own computer program for this type of piercing and for those who wish additional
information about this, reference is made to the company's open reports on the subject.
In Fig. 6 we have set up a general diagram that shows the pressure increase as a function of the tunnel length and static water pressure. Examples are chosen with tunnel area 10 m^2, concentrated loss equal to twice the velocity head and Manning's number equal to 30. The intervals in the computer calculations are chosen to be equal to 5 m. The diagram clearly shows that the pressure increase is small for long tunnels but increases greatly when the tunnel reaches a certain minimum length. One should be careful w hen using the method for particularly short tunnels on the border or outside one's range of experience.

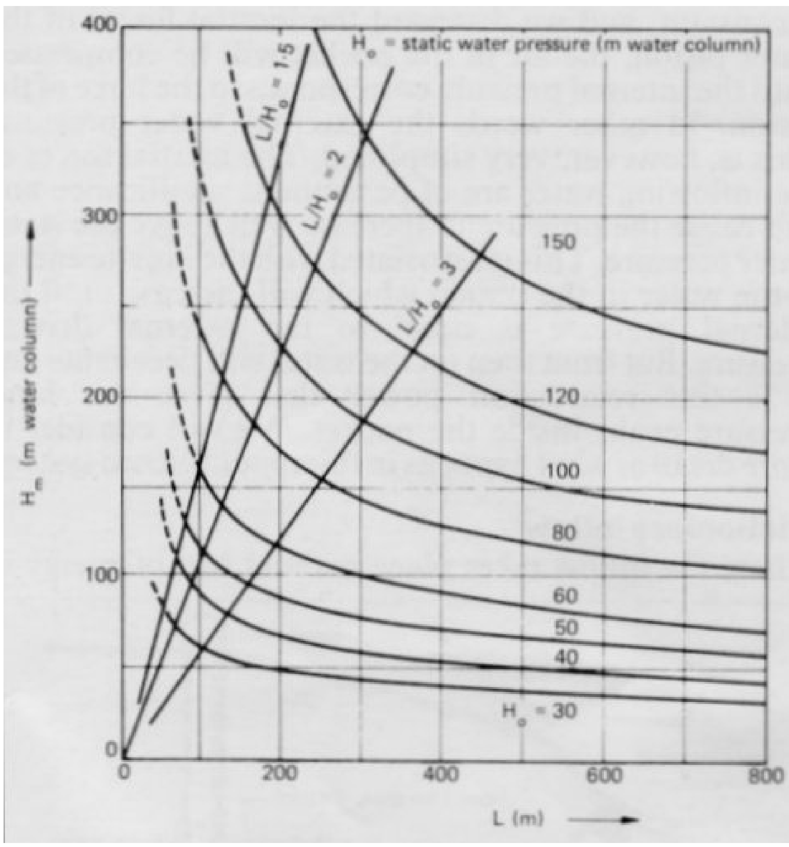


Fig. 6. Piercing with tunnel closed to atmosphere; dimentioning maximum pressure H_m at static pressure H_0, where the lenght of the tunnel = L, and the tunnel area, A = 10 m^2. (C = 2, M = l/n = 30)

Collection debris
With closed piercing the water velocities will be very high immediately after the penetration, (v = √[2gH]). This makes the collection of debris very difficult. There are possibilities of collecting it just under the plug if a large enough cavity can be created outside the incoming jet of water or in a deep cavity, see Fig. 2. But with this type of piercing the debris must be allowed to spread into the tunnel. However I it must be prevented from reaching the closing system so that this is not damaged. Generally water reaches the gate first at the bottom of the tunnel after which water jets are deflected and fill the cavity under the overhang. During this process large boulders may be carried all the way to the closing system.
Prior filling of water can be a solution, for example in cases where it will submerge the closing system. One should in any case be aware that the filling of water beforehand reduces the total volume and will have the same effect as shortening the tunnel, which again increases the maximum

pressure. It is difficult to give any reliable recommendation w hen it applies to collecting debris at such high velocities that occur.
The details in the geometry will greatly influence the effectiveness of collecting debris, for completely identical piercing designs very rarely repeat themselves. This problem must be solved before the piercing is done, and a model test is an important indicator. With an artificial solution as, for example, dead-end tunnels, it is difficult to predict the effect of these and here the model test is strongly recommended before a final decision is made. .
Covering the closing system with prior filling without causing 100 much of a reduction of the tunnel's air pocket, can be a good solution. But it has to be emphasized that such evaluations and tests must be done at an early stage before plans are consolidated for other reasons.

Lake tap - the Norwegian method.

B. Berdal
Ing. A. B. Berdal A/S, Sandvika, Norway
B. Buen
Ing. A. B. Berdal A/S, Sandvika, Norway
J. Johansen
Dyno Konsulent A/S, Gullaug, Norway

Summary
Some 400-500 successful submerged tunnel piercings have been performed in Norway since the 1900s - mostly to create economical draw-down reservoirs for hydroelectric power plants. The deepest piercing is at 105 m at Lake Jukla and the largest (95 m^2) at Rygene. A piercing is located where rock quality is favourable and soil overburden limited. Exploratory drilling, grouting and careful blasting are used to bring the tunnel close to the rock surface. The final blast has at least two parallel hole cuts and high-strength water-resistant explosives are used. Specific charge varies between 3 and 8 kg/m^3, depending mainly on cross-section and depth. In the most common piercing method an air cushion is established below the final plug to reduce shock waves on gates and bulkheads.

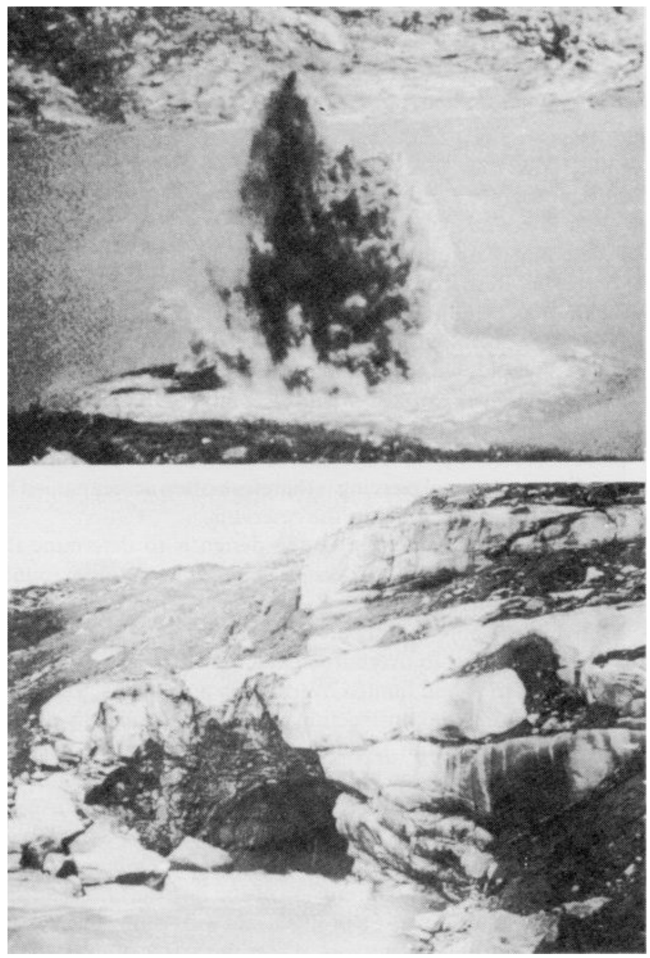


Fig. 1 Lake Rembesdal during blasting and after drawdown.
(depth, 25 m; cross-section, 54 m^2; plug thicness, 2,5 – 7m; soil cover, 3 – 5 m)

Introduction
Submerged tunnel piercing, the process of piercing the bottom of a lake with a tunnel, has been a regular practice in Norway for more than 75 years. During this period it has been under- taken somewhere between 300 and 500 times. The firm and documented number is approximately 300, but from the number of hydroelectric power schemes that are in existence, and in view of the haphazard way in which the early cases were registered, the estimate of 500 is considered fairly accurate. The final blast for a lake tap and the opening after draw down are shown in Fig. l.

One reason for the failure to register lake taps is probably the fact that nobody considered it to be a feat that merited reporting -it was merely common construction practice. To date, the practice has involved the piercing of a lake bottom at a depth of 105 m and the breakout of a tunnel with a cross- section of 95 m^2. A list of some of the more noteworthy piercings is given in Table l.

Table 1.

Location	Hydroelectric power scheme	Depth, m	Cross section m^2	Year
Skjeggedal	Tysse	---	---	1905
Tverrelvvatn	Simavik	---	---	1913
Storglomvatn	Glomfjord	---	16	1920
Krokvatn	Skarsfjord	---	4	1922
Tafjord	Tafjord I	---	---	1923
Storbotnvatn	Svelgen IV	70	8	1971
Jukla East	Folgefonn	80	13	1973
Jukla West	Folgefonn	105	10	1973
Jukladalsvatn	Folgefonn	93	60	1977
Selbusjøen	Bratsberg	10	65	1977
Nidelven	Rygene	9	95	1977
Lomvivatn	Lomi	70	17	1978
Ringedalsvatn	Oksla	86	40	1980
Tyee Lake	Tyee Hydro	50	8	1983

The main use of submerged tunnel piercings are lake taps, but the principle has found application for some tailrace tunnels and for sewer outfalls.
The large number of lake taps that have been executed in Norway relate to the combination of topography and economics. The economic part is the possibility of creating a reservoir by draw down of a natural lake or by combining a low dam and a draw down. The topographic element is that the many small lakes in high valleys and in the mountains lend themselves to such a solution.
The process of tapping a lake can be divided into the design of the tunnel system, the excavation of the tunnel and the design and execution of the final blast.

Design of tunnel system
The main components in an intake tunnel system are shown in Fig. 2. The rock trap will contain the debris from the final plug and prevent rock fragments from being swept far into the tunnel and even jamming and damaging the gate. Temporary bulkheads are used when work is still proceeding in the down-stream tunnel system or if the installations cannot take the blast shocks. In many cases the ordinary gate can be used during the blasting.

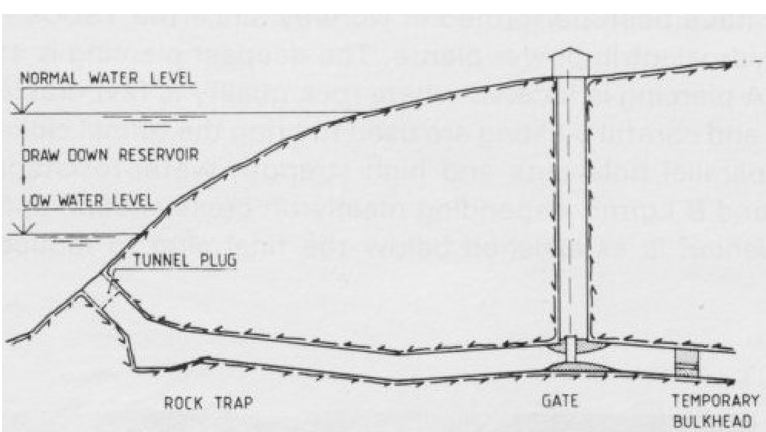


Fig. 2. Typical layout for a lake tap.

The gate shaft to the surface shown in Fig. 2 is, of course, not a requirement. A gate chamber in the rock connected to a suitable surface location by a tunnel is a much used alternative.
The depth of the piercing in a hydroelectric power reservoir is determined mainly by the hydrology and the topography of the basin. Restrictions on the draw down, both on rate and depth, may be imposed by unstable slopes in sediments and in tallus slopes. Rapid changes in pore pressure might trigger slides. The slides in themselves can be a problem and the slide- generated waves can also be unacceptable. Other factors are wave erosion on newly exposed surfaces and the erosion taking place in the tributaries to the reservoir now at a lower base level. A submerged tunnel piercing is therefore often accompanied by erosion-protection work in the reservoir .
A very important factor in the design is to determine the location of the piercing. Extensive geological mapping, soundings, refraction seismic surveys, exploratory drilling, inspection by divers and core drilling are used to assess the suitability of a site with respect to overburden and rock mass quality. Fairly competent rock and limit ed overburden are sought. The main problem during the construction of a lake tap is usually that of leakage, so sites with faults and open joints are avoided, if possible. A piercing can be done through some soil overburden, but an overburden larger than the tunnel diameter is usually considered to be risky. A limited overburden of 1-3 m will, on the other hand, often help to seal cracks and joints and may therefore be a benefit. The erosion of soil into the tunnel during draw down may also pose a problem and will require an oversize rock trap. For this reason rock slopes fairly free of overburden are considered to be those most suitable.
Two basic types of piercing are normally applied, the differences between open- and closed-system piercing being shown in Fig. 3. In an open-system piercing the gate or bulkhead is placed on the downstream side of the gate shaft , leaving a direct communication between the tunnel face and the atmosphere.

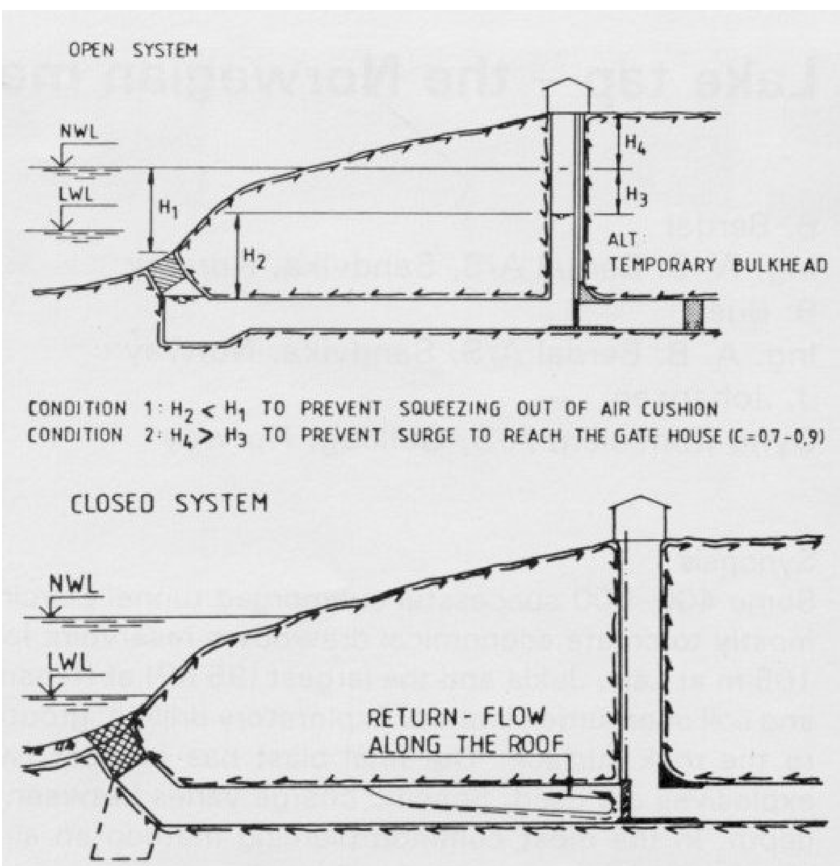


Fig. 3. Open and closed systems of submerged tunnel piercing. In the open system, (top), the trapping of plug soil is easy. In the closed system, (bottom), it is difficult. After Solvik[1].

To prevent rock debris from reaching the gate it is essential to fill the tunnel partly with water prior to the blasting. The degree of filling must be weighed against the upsurge in the shaft, which should not be allowed to reach the floor of the gate house (condition 2, Fig. 3). This upsurge can be calculated quite accurately. W hen filling the tunnel before piercing an air pocket must be left against the tunnel face to prevent any harmful shock from being transmitted to the gate. This will normally restrict the level to which the tunnel may be filled to avoid the air in the pocket being squeezed out through the plug into the atmosphere (condition l, Fig. 3). The pressure conditions in this air pocket just after the detonation are presented in Fig. 4 (both theoretical pressures and some recorded data).

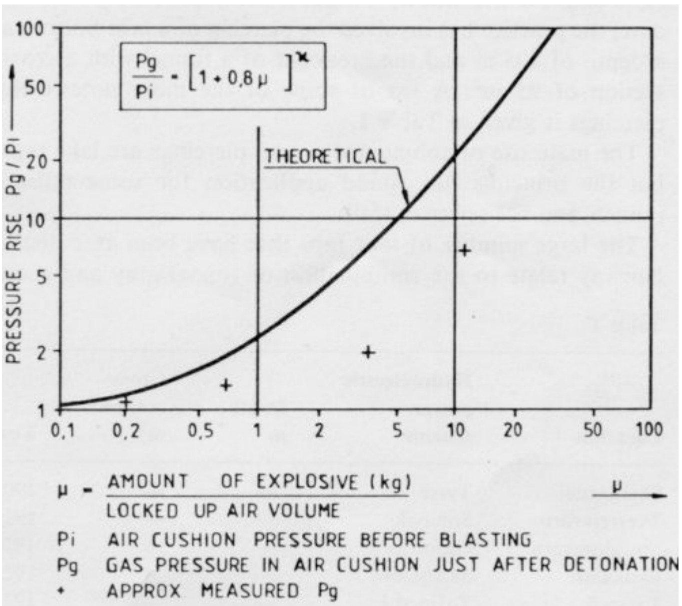
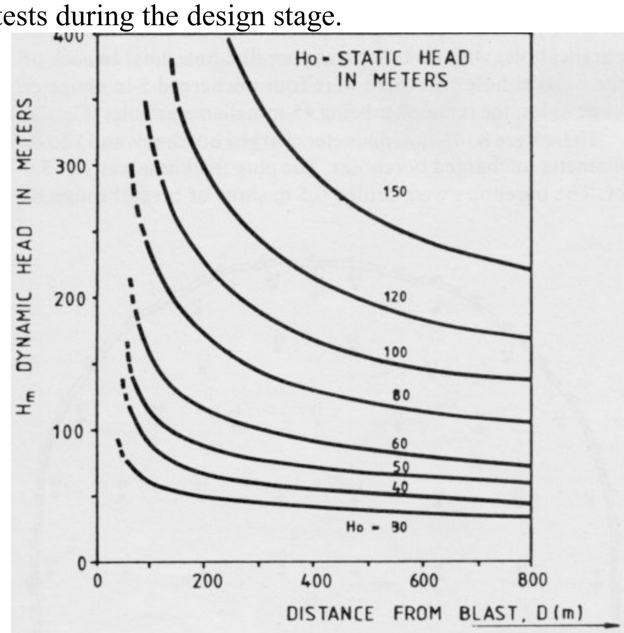


Fig. 4. Gass pressure in air pocket (open piercing). After Solvik[1].

To obtain an air pocket of the required volume the final plug is usually situated at the end of a short shaft. Very often, the tunnel is excavated at a gentle incline close to the breakthrough. In a closed-system piercing the gate or valve is placed so that the tunnel volume is confined from the atmosphere. This system requires a relatively long stretch of tunnel between the plug and the gate to prevent damage to the closing structure from the maximum pressure produced by the detonation. Fig. 5 shows how this dynamic pressure, H_m, from inrushing water against the gate decreases with increasing tunnel length. Closed piercings can be made with empty or partially filled tunnel. There are records of closed piercings in which the increase in pressure was so high that the closing structure was damaged. Even if the results have normally been good, closed piercings are used less frequently than open piercings and they have more frequently required model tests during the design stage.

Fig. 5. Dynamic pressure, H_m *(closed piercing). After Solvik*[1].



Construction

Construction of a tunnel system for a submerged piercing is done by drill and blast methods. Because extensive drilling of exploratory holes is required and because grouting is generally necessary, only drill and blast methods have the required flexibility and space at the face.
W hen the tunnel is approaching the location of the piercing a systematic drilling of exploratory holes is begun. This drilling starts w hen the tunnelling under the reservoir begins or w hen the tunnel reaches to 30-100 m of the breakthrough, depending on local conditions. The main objective is to detect water- bearing faults and joints ahead of the face. For this percussion drilling of three to five holes is used. The drilled length is dependent on round length and available equipment, but the rock mass one round ahead of that which is being drilled and blasted should always be explored (Fig. 6).

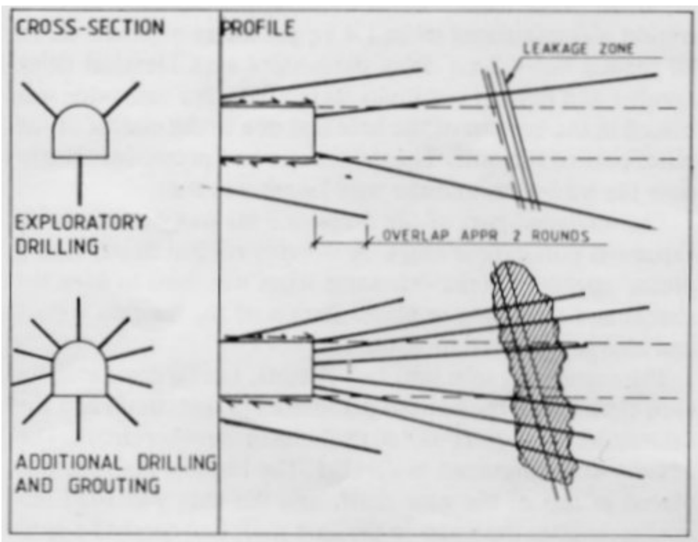


Fig. 6. Exploratory drilling and water-sealing procedures

Major leakages are grouted ahead of the face, minor leaks not seriously affecting working conditions being left untreated. Fast-setting cement-based grouts are commonly used. Special coarse-grained types are used to plug large leakages. Chemical grouts of types that react and expand on contact with water have proved of value, especially close to the surface where only low grouting pressure can be applied. The use of low grouting pressure dose to the surface is important because too high a pressure will jack open joints and faults.
Close to the surface the round length is reduced and, if necessary, the drilling pattern is changed to produce a light breakout. W hen dose to the surface a number of holes that depends on the cross-section is drilled through to the surface to locate it exactly. On this basis the final trimming, design and drilling out of the final round are done.
The rock trap is excavated carefully after the drilling of the final round -the rock trap preludes the use of heavy equipment at the face without the installation of heavy ramps and scaffolding. Conditions at the face are very often wet and the water is cold, so survival suits can be a necessity. To push a packer into a drill-hole jetting ice-cold water at high pressure is almost impossible without the right protection and equipment. At lower pressures the holes can be plugged by long tapered wooden plugs, but at higher pressures packers or even large 'drill-through' packers have to be used.

Design of final blast and blast control system

The final blast is a crucial part of the tunnel piercing. A failure can lead to long delays because part of the -old tunnel system may have to be sealed off and a new approach constructed and new gates may have to be installed. This can be very costly in terms of construction and because of the delayed start-up of the project For this reason detailed planning of the final blast is done and special water-resistant explosives and detonators are used. The explosive used is of a high-strength gelatine type. Before use in a piercing samples are taken from the batch and tested- The standard test requirement is failsafe and full- strength detonation after a 72-h storage in water at a depth equivalent to the final blast.

Electric detonators of the millisecond delay type are used. The detonators have heavy-duty insulation. The test requirement for detonators is the same as that for the explosives.
The drilling pattern is designed on the basis of the tunnel cross-section, the plug area as mapped by exploratory holes, the thickness of the plug, the rock type, the thickness of the over- burden, the water pressure and the water leakage. Parallel-hole cuts are used and the minimum number of cuts is two (in large cross-sections three). There are usually four large-diameter unloaded holes in each cut The holes are drilled 0.3-0.5 m short of breakthrough.
The blast holes are loaded and tamped for maximum density. Depending on the factors mentioned above, the specific charge will typically vary between 3 and 8 kg/m^3. This is, as a rule, 50-100% more than the usage in normal tunnelling. The high specific charge gives high shock loads on gates and bulkheads. To reduce the specific charge would, on the other hand, increase the risk of full or partial failure and, as the lesser of two evils, high charges are used.
Double and independent ignition systems are used. One detonator is placed in the bottom of the hole and the other in the outer part of the hole.
Present-day piercings are usually of the "open-system" type, and the tunnel system is designed to entrap an air volume to act as cushion. If this volume is not sufficiently large, additional air is pumped in by compressor. Compressors are also kept at standby to replenish any air leaking out of the tunnel. The water level in the tunnel is monitored by a simple device where the water short-circuits two sensors: -one for high and one for low water level.
Detailed descriptions of two piercings, Tyee Lake and Lake Lomi, at depths of 50 and 70 m, respectively, are given in Appendix 1 and Appendix 2.

Conclusion

Underwater piercing is a proven method of obtaining economical reservoirs for hydroelectric power projects. A successful breakthrough is dependent on a well-selected site, a carefully planned and executed construction and a well-designed final blast. With these requirements fulfilled, the hitherto maximum depth of 105 m and a maximum cross-section of 95 ml are not the limits of this method of ten termed the 'Norwegian method'. Most lake taps to be done in the future -and there are many planned in Norway, and there is no reason why they should not be used elsewhere -will, however, be in the inter- mediate range both with respect to cross-section and depth.

Reference

Solvik Ø.
Underwater piercing of tunnels. In Fjellsprengnings- teknikk (Trondheim: Tapir, 1983). (Norwegian text)

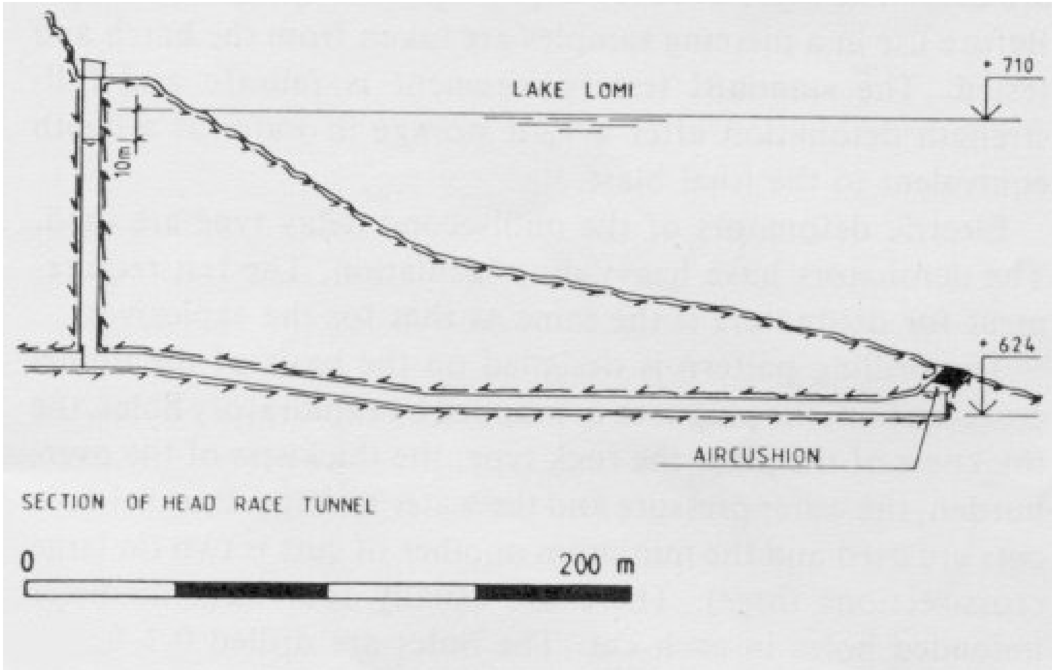


Fig. 1

Appendix 1
Plug blasting
Lomi hydroelectric power plant

General description
The Lomi hydroelectric plant is located in the northern part of Norway, approximately 100 km east of Bodø. The headrace tunnel was connected to Lake Lomi by the 'lake-tap' method. The static head on the plug was 75 m. The cross-section of the final plug was 18 ml and the thickness of the plug about 4.5 m. The plug was covered by 2-3 m of loose deposits. The distance between the plug and the gate shaft was 280 m (Fig. l). The rock type was quartzitic mica schist. Leakage of water through the rock close to the final plug was insignificant.
At the time of blasting the gate was closed, and the tunnel system was filled with water to lo m below the level of the lake. Below the plug an air cushion was established. A high-strength gelatine explosive and millisecond delay detonators were used for the final blast.

Drilling pattern
The drilling pattern was based on the use of parallel holes with three separate parallel-hole cuts. In each of the parallel-hole cuts there were four uncharged 5-in diameter large holes, the remainder being 45-mm diameter holes (Fig. 2).
There were 80 45-mm diameter charged boreholes and 12 5-in diameter uncharged boreholes. The plug thickness was 3.9-5.0 m. The boreholes were drilled 0.5 m short of breakthrough of

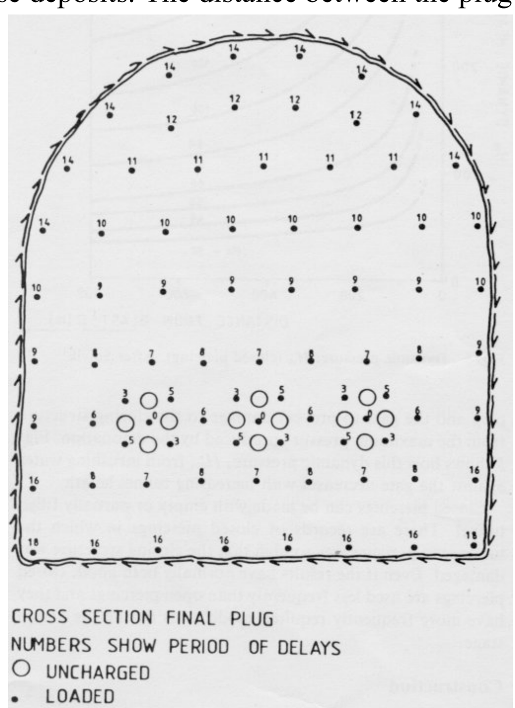


Fig. 2.

the plug, i.e. the average length of the boreholes was 4.0 m. Charging and ignition system The boreholes were loaded with 35 mm x 600 mm cartridges in plastic film with 60% NO explosive (trade name, 'Extra Dynamit'). The charge concentration was calculated to be 1.4 kg per metre of borehole. In all loaded holes 2-ms delay detonators with identical delay number and normal sensitivity were used. One detonator was placed in the bottom of the hole and one in the middle of the outer part of the hole. The detonators had protective sheaths over the lead wires and the wire length was 6 m.
The unloaded part of the holes (0.3 m) was stemmed with expanded polystyrene plugs. A wooden conical dowel with a pre-cut opening for the detonator wires was used to keep the charge and stemming in place. Details of the ignition system and charge are given in Table l.
The round was split into two circuits, i.e. 80 detonators in each circuit with the bottom detonators in one circuit and the detonators in the outer part of the holes in the other circuit. The circuits were connected in parallel. The blasting machine was placed at top of the gate shaft, and the shot was fired immediately after the water in the gate shaft had reached a level 10 m below the lake level.

*Table 1 Delay detonators- charges**

Delay no.	**No. of holes per interval**	**No. of short-delay detonators**	**Loading lenght, m per interval**	**Extra Dynamit 35 x 600, wt of explosives, kg / interval**
0	3	6	10,3	14,4
3	6	12	20,6	28,8
5	6	12	20,6	28,8
6	4	8	14,5	20,3
7	6	12	21,0	29,4
8	8	16	28,8	40,3
9	10	20	26,5	51,1
10	8	16	29,8	41,7
11	5	10	18,4	25,7
12	4	8	15,0	21,0
14	10	20	37,5	52,5
16	8	16	28,1	39,3
18	2	4	7,4	10,4
	80	**160**		**403,7**

*) Total quantity of rock, 80 m^3; total explosives erquired, 404 kg;
powder factor, 404/80 = 5,1 kg/m^3

Appendix 2
Plug blasting

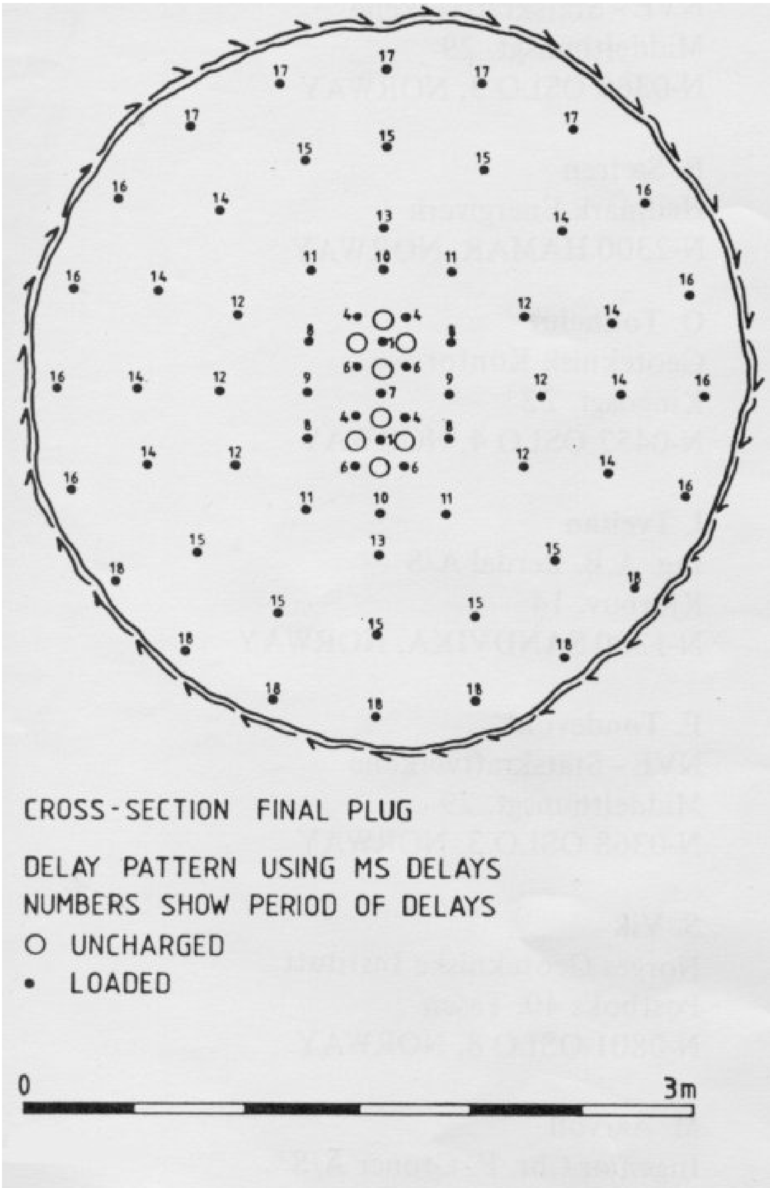
The Tyee Lake Hydroelectric plant,

General Description.
The Tyee Lake Hydroelectric plant is located near the town of WrangelI in Southern Alaska. Break- through of the final plug was carried out in September, 1983. The static head on the plug was 50 m and the distance between the plug and the gate shaft was 85 m. The plug cross-section was 8 m^2 and the plug thickness was 2.5-3 m.
The rock type was granodiorite and extensive grouting was necessary to seal the plug area.
Drilling pattern The drilling pattern was based on the use of parallel holes with two separate parallel-hole cuts on the symmetric line through the centre of the tunnel face. In each of the parallel-hole cuts there were four uncharged 76-mm diameter holes, the remainder being 35-mm holes (Fig. 1). The boreholes were drilled 0.3 m short of breakthrough of the plug, i.e. the average length of the boreholes was some 2.5 m.
Charging and ignition system The boreholes were loaded with 25 mm x 200 mm cartridges, containing 60% NO explosive ('Extra Dynamit'). The charge concentration was calculated to be 1 kg per metre of borehole.
In all the loaded holes there were 2-ms delay detonators with identical number and normal sensitivity. One detonator was placed in the bottom of the hole and the other was used in the middle of the outer part of the hole. The detonators had protective sheaths over lead wires and the wire length was 4 m.

*Table 1 Dealy detonators – charges**

Delay no.	**Interval ms**	**No. of holes per interval**	**No. of short-delay detonators per interval**	**Loading lenght m per interval**	**Explosives, 25 x 200 mm kg / interval**
1	25	2	4	4,4	4,4
4	100	4	8	8,8	8,8
6	150	4	8	8,4	8,4
7	175	1	2	2,2	2,2
8	200	4	8	8,8	8,8
9	225	2	4	4,4	4,4
10	150	2	4	4,4	4,4
11	275	4	8	9,0	9,0
12	300	6	12	13,2	13,2
13	325	2	4	4,4	4,4
14	350	8	16	17,6	17,6
15	375	8	16	17,0	17,0
16	400	8	16	17,6	17,6
17	425	5	10	17,0	14,5
18	450	7	14	13,3	13,3
		67	134		143,6

*) Total quantity of rock, $1{,}6^2\pi \cdot 2{,}8 = 22{,}5$
total explosives required, 143,6 kg
powder factor, 143,6 / 22,5= 6,38 kg/m^3



A wooden conical dowel with a pre-cut opening for the detonator wires was used to keep the charge in place.

Details of the ignition system and charge are presented in Table 1. The round was split into two circuits, 67 detonators in each circuit, the central detonators being in one circuit and the detonators in the outer part of the holes in the other .

Fig. 1.