INTRODUCTION

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In Norway underground-space is being utilized for a wide variety of purposes.
First of all, the topographical conditions are especially favourable for the development of hydro-electric energy. More than 99% of a total annual production of 90 TWh of electric energy is generated from hydropower .
Almost all post-war power stations are situated underground, simply due to the fact that this is the cheapest solution. Of the world's 300-350 underground power houses, almost one half i.e. 150, are situated in Norway. The hydropower development has also resulted in the excavation of approximately 2500 km of tunnels, and new tunnels are excavated at a rate of 100-150 km per year .
One Norwegian speciality is the use of unlined pressure shafts and high pressure tunnels in hydropower development. Altogether 64 unlined shafts with water heads varying between 150 and 780 metres will be in operation this year.
The most recent innovation in power plant design includes the replacement of the conventional surge shaft with an unlined air cushion surge chamber close to power station level. A total of 8 such chambers are currently in operation, with air pressures varying from 18 to 75 bar.
Tunnelling, however, is not only restricted to hydropower development. The mountainous nature of the country has resulted in the construction of more than 750 road and highway tunnels, and an equal number of railway tunnels. Throughout the country numerous large underground openings are used for storage of different products, such as oil, gas, ore, paints, flour, frozen foods etc. Underground space is also utilized for the construction of drinking water reservoirs, sewage plants, factories, telecommunication centres, military installations, car parks, swimming pools and gymnasiums etc. some of which are mentioned in this publication. Underground mining has been going on for the last 300 years, and still plays an important role in the utilization of the country's natural resources.
Norway is typically a hard rock province, forming part of the Fenno-Scandian Precambrian shield. Approximately two thirds of the country is Precambrian rocks, with gneisses as the dominating rock type, but also including granites, gabbros and quartzites. Most of the remaining third of the country is made up of rocks of Cambro-Silurian age. Due to the Caledonian oregeny these are metamorphosed to a varying degree. Both gneisses, schists, phyllites, greenstones and marbles as well as granites, gabbros, sandstones, shales and limestone are found.
Scandinavia is known not only as a typical hard rock province, but also as having extremely favourable rock conditions. Tunnellers in other parts of the world may feel that Scandinavian underground construction experience has little relevance in connection with their own problems, but this is not strictly true. Bad rock conditions do exist and can be particularly difficult in certain places.
Admittedly, in the case of hydropower tunnels, it is not unusual to find that 95-98% of the tunnel length is left unlined or without other permanent support measures. On the other hand we have examples of tunnels where 20-30% of the length has to be concrete or shotcrete lined due to adverse rock conditions. Also major parts of some tunnels have to be systematically bolted due to rock burst phenomena.
The vast majority of rock excavation in Norway is achieved by drill and blast methods. The tunnelling is highly mechanized and may include the application of electro-hydraulic drills, computer guided drill rigs, mechanized scaling and shotcrete robots to give maxi- mum advance rate.
A typical tunnel crew will consist of 3-6 men, who carry out all operations from drilling to hauling. Today's prices for drilling, blasting, scaling, mucking and hauling vary from 600 U.S. Dollars per metre for a 10 m^2 tunnel, up to 1200 U.S. Dollars per metre for a 70 m^2 tunnel. Typical advances made per week (i.e. 10 shifts of 7.5 hours) would be 40- 70 metres.
The first full-face tunnelling machine was introduced in Norway in 1972, and currently 10-15 kilo metres of bored tunnel is excavated annually. A major part of these tunnels pass under urban areas and include extensive systematic pregrouting to prevent ground- water drainage and consequential damage to buildings.
Annually 4-5 million cubic metres of solid rock is excavated for tunnels and subsurface caverns in Norway. With a population of only 4 million, this gives the Norwegians an out- standing record in tunnel excavation per inhabitant. It is .hoped that some of the experiences gained through this work may be of interest to fellow tunnellers and planners abroad.
Following this brief introduction are 17 papers presenting current Norwegian underground design methods. These illustrate the utilization of subsurface space in meeting modern societies needs for shelter, storage, transport, production, recreation and environmental conservation.

GENERAL DESIGN PROCEDURE FOR UNDERGROUND OPENINGS IN NORWAY

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The general procedure followed by Norwegian engineering geologists when designing a permanent underground opening may be divided into the following stages:
I -Location of place,
II- Orientation of length axis,
III- Shaping of the opening,
IV -Dimensioning of the opening.
After brief descriptions of the use of underground space and the geology of Norway, these four stages are described in detail. It is emphasized that the first stage is of crucial importance for the cost of project. Besides the discontinuities of the rock masses, it is especially the stress conditions that in general may influence the design. The paper concludes with a belief in a step-by-step design procedure where all factors which may influence the stability of an underground opening are carefully evaluated at each stage.

THE USE OF UNDERGROUND SPACE.

In Norway underground space has been taken into use for a wide variety of purposes.
In the first half of the century railway tunnels dominated the underground workings. During the first decade after World War II a number of underground openings were made for different military purposes. An extensive development of hydro-electric power has resulted in a total of approximately 125 underground power plants, the oldest dating back to 1916. During the last two decades, underground power plants have been constructed at an average rate of 5 per year. The annual length of tunnels excavated for these hydro-power projects has been in the order of 150 km. (A comprehensive survey and descriptions of Norwegian hydro-power plants are given by VOGT and SOLEM (1968))
Construction of this great number of underground power plants, which are situated all over the country under varying geological and topographical conditions, has given considerable experience to the designers as well as to the contractors. Hence, it was only natural that underground space started to attract interest outside the circles of the hydro-power companies. This attraction to the use of underground space has increased as the cost of surface building sites has risen, and as the demand for a more restrictive use of agricultural land has increased.
Throughout Norway one can today find numerous large underground rooms for storage purposes for different products such as oil, gas, ore, flour, paints, frozen, food etc. Drinking water reservoirs, sewage plants, parking lots, factories, telecommunication centrals, swimming pools and an athletic hall are also to be found in the Norwegian underground - the last ones are dual purpose public shelters, BREKKE (1970) , BROCH and RYGH (1976) , LIEN and LØSET (1977) .

GEOLOGICAL AND TOPOGRAPHICAL CONDITIONS .

Norway forms part of the Fenno-Scandian Precambrian shield. Approximately two thirds of the country is Precambrian rocks with different types of gneisses dominating. Other major types of rocks from this era are granites, gabbros and quartzites. From the so-called "Eocambrian". period there are provinces dominated by arkosic sandstones and shales. Approximately one third of the country is covered by rocks of Cambrian, Ordovician and Silurian age. As a result of the Caledonian movements the greater part of these rocks are metamorphosed, but to very varying degrees. Rocks such as different schists, phyllites, greens tones and marbles as well as granites, gabbros, sandstones, shales, limestone and other unmetamorphosed rocks form the root zone of the Caledonian mountain range which runs through the central parts of Norway.
In the geologically unique Oslo region, the rocks are partly made up of unmetamorphic Cambro-Silurian shales and limestone and partly of Permian intrusives and extrusives. These Permian eruptives are the youngest rocks of on-shore Norway
After this rather brief description of the geology of Norway, one may conclude that the Norwegian bedrock is old -or even very old. From a engineering geology point of view, one may generally describe Norway as a typical hard rock province. Also typical for Norwegian rock types and rock masses is that they are in general anisotropic in mechanical properties and with a great variation in jointing. The rock masses have been subjected to folding and faulting during different eras from Precambrian to Tertiary. Especially the faulting may have a great influence on the stability in underground openings.
As a secondary result of the Alpine orogenesis, the Scandinavian peneplane

was uplifted about 1500 metres along the West Coast, with declining height towards the east. During the later Quaternary glaciations, the ice wore away almost all the weathered top of the rock masses, dug out weakness zones in the bedrock, and widened and deepened valleys and fjords. Thus, Norway today may be topographically characterized as a mountainous country, with young soils (less than 12.000 years) on top of almost unweathered rock.
From an engineering geology point of view, this bedrock, often exposed in outcrops, makes mapping and sampling fairly easy.
Due to glacial erosion, weakness zones, faults and gouges are in general well exposed. A complicating factor due to the steep and irregular topography, is the irregular stresses in the rock masses.
Also high tectonic and residual stresses are encountered.

DESIGN STAGES.

With knowledge about the geological conditions, the engineering geologist may be able to give advice so the construction problems for underground openings in rock may be avoided or solved in an economical way. For such works the term "geological conditions" should include:
-the type of rocks and the mechanical properties of these rocks.
-the joining of the rock masses
-the weaknesses in the zones of bedrock, (faults, gouges, crushing zones etc.)
-the stress conditions of the rock masses
-the water conditions in the rock masses
These are all conditions that may have serious influence on the construction work as well as on the later use of underground openings. Advanced equipment and construction methods are of little use if the material itself -in this case the rock masses -do not satisfy certain demands of quality, BREKKE AND SELMER-OLSEN (1966) .
The general procedure followed by Norwegian engineering geologists when designing permanent underground openings may be divided into the following stages:
A location is selected which from a stability point of view shows the optimal engineering geological conditions of the actual area.
II. The length axis of the openings are oriented so as to give minimal stability problems and overbreak.
III. The openings, including both halls and tunnels, are shaped taking into account the mechanical properties and the jointing of the rock masses as well as the local stress conditions.
IV. The different parts of the total complex are dimensioned so as to give an optimal economic solution.
As decisions in one stage will influence on the possibilities in the other stages, it is often necessary to run through the whole procedure several times. Mistakes in one stage will always give economical consequences. The extent of these may vary with local conditions and type of project. In general, however, the greatest risk for technical and economic calamities in connection with underground openings, lays in a wrong or bad solution for stage I- the location. This is important to keep in mind, as decisions on where the underground opening should be placed, all too often are made on a too early stage and on too uncertain terms.
This paper will in particular discuss design of underground openings in situations where the engineering geologist still have some possibilities of making changes or revisions of a project. Fortunately, this is very often so when storages in excavated rock caverns are in question. Furthermore, in the discussions of the different design stages, emphasis will be put on avoiding or solving the stability problems.
In unlined caverns which a going to contain fluids or gases, special demands to the permeability of the rock masses are necessary. It is a general experience in Norway that stiff rocks like granites, quartzites etc. have a tendency to give greater leakages than more deformable rocks like micaschists, phyllites etc.
It is also an experience that leakages are decreasing with increasing rock pressure. Pressure shafts and so-called II air cushions II for water power schemes , have shown that within the right rock masses and with the necessary tangential stress, it is possible to make completely unlined underground openings with internal air or water pressure of up to 50 atmospheres, SELMER-OLSEN (1970, 1974) .

SHALLOW SEATED AND DEEP-SEATED OPENINGS.

Before starting on the design procedure it is advisable to clarify whether the underground opening may be classified as shallow seated or deep-seated, as this will influence the design. The shallow seated openings are characterized by a short distance to the surface combined with low stresses in the rock masses. Under very low stresses the interlocking effect between the blocks that make up the rock mass is reduced. This may lead to difficulties in establishing the necessary arching effect for a self supporting roof.
The deep-seated openings are situated in areas where the stresses in the rock masses are so high or so anisotropic that they may locally exceed the strength of the rock or the rock masses. This will cause rock bursting, squeezing or other stress induced stability problems. Such problems may be encountered in openings situated at depths of several

hundreds of metres, but also in areas where stresses due to the topography are especially high or especially anisotropic. For instance, in steep valley- and fjord-sides stability problems caused by stresses may occur in tunnels even where the shortest distance between tunnel and ground surface is only tens of metres. In fact, rock bursting in the valley- or fjord-side itself is often observed as the picture in Fig. 1 shows (lift joints or exfoliation) .

Implicit in the descriptions of what characteristics shallow seated and deep- seated openings have, is the fact that underground openings situated at inter- mediate depths will have the best stability conditions. At such depths the discontinuous material as rock masses normally are, will exhibit its best behaviour. It is hence natural to look for such positions when a location for an underground opening is to be established.

Figure 1:
Spalling in a steep fjord-side at Simadalsfjorden, Norway, still active due to weathering. The mountain rises to 1200 metres above sea level at an approximate angle of 45°

LOCATION OF PLACE.

The most important part of the feasibility study for an underground opening is to find the right place and position. As the choice of place also decides the quality of the rock masses in which the underground opening is to be excavated, one should regard this choice as the most important during the whole design and construction period. The highest possible competence should therefore be consulted for this crucial decision. First of all it is important that certain unfavourable types of rock are avoided. In Norway soapstone, serpentines and peridotites, together with jointed tectonites with clay coated joints and fissures, have caused severe stability problems in under-ground openings.

If, on the other hand, there is a market for rock aggregates, this should be taken into account when the right location is to be found. Placing the opening in an area of the highest priced rocks may reduce the net costs of the project.

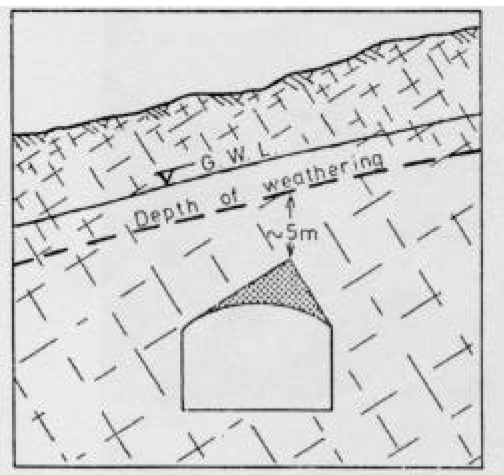

Figure 2 -Necessary rock mass over- burden for a shallow seated underground opening.

Normally, however, the situation is that a favourable location has to be found within a limited area. This may be due to the possibilities for access tunnels, external traffic, hydraulic conditions in a waterway and other more or less economically determinated conditions.

For an underground opening that may be regarded as shallow seated, the first question should then be: What rock mass overburden is needed? A general answer to this is that the opening should be placed deep enough to leave a reasonable layer of unweathered rock masses above the roof. Preferably the overburden should also be thick enough to give the normal stresses on joints and fissures which are necessary for a self supporting roof. In a hard rock province like Norway a layer of approximately 5 metres is regarded as reasonable for spans up to 20 metres when the layer is measured from what is the theoretical maximum overbreak above the roof line, see Fig. 2.

In general the weathering zone in Norway goes some few metres below today's ground water level. The depth of the zone may be measured by studying fissure- and joint- surfaces in drill cores as these will be oxidized and often covered with rust in the weathering zone. Borehole permeability tests may also be used to get information about the decreasing openness of the joints with depth.

For an underground opening that may be regarded as deep seated, the first question should be: Is it possible within the actual area to find a part of the rock mass that locally is so de-stressed that rock pressure problems can be avoided or considerably reduced? In deep valleys, areas with reduced horizontal stresses may be found in protruding "noses" or corners along uneven valley-sides. De-stressed areas are also found outside faults, gouges or crushed zones with strike parallel to the valley and dip steeply towards the valley, see Fig. 3.

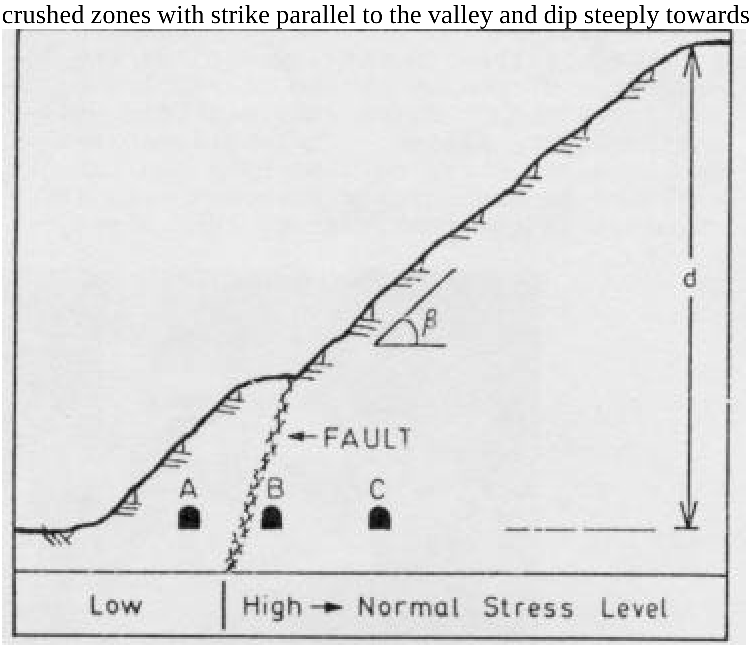

Figure 3 -Stress situation for an unde ground opening in a steep valley-side with a fault zone.
A- de-stressed area -no rock bursting.
B -highly anisotropic stress situation- intensive rock bursting.
C -normal stress situation according to the topography -moderate rock bursting.

According to Norwegian experiences rock pressure problems will, as a rule of thumb, occur in underground openings in valley-sides without de-stressing fault zones if the depth of the valley, d, is more than 500 metres and the over all inclination, B, is more than 25°. In weak rock masses rock pressure problems have been experienced in valley-sides of heights down to 300 metres, as on the other hand only minor rock pressure problems have been observed in underground openings in valley-sides of up to 800 metres when the rock is very strong. Unusual high tectonic and residual stresses are, of course, factors that may have an influence on these approximate numbers,. SELMER-OLSEN (1965).

Whatever will be the stress situation in the rock masses where an underground opening is going to be constructed, it is of vital importance to the stability of the opening that weakness zones such as faults, clay gouges and crushing zones are avoided or at least that the crossings of such zones are as short as possible.

A thorough survey of such zones is thus needed. Weakness zones in rock are usually exposed as depressions in the terrain, and aerial photos are very useful when viewed with a stereoscope as the vertical scale is exaggerated. Air photos are especially useful for the feasibility study when a large area is under consideration.

After mapping of all weakness zones in the terrain, their strikes and dips are calculated, and they are projected down to the level on which the opening is going to be. The final place for the underground opening is now at the best in an area where no such zones will cross the opening. If the space between faults, gouges or crushing zones is too small for the opening, the different zones have to be evaluated and the smallest chosen for crossing the opening. These crossings should, of course, be made as short as possible. Furthermore, it should be kept in mind that steeply dipping discontinuities in the rock masses especially influence the stability of walls, while flat lying discontinuities are a threat to the roof stability.

ORIENTATION OF LENGTH AXIS.

Just as the major discontinuities of the rock masses has a dominating influence on the choice of the location for an underground opening, the detailed discontinuity pattern is of vital importance when an orientation of the opening that gives minimum stability problems and over- break is to be decided. This requires a survey of the bedding or foliation partings, the fissures and the joints of the rock masses. It is advantageous to plot all these observations in a joint diagram as for instance the joint rosette.

For openings situated at shallow or inter- mediate depths, the basic rule is to orientate the length axis along the bisection line of the maximum intersection angle between the two dominating joint directions, bedding or foliation partings included, see Fig. 4. Close parallelity to an eventual third or fourth joint direction must, of course, be avoided. In evaluating the joint sets, not only the number and dips of the joints should be taken into consideration, but also the character of the joints, especially their friction properties. For long and high walls it is important to have an angle of at least 25 to steeply dipping smooth planes or clay filled joints.

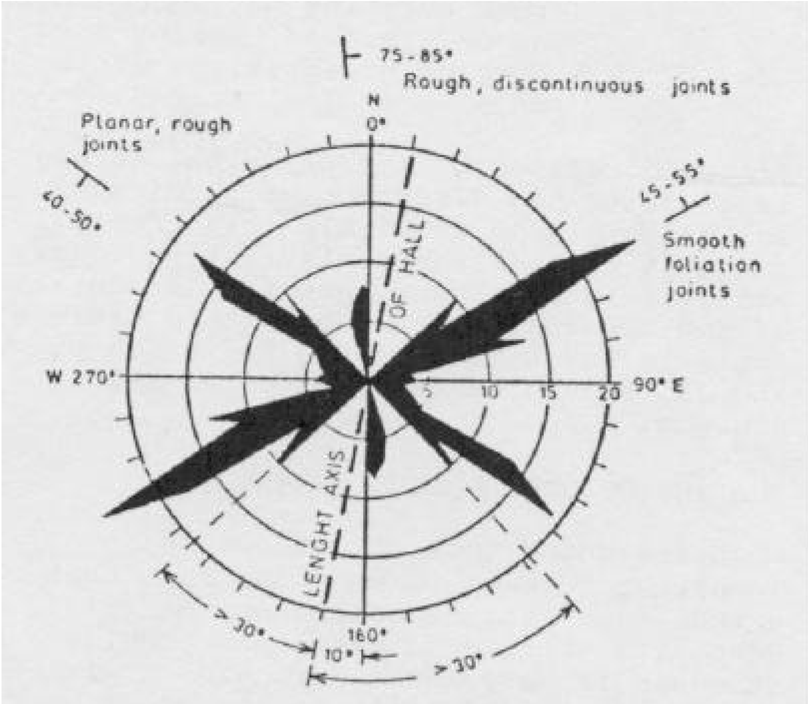

Figure 4 -Joint rosette with short descriptions. Dotted line represents the orientation of the length axis of an underground opening that. will give least stability problems.
When orientating deep seated underground openings an additional factor has to be taken into consideration, namely the direction of the major principal stress

In areas with high anisotropic stresses it is a repeated experience that the parts of the periphery of an opening which are tangential to the plane through the major and intermediate principal stress, are exposed to stress problems like rock bursting and spalling, see Fig. 5. It is thus important that the opening is oriented so that a minimum of its periphery will have such "touching" of the stress plane. The most stable orientation will, from a rock pressure point of view, be obtained when the length axis of the underground opening makes an angle of 15- 300 to the horizontal projection of the major principal stress. Once again parallelity with foliation or important joint sets should be avoided, because in an underground opening with high tangential stresses which is badly oriented with respect to important partings or joints, the overbreak may increase to two to four times what is considered as normal overbreak.

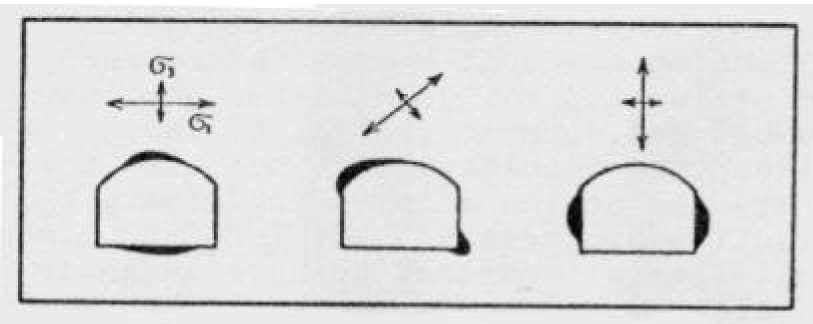

Figure 5 -The black areas show were rock bursting may occur in an underground opening for varying orientations of the major principal stress.

If the direction of the principal stress is close to the direction of bedding or foliation planes in highly anisotropic rocks as crystalline schists, flagstones, etc., it is important that the length axis of the opening is oriented with a maximum angle 5° the strike of the foliation plane, and 35 should be regarded as an absolute minimum.

SHAPING OF UNDERGROUND OPENINGS.

When designing rock structures it is important to keep in mind the fact that a rock mass is a discontinuous material. Hence its ability to withstand tensile stresses is very low. Furthermore is the stability dependent on the shear strength of the discontinuities, which in turn is a function of the normal stresses which may be mobilized on these.

A basic design concept for underground openings is therefore not only to find a favourable orientation, but also to aim at evenly distributed compressive stresses along the whole periphery of the opening. This is best obtained by giving the room a simple form with an arched roof. It is important that intruding corners are avoided. The rock masses in such corners will be in a de-stressed state, resulting in overbreak during blasting or unstable situation after blasting, see Fig. 6.

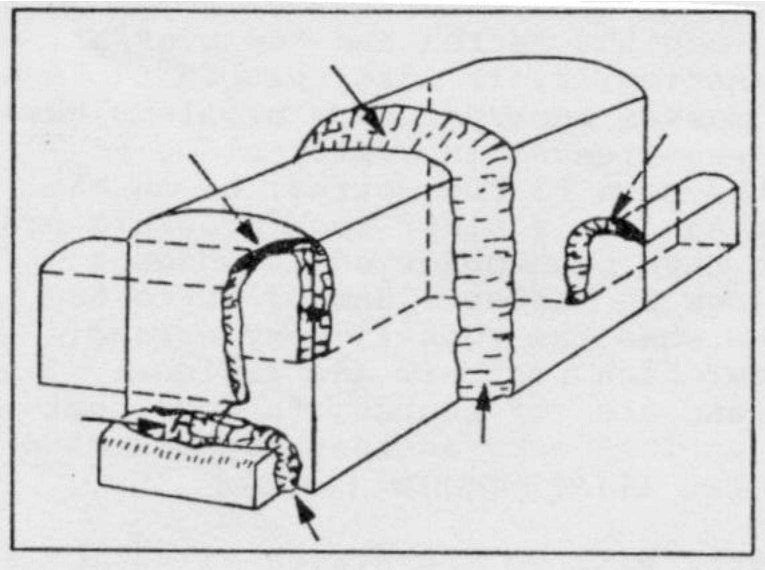

Figure 6 -Example of a wrongly designed underground opening. The arrows are pointing to unstable corners.

The shape of the roof of an opening at shallow or intermediate depth is designed taking into consideration the orientation, the number and the character of the joints and foliation or bedding partings. In horizontally bedded rocks it has been shown possible in Norway to make flat roofs along the bedding partings in openings of approximately 10 metres span if the thickness of the rock layers are 1 metre or more, and the opening has a proper orientation (demonstrated in Fig. 4) .If the distance between smooth bedding partings is less than 0,5 metre, the roof is normally profiled with a high arch. For spans greater than 10 metres the use of rock bolts in the sides of the roof is usually necessary under such conditions.

In deeper situated underground openings the tangential stresses may locally exceed the strength of the rock, thus resulting in spalling or rock bursts. If the stress level is not too high or anisotropic, it is advisable in the design of the opening to avoid small curvature radii, as these will lead to unnecessary concentrations of stresses. If, however, the stress level and the anisotropy is so high that rock bursts or spalling may be expected, it will usually be economical to give the opening a design which will concentrate the stability problems and in this way reduce the areas which have to be supported.

The design concepts based on these two different principles are shown schematically in Table 1. Note that the principles in the table imply that a stable situation for an opening with high walls is obtained when the opening is situated in rock masses dominated by moderate horizontal stresses, and the length axis of the opening is oriented normally to the direction of the major principle stress. This is also a favourable combination of level and direction of stresses when a large span for an opening is desirable.

STRESS LEVEL	DIRECTION OF MAJOR PRINCIPLE STRESS		
DESIGN PRINCIPLE	VERTICAL	HORIZONTAL	INCLINED
MODERATE EVEN DISTRIBUTION OF STRESSES TO AVOID LOCAL STABILITY PROBLEMS.	HIGH WALLS SHOULD BE CURVED TO AVOID BUCKLING.	HIGH WALLS CAN BE STRAIGHT.	ASSYMETRIC PROFILE WHEN LARGE ANISOTROPY IN STRESSES.
HIGH CONSENTRATION OF STRESSES TO REDUCE UNSTABLE AREA AND COSTS OF SUPPORT.	ROCKBOLTS HIGH WALLS SHOULD BE AVOIDED.	THE ROOFARCH SHOULD BE POINTED.	ASSYMETRIC PROFILE WITH CURVED WALLS.

Table 1. Design principlesfor underground openings in rocks with varying stress levels and with varying directions of the major principle stress when this is normal to the length axis of the opening.

DIMENSIONING OF UNDERGROUND OPENINGS.

Dimensioning of underground openings based on detailed static calculations are usually not carried out in Norway. The reasons for this being partly the problems of inserting reliable parameters of the material, and partly that a transforming of the problems from a three- dimensional to a two-dimensional situation, which usually has to be done, is not al- ways either easy or correct when the materials are as complicated as rock masses. Dimensioning is therefore to a large extent, based on rules of experience from the numerous openings situated in varying types of rock masses under varying stress conditions.

In general excavation spans for Norwegian permanent underground openings vary between 10 and 20 metres. One of the largest span so far has a sports hall with 25 metres. In mines, abandoned openings with spans of more than 60 metres can be found. Studying of the conditions in and around such enormous rooms can be of great value when the spans for permanent rooms have to be extended beyond today's experience.

The increasing interests for taking under- ground space into use have accentuated the discussion for spans of 50- 60 metres. However, although the leap from today's 25 metre span up to a future 50- 60 metre span in a permanent underground opening should be overcome from a geotechnical point of view, there is a psychological factor which should not be overlooked. Somebody has to excavate the rooms, and these persons have to be convinced that their working places are safe at any time. Such large openings will require extremely thorough engineering geological investigations and design to be carried out. Special considerations should be given to the unknown scale effects of deformations and joint spacing in relation to span width. Also a very carefully weighed excavation plan will have to be worked out and followed.

Stability problems underground are increasing with increasing excavation span. Due to the influence of the broken zone after blasting, this is especially pronounced when spans are exceeding 5 - 6 metres. The need for large spans should therefore carefully be examined.

It is preferable to meet the need for increased volumes with extensioning of the opening along its length axis. One should also keep in mind that the arch height has to be increased when the span is increased so that the curvature radius is kept constant. For many rooms this will result in unnecessary additional volume.

It is the height of the underground openings together with the quality of the rock mass and the local stress situation, that are dimensioning the thickness of walls between adjacent rooms. Under reasonable conditions with properly oriented rooms and normal demands for stability, it is a rough rule of thumb that the thickness of the walls should be equal to the height of the rooms. For low rooms, it is preferable to have some- what thicker walls (minimum thickness 5 metres) while somewhat thinner walls may be accepted for high rooms.

For stability of walls, a horizontal bedding or jointing is favourable. On the other hand, this has an unfavourable influence on the stability of the roof.

As is often the case during the design of underground openings, different conditions may have contradicting influence on different parts of the total structure. It is the designer's task through thorough collection of all information needed, and by use of all his experience, to care- fully weigh all such contradiction relations in his effort for the best and most economical solution.

CONCLUSION.

A great number of geological factors may influence the stability of an underground opening, and from case to case the different factors may be of different importance. General design formulas covering all £actors and cases are thus bound to be too complicated for practical use. On the other hand, by using simplified design formulas there is always the danger of in some cases, may be few, one important factor may be neglected. It is especially dangerous if reinforcement and supporting systems are combined with such design formulas.

So far Norwegian experiences from the designing of underground openings have lead to a belief in the outlined step by step procedure, where all factors which may influence the stability of the opening, are carefully evaluated at each stage. This procedure makes it possible to "build in" the geological conditions into the design in a way that will give the cheapest and also the safest under- ground openings.

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ENERGY ECONOMY IN SPORTS HALLS AND SWIMMING POOLS IN ROCK

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SUMMARY
In Norway, during the last decade, several dual purpose rock installations have been constructed. In time of peace they are serving as sports halls and swimming pools and in war events as public shelters. Running experiences have shown remarkable low energy consumption and maintenance costs compared to conventional installations. A sports hall built in 1972 and a swimming pool built in 1975 are referred to.

INTRODUCTION
Since the last World War, Norway has had ambitious plans in building wartime shelters for greater part of the population.
However, if the constructions can not be given a reasonable application in time of peace, this would be a very expensive, if not an impossible task.
During the last decade, several large dual purpose rock installations have been erected. In 1972 a sport hall was taken into use in the western part of Norway, and in 1975 a swimming pool in the inland north of Oslo (capital of Norway).
For the swimming pool an extensive study was carried out to elucidate the energy aspects compared to a conventional indoor swimming pool.
The calculations indicated very low energy consumption for the rock alternative.
The running experiences have been very satisfactory , and have fully confirmed the theoretical estimates.
In the following I will give a short summary of the theoretical and practical results and try to elucidate the reason why a rock installation is so favourable compared to a freestanding alternative.

ENERGY BALANCE
A conventional building is influenced by following main factors, determining the heating demand:

-Heat transmission through building structures owing to the difference between inside and out- side temperatures.
-Heating (conditioning) of ventilation air volume. -Heating (conditioning) of infiltration air volume, owing to the leakage through building structures, windows and doors.
-Solar heat gain.
-Electric illumination and motors.

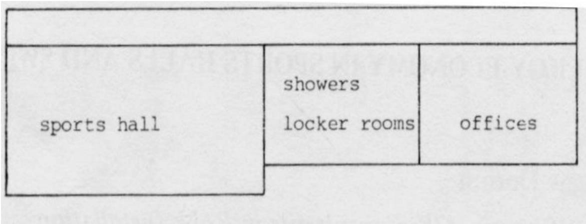
By means of modern insulation materials, it is possible to obtain very low heat transmission through walls and roofs. The windows, however, will always be a tangible problem, especially where a high indoor air humidity is desirable, par example in a swimming pool.
In a swimming pool, owing to high indoor air humidity and temperatures in combination with low environmental temperatures during the winter, special attention has to be paid to the vapour barrier in wall or roof.
Undesired ventilation owing to infiltration is practically proportional to wind velocity. In Norway, the highest wind velocities occur during the winter in combination with low ambient temperatures.
The heat loss due to infiltration can not be re- covered.
In theory it is not difficult to handle the infiltration problem. Most of the constructions suffer from unfortunate technical solutions and bad workmanship. And a present leakage has a tendency to be constantly increasing as the time elapses.
In a building with high air humidity contents and temperatures, the leakage air transports large quantities of vapour condensing into the insulation layer. Primarily this is causing a reduction of the heat resistance of the structure, secondarily the building structure is damaged.
The heat dissipation from electric illumination and motors is considered as a part of the heating system. Eventual excess heat during the day can only be partially stored, and the surplus is considered as part of the cooling load.
Rock installation
A rock installation is influenced by the following main factors determining the heating demand:
-heat flow through rock
-heating (conditioning) of ventilation air -electrical illumination
-motors and other heat sources
Owing to the insulation ability of rock, no additional insulation is necessary.
The heat flow through rock is depending on the following main factors:
-type and homogeneity of rock (heat transfer factor)

-difference between the initial rock temperature and the air temperature in the cavern
-form and dimension of the cavern.
The rock installation has Do undesired ventilation.

The ventilation system for the rock installation is similar to the conventional building. The total circulated air volume, however, will be far less because of less heating and cooling demand.
The heat dissipation from electric illumination and motors is considered as part of the heating system. Because of the accumulation ability in rock and in insulated concrete constructions, eventual excess heat in daytime is stored and automatically released at night.

ODDA SPORTS HALL

General
The first sports hall in rock was built in Odda and was finished in 1972.
The installation consists of a sports hall 25x50 m, a running track 120 x 5 m and necessary locker rooms, showers and technical rooms. In addition the local civil defence has the disposal of a separate department.
The total net floor area is 3100 m2 and net volume 19800 m3 .The installation is fit out as a shelter for 1000 persons, but may easily be extended to 2000 persons.
Climatic conditions
The calculations are based on following main data:
-Sports hall and running track 20 °c
-Civil defence department 20 »
-Changing rooms and showers 22 »
-Minimum design temperature -15 »
-Average design temperature 6.1 »
-Design wind velocity 12 m/s
-Average wind velocity 5 »



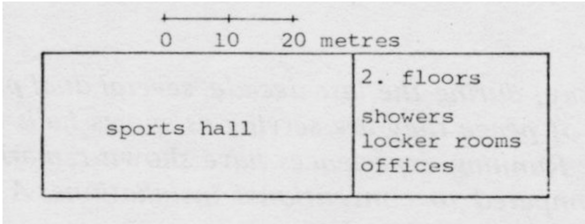


Fig. 1.(left) Conventional sporst hall with running track.
Fig. 2.(right) Conventional sports hall.

Ventilation system.
The capacity of the installed ventilation system corresponds to one air change per hour related to the net volume. The fresh air quantity is varied according to the immediate requirement. No heat recovery system is installed (in 1970- 72 the energy prices were low).
In table I the effect of an installation of a heat recovery system with 50% temperature efficiency is illustrated.

Conventional sports hall
The building is presumed to be of high standard, with walls and a wooden roof being carried by arched beams. The windows are triple pane glass in wooden frames, covering approximately 5% of the floor space.
The most important heat transmission factors are :

-walls 0.4	w/nr
-roof 0.3	»
-windows 2.5	»

Two different alternatives have been evaluated :
-A building similar to the rock alternative with an indoor running track 120 x 5 m. (Fig. 1)
-A building where the running track is omitted, and given a more favourable form in thermal sense. (Fig. 2)
Artificial cooling ought to be installed.

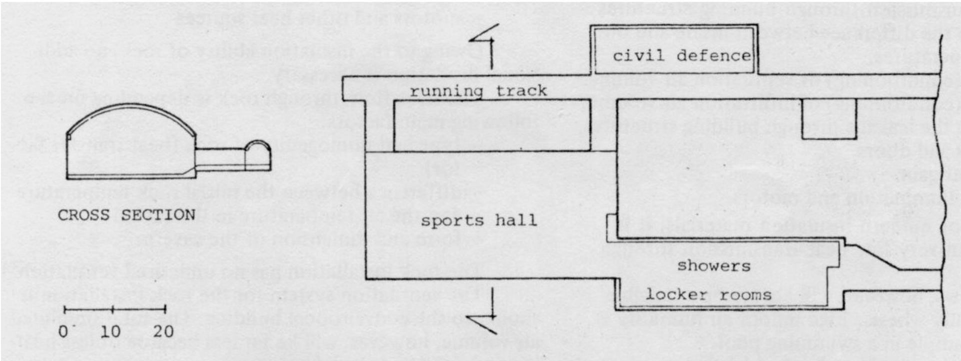


Fig.3. Odda sports hall in rock.

Sports hall in rock

The rock installation consists of one cavern containing the sports hall and service rooms, locker rooms, showers and technical rooms, and a tunnel parallel to the main hall, serving as public entrance and running track. The civil defence department is built as an extension of the tunnel.

In the sports hall and the running track, the walls have exposed rock surface. The roof is arched and made of precast concrete elements being carried by the site cast beams. The rest of the rooms have concrete walls and roof.

No additional insulation has been fitted.

The rock is homogeneous granite with thermal conductivity factor 3.5 w/m ºC.

Experience has shown that no cooling installation has been necessary even at extreme summer temperatures and with a crowded hall.

Summary

Table I shows an abstract of the calculations and the real energy consumption for 1978-79.

By installation of a heat recovery system even better proportional factors are obtained.

TABLE I

SUMMARY OF ENERGY REQUIREMENT AND CONSUMPTION

NOS	SPECIFICATION	UNIT	ROCK INSTALLATION		CONVENTIONAL INSTALLATION			
			Running track incl		Running track incl		Running track excl	
1	Floor space	m^2	3100		3050		2500	
2	Net volume	m^3	19800		19500		16800	
	ENERGY REQUIREMENT							
3	Heat recovery system	η	0	0,5	0	0,5	0	0,5
4	Transmission	kW	22	22	112	112	99	99
5	Infiltration	kW	0	0	40	40	37	37
6	Ventilation	kW	164	82	164	82	143	72
7	Design energy requirement	kW	186	104	316	236	279	208
	ENERGY CONSUMPTION							
8	Transm. and infiltration	mWh	187	187	457	457	410	410
9	Ventilation	mWh	418	209	418	209	360	180
10	Total pr. year	mWh	605	396	875	666	770	590
11	Total pr. year	kWh/m^3	30,6	20,0	44,8	34,2	45,8	35,1
	Total in 78/79	kWh/m^3	31,9					

GJØVIK SWIMMING POOL

General

The first full scale swimming pool was finished in 1975.
The installation has two swimming pools, one main pool 25 x 12.5 m and a children's pool 8 x 4 m. In addition there are a small gymnasium and necessary locker rooms and shower rooms. The total floor space, technical rooms included, is 1780 m^2 and net volume 7730 m^3 .
The installation is designed as a shelter for 1750 persons.

Climatic conditions

The calculation of heat requirement and consumption is based on following main design data:
Swimming pool:

	Alt. 1		Alt. 2
-Water temperature	24°C		28°C
-Room temperature	26°C		30°C
-Relative humidity max		65%	
Locker rooms and showers		24°C	
Sports hall		20°C	

Environmental conditions

-Design winter temperature	25°C
-Mean temperature (year)	4.2°C
-Design wind velocity	10 m/s
-Average wind velocity	5 m/s
-Initial rock temperature	8°C

Operation time

The installation is generally in operation from August 15th to June 15th, 6 days a week and 12-14 hours a day.

Ventilation system

The ventilation system for the swimming pool is designed to remove excess humidity due to evaporation from the pools and other wet surfaces. The fresh air quantity is continuously adapted to desired relative humidity in the hall.
The ventilation system for locker rooms and sports hall is designed for respectively 12 and 2.5 fresh air changes during day operation.
An installation of a heat recovery system with an average temperature efficiency factor of 0.5 is considered. Concerning the swimming pool, the factor is reduced to 0.3 at extreme low winter temperature due to freezing problems.

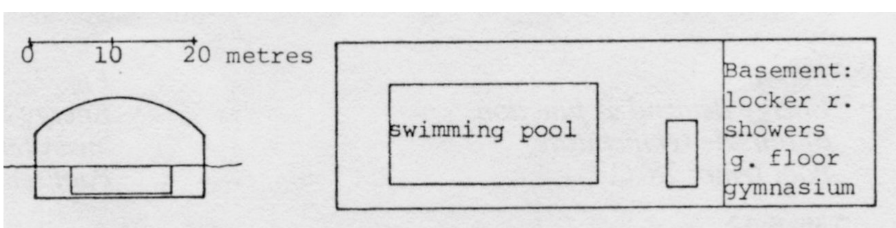


Fig. 4 Conventional Swimming Pool

The building is assumed to be well thermal insulated and of high constructional standard. The window area is limited to 5% of the floor space.
To obtain an optimal installation in thermical sense, the locker rooms and shower facilities for the swimming pool are arranged in the basement.
The main heat transmission coefficient are :

-Walls	0.35 w/m^2
-Roof	0.25 w/m^2
-Windows	2.1 w/m^2

The relative humidity in the main hall must continuously be adapted to a dew point corresponding to
the inside surface temperature of the windows. This is primarily done to avoid condensation on the windows

and to reduce the risk of damage to the building at low environmental temperatures due to condensation within the structures. (According to a study carried out by the Norwegian Building Research Institute, 70% of Norwegian indoor swimming pools has serious damages because of condensation.) The evaporation from the pools increases by decreasing relative humidity of the room air. Accordingly the fresh air volume, necessary to re- move the excess water vapour, is increasing by sinking outside temperature.

Swimming Pool in Rock

The installation consists of two caverns with common admission tunnel and connection tunnels.

The largest cavern contains the swimming pools and the gymnasium. The smaller cavern contains the locker rooms and showers for the swimming pools. Technical rooms are situated partly in the basement of the swimming pool, partly on the roof above the gymnasium and the locker rooms.

The cavern containing the swimming pools has rock walls covered with sprayed concrete and an arched roof of precast elements. The rest of the installation is freestanding concrete buildings erected inside the caverns. No additional insulation has been supplied.

The rock is limestone with a thermal conductivity factor = 2.5 w/m^2.

The heat flow was estimated as follows (based upon experience from other caverns in the same structure) :

-2.25	w/m at	12°C	temp. difference air/rock
-3.9	» »	18°C	» » »
-4.8	» »	22°C	» » »

The ventilation system is similar to the conventional alternative. The capacity, however, is approximately 65% of the former.

The relative humidity in the swimming pool is constant = 65%. At the outer entrance a dry zone is arranged.

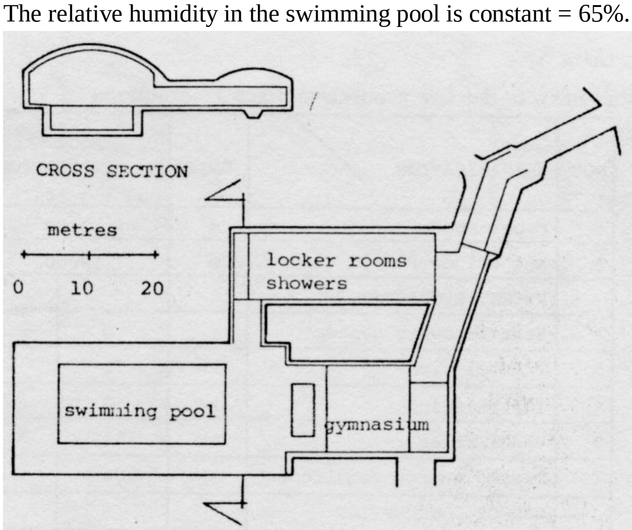


Fig. 5. Swimming pool in rock.

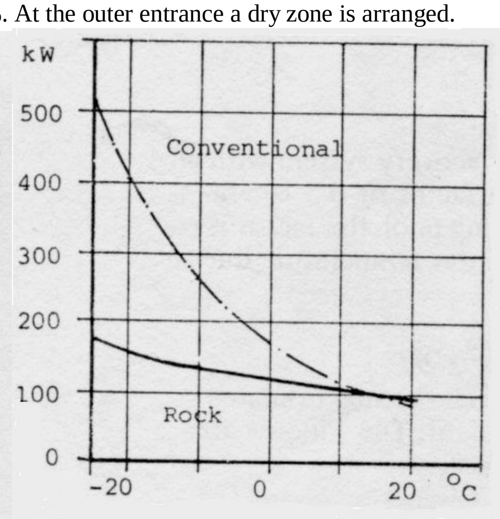
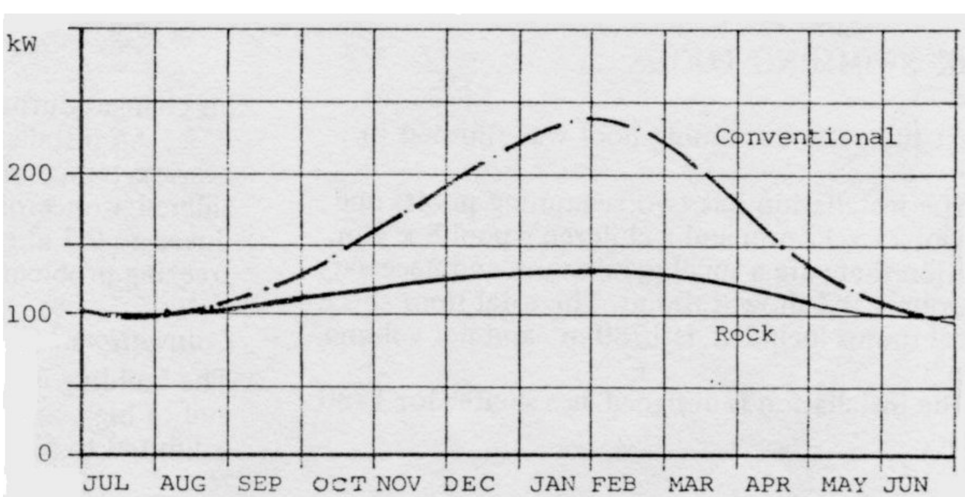


Fig. 6. Energy demand as function of outside temperatur. Pool temp. 28º.

Fig. 7. Energy demand as function of the monthly mean temperatur. Pool temp. 28º



The energy requirement as function of the outside temperatures is shown in fig. 6 and fig. 7.

NOS	SPECIFICATION	UNIT	ROCK INSTALLATION						CONVENTIONAL BUILDING					
			D	N	D	N	D	N	D	N	D	N	D	N
1	Floor space	m^2			1780						1800			
2	Net volume	m^3			7730						7800			
3	Water temp. (pool)	°C	24		28		28		24		28		28	
4	Hall temp. (air)	°C	26		30		30		26		30		30	
	ENERGY REQUIREMENT													
5	Heat recovery system	η	0,3-0,6		0,3-0,6		0		0,3-0,6		0,3-0,6		0	
6	Transmission	kW	15	15	17	17	17	17	73	73	77	77	77	77
7	Evaporation	kW	40	10	57	14	57	14	96	24	131	32	131	32
8	Ventilation	kW	92	25	102	27	197	53	244	63	295	76	468	120
9	Design energy requirem.	kW	147	50	176	58	271	84	413	160	503	185	676	229
	ENERGY CONSUMPTION													
10	Annual energy consump.	mWh	456		551		784		661		884		1095	
11	Energy consump. 78/79	mWh					736							

Table 2. Summary of energy requirements and consumption.

Table II is showing a summary of the calculations and the total energy consumption at Gjøvik Swimming Pool in 78/79 adjusted to the design mode of operation.

SUMMARY

Table I &II are showing following highlights:
Energy demand:
A sports hall in rock has a maximum energy demand for heating and ventilation less than 58%
and an energy requirement for heating only which is 19.6% compared with a conventional sports hall.
A swimming pool in rock has a maximum requirement for heating and ventilation which is 35% of the conventional swimming pool. (28°C pool temperature.)
Energy consumption:
A sports hall in rock has an annual energy consumption for heating and ventilation less than 69% and an energy requirement for heating only which is 41% compared with a conventional sports hall.
A swimming pool in rock has an annual energy consumption for heating and ventilation which is 65% of an conventional swimming pool. (28°C pool temperature.)

CONCLUSION

The considerations above show that by choosing a rock alternative following advantages are obtained:

- great saving in energy consumption
- low peak energy requirement
- saving valuable site area
- safe public shelters in war events
- low installation costs owing to dual purpose utilization

And these are the key-words for the 80's.

UNDERGROUND HYDRO-ELECTRIC POWER STATIONS IN NORWAY

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ABSTRACT

Most of the 18000 MW of hydro-electric power installed in Norway, come from plants located underground. The course of development from 1950 until today has been from steel-lined shafts in the supply system to unlined shafts and tunnels. This type of design entails significant economic advantages. The tendency has been to build constantly larger units, and power stations with installed capacity of 1 200 MW are now under construction. A total of about 150 km of tunnels are driven annually. Nearly half the cost of the power plants is accounted for by the cost of blasting alone. A description of the arrangement of machinery and equipment is given in the following, and methods of excavating underground power stations and supply systems are discussed. Pelton turbines with a capacity of 300 MW are being installed, and shafts up to 1 000 m in length are driven, using Alimak raise climbers.

Necessary geological studies, when using unlined shafts subject to high water pressure, must concentrate on ensuring safety against hydraulic fracturing. High rock stresses occur frequently in connection with Norwegian hydro-electric power plant construction.

INTRODUCTION

99 per cent of all electric power in Norway is hydro-electric power. In 1950, power stations with total installed capacity of approximately 3 GW had been built. By the end of 1979, total installation had increased to approximately 18 GW, with a mean annual production capacity of 85 TWh, or about 20000 kWh per capita. For the last 30 years, practically all installations have been located underground. 15 out of 18 GW (Fig. 1), are installed in 150 underground power stations, ranging from 10 to 640 MW each. of the 5-6 GW presently under construction, 99 per cent will be located underground, including two stations of approximately 1 200 MW each. Norwegian power stations are situated in areas under different geological and topographical conditions. Most of them are high-pressure plants with total heads ranging from 200 to 1 000 m. The plants are generally located in solid rock, however, frequently intersected by a number of distinct weakness zones. Topographical extremes - especially in the western part of Norway -also give rise to stability problems, mainly due to high rock stresses.

TRENDS IN DEVELOPING A PROJECT'S POTENTIAL HEAD

It is evident from Fig. 1 that until 1950 most of the power stations in Norway were built in the open. The policy of locating stations underground, was adopted after the second world war. This policy was mainly based on economic considerations, however, increased security against war-like acts, and less maintenance also played a part. Subsequently, importance has also been attached to conservation of the environment. Before long the course of development was such that for most projects, clear economic advantages accrued from locating power stations underground.

There are essentially three factors that have caused this outcome: The first is the expansion of the electric power transmission network, which has made it possible to transmit large quantities of electric power over great distances. The second is the progress made in blasting techniques. Last but not least, increased knowledge of the mechanical qualities of rock and rock masses has had a significant effect.

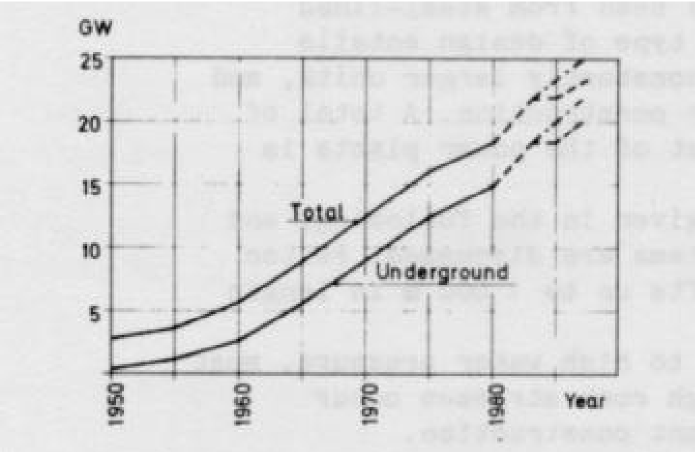

Fig. 1. Installed power from Norwegian hydro-electric power stations.
constructed
under construction
planned

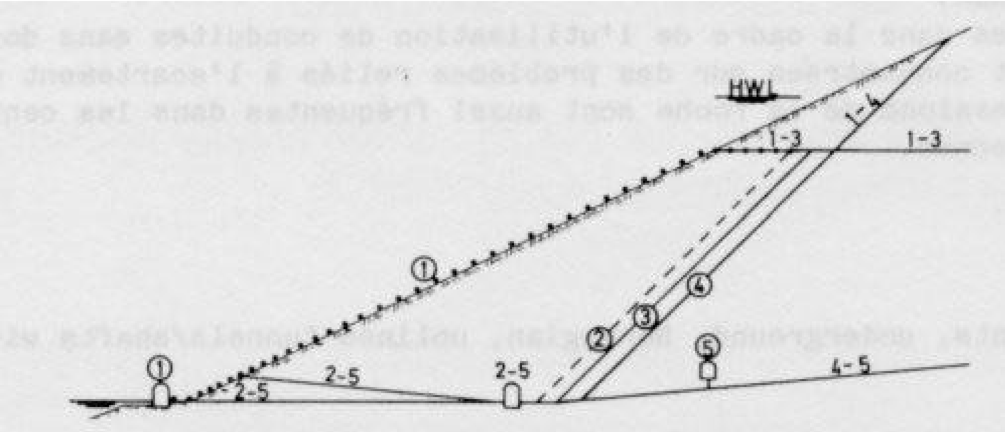

Fig. 2. Alternatives for developing a potential head.

Alt. 1 Station with penstock in the open, supply tunnel, surge shaft,
Alt. 2 Underground station with steel-lined shaft and adit at supply tunnel level
Alt 3 Underground station with unlined shaft in common.
Alt. 4 Underground station with unlined tunnel and surge shaft Supply tunnel in common.
Alt. 5 Underground station with unlined tunnel and air-pressurized Access only at station level. surge chamber

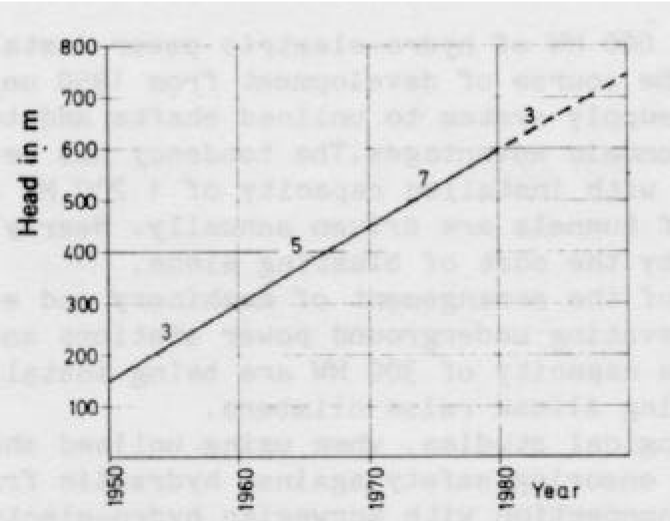

Fig. 3. Development in constructing unlined supply-systems with increasing head.
The figures above the sloping line indicate the number of projects constructed in the decade with a maximum pressure higher than the record of the previous decade.
In the last 30 years, 50 unlined supply-systems with a head larger than 150 m have been constructed, including 31 in the 70's with average maximum head of 320 m. In 1980 there are 14 under construction with average maximum head of 440 m.

These developments can best be illustrated by considering how the methods of utilizing a projects potential head (headrace-power station -tailrace) have changed in the last 30 years (Fig. 2).
Before 1950 the usual project configuration was as shown by alternative 1, with the power station located in the open. Water flowed from the intake reservoir through a nearly horizontal tunnel to a point above the station. From there the water was conveyed by penstock to the station. Outflow was channelled into a nearby river or lake. Projects could be expanded in steps, by adding penstocks and generating units as the need for power increased.
In the 50's the trend changed to locating power stations underground. The penstock was replaced by a steel-lined shaft, and the tailrace was in the form of a tunnel (alternative 2). Lyse power station near Stavanger was the first typical example of this type of lay-out. The entrance to the power station is always through a vehicle access tunnel, which also frequently carries the power cables.
Even before 1950, a couple of smaller power stations had (with varying degree of success) been constructed with the headrace in the form of an unlined tunnel, with water pressure up to 150 m. By the end of the 5~s, steel-lining was omitted in shafts with up to 300 m water pressure (alternative 3), with good results. In the 60's and 70's, this line of development has steadily been pursued (Fig. 3), so that today there are stations in operation with unlined shafts/tunnels with water pressure up to 600 m. (For cases of accidents, see the final chapter). Current plans call for pressure as high as 750 m.

The increased tunnel pressures are mainly accommodated by lowering the distribution chamber area, and inclining the tunnel towards the intake. This allows great freedom in selecting a workplace near the distribution chamber, and causes a considerable reduction in required length of steel-lined shaft. In several cases the shaft has been dispensed with entirely, with the pressure tunnel running directly from station level to intake (alternative 4). A longer surge shaft is required, but the workplace at the distribution chamber level can be omitted. This lay-out has been chosen for power stations with water pressures in the range of 600 m. The solution often entails great advantages from both economic and conservational points of view. However, in cases where there are several intakes from rivers or streams along the supply tunnel, this alternative is less attractive.

Air-pressurized surge chambers have replaced surge shafts for some power stations in the 70~ (alt. 5). The material to be excavated is thus limited to a relatively confined area, as compared to a surge shaft solution. However, in its place a certain amount of mechanical equipment has been introduced. Four power stations of this type are now in operation, with surge chamber pressure up to 50 atm. Several more are under construction, one of which will have a pressure of 75 atm.

ALTERNATIVE PROJECT CONFIGURATIONS AND THEIR COSTS

We shall now examine the economic factors that have played a part in these developments. This can best be done by presenting construction costs in kr/kW installed capacity for the various project components. (Fig. 4 and 5). The curves are based on data from projects completed during the period 1960-1979. The figures include all project expenditures until the plants are operational. Administrative costs, as well as investment tax and finance charges are added to the direct costs. (This is done for all cost figures in this article). The cost-level of January 1980 is used throughout.

The Power Station

Figure 4 illustrates the costs of power stations located underground. Costs include all permanent installations, except switchyard and cable-connection equipment. The power cavern, with the necessary tunnels for removing excavated material during construction, is also included under civil-engineering works. Costs, however, do not include access, tailrace, or cable tunnel construction.

Building expenditures constitute 20-25 per cent of total costs, whereas the cost of rock excavation, including rock support, in turn, accounts for approximately one-fourth of the building costs, or about 5 per cent of total costs. Average excavated volume in the power stations is 0,1- 0,3 m3/kW installed capacity, depending on the size of installation, and the head. With a total excavation cost of 300 kr/m3, this corresponds to 30-90 kr/kW.

Power stations located in the open as compared to underground stations, will incur higher building expenditures {foundations, walls, roof, and outdoor construction during winter). These expenditures are generally larger than the additional costs of blasting, when locating a power station underground. This is still true considering that underground stations will incur the added costs or access- and cable tunnels. As for Norwegian power stations, the length of these tunnels varies from 200 to 1 000 m, depending on head, and the choice of lined or unlined shaft.

Figure 5, curve 4, shows costs in kr/kW per km of supply tunnel. Even though the cost per metre of access tunnel, may be higher than for the supply tunnel, the resulting increase in kr/kW will be modest. Added costs resulting from more expensive ventilation and necessary cable extensions 1n the case of underground construction, will usually have even less effect.

The Penstock/Tunnel System

Construction costs are illustrated in Fig. 5 for a total head of H : 500 m. Shaft and penstock are assumed to start at elevation 100 m, and to descend at a 45 degree inclination. Costs are based on economically representative cross-sections for a typical Norwegian power station (annual operating time of 4500 hours, and an energy value of 0,15 kr/kWh). In comparing alternatives, consideration must be given to energy losses. The capitalized value of the energy loss corresponds to 20-40 per cent of total construction costs, and is proportionally larger for projects with the higher construction costs. (high costs tend to make smaller cross-sections economical). The cost of energy losses added to construction costs, therefore results in a somewhat greater difference between alternatives than the construction costs alone indicate.

The curves show that total construction costs for a penstock in the open are greater than the costs of a steel-lined shaft. The steel in the penstock costs 2- 2,5 times more per metre than a steel-lining of the same diameter. In cases of high pressures (H >200m), internal water pressure normally controls the design of the steel-lining. More than half this pressure, however, is transferred to the surrounding rock. Although civil-engineering costs pertaining to a steel-lined shaft (blasting, rails and concrete work) are twice those of a free-laying penstock (grading, trolley-line and foundation-blocks), the added costs amount to little in relation to the savings in steel.

Furthermore, the total length of a penstock is often greater than that of a steel-lined shaft. The cost of the added length is usually higher than the cost of extending the access- and tailrace tunnels in the case of an underground location. In the last 30 years therefore, only a few smaller stations (<20 MW) have been built with penstocks in the open.

The construction costs for an unlined shaft are, as seen from the curves, about half those of a steel-lined shaft. The costs are, however, more dependent on rock conditions, and vary to a greater extent than do the costs of the steel-lined shafts.

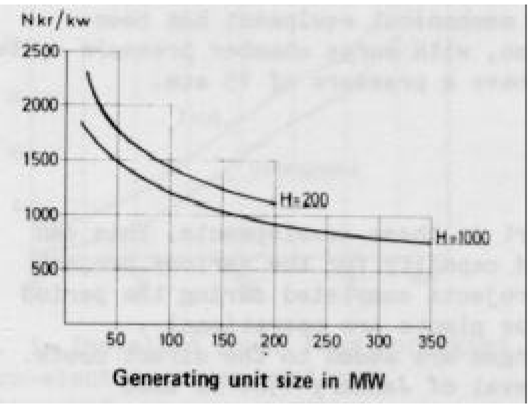

Fig. 4.
Construction costs for underground power stations. Heads H = 200 and 1000 m.
Costs in the case of several similar units:
1st. unit = 100%
2nd. unit = 85%
3rd. and 4th., each = 75%

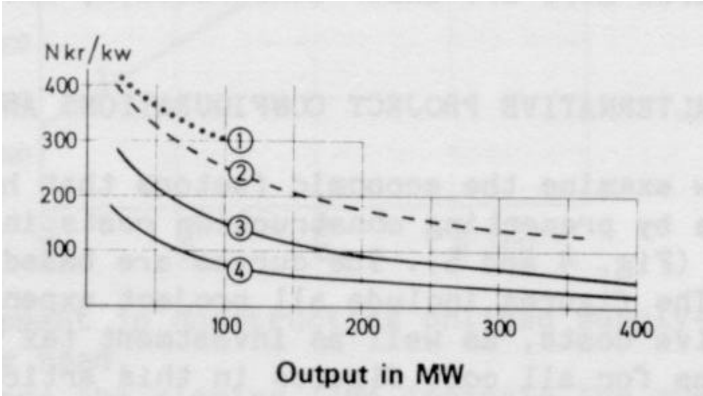

Fig. 5.
Construction costs for a head H = 500 m
1. Penstock in the open
2. Steel-lined shaft
3. Unlined shaft
4. Supply tunnel per km
all: inclination 45 degrees

Power Project Construction -An Example of Old & New Practice

A good insight into the course of development in the field of Norwegian hydro-power construction, from its inception to the present day, can be obtained by studying how a recently completed power project in western Norway has been designed. Oksla power station was brought into operation in March 1980. It utilizes a total head of 470 m from Ringedalsvatn to the sea-level, a total distance of about 4 km. The project has been built in accordance with alternative 5 (Fig. 2), with a 200 MW installation (one unit). Construction costs amount to 300 million 1980-kr., or 1500 kr/kW.

The Oksla power station replaces an older plant, utilizing the same head. The older station was built in accordance with alternative 1 shortly after World War I. It had 14 generating units with a total installation of 100 MW, 5 penstocks, and 2 headrace tunnels. Utilizing this design today, would result in costs in the order of 5000 kr/kW. There is half a century of development between the old and new approach.

In the 50's, or in the middle of this process of development, the project would most likely have been modelled in accordance with alternative 2, with a steel-lined shaft, and a need for an upper adit. This would have resulted in 20-25 per cent higher construction costs, as compared to the chosen configuration, and also larger energy losses.

GENERAL TRENDS IN HYDRO-POWER DEVELOPMENT

We have seen that the construction of the Oksla power station with one unit of 200 MW cost 1500 kr/kW. The cost-curves for all components included in a project, show a marked dimensional effect, that is, the cost per kW diminishes greatly with increased installation. Thus, the same project designed with 4 units of 300 MW each would cost approximately 1000 kr/kW. This is partly due to the fact that the electrical and mechanical engineering industry has, by research and development, succeeded in producing constantly larger units with diminishing volume and weight per kW. Furthermore, as we have seen above, solutions that provide less costly shaft-and tunnel-systems have been chosen. This effect has made it feasible to spend more on conveying water to a suitable location. The run-off from a number of watercourses is collected by means of excavating more and longer tunnels.

It can be said that cheap potential heads have made it possible to excavate a great many tunnels. It can also be said that the mechanization and rationalization of the tunnelling process has made it possible to collect water more economically than before, so that natural potential heads can be developed to greater advantage. This is a reciprocal effect. The result is that in the case of several of our larger power plants, around 100 km of tunnels with accompanying shafts, have been driven. The excavation work accounts for nearly half the total costs of a medium-sized Norwegian power project.

The maintenance costs for tunnels and underground power stations are low. The cost of operation is furthermore kept at a reasonable level through rationalization. It is the capital expenditures that constitute by far the most important part of the production costs. Altogether, administration, operation, and maintenance of all power stations built after the last war now cost on the average 0,005 kr per kWh produced.

LAY-OUT OF MACHINERY AND EQUIPMENT IN THE POWER STATIONS

As previously mentioned, modern technology has lead to the construction of fewer and larger power stations, with large generating units. These units are normally designed with vertical axles, as is the case for units installed in stations referred to in Table 1. Maximum output of both Francis and Pelton turbines has increased by 100 per cent in the course of a decade. The Pelton unites at Sima Power station, at present, hold the world record for output on a single axle.

Large Pelton turbines are equipped with 5 or 6 nozzles, and are designed to run with one or several of these closed at times when the load is low. Francis turbines are being built for constantly increasing heads, and as the number of revolutions per minute increases, dimensions are reduced. New methods of cooling the generators have been introduced, with direct water cooling of the active parts in the rotor and stator, coupled with improved air-cooling methods. This advance has reduced electric losses, and further reduced dimensions.

In arranging machinery and equipment, the aim has been to achieve a compact lay-out. The transformers have normally been placed in the same cavern as the generating units, either over the machine-hall floor {Tonstad, Fig. 6) or below (Sima, fig. 7). The Sima arrangement has led to a reduction in the required width of the hall. In several cases the transformers have been located in separate caverns parallel to the machine hall, especially when it has been advantageous to combine such arrangements with the excavation of tunnels for other purposes. Explosions and fires in some transformers in the last decade have, to a certain extent, given impetus to this solution. Moreover, a number of steps have been taken to improve precautions against, and safety during, a fire. The transformer enclosures are designed to withstand explosive loads, and explosive gases that may be formed, are led from the machine hall through separate ducts (often through the cable-tunnel)

Improvements have also been made on the equipment side to eliminate, or reduce damage. Circuit breakers are installed between generator and transformer, which immediately cut the connection to the transformer in the event of a fire. Electrical connections between generator, transfomer and junction box are gas-isolated. The necessary volume of switch gear can be reduced considerably (from 1/10 to 1/30 of that required by outdoor switch yards, depending on the voltage) by using SF6-isolated components. The switch gear can thus be placed indoors or underground, with greater operational safety and less maintenance. In the case of smaller stations, power is transmitted by cable, laid in the access tunnel. The cables are preferably separated from the tunnel itself by a concrete wall, or placed in a culvert. As for the larger stations, the cables are laid in separate tunnels or shafts.

Air for ventilation purposes is brought in through the tailrace tunnel, when free-flowing (Pelton units), and out through the access tunnel, or cable canal. Otherwise, air is brought in through the access tunnel, and out via the cable canal. The larger and more modern power stations are equipped with reinforced shelters, with their own supply of oxygen.

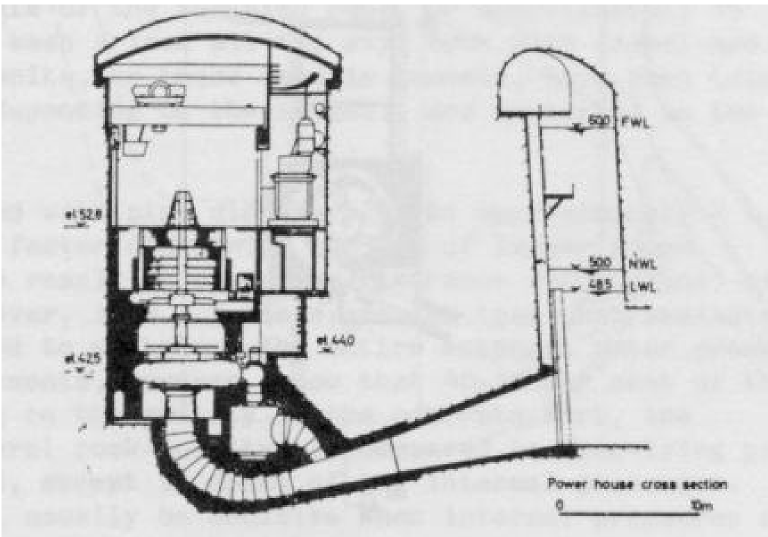

Fig. 6 Cross section throught the Tonstad power station

TABLE 1 Stations Constructed or Under Construction

Station	Project	Production (GWh/year)	Installasjon (MW)	Maximum gross head (m)	Turbine type
Tonstad	Sira-Kvina	3 800	4 x 160 = 640	448	Francis
Kvilldal	Ulla-Førre	2 900	4 x 310 =1 240	538	"
Saurdal	"		turbin = 2 x 160 pumpe/turbin = 2 x 160	ca. 465	"
Sima	Eidfjord Nord	2 800	2 x 310 2 x 250 = 1120	900 1034-1152	Pelton "
Sarp*	Glomma		1 x 80	21	Kaplan

* Partly underground

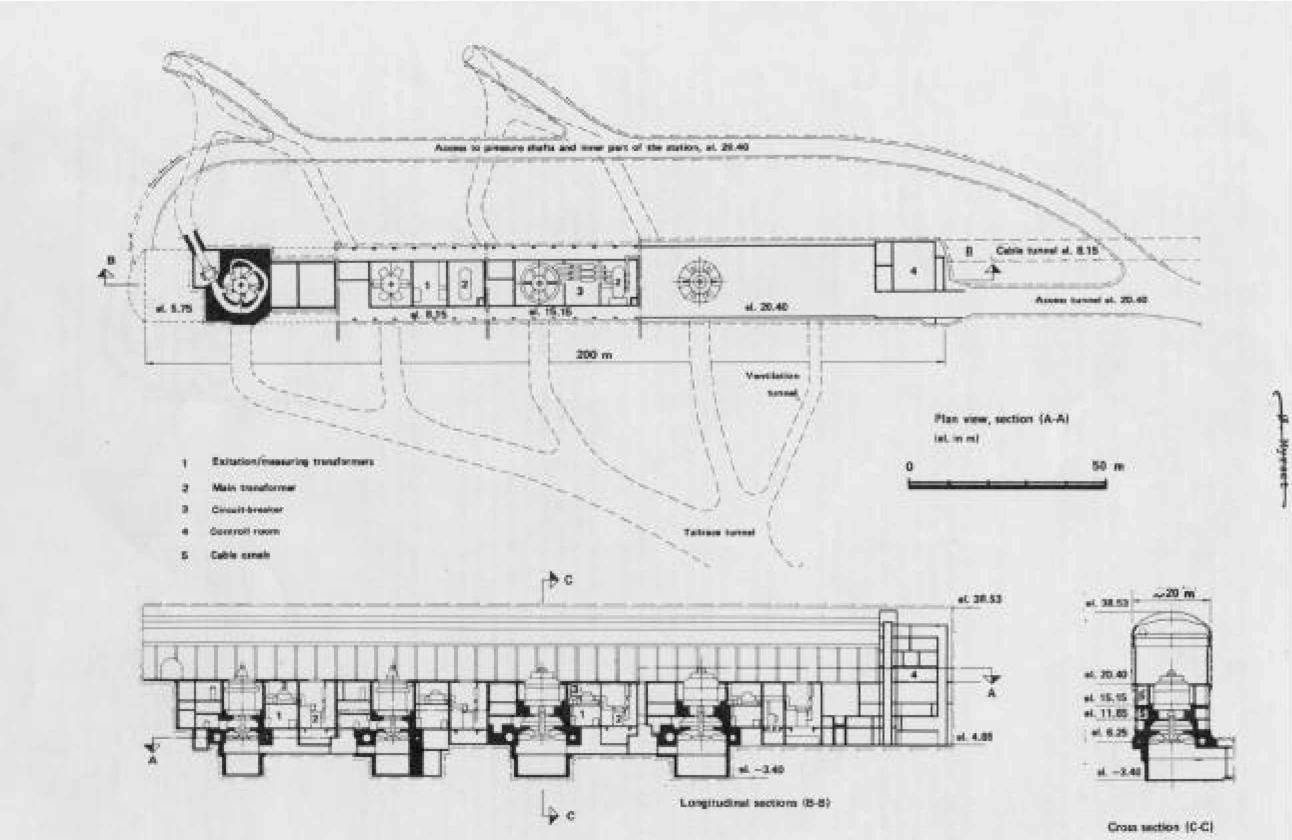

Fig. 7. The Sima power station (see table 1).

POWERHOUSE EXCAVATION AND STRUCTURAL DESIGN

Excavating the powerhouse starts with the top gallery, followed by a benching-down operation. Access to the top gallery is gained either through an extension of the access tunnel, or by driving an adit from the access tunnel level. The adit often houses machinery for the tailrace gates, and in some cases serves as part of the transformer hall. The top gallery is driven with the same rig as is used in the access tunnel, whereas the benches most often are drilled with a vertical benching rig. In benching, each bench (5- 6 m in height) is respite, using a light distributed charge along the periphery of the walls. (Fig. 8). Where rock-bursting is prevalent, the walls are systematically bolted. (Fig. 9). Loading- and transport-equipment is generally interchangeable with that used for tunnel-driving in the area.

A usual method of securing the roof arch nowadays is by systematic bolting, combined with reinforced shotcrete, 10-15 cm thick. A light ceiling of corrugated metal plates is often erected beneath the reinforced rock roof, to protect against water-drips, and in order to obtain a more attractive roof surface. The area thus created is often used as passage for ventilation purposes.

Fig 8. (left)
Åna-Sira power station with presplit walls. The station is part of the Sira_kvina project.

Fig. 9. (right)
Sima power station under construction (see table 1. The walls are systematically bolted due to rockburst.

Concrete beams for the travelling crane are usually cast in place and anchored to the walls by rock bolts. The beams are concreted following the first benching operation, ready for use by the "temporary" construction crane. The crane should preferably be operational as early as possible in the building phase. Structural design above the machine-hall floor, vary to some extent from station to station, depending on rock-quality, the tunnelling schedule, and tradition. Opinions also vary as to whether rock surfaces should be covered or exposed (Fig. 8). If covering is chosen, which is usually the case where the rock is unsound, prefabricated elements, with an acoustic surface facing the hall, are generally used (Fig. 7).

CONSTRUCTION OF LINED AND UNLINED PRESSURE SHAFTS

In the 508, shafts were frequently driven with a slope of 35-40 degrees. Excavated rock was scraped down to a loading station. Nowadays, nearly all shafts are driven with Alimak climbers, at an inclination a little steeper than the friction angle of the tunnelled rock, or approximately 45 degrees. Several shafts of 1 000 m in length have been driven all the way, both with diesel-and electrically operated hoists. Hydraulic drilling units, as those used in tunnels, have been tried. Shafts with cross-sections larger than 15-30 m2 (depending on the length), are excavated in two operations.

In Norway, steel-lined shafts have been constructed with pipe diameters up to approximately 4 m. Transport problems have usually been the limiting factor concerning the use of larger pipes. Theoretically, blasting operations are supposed to result in a minimum clearance (steel-rock) of 15-20 cm. The actual consumption of concrete, however, is 2- 3 times greater than that indicated by the minimum clearance. The steel-lining is designed to withstand the entire internal water pressure, with stresses approaching the yield point. Measurements, however, show that 40-70 per cent of the load is transferred to surrounding rock, depending on "the quality of the concrete work, the blasting, the degree of rock scaling, and the general rock-quality. As compared to free-lying pipe, the weight of steel can be correspondingly reduced, except in cases of low internal pressures. External water pressure, or minimum thickness will usually be decisive when internal pressures are low. Fine-grained materials with high yield points are used for high heads and large diameters. This makes great demands on the execution of welding-operations.

Unlined shafts with high water pressure require sound rock. Where weakness zones are encountered, these are strengthened and sealed by various methods. Reinforced concrete or shotcrete, preferably combined with injection and bolting, is frequently used. The purpose of sealing and injection is to reduce the outward directed hydraulic seepage pressure as much as possible at or near the shaft wall

AIR-PRESSURIZED SURGE CHAMBERS

A new feature in the last decade is the use of air-pressurized surge chambers in place of surge shafts, to reduce the effect of the mass inertia of the water at load rejection or admittance. As the dynamic viscosity of air is far greater than that of water (it is estimated to be 70-80 times greater under relevant temperature conditions), stringent demands must be made regarding the permeability of the rock mass. Furthermore, it is preferable that the pore pressure in the surrounding rock mass is greater than the air pressure in the chamber. As the extent of a chamber is limited, and since there is relatively great freedom of choice of locating the chamber along the pressure tunnel, all four chambers now in operation have been located in favourable, nearly impermeable rock masses. Necessary support-measures have been kept to a minimum. Attempts are made, however, to locate a chamber as near the access tunnel as possible, to reduce the distance between the instrument-house and air compressors, and the surge chamber. This reduces the length of the necessary conduits and air pipes. Furthermore it is desirable to locate a chamber near the turbine, in order to improve regulation stability.

SPECIAL CONDITIONS PERTAINING TO TUNNEL-EXCAVATION

So far, we have essentially concerned ourselves with the excavation and design of stations and shafts. However, as previously mentioned, tunnels account for by far the greater portion of the excavation-work in Norwegian power plants. Approximately 150 km of tunnel is excavated annually. Tunnels with cross-sections up to 80-90 m2 can be economically driven full face. Larger cross-sections are encountered more rarely (max. 220 m2). Equipment and methods used for power tunnel excavation do not differ significantly from what is used in excavating tunnels for other purposes. Careful blasting of the contour is required, however, in order to achieve a smooth surface. The increase in energy prices will, no doubt, cause even greater attention to be paid to this fact in the future. Measurements of head losses in a number of tunnels have shown differences in hydraulic roughness corresponding to a capitalized head loss in the order of one-fourth of the excavation costs.

Due to the relative hardness of Norwegian rock-types, full face tunnel-boring is mainly used for shafts or tunnels with small cross-sections. The increased price of energy, however, will probably encourage further use of this method in the future. It appears that full-face boring primarily is competitive in the case of longer shafts. A special challenge for planners and constructors of Norwegian hydro-power tunnels has been underwater piercing of deep lakes (about 100m), and, in the last decade, intakes of rivers beneath glaciers.

GEOLOGICAL STUDIES

We have seen above that the ability or the rock-mass to resist great water pressure in an unlined supply system, has a decisive effect on the level to which the water is to be conducted from the intake to the station area. Studies for establishing to what extent the rock mass can resist these pressures, must therefore be undertaken at an early planning stage. The first step in such a study is to ensure safety against hydraulic splitting. Here, the primary requirement is that the prevailing water pressure shall be less than the minimum principal stress at the point in Question. A rule of thumb,

that is being applied, is that the minimum rock-cover must be 0,6-0,7 times the maximum static water pressure. More comprehensive calculations are made by using the finite element method (FEM).
Concerning hydraulic splitting of the rock mass, and possible leakage, the degree of rock-jointing and faulting also plays an essential part. Detailed mapping and analysis must be made of possible weakness zones, with special attention being paid to clay materials. During construction, these studies must be followed up with pore pressure-and rock stress measurements, coupled with related rock-mechanic studies of the rock-types and any class filling-materials. Most often these studies will, and should, lead only to modest adjustments of the actual design of the supply system. An example would be the minor shifting of the transition between the lined and unlined section of the pressure tunnel. In the station area, primary attention must be paid to rock stresses in the first phase of planning. Our larger stations are usually located deep underground, in rock subject to considerable rock stresses. In order to reduce the stability problem, attempts are made to position the longitudinal axis of a station, and the adjacent tunnels, at about 15-30 degrees to the alignment of the maximum principal stress. Support measures (bolting) occasioned by rock-bursting, will then usually be of minimal extent. The optimum orientation with respect to stress may need to be modified in order to avoid paralleling major joint-sets or weakness zones, if present.

ANALYSIS OF THE CONSEQUENCES OF FAILURE

Hydraulic splitting or dislocation of the rock mass has occurred in three unlined systems constructed since the last war. All three cases concerned smaller power stations. The cracks were due partly to deficient rock cover, and partly to troublesome, improperly sealed, joint systems. In two cases the cracks occurred in the distribution chamber area, at the transition point between the horizontal tunnel and the steel-lined shaft. The prevailing water pressures were approximately 200 m, and 70 m, respectively. In these two cases the consequences of failure were relatively slight. In the third case, however, the consequences were far more drastic. A crack occurred along the whole length of the shaft, under a maximum water pressure of approximately 300 m. As a result, the Byrte power station with an installed capacity of 20 MW, was put out of operation for about a year.
In selecting alternatives, the consequences of failure must be thoroughly investigated, especially when operating with new or untried techniques. If an unlined system is chosen, one stands to gain from reduced construction costs. The head loss is also reduced, through an increase in cross-section. These gains must be balanced against the risk and consequences of an accident. In the case of a medium-sized Norwegian hydro-electric power station, with an output of 100 MW and a total head of 400-500m, construction costs can be reduced by approximately 10 million kr. by choosing unlined over steel-lined shaft. The reduction in energy losses (by using larger cross-sections) will increase the total gain to 14 million kr. However, with total power plant costs of 500 million kr., the saving of 14 million kr. is only equivalent to a few months loss in production in case the power station is put out of operation. Should conditions, however, be suitable for design according to alternative 4 or 5, the saving (compared to alternative 3) can be multiplied, and there is more to fall back upon should an accident occur.

CONCLUSIONS

Prior to 1950, most of the Norwegian hydro-power stations were placed outdoors. Since then, however, the trend has been, at an increasing scale, to build under-ground stations. For projects presently under construction, 99% of the installations will be underground. The course of development through the last 30 years, has led to utilization of larger units (plants, stations and generating units), and the supply systems are being built unlined for steadily increasing water pressures. Placing the stations underground, has reduced construction costs and energy losses as compared to outdoor stations. Furthermore, significant reduction has been made in adverse environmental effects and general maintenance.

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COMPARISON OF CALCULATED, MEASURED AND OBSERVED STRESSES AT THE ORTFJELL OPEN PIT (NORWAY)

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SUMMARY:
During the design period for the Ortfjell open pit in Rana (Norway) the stress situation was investigated by the use of finite element models. At a later stage an exploration tunnel made three-dimensional stress measurements possible. Brief descriptions of the techniques used are given. The results are compared and correlated with observations of rock bursts in the exploration tunnel. It is concluded that high horizontal stresses oriented parallel to the strike of the rock and the long axis of the open pit, may cause severe stability problemsin the benches of the foot wall.

INTRODUCTION.
In 1974 work was started for the opening of a new iron ore pit at 0rtfjell in Rana.
The pit is situated close to the Artic Circle and is operated by Rana Gruber, a division of the government owned company Norsk Jernverk (Norwegian Steelworks) .The pit is planned to be mined down approximately 500 metres and will thus be by far the deepest open pit in Norway.

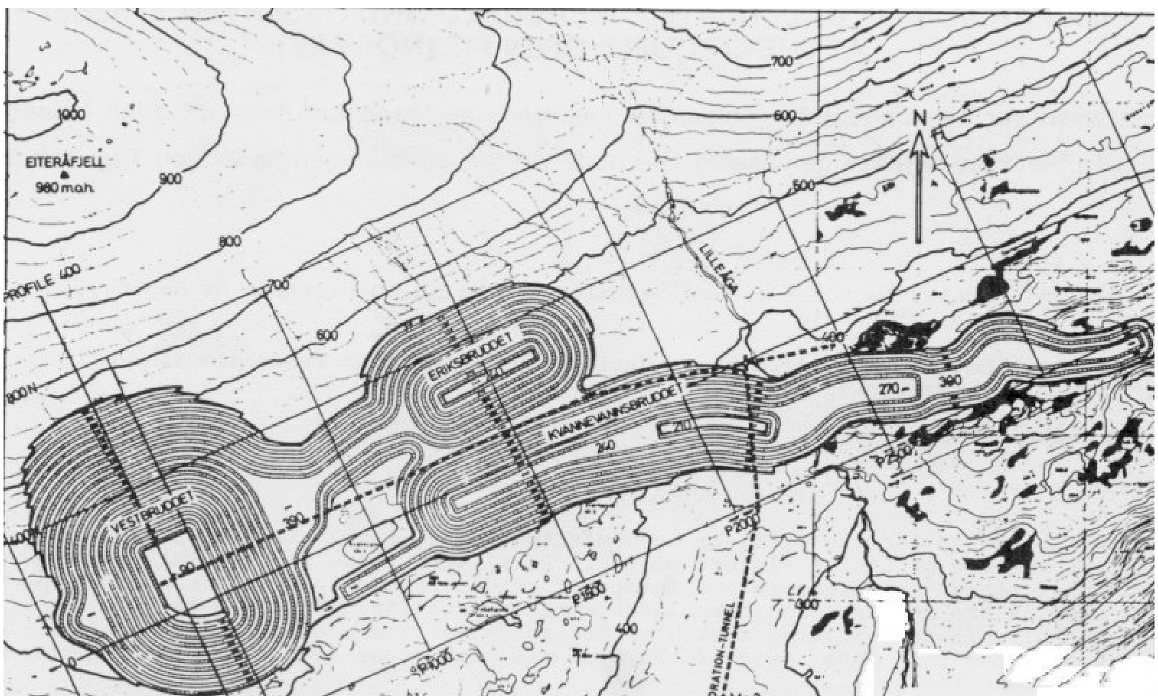

Fig 1. Map of the planned Ørtfjell Open Pit Mine in Rana, Norway. The exploration tunnel with the points stresses were measured, Mp 1 and Mp 2, is shown with dotted lines.

Fig.1 gives the dimensions, the elevation of the planned floor levels and the topography of the pit and the immediate surroundings. The mining plans include excavation of 150 mill. tons of iron ore and 380 mill. tons of waste rock.

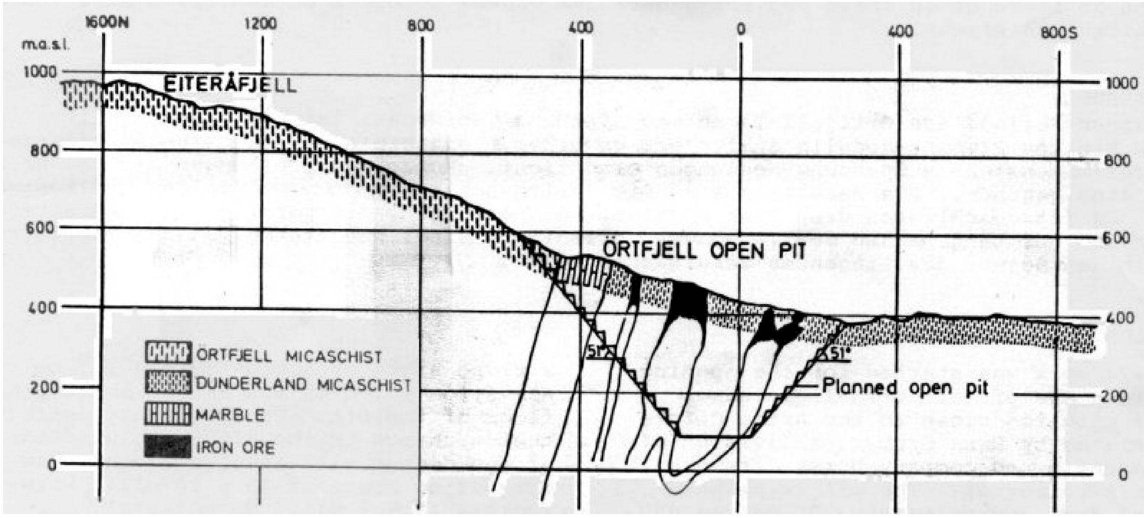

Fig 2. Cross section along profile 400 through the deepst part of the Ørtfjell open pit.

Fig.2 shows a cross section through the deepest part of the planned pit along profile 400 (see fig. 1) Mica schists of Cambro-Silurian age are the dominating rocks. As the pit is situated in the central part of the old Caledonian mountain range, the rocks are intensely folded with a number of complicated tectonical features. However, the strike of the schistosity is roughly parallel to the long axis of the pit and the dip is steep to the north. The north- wall of the pit is thus the hanging wall and the south-wall the foot wall. As the figures show the north-wall of the pit continues as a natural slope with a slope angle of approximately 25 up to an elevation which is 900 metres above planned floor of the pit. It is roughly estimated that a change in the slope angle of the pit of one degree will involve a change in excavation costs of 15- 20 mill. Norwegian kroners (3- 4 mill. us dollars).

As a starting point for the design of the Ortfjell pit the mining c8mpany chose an overall slope angle of 51° .This was purely based on experiences from the operations of nearby pits in the same geological environment where the final walls are left with benches of 13 metres width, 30 metres height (two benches) and bench surface slopes of 70.

The dimensions of the Ortfjell pit will greatly exceed the dimensions of existing pits. Thus it was felt that a closer examination of the stability of the slopes was needed. One was especially concerned of the influences high stresses might have on the north-wall. From 1974 a research project was carried out as a joint venture between the mining company and Department of Geology at the Norwegian Institute of Technology in Trondheim.

The main conclusion from the comprehensive and various stability analyses is that large and deeply cutting slides of the whole pit walls are unlikely to occur, BROCH & NILSEN [1977]. However, to avoid undercutting of schistosity planes in the foot wall, the slope angle of this will have to be somewhat reduced from the original estimate of 51° .In the hanging wall toppling may occur locally. A slight increase (2- 3) of the slope angle for this wall will have only minor influence on the toppling problems, As the savings on reduced excavation of waste rock are considerable, a slightly steeper slope is therefore now carefully considered. As an integrated part of the slope stability analyses stresses in the pit walls and the surrounding rock masses have been studied in different ways. This paper describes calculations and measurements of stresses, as well as the observed signs of stresses in an exploration tunnel. Correlations between stresses calculated by the finite element method and stresses measured with a three-dimensional borehole deformation gauge were carried out. The results have been evaluated in the light of the observations of rock bursting actually taking place. The influence of high stresses on the stability of the slopes is finally discussed.

CALCULATIONS OF STRESSES.

To get information about the distribution of stresses in the surroundings of the pit, numerical stress analyses have been carried out by using two-dimensional computerized finite element models. Fig.3 shows a simplified cross section through the deepest part of the pit with four different types of rocks. The necessary mechanical properties are given in the upper right table, i.e. specific gravity (I) , modulus of elasticity (E) and Poisson's' ratio (\1) .

Based on this simplified geological model a finite element model as shown in figure 3 has been constructed. Each element is entirely within one type of rock and is given the properties of that. As can be seen the size of the elements gets smaller close to and with- in the pit itself. This is to get a best possible approximation of the stresses in these areas. The model which is a plane strain model, contains 103 elements (260 nodal points). It allows a stepwise excavation of the lowest 200 metres of the pit thus giving information about changes in stress level as the pit is excavated.

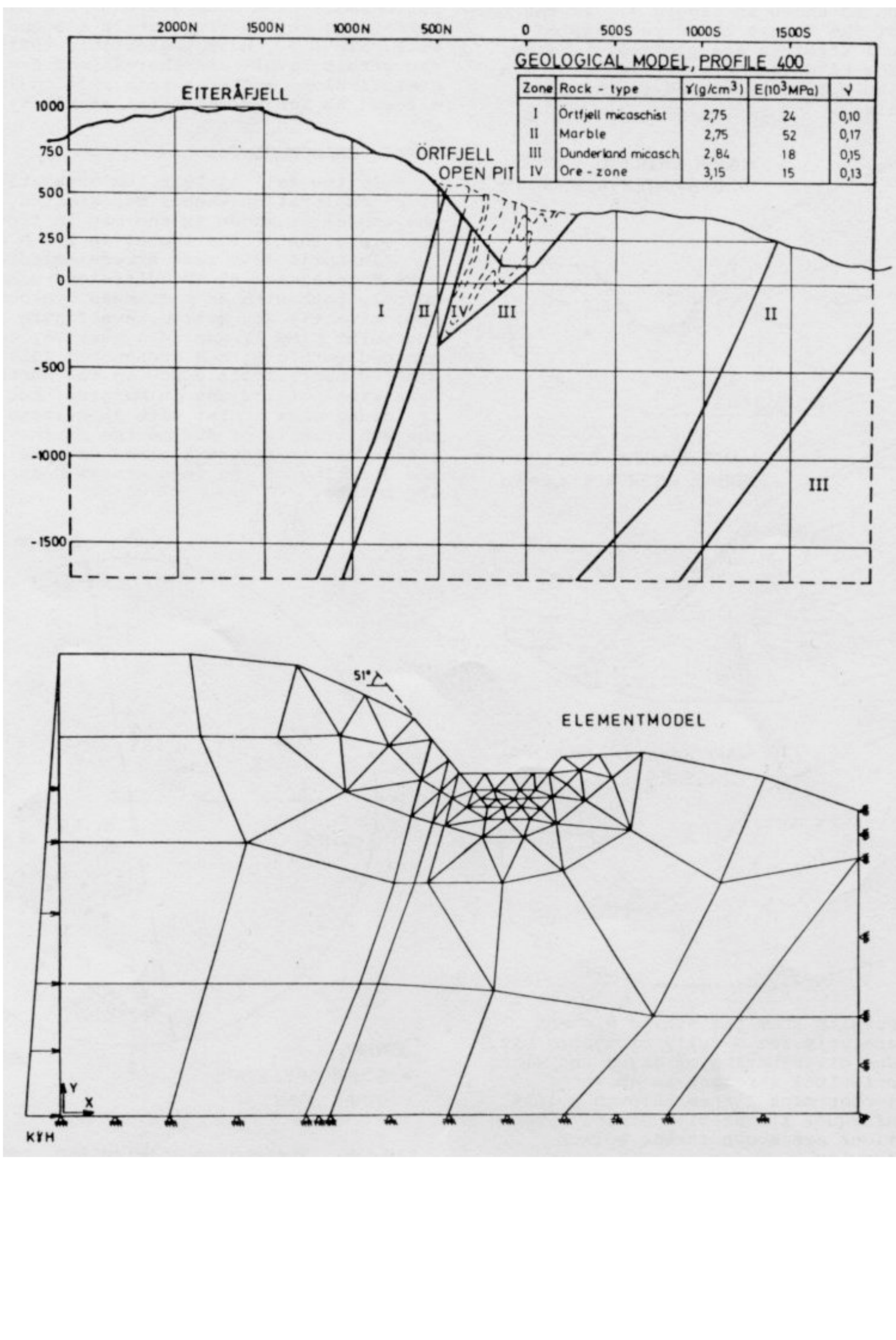

Fig. 3. Simplified geological model (above) and two dimentional finite element model (below) of the Ørtfjell open pit. (profile 400)

The nodal points at the bottom are only allowed to move horizontally, and the nodal points along the right hand side are only allowed to move vertically- The model is loaded vertically with gravity forces only, γ· H·Horizontal load is applied on the left hand side of the model and is given as K· γ·H. This will include forces resulting from elastic deformation as well as tectonic forces. The finite element analyses were carried out before it was possible to do any stress measurements (no underground access). A K- value had thus to be estimated. According to SELMER-OLSEN [1974] who has evaluated the risk of hydraulic splitting in a number of unlined pressure shafts, a K-value of 0,5 seems to be fairly common in Norway. This value was thus used in the calculations.

The finite element calculations give the stresses in all nodal points. It is convenient to plot by computer the results as isobar-maps for the major and minor principal stresses such as shown in fig. 4.In the lower part of the figure the directions of the principal stresses are plotted, lengths indicating the magnitude of the stresses. In this model the pit is excavated to the final floor 90 m.a.s.l. The analyses show that the stress level in the south-wall is just as high as in the much higher north-wall. The map in figure 4 also show that the minor principal stress may be tensile in parts of the pit walls. That is the parts outside the dotted lines. Rock masses are in general not able to take tensile stresses Hence the "tension"-parts will most likely be experienced as "no-stress" areas where the nor- mal stresses on discontinuity planes are so low that the risk for failures in the benches will increase.

The finite element analyses show, as expected, a continuous increase in stresses as the pit is excavated, the stresses near the toe of the slope always being the greatest. Furthermore, the analyses show the minor variations in the slope angle for the north- wall, 51º + 4º , have neglectable influence on the. stress level. An increase of 2 -4 of the overall slope angle in this wall could thus be allowed as far as the total stability concerns.

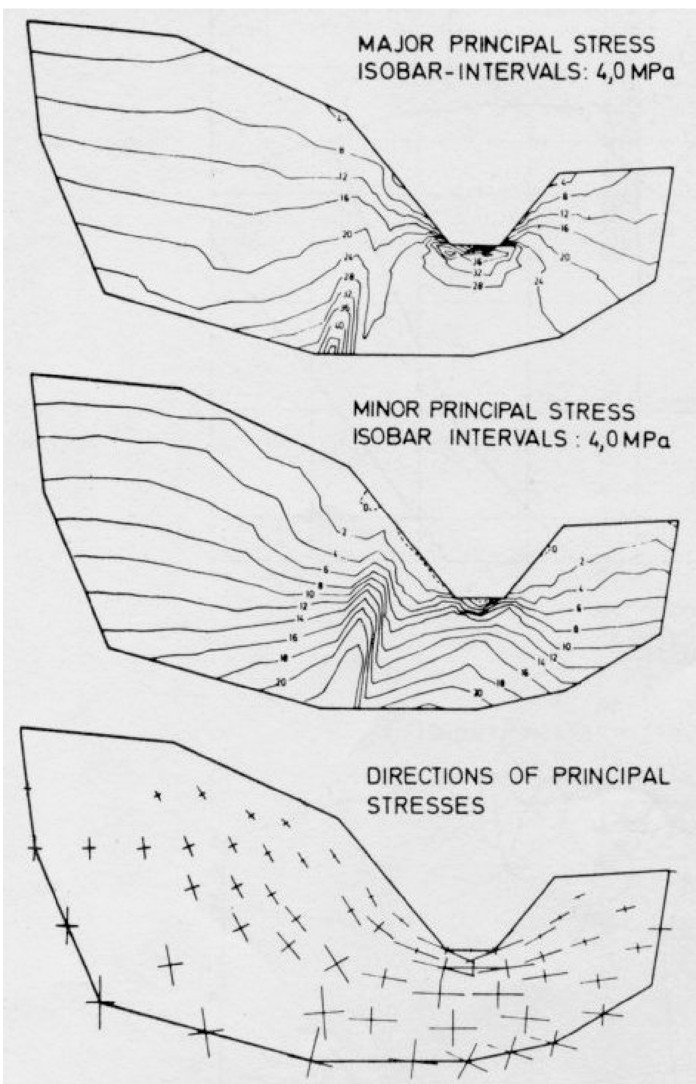

Fig 4. Results from the finite element analasys for a fully excavated pit. The distribution of major and minor principal stresses is shown as isobar maps (lines through points of equal stresses). Stress direction are shown in the bottom drawing.

STRESS MEASUREMENTS .

In the fall of 1976 the excavation of a 20 m^2 exploration tunnel was started. Part of the tunnel is shown in the map in figure I. The elevation of the tunnel is loo m.a.s.l. In April 1977 rock stress measurements were carried out at two different places in the tunnel, both with an rock mass overburden of approximately 250 metres, see figure I. Measuring point 1 (Mp I) was in a massive, coarse- grained marble with a pronounced foliation dipping approximately 30° to the north-west. Measuring point 2 was in massive, but intensely folded mica schist with an average dip of the schistosity of 300 to the north. The stereogram in Fig. 5. gives details about the jointing of the rock masses near the measuring points. The stress measurements were carried by the Rock Mechanics Laboratory of the Mining Department, University of

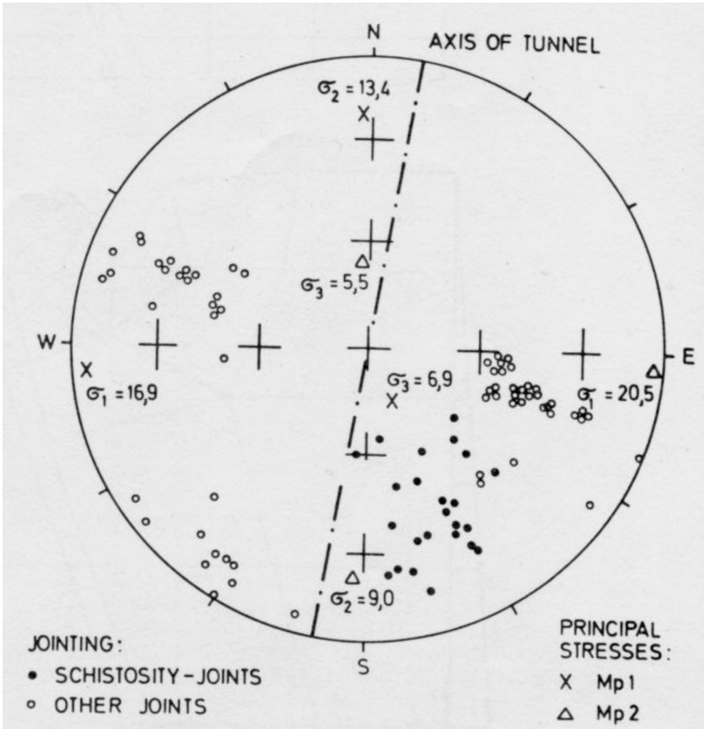

Fig 5. Stereogram showing the results from the stress measurements and the jointing of the rocks near the measuring point. (Schmidt-net, equal area projection of the lower half of a unit sphere.)

Trondheim, using their modified version of the C.S.I.R. three-dimensional gauge. Equipment and method is described in detail by MYRVANG [1976]. To get results as reliable as possible 8- 9 measurements with a spacing along the bore- hole of approx. 0,5 m were taken. The results from the stress measurements are plotted in the stereogram in figure 5, crosses indicating stresses measured in point 1 and triangles in point 2. As shown the major principal stress, δ_1 , is near horizontal and oriented in a east-westerly direction at both points. The minor principal stress is close to vertical. There are very good correlations between the measured vertical stresses and the stresses calculated from the weight of the rock mass overburden, $\delta_z = \gamma_{rock}$. z. (δ_z = 2,75 x 250 = 687,5 tons/m^2 ≈6,8 MPa) .This fact together with a minimum variation in the readings from the axially oriented strain-gauges at the same points, indicate measurements of good reliability. The stress ratios, δ 3: δ2 : δ l are roughly 1:2:2,5 and 1:2:4 for measuring points 1 and 2 respectively. This shows that there is a slight change which may be due to variations in rock type and directions of foliation. Most of all, however, the stress ratios show that the rock masses in the 0rtfjell area are subjected to high tectonical stresses. The flat lying major principal stress has an orientation which is close to parallel to the strike of the rocks and hence the long axis of the pit and the hanging wall and foot wall.

OBSERVATIONS OF ROCK BURSTS.

High and anisotropic stresses close to the surface may often cause intense sheeting and "popping" in steep valley sides, a phenomena which is easily observed, BROCH & RYGH [1976]. In the 0rtfjell area such "rock bursting" is not observed in spite of the high tectonical stresses. The reason for this is probably that the major principal stress is oriented along the very pronounced schistosity of the rocks and the schistosity is dipping steeply into the valley side. In the exploration tunnel, however, not only is rock bursting observed, but it also gives rise to severe stability problems. Comprehensive rockbolting is thus necessary. The problems started when the tunnel had been excavated some 400 metres and the overburden had reached approx. 150 metres.

The intensity of the rock bursting varies along the tunnel from just loosening of blocks and flakes of rocks to more violent and noisy rock bursting. The reasons for the variations are probably more to be found in the variations of the rock masses than in variations of the stress field.

The strength of the rocks and the jointing of the rock masses varies along the tunnel. However, rock bursting is observed in all types of rocks.

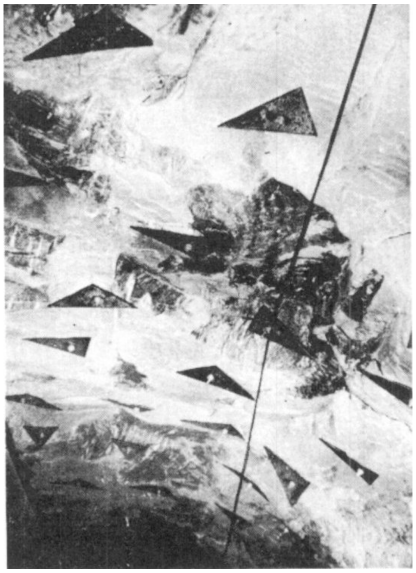

Fig 6. Rockbursting and bolting in the exploration tunnel.

Fig 6. shows an example of the rock-bursting in the east-west-tunnel. In the worst parts, such as shown in the picture, it has been necessary to install up to 10 rockbolts per metre of the tunnel. All together 13.000 rockbolts have been used on the 4.000 metres long tunnel. As the over- burden in general varies between 250 and 350 metres, the observations in the tunnel clearly indicate strong horizontal stresses in the area.

EVALUATION OF OBSERVATIONS AND MEASUREMENTS.

In the north-south-running part of the tunnel, see figure 1, the rock bursting activity with only few and local exceptions may be characterized as moderate. The bursting is concentrated to the crown of the tunnel. This is in good accordance with the stress measurements, as horizontal stresses normal to the long axis of an opening will create maximum tangential stresses at top and bottom of the opening. In the east-west-running part of the tunnel the rock bursting activity is generally higher in spite of a more favourable direction of the tunnel with respect to the direction of the major principal stress, close to parallel. The main reason for this is, obviously, the fact that the tunnel not only runs parallel to the major principal stress, but also to the schistosity of the rocks. With very weak bonds between the layers of the mica schists spalling will easily take place. An increased overburden may add to that effect. Also in this part of the tunnel the crown is where the rock bursting occurs. However, there is a tendency that the bursting takes place in the left hand (or the south) side of the crown where the roof and the wall meets. This too, is in good accordance with the measured stresses as also the intermediate principal stress is near horizontal and fairly high. An empirical method for the evaluation of rock bursting activity in underground openings has been developed by RUSSENES [1974].

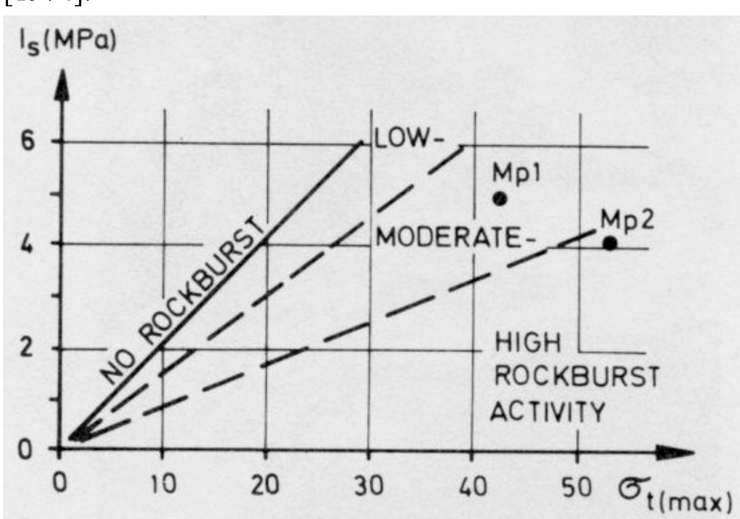

Fig 7. Rockbursting activity as a function of maximum tangential stress, $\sigma_{t(max)}$,in the pheriphery of a tunnel and the point load strenght, I_s, of the rock. I_s is measured om 32 mm wet cores drilled normal to the direction of $\sigma_{t(max)}$. (After Russenes 1974) Mp1 and Mp2 mark results from the exploration tunnel.

A number of valley sides have been modelled and the stresses calculated by the finite element method in the same way as

described in a former chapter of this paper. For tunnels in these valley sides the maximum tangential stress at the periphery is calculated as $\delta_{t(max.)} = 3\,\delta_1 - \delta_3$ (Kirsch's formula)
The shape of the tunnel is assumed to be circular and the long axis horizontal and perpendicular to the modelling plane.
32 mm rock cores are drilled normal to the direction of the maximum tangential stress and are diametrically point load tested in fully water saturated conditions. By plotting these point load strength indices against the maximum tangential stresses for approximately 45 tunnels, Russenes has succeeded in dividing the diagram into sectors with varying rock- bursting activity such as shown in fig.7.
In the diagram the results from the measuring points in the Ortfjell exploration tunnel are also plotted. In this case the calculation of the tangential stress is based on the measured principal stresses.
The points, Mp 1 and Mp 2, fall in the sectors of "moderate" and "high" rock bursting activity respectively. This is in full accordance with the rock bursting activity as it has been observed in the tunnel.

EVALUATION OF THE VALIDITY OF THE FINITE ELEMENT MODEL.
As earlier mentioned the finite element analyses were carried out before measuring of stresses was possible. Thus a number of simplifications and assumptions had to be made. The most important, but also most difficult to choose, is the ratio of the horizontal to the vertical stresses, the so-called K-value. The influence of this on for instance the tangential stresses at the toe of the pit-walls for a fully excavated pit is demonstrated in fig. 8. In this case a simplified model with uniform geology is used.
(γ= 2,8 t/m^3, E = 25.10 Mpa and v = 0,2 .
From a stability point of view the most important part of the mine is the deep, Western pit (Vestbruddet) .Hence the two- dimensional finite element model was made for profile 400 which runs through the deepest part of this pit, see figure 1. The figure also shows that the stress measurements are done close to the extension of profile 1600.

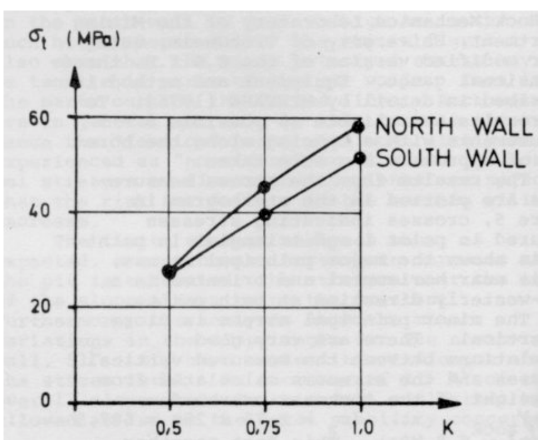

Fig. 8. Tangential stresses σ_t, at the toe of the pit walls as a function of the ratio of the horizontal to vertical stresses, K, in the finite element method.

In fig. 9 vertical sections along the two profiles are shown. As may be seen both the topography and the geology of the two profiles are almost identical. A projection of the measured stresses from profile 1600 to profile 400 should therefore be justified. The measuring points are marked with black dots. The measurements showed that the orientation of the major principal stress is almost parallel to the foliation of the rocks. Hence it will be normal to the plane of the finite element model, and thus not involved in the stress modelling. The major and minor principal stresses of the two-dimensional model as referred to in figure 4, are therefore the intermediate and minor principal stressesin the actual pit situation.
In Fig. 10 the stress vectors from the measuring points are plotted on a map of the calculated principal stresses for the pit area. One can see that there are slight variations in the direction and the magnitude of the stresses from Mp 1 to Mp 2. However, if one takes the average values, as indicated on the figure, one finds these in good accordance with the calculated stresses from the finite element model. The assumption of a K-value of 0,5 seems thus justified, and there is then

good reason for accepting the calculated stresses to be representative for the stress distribution around the planned pit.

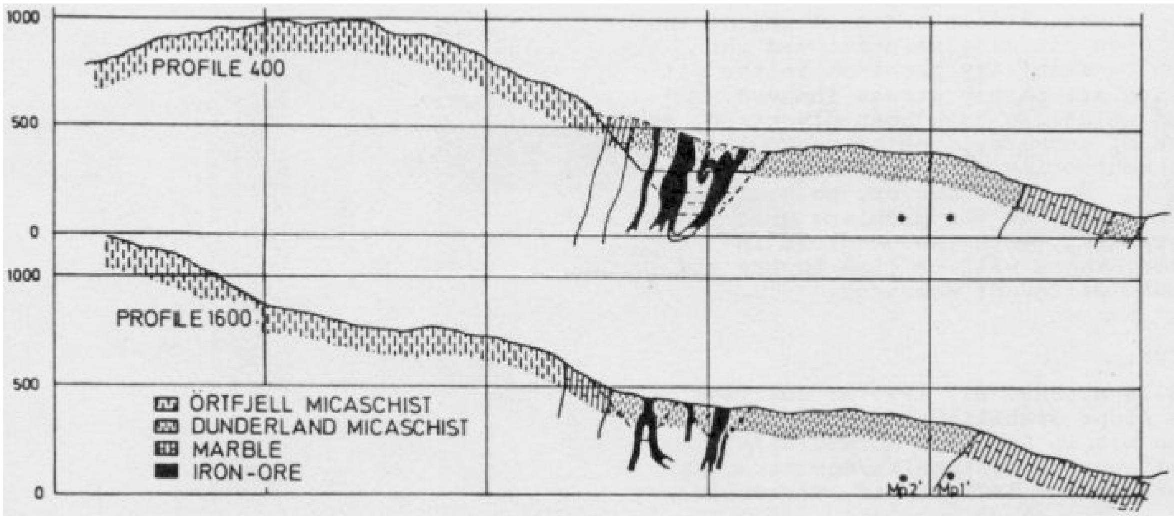

Fig 9. Section along profile 400 and 1600 of the Ørtfjell open pit.

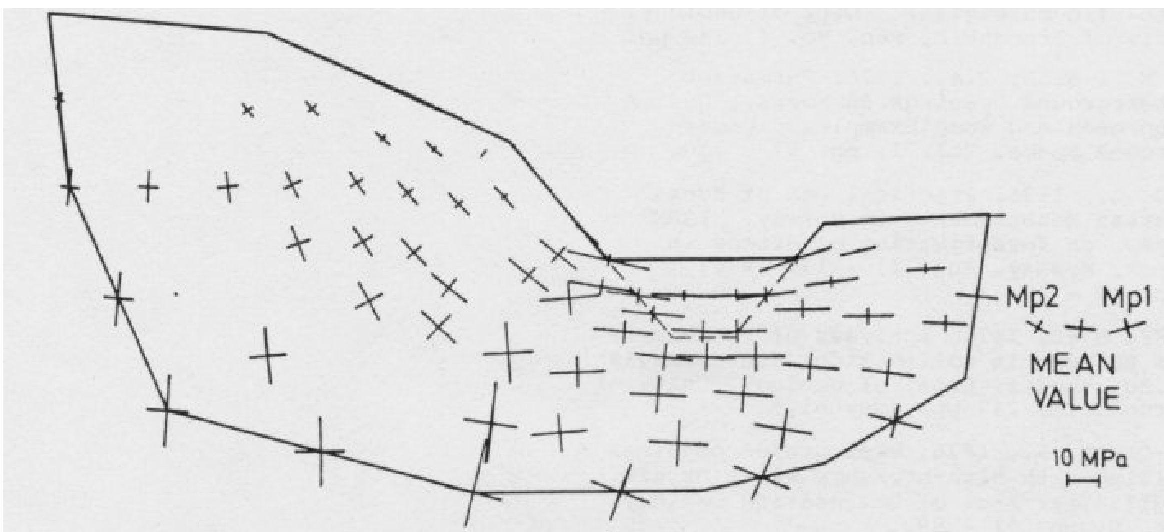

Fig 10. Measured stresses, Mp 1 and Mp 2, and their mean values plotted on the map of the principal stresses as obtained from the finite element method.

STABILITY PROBLEMS CAUSED BY THE STRESSES.

The stability analyses for the 0rtfjell open pit mine were initially started because of the uncertainties about the high north-wall. The primary concern was the stability of this wall. It was also considered important to get information about the influence of the stresses on the stability in a pit of this size.

Abnormal stress situations had earlier been experienced in a similar geological environment in the Rana-district.

Based on the results from different stability analyses one now feels reasonably confident that deeply cutting slides which damage entire pit walls will not occur. During the work with the different analyses the attention was gradually drawn towards the local stability problems, i.e. the

stability of one or a few benches. At the same time the results of the field investigations and the stress analyses have focused the attention on the south-wall (the foot wall) rather than the high north-wall (the hanging wall). In the foot wall, which eventually will be 300 metres high, it will be necessary to adjust the angle of the bench surface slope to the dip angle of the schistosity to avoid undercutting of schistosity planes. This probably means that this angle locally will have to be reduced to 60º .It also means that where the benches are parallel to the strike of the rocks, the dip of the schists will correspond to the incline of the bench. In these very schistose rocks one will thus be left with benches consisting of what may be characterized as a steeply dipping "deck of cards". The comparisons and correlations of the calculated, measured and observed stresses have clearly demonstrated the strong evidence of high horizontal stresses in the pit area. With the major principal stress almost parallel to the strike of the rocks, i.e. along the cards in the above mentioned deck of cards, it is easy to imagine how buckling of the cards (or plates of mica schists) can occur. For the deeper part of the pit and under unfavourable conditions popping or rock bursting may even be expected. In the hanging wall (the north-wall) the rocks are dipping into the slope, and thus the stability situation should be more favourable. However, if the tangential stresses are in- creasing too much, new cracks may develop. These cracks will normally be initiated at the toe of the slope. Under certain conditions they, may then combine with existing cross joints so that a sliding plane is developed, and a stability problem is created.
The investigations and analyses of the Ortfjell open pit mine have focused the attention on stability problems in the pit walls which are partly stress induced. Different solutions have been discussed, such as change of geometry, use of rockbolts or anchors, monitoring and controlled sliding, and others. So far, however, no decisions have been made. As the problems will in- crease gradually with the progress in excavation, there will be time to try and to evaluate different measures.

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A REVIEW OF NORWEGIAN ROCK CAVERNS STORING OIL PRODUCTS OR GAS UNDER HIGH PRESSURE OR LOW TEMPERATURE

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Twenty three Norwegian rock caverns are briefly described. Fourteen of these are constructed in Precambrian gneiss, seven in rocks of Cambro-Silurian age and two in Permian syenite. There are three types of caverns: Storage for liquids (oil, water etc.), storage for gases under high pressure (propane, ammonia and caverns for compressed air) and thirdly storage for food at low temperature. Most of the caverns are constructed in rocks of good quality and few stability and operating problems have occurred. The caverns are as a rule unlined, but bolting and shotcrete are frequently used as security measures. Some injection is performed to reduce leakage.
Underground constructions have been in use for a long time in Norway. This started with the mining industry in the beginning of the 17th century. Parts of our transportation network went under- ground some 75 years ago in connection with the construction of railways in mountainous areas where rather long tunnels were needed. The development of hydro-electric power further increased the use of the underground with long headrace tunnels and large machine halls. Statistics on excavations performed in Norway for the years 1971 and 1972 show as a mean yearly magnitude 175 km of tunnels, representing 3.5 mill. m^3 of excavated rock.
Storage in rock based on experiences from earlier underground constructions is the latest development in the use of underground space.

The theme "ROCK STORE 77" is understood to include caverns exposed to extraordinary inner conditions as high pressure, low or high temperature, tightness etc. Thus there are three main types of rock store to be described in this review. First there is the storage of liquids, oil and water etc.
Secondly there is the storage of gases under high pressure; for instance air, propane, ammonia etc. and thirdly there is the low temperature storage of food. Low temperature storage of gases is not yet performed in Norway.
A table is presented giving data for 23 underground plants, including rock type, critical dimensions, type of storage, pressure, temperature, lining, injection, years in operation and some remarks on experiences gained.
Fourteen of the caverns are constructed in Precambrian gneisses, seven in Cambro-Silurian rocks and two in Permian syenite. Younger rocks are practically non-existent in Norway.
Span widths range generally from 10 to 20 m, but two oil storage caverns are constructed as cylinders with diameters of 32 m and dome shaped roofs. The caverns are as a rule unlined, but bolting and shotcrete are frequently used as security measures The two domes referred to, being constructed in rather poor rock, have concrete domes of a minimum thickness of 20 cm. Steel tanks are freely installed inside rock caverns at one oil storage but due to the chemical environment (pyrite-bearing rocks) corrosion has become a problem. Experience from oil storage in rock are generally good. Only minor stability problems have occurred. The permeability is generally low in the rocks concerned and as all storages are placed underneath the ground water table it is only a question of leakage of water into the caverns. Problems might thus occur if sea-water is encountered. Another problem is leakage between adjacent chambers filled with different types of oil. A water curtain in the pillar between the chambers is used to prevent such leakage. Storage of water in rock caverns is a convenient local solution to water supply as ground water generally provides good quality drinking water. Three examples are described. The storage of gases under high pressure started with chambers for compressed air in the mining industry. two examples of which are given in the table. The latest development in this field. which is possibly unique. is the use of an unlined rock chamber half filled with compressed air. the so-called "air cushion". instead of a conventional upstream surge chamber in hydroelectric plants.
Two examples are given in the table. the air pressure being 24 and 42.5 atmospheres respectively. The plants have been in operation for more than 3 years and operate well. practically without any leakage of air out of the chambers. The table also gives data concerning three new plants now under construction. where the air cushion concept is to be used. For these plants the pressure ranges from 43 to 50 atmospheres. Experience from gas storage is limited to one single plant storing ammonia at a pressure of 7 atmospheres.-The rock at the site consists of Cambro-Silurian limestone where no support was needed, but injection has been performed. The cavern is unlined and has been in operation for more than 10 years. No leakage of gas has been registered. An underground storage cavern for propane gas at a pressure of 7 atmospheres has recently been constructed in Precambrian gneissic granite where bolting and shotcrete are used as security, measures. Injection is performed to reduce water leakage into the cavern and thus help maintain the ground water pressure outside the cavern. Another precaution is the provision of a water curtain above the cavern for injection of water into the surrounding rock at a pressure 3 atmospheres above that of the surrounding ground water pressure. This storage caverns has just been brought into operation (June 1977) and seems to operate well.
Storage of food in rock caverns at temperatures down to -30 C, seems to be a conventional solution. Three examples are given and experiences are reported to be good.

A TABLE SHOWING NORWEGIAN ROCK CAVERNS STORING OIL PRODUCTS OR GAS UNDER HIGH PRESSURE OR LOW TEMPERATURE

LOCATION	OWNER	TYPE	DIMENSIONS IN METERS (Width x height)	TEMP °C	ABSOLUTE PRESSURE MPa	ROCK - TYPE	SECURITY MEASURES	OPERAT YEAR	EXPERIENCE	REFERENCE
Mongstad Bergen region	Rafinor A/S	Oil storage	22 x 30 15 x 15	Ca 7	0,1	Precambrian meta - anorthosite	Roof: Spot bolting and shotcrete (ca. 13%) Some cement grouting	1975	Some leakage	
Ekeberg I Oslo	Ekeberg Olje-lager	Oil storage	12 x 10		0,1	Precambrian granitic gneiss with some Permian diabase dykes	Roof: Spot bolting	1969		
Ekeberg II Oslo	Ekebergtank A/S (Norol)	Oil storage	15 x 10		0,1	Precambrian granitic gneiss with some Permian diabase dykes	Roof: Extensive bolting	Under constr.		
Ilsvika Trondheim region	Norske Esso A/S	Oil storage	12 x 15		0,1	Caledonian quartz - diorite (Trondhjemite)	Roof: Bolting Grouting	1976		
Ilsvika Trondheim region	Norsk Olje A/S	Oil storage	Ø = 32 H = ca. 15	40	0,1	Caledonian quartz - diorite (Trondhjemite)	Roof: Cast concrete Walls: Cast concrete pillars	1955		
Stavanger	Norske Shell A/S	Oil storage	Ø = 15		0,1	Cambro - Silurian mica schist	Roof: Bolting + mesh reinforced shotcrete	Ca 1960	Corrosion on steel tank	
Kristiansand	Norsk Olje A/S	Oil storage	Ø = 32 H = ca. 15	40	0,1	Precambrian gneiss - granite	Roof: Cast concrete. Some bolting Walls: Some bolting	1951		
Rafnes Porsgrunn district	Norsk Hydro A/S	Propane storage	19 x 22	Ca 9	0,8	Precambrian granite	Roof: Bolting + shotcrete, partly mesh reinforced Grouting	1977		Pettersen, 1976
Herøya Porsgrunn district	Norsk Hydro A/S	Ammonia storage	10 x 10	Ca 8	0,8	Cambro - Silurian limestone	Grouting	1968	No leakage	
Driva Sør - Trøndelag	Sør - Trøndelag Elektrisitetsverk	Air cushion as surge chamber	10 x 12 (5000 m³)		4,3	Precambrian biotite - gneiss	Some cement grouting	1973	No leakage	Rathe, 1975
Jukla Hordaland	NVE Statskraftverkene	Air cushion as surge chamber	10 x 12 (6200 m³)		2,5	Precambrian gneiss with faint schistosity	Roof: Systematic bolting with spacing 1,5 m	1974	No leakage	Stokkebø, 1976
Sima Hordaland	NVE Statskraftverkene	Air cushion as surge chamber	12 x 12 (ca. 6500 m³)		5,1	Precamrian granitic gneiss with quartzite		Under constr.		(i)
Kvilldal Rogaland	NVE Statskraftverkene	Air cushion as surge chamber	16 x 21 (ca. 100 000 m³)		4,3	Precambrian quartz - dioritic gneiss with some bands of amphibolite		Under constr.		(ii)
Oksla Hordaland	NVE Statskraftverkene	Air cushion as surge chamber	18 x 13 (ca. 18 000 m³)		4,5	Precambrian granitic gneiss		Under constr.		(i)
Fosdalen Gruver Nord - Trøndelag	Fosdalen Bergverk A/S	Chamber for compressed air	Ca. 4000 m³		1,3	Cambro - Silurian schistose greenstone		1939	Some leakage	Tessem, 1976
Rausand Gruver Romsdalen	Elkem - Spigerverket A/S	Chamber for compressed air	Ca. 2500 m³		0,8	Precambrian gabbro	Grouting of a waterbearing joint	1948	Increasing leakage	Tessem, 1976
Stavanger	Statens Kornforretning	Molasses tanks	Ø = 22	Ca 35	0,1	Cambro - Silurian banded phyllite	Walls: Bolting	1974		Meisingset and Larsen, 1977
Jordalen Bergen	G.C. Rieber & Co. A/S	Cold storage	20 x 11	+ 30	0,1	Precambrian gneiss	Roof: Bolting (one per 5 m²)	1974		Bollingmo, 1975
Ekeberg Kjølelager Oslo	Oslo Fryseri A/S	Cold storage	14 x 8	+ 25	0,1	Precambrian granitic gneiss with some Permian diabase dykes		1956		Lorentzen, 1959
Gjelleråsen Oslo	Diplomis	Cold storage	10 x 5 m (10 000 m³)	+ 27	0,1	Permian syenite	Roof: Spot - bolting Local grouting	1967		
Hamar	Diplomis	Cold storage		+ 22	0,1	Cambro - Silurian sediments		1967		
Oset Renseanlegg Oslo	Oslo kommune	Water storage	13 x 16		0,1	Permian alkalic - syenite (Nordmarkite)	Roof: Bolting + cast concrete Walls: Bolting + cast concrete	1971		Sæther, 1974 Sætersmoen, 1977
Kristiansund vannverk	Kristiansund vannverk	Water storage	7,5 x 11		0,1	Precambrian gneiss	Roof: Some bolting and shotcrete	1975	No leakage Good stability	Broch and Rygh, 1976

(i) Operating year 1980. No leakage reported. (ii) Operating year 1981. No leakage reported.

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EXPERIENCE FROM COLD STORAGE PLANT IN ROCK CAVERN

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A cold storage plant in Bergen, Norway has an 11000 m^3 rock cavern as store room. It is cooled to -22ºC. The cavern has exposed rock in the roof and walls. These have been stable and dry in the two years of operation. Comparison between this plant and similar plants in free standing buildings shows that construction- and operating costs are in favour of the underground alternative for cold storage.

1. General

A cold storage plant owned by G.C. Rieber & Co. A/S, Bergen (Norway) was completed in 1974. It is located outside Bergen where the climate is fairly mild and wet. The plant is situated at the foot of a steep mountain at elevation 47 m. The mountain is inclining some 35º. There are almost no overburden deposits in the area.
The bedrock consists of Precambrian granitic gneiss.
To establish an area outside the storage room, some 20.000 m3 rock are excavated, with a 23 m high cut at the most.

2. Description of the plant

The storage plant consists of a 12 m long access tunnel and a storage cavern 51 m long, 20 m wide, and total height 10.8 m. The storage room is approx. 11000 m^3. The cavern is outlined in fig. 1.

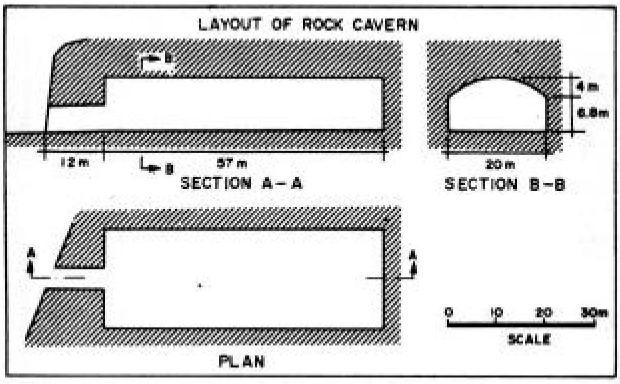

FIG. 1

A machine room is situated inside the rock cavern. Outside there are office- and service buildings, also containing some ventilation equipment for the cavern. The cavern is cooled by two electric compressors, 72 Hp each. The temperature was initially -28oc, later raised to –22ºC, which is now the operating temperature.
In the entrance, a concrete wall with gates is installed. In the cavern there is a concrete floor with rails on which racks on wheels can move, thus allowing maximum utilization of the volume. Containers are moved to and from the racks by electrical trucks.
The rock surface is exposed in the cavern roof and walls, and rock bolts are the only rock supporting means used.

3. Experience from the construction period

The orientation and design of the cavem was based on proper engineering geological pre- investigations and the blasting works were programmed and supervised in detail. No surprises or unexpected difficulties influenced the excavation.

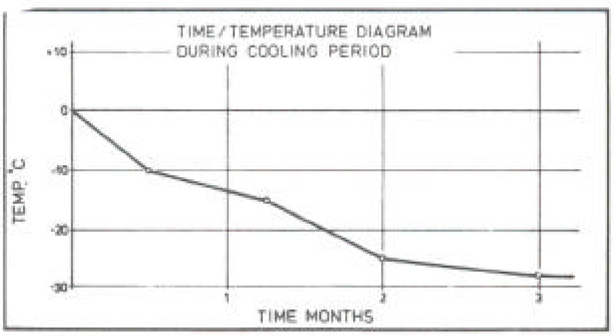

FIG. 2

As mentioned above, only rock bolts are used for rock support. Grouted bolts with length 3 m were used. In the roof a total of 305 bolts were in- stalled. (1 bolt per 5 m2). In the walls 40 bolts were installed.
Water seepage occurred as dripping at some 15-20 spots. During the cooling-down period most of the water seepage disappeared very soon, but at 2 - 3 spots the seepage increased, and for about 3 weeks there was a busy time picking and removing ice. To collect the water at concentrated spots some holes were drilled. Fans blowing cold air were placed up to these spots, and the water froze rather quickly. The air temperature in the cavern was then -14°c. The time/temperature diagram during cooling period is shown in fig. 2.
The temperature decreased rapidly to -10ºC. From -10ºC to -15°C the fall in temperature was rather slow, probably due to the water freezing on the rock surface which stopped at -14°c. From that moment, the temperature was falling more rapidly.

4. Experience from plant in operation

The temperature in the rock surrounding the cavern has been measured for 3 months by means of eleven sensors attached to electric wires installed and grouted in a 20 m long borehole. The temperature in each of the eleven points was constant thus giving the gradient as shown in fig. 3.

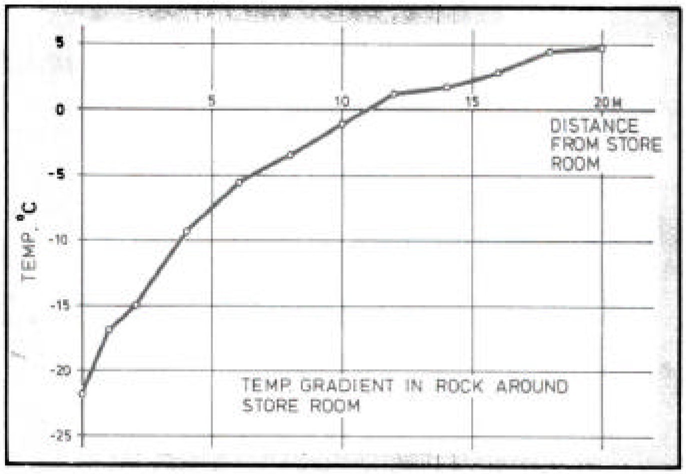

FIG. 3

The temperature in rock has also been recorded some 100 m from the store room, showing constant +4.9°C at 6 to 12 m distance from terrain surface

The temperature in the cavern varies between -22 and -23°C. There has been no need for maintenance of the walls and roof, and they are stable and dry. The only cleaning required is removal of dust from the floor at intervals. During the planning period the relatively long transportation length was considered as a draw- back. The experience is however, that the hand- ling time of the containers for loading/unloading is far the most important part of the total time consumption, and that the transportation time is a minor part of the total.

5. Construction

The construction costs are shown in fig. 4. These include the access tunnel and cavern with all constructions and installations. Service and office building is not included. The costs are per 1 January 1975, and converted from Norwegian kroner (Nkr) to US$ at a rate of 1 US$ =Nkr. 5.25. The construction costs in Norway are approximately 20% higher in 1977.

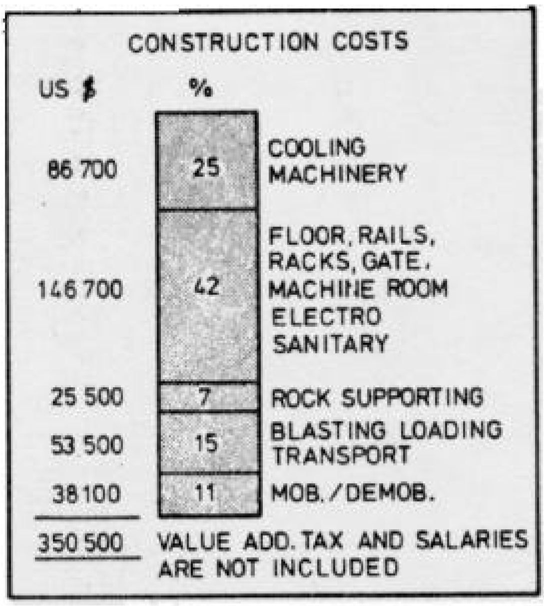

FIG. 4

It has been of interest to compare construction costs of the plant presented here, and those of a free standing store building. It is as follows (per 1.1.75):

ROCK CAVERN STORAGE

Total costs	$ 350.500.-
Total volume	l1000 m^3
COST PER M^3	$ 31.80

Approx. 35% of total volume can be utilized as net storage volume.

Net storage volume: 3850 m

COST PER M^3 STORAGE VOLUME: $ 91.

SURFACE STORE BUILDING

Information from comparable projects indicate costs of $ 62 per m^3. This does not include land- and foundation costs.

Approx. 50% of total volume can be utilized as net storage volume

COST PER M^3 NET STORAGE VOLUME: $ 124.

This comparison indicates that construction costs of an insulated free standing building are approx. 35% higher than those of a rock cavern store. Land- and foundation costs are not included.

6. Energy consumption

The operating costs have not been analysed as the number of staff, trucks etc. are being considered the same, independent of the construction method of the store. However, the electric energy consumption was expected to be favourable and data have been recorded and analysed. So far, the compressor energy consumption is approximately 45 kW 18-20 hours a day. The mean outside temperature from May to October is l1ºC, from November to April 3ºC, and the annual mean is 7°C.

This consumption is approximately 50 % of the energy required for a cold storage building of equivalent volume.

However, it should be noticed that the energy demand will be higher for deep freezing purposes than for storage of already frozen goods This demand will also be influenced by the frequency of opening and closing of gates.

7. Comments and conclusion

Safety against destruction of stored products
in case of cooling machinery breakdown or power supply failure is an advantage of underground storage. Cooling machinery may be switched off
for weeks without any critical raise in temperature.
There are no foundation problems or -costs and
no external maintenance expenses as for buildings,
Construction- and operating costs are in favour of the underground alternative for cold storage.

THE PERFORMANCE OF A HIGH PRESSURE PROPANE STORAGE CAVERN IN UNLINED ROCK, RAFNES, NORWAY

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1. INTRODUCTION

The Rafnes high-pressure propane storage cavern is situated some 200 km south-west of Oslo at the western shore of the Frier Fjord. The cavern is designed for a propane volume of 100,000 m3 at a pressure of 0.79 MPa (7.9 bars abs.) .It is excavated in Precambrian granitic rocks with its roof 90 m below sea level, and is principality unlined, except for bolting and mesh reinforced shotcrete in the roof. Fig. 1, 2.
The rocks are practically impermeable, but due to jointing and a few minor weakness zones, there was a need for grouting to prevent gas leakage and also to reduce the inflow of water.
The design criteria for the Rafnes Propane Cavern was that the hydraulic gradient towards the cavern must be greater than one. For the worst case, a vertical joint, the pressure distribution to fulfil this criteria is shown on Fig. 3.
We recognize that the design criteria of $i > 1$ is very much on the safe side. The condition for gas escaping from the cavern is not only dependent upon the gradient, but also on the joint width, the orientation of the joint and the capillary forces (Aberg 1969 and 1978 and Noren et al. 1970). Furthermore, it was important to maintain the groundwater level as high as possible.

2. DURING THE EXCAVATION PERIOD, BOTH CEMENT GROUTING AND WATER INFILTRATION KEPT THE GROUNDWATER LEVEL REASONABLY HIGH.

Excavation of the access tunnel started in February 1975. During the excavation inflow of water was reduced by cement injection. during May the 5 observation wells drilled above the cavern showed a general fall in the ground water level. At this stage it was decided that the water infiltration holes had to be bored. (Fig. 4) .The first 4 holes were put into service during the last week of May. In July the ground water level was at its minimum, observation well no. 7 showed a ground water level of below -20 m (see Fig. 5)

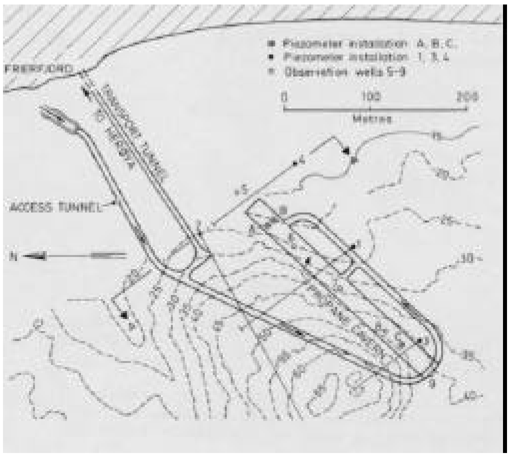
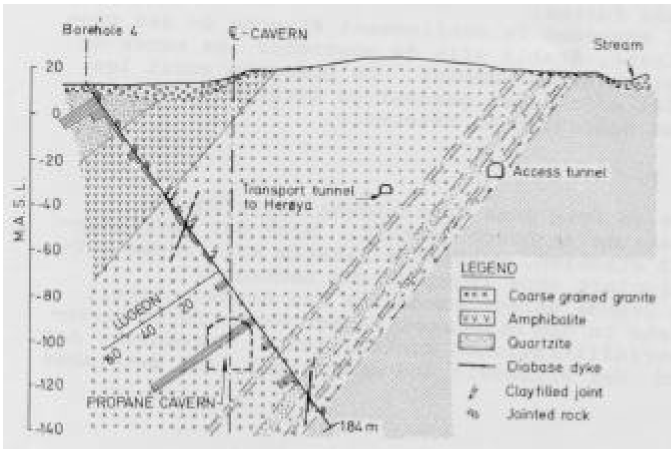
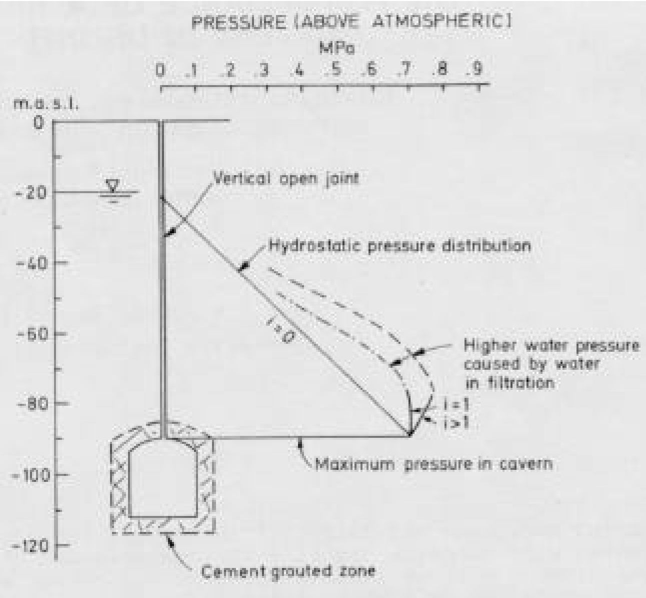


Fig. 1 (left) Plan view of propane cavern with observation wells and piezometer installation.

Fig. 2 (right) Cross section A-A, through bore hole No. 4.



By August 1975, 26 water infiltration holes were supplying the surrounding rock mass with approximately 200 litres/min.
The ground water level now rose to about the same level as measured before the excavation started. By October/ November 1975 the first grouting of the cavern was finished. The cavern was completely excavated by May 1976. The water infiltration system then consisted of 34 holes giving a total of 380 l/min. of water. The ground water level was still about the same as before the construction period. A detailed description of the excavation works is given by A. Pettersen (1976) .



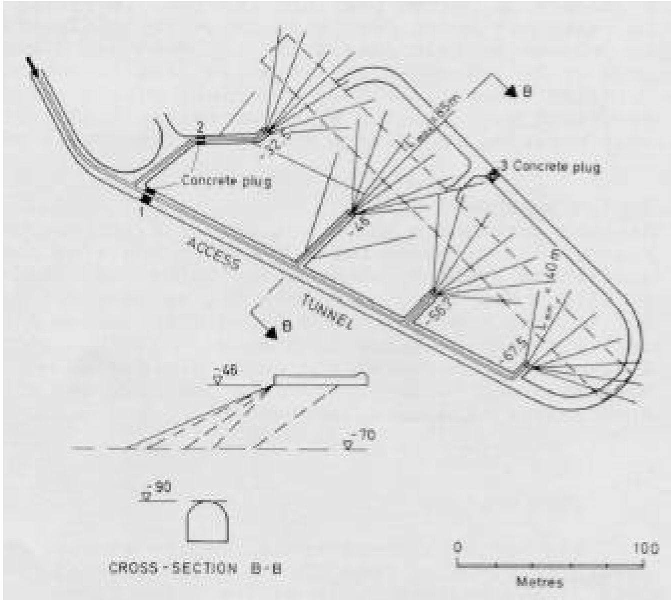
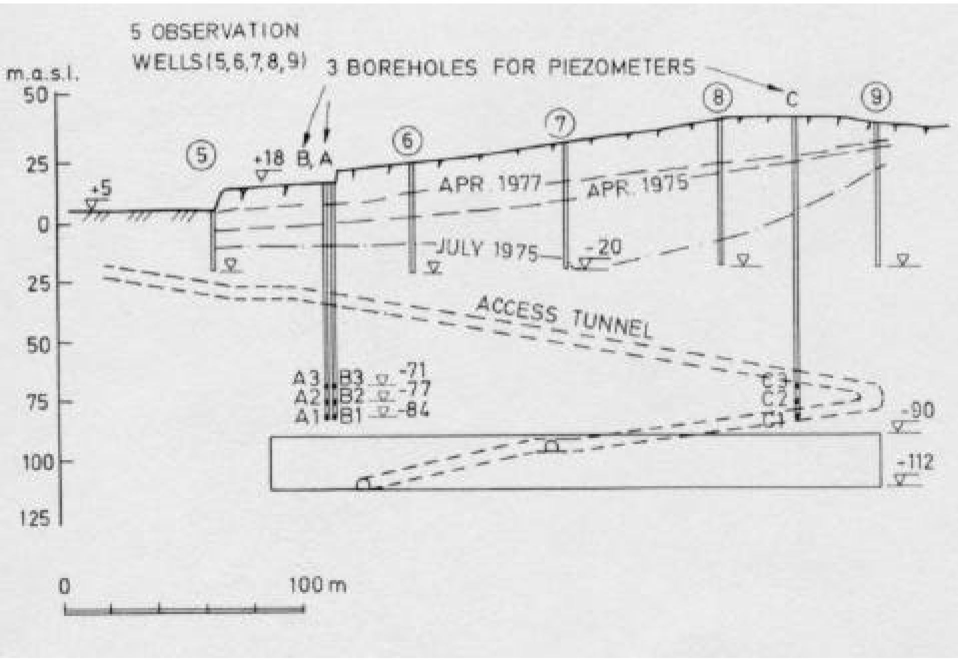


Fig. 3(left) Pressure distribution on a vertical joint.
Gradient, i=0; no water flow, i> 0; inflow of water (no gas leakage).

Fig. 4 (right) Plan view of water infiltration holes. 36 holes, diameter 2 ½ inch. Total lenght 2110 m. Infiltration pressure 0,6-0,7 Mpa. pumped volume; 350 l/min. (July 1977)

3. INSTALLATION OF PIEZOMETERS TO MONITOR THE GROUND WATER FLOW GRADIENT.

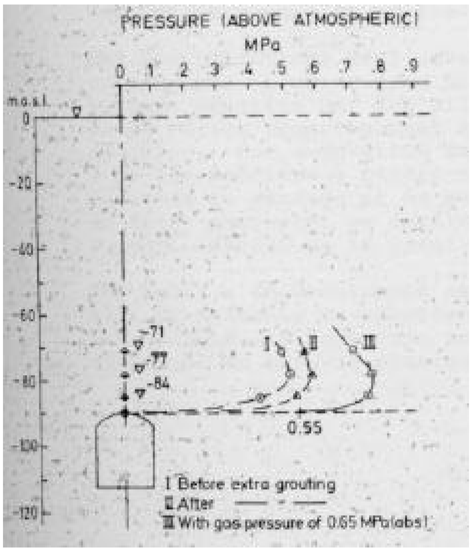
In November 1976, piezometers were installed in three boreholes (A, B and C) at three different levels (6, 13 and 19 m above cavern roof) for the purpose of observing the gradient. just above the cavern (Fig. 5) .



The positions of these piezometers were carefully chosen based on the "thickness" of the injected zone around the cavern and permeability tests in the boreholes before installation. Borehole B was placed in the most permeable zone giving relatively high inflow into the cavern, borehole A was placed just outside this permeable zone and borehole C in a less permeable zone at the other end of the cavern.

Fig. 5 Longitudal section through cavern with piezometers and observation wells.

The water pressure in the 34 water infiltration holes could be raised by means of a reduction valve if the pore pressure measured gave a lower gradient than anticipated during the test period.



4. EXTRA GROUTING

The piezometers installed in borehole B showed relatively low pore pressures due to rather high local water inflow into the cavern at this point. It was therefore decided to do more grouting in this location. The effect of this extra grouting can be seen in Fig. 6.

Fig. 6 Pressure distribution in bore hole B.
I Before extra grouting.
II After extra grouting
III With gas pressure of 0.65 MPa (abs.)

5. FILLING OF ACCESS TUNNEL WITH WATER

During April/ May 1977 the access tunnel was put under water pressure between the concrete plugs 1 & 3 (Fig. 4) .Also the gallery behind concrete plug No.2 is under the same pressure as the access tunnel. The transport tunnel to Herøya was also filled with water (up to +/- 0 m) .

6. TESTING OF THE CAVERN

On the 13th May, 1977 nitrogen gas was pumped into the cavern. By 4th June the pressure had reached 0.79 MPa (abs.) .During this period the water pressure at A1, B1, and Cl, 6 m above the cavern roof, rose between 0.21 and 0.36 MPa. Fig. 7 shows the gradient between level 1 (6 m above the roof) and the cavern roof, and the gradient between level 2 (13 m above) and the cavern roof.

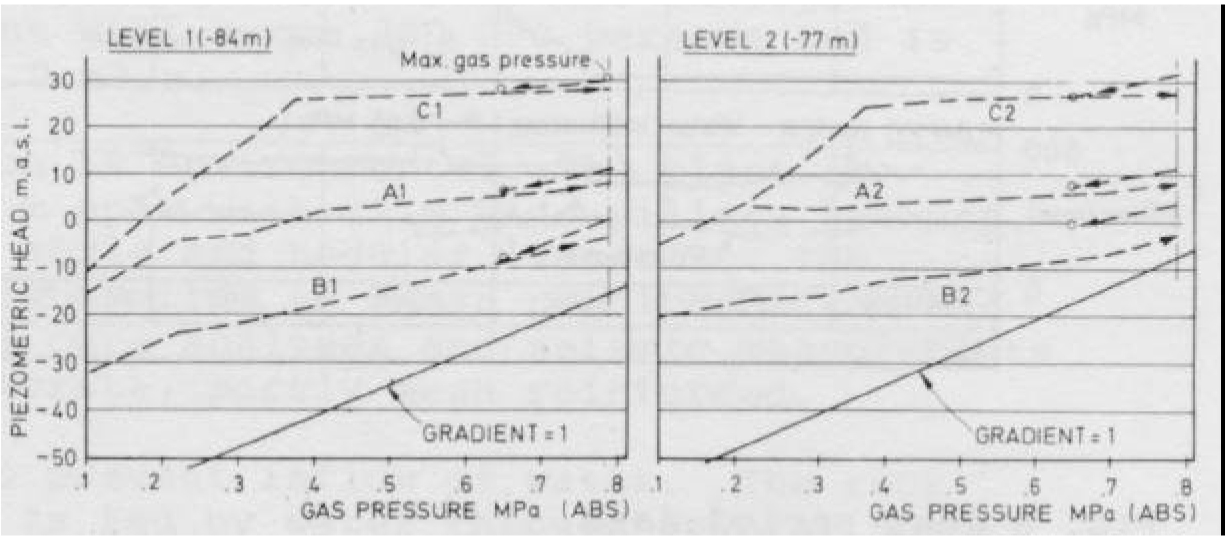


Fig. 7 Piezometric head of water vs. gas pressure in the cavern for level 1 and 2 at A, B and C.

During the test period the water level in the observation wells did not change appreciably. It seems that the ground water level is more sensitive to rain fall than changing pressure in the cavern. This is probably due to the water infiltration system which has an excess pressure of 0.65 MPa at the level of :t ° m. The quantity of pumped-in water in the system did not change much during the pressure test nor did the inflow of water into the cavern.

Gas pressures and temperatures were measured very accurately in the cavern when the maximum pressure of 0.79 MPa (abs.) was reached. These measurements were carried out during a 4 week period as the inflow of gas was closed. The conclusion of this test was that no leakage of gas had occurred.

After this test period the gas pressure was lowered to 0.65 MPa (abs.) and propane gas was pumped in while the nitrogen was driven out (see Fig.8) .

The water infiltration system was shut off between the 28th and 29th July 1977. During the 24-hour period the pressure at Al, B1 and C1 fell 0:12, 0.05 and 0.06 MPa respectively. It seems that the communication between the infiltration system and Al is better than for the other two piezometers.
On the other hand piezometer B1 immediately changes when the pressure in the cavern changes and this shows that the zone between level 1 and cavern is much more permeable at B than near the other two piezometers.
During the remainder of 1977 and in 1978 the pressure in the cavern has been about 0.65 MPa (abs) .Fig. 8 shows the piezometric levels of Al, B1, and C1, observation well 6, quantity of pumped water, rainfall, and pressure in the cavern during the test period. No leakage has been observed or measured during this time.

7. ACKNOWLEDGEMENT

The authors are grateful to the owner of the Rafnes Plant, Norsk Hydro A/S, for giving them the opportunity to write this paper by kindly putting at their disposal data from the test period, and data from the operation of the storage cavern.

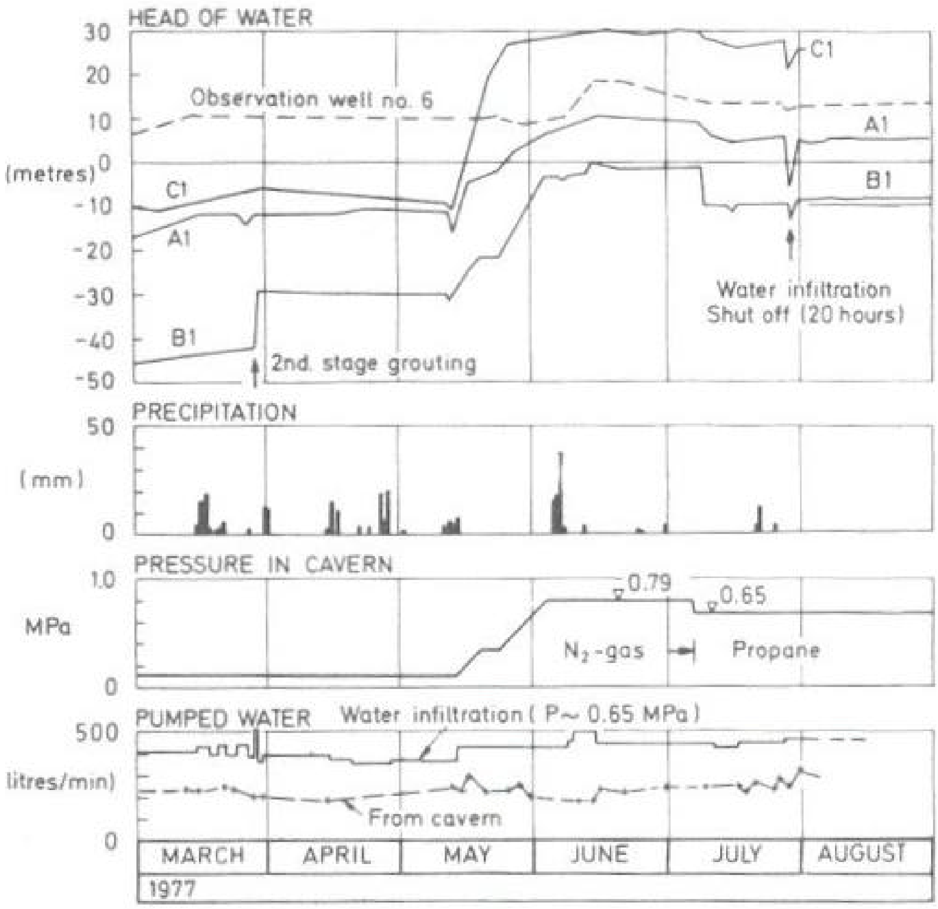


Fig. 8 test period data.

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SUBSURFACE LOCATION OF A MAJOR WATER POLLUTION CONTROL PLANT PRESENTLY UNDER CONSTRUCTION IN THE OSLO AREA

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SUMMARY
The largest sewage treatment plant in Norway is being constructed underground at Bjerkås, 30 km south of Oslo. The plant will serve 300000 persons and is designed to treat a dry weather flow of 3.0 m^3/s.
The main reason for the subsurface location is environmental. The plant is being placed in 11 parallel caverns of 16 m span with 12 m wide pillars between. The rocks in the area are Cambro-silurian shale and nodular limestone. The conditions revealed by excavation were very similar to those predicted from the combined results of surface mapping, drill core analyses and seismic measurements The support is systematic bolting and shotcrete, partly mesh reinforced.
Some injection work has been undertaken to prevent inflow of water. The rock mass underlying an industrial area nearby is fed by water through several bore- holes to prevent lowering of the ground water table in the overlying clay.

INTRODUCTION

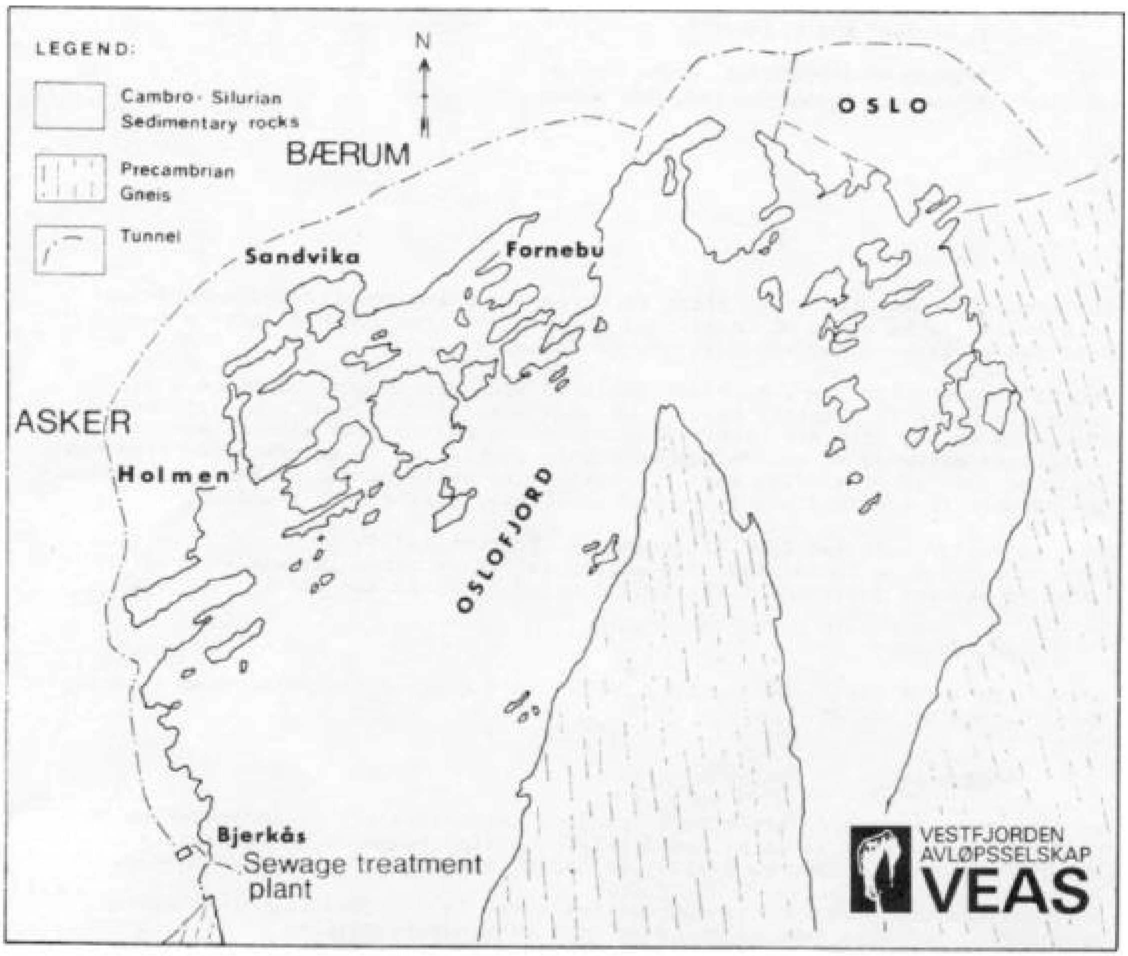
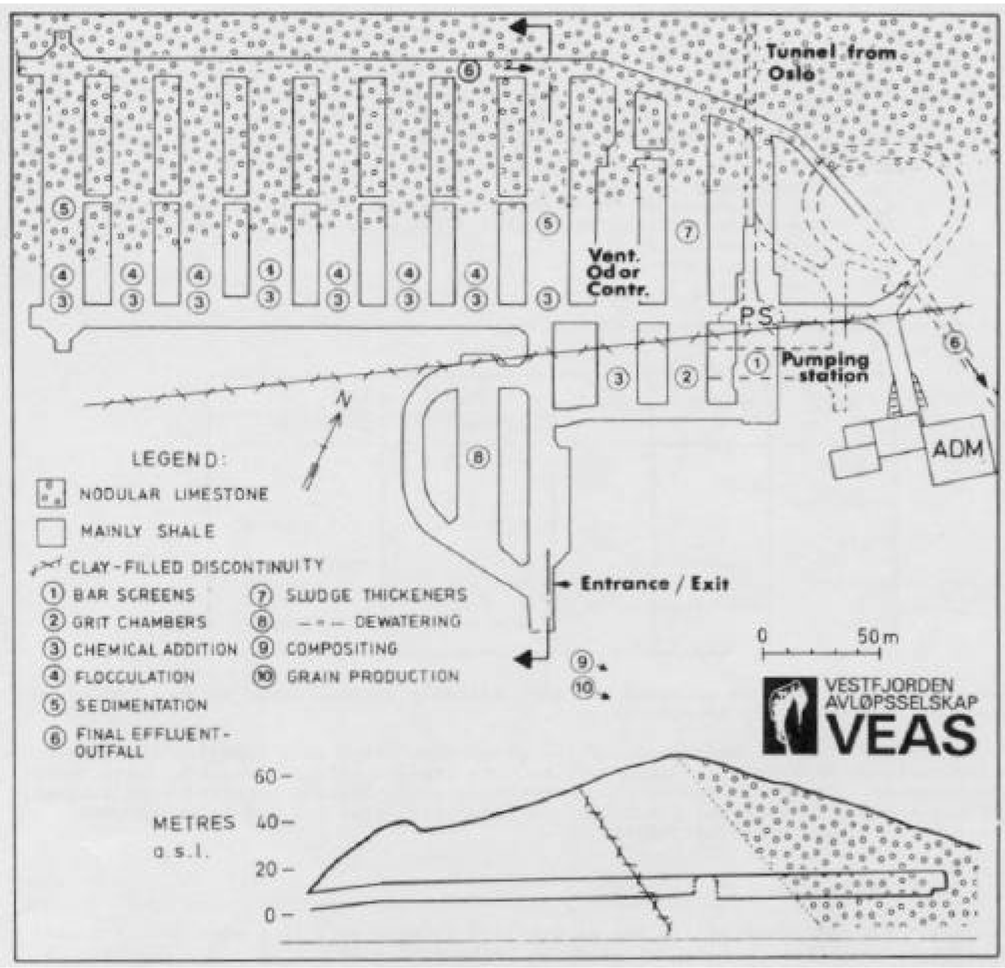


Fig. 1 The Oslofjord Sewerage Scheme.

Norway has long traditions of building underground. Our hydro-electric projects make up the corner stone of Norwegian expertise regarding major subsurface constructions. This technology has been converted into use for several other purposes: air raid shelters combined with sport arenas, subway systems, oil storages, general storage and public utility facilities such as water- and sewage treatment plants.

Although representing a rather unique feature it was not out of the ordinary when the communities of Oslo, Bærum and Asker in 1976 decided to go ahead and build a large regional sewerage facility consisting of 40 km full-face bored tunnels and a mechanical/chemical treatment plant excavated into the hill of Bjerkås in Asker, 30 km southwest of Oslo -adjacent to Oslo fjord (Fig.l).
During the early surveys -and the preliminary reports -several alternatives were considered- also surface locations. The main reason for choosing the subsurface alternative was of an environmental nature. A surface plant was considered less ex- pensive. However, the difference was eventually considerably less than originally expected.
The Bjerkås area was finally chosen for the plant mainly because of political considerations. The rocks were estimated to be of poorer quality here than at the other possible localities, but Bjerkås was already an established industrial area with few private homes.

GEOLOGY
The project is situated in the geological formation called the Oslo Graben. The bed- rock at Bjerkås consists of sedimentary rocks of Cambro-Silurian age, mainly shale and limestone, cut by some Permian dykes.



The Bjerkås area is a hill about 70 m high. Due to limitations of space the orientation could not be chosen in the most favourable direction and the minimum rock cover is approx. 15 m.
Geological mapping showed that the northern slope of the hill consisted of nodular limestone, in the southern slope there was

Fig. 2 Water Pollution Control Plant West - SRV.

shale. Seismic refraction measurements showed velocities of about 5000 m/s in the nodular limestone and 2500- 4000 m/s in the shale. Some crushed zones gave 2000- 3000 m/s, the same as a near surface weathered zone of about 10 m thickness.
Core drilling showed that the nodular limestone was of rather good quality. The joints had usually a high roughness with little or no clay coating. The point load strength was usually 2- 4 MPa. The shale had a more variable quality. It was intersected by some crushed zones some tens of centimetres thick. The joints were often planar and slickensided with a graphitic coating. in some beds only 0.1- 0.2 MPa.
The point load strength was low. Water leakage was measured in the boreholes. Usually the leakages were less than 1 Lugeon. However, in the near surface zone and along some breccia zones the leakages were 20-60 up to loo Lugeon (1 Lugeon = litre per min. per metre borehole for an excess pressure of 1.0 MPa).
From the pre-investigations two questions had to be answered: Was the rock quality good enough to give an economically acceptable and safe plant?
Would the excavation lead to a lowering of the ground water table and damage due to settlements in an industrial plant near by?
Analysis of the rock quality was performed using the Q-method (Barton, Lien, Lunde 1974) "
The formula $Q = RQD/J_n \cdot J_r/J_a \cdot J_w/SRF$ describes the stability of the rock mass by means of 6 parameters. The parameters are:

- RQD -describing the joint spacing (Deere 1963)
- J_n -number of joint sets
- J_r -the roughness of the most unfavourable joint or discontinuity
- J_a -the degree of alteration or filling along the most unfavourable joint or discontinuity
- J_w -water inflow
- SRF- rock load

The result of the Q-analysis was as follows:

Rock	Q	Class	Support: Bolting	Support: Shotcrete
Nodular limestone	2-20	18	c/c 1.5 m l=4m	--- ---
			c/c 1.25m	7.5cm mesh
Shale	0.35-10	23	l=4m c/c 1.0m	reinforced 20 cm mesh
Crushed zones	0.02-0.1	35	1=4	reinforced

The conclusion from this analysis was that suitable support could be achieved by conventional methods at an acceptable cost.
It was feared that the excavation of the plant might lead to a lowering of the ground water table in an industrial area near by. To prevent this a so-called water curtain was proposed. The locations in the excavations where leakage occurred were grouted, and the bedrock between the plant and the industrial area was fed by water under pressure through several boreholes.

PROCESS

The plans for the plant called for 11 parallel caverns with 16 m span and 12 m wide pillars between (Fig. 2) .A typical cross section of a cavern is 150- 160 m^2. In addition there are several smaller connecting and traffic tunnels and an 800 m long outlet tunnel beneath the fjord. The total volume of rock excavation is about 350000 m3, while concrete installations constitute 25000 m^3.

The treatment plant includes conventional mechanical -chemical processes, designed to remove phosphorus from the waste water collected from 600.000 persons, including industry in the area. This amounts to a flow of 3.0 m^3/sec (66 MGD). Phosphorus is considered the limiting factor on the algae growth in the Oslo fjord -a growth that in turn causes secondary pollution problems.
The wastewater arrives at the plant through the trunk tunnel 14 metres below sea level while the major part of the plant is located 7 metres above sea level. Pumps will lift an average of 3.0 m^3/sec. 21 metres.
The unit processes are:
Bar screens -removes sticks, rags, large objects
Grit chambers -removes particles with high specific gravity (gravel, sand etc.)
Chemicals are added to precipitate and coagulate phosphorous and other pollutants
Flocculation tanks -slow agitation, aggregate smaller particles to heavier flocks.
The coagulated pollutants are removed by settling in the sedimentation basins.
The flow of the effluent is measured before being led to the Oslo fjord through a tunnel and diffuser pipes beneath the fjord.
Sludge collected from the settling basins are thickened in gravity thickeners.
The thickened sludge are dewatered further in filter-presses in the solid handling cavern.
The filter-pressed sludge is then converted to compost outside the plant area
Parts of the pressed sludge are trucked directly to farmland for grain production.
It may well be said that a modern industrial processing plant is located in the hill at Bjerkås. The subsurface location makes specific demands for the design and layout to achieve the internal environment for sound operation of the plant.

COMMUNICATION
The lay-out is stretched out considerably compared to surface plants. As mentioned the caverns are generally 16 m wide with 12 m wide pillars. It is therefore some 3- 400 m from the administration building to the farthest point. Transportation will be by foot, bicycle, or forklift. All walkways shall be passable with a forklift. There will be heavy trailer traffic hauling chemicals to the plant and waste/sludge materials from the plant. This transport is concentrated in the outer regions of the plant.

FLEXIBILITY
Subsurface location has some disadvantages as far as alterations and enlargements are concerned. However, a section in the southwestern corner of the hill is reserved for future expansions.
Ventilation is an important part in the design of a subsurface plant. Heating, moisture, and odour control are important factors for creating a good working environment necessary in a water pollution control plant.

HEATING
The energy source for heating the plant and outside buildings is waste water. The use of heat pumps for converting the energy in the waste-water will represent a net saving of about 450000 Norwegian kroner/year. Approximately 4 mill kwh/year will be removed in this manner. The raw sewage will perhaps be used in similar manner before it arrives at the plant -heating a "new town" on the way.
The cooling of the waste-water will not seriously effect the plant use -as the waste-water travels in the rock tunnel with constant temperature, about 80 C, before arriving at the plant.
The potential energy output from 3.0 m^3/sec waste-water is about 60 MW. Full utilization for heating purposes will demand large new industrial and/or housing projects but the initial costs are high.

EXCAVATION AND SUPPORT

The plant is estimated to be completed early in 1982. By now the excavation of the rock is finished. The environmental impact on the neighbourhood from the plant construction was much feared - before the work began. 350000 m^3 of hard rock was to be excavated, mined, and blasted resulting in vibrations that might cause damage. The rock volume itself would create problems -where should it be dumped? Transportation on small neighbourhood roads was out of the question. However, nearly all the excavated rock were dumped along the shore -creating new land, and most important to the neighbourhood -the pleasure boat harbour was replaced and modernized with two long breakwaters, utilizing most of the mined rocks.

The rock quality has been close to that predicted. With some simplification three categories of support have been used in the caverns. In the nodular limestone bolting c/c 1.5 m (1 = 3.5 m) and 5 cm shotcrete has been used. In the middle zone of the plant (Fig. 2) between the nodular limestone and a clay filled discontinuity, bolting c/c 1.5 (1=3.5 m) and 12- 15 cm mesh reinforced shotcrete has been used.

In the southern part of the plant is shale of rather poor quality. The support here is usually bolting c/c 1.25 m or 1.0 m (1 = 3.5 m) and 12- 15 cm mesh reinforced shotcrete. Along some especially poor zones an even thicker layer of shotcrete sup- port has been used.

Some water leakage has occurred in the halls, especially along a Permian dyke, and some grouting has been undertaken. Up to now the water curtain has worked well and the ground water table in the industrial area has been normal.

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SPORT HALLS AND SWIMMING POOLS IN ROCK IN THE OSLO REGION (NORWAY)

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Fortifikasjon A/S, Consultants in Rock installations, Oslo, Norway

In the Oslo region (Oslo = Capital of Norway) many dual purpose rock installations are under construction. (1982) In peacetime they will be used as sport halls and swimming pools. In the event of war they can easily be converted into very good civil defence Shelters. Five such installations are shortly described here.

Norway is a relatively large country .It is approximately 50% larger than Western Germany. But while W. Germany has 62 mill. inhabitants, Norway has only 4 mill. 70% of the land is covered by rock. Installations in rock are therefore a Norwegian tradition. The activity in tunnelling and rock blasting has several decades been very high. Planned tunnel length per year and per million inhabitants for the period 1970-79 was 19 km compared to the average value for OECD-countries which was 3,2 km. Since 1916 nearly 150 hydro-power plants have been constructed in rock. This number is more than 50% of the total number in the world.

Fig. 1. From a hydropower plant in rock

It has given considerable experience to the designers as well as to the contractors. This has benefited planning, design and construction of a large number of other under- ground installations throughout the country ranging from caverns for storage purposes for different products such as oil, gas, ore, flour, paints, frozen food to drinking water reservoirs, sewage plants, parking lots, factories, telecommunication centres, swimming pools and sport halls.

The defence and civil defence have also long tradition in rock installations.

The first sport hall in rock was finished in 1972 in the small town Odda on the west coast. The main hall (fig. 2) is 25 by 60 m which makes it adequate for playing international handball games. The total volume of this installation is 25.000 m3 .

The first swimming pool in rock was finished in 1975 situated in the small inland town Gj0vik. The installation contains a 25 m pool with 6 lanes, a smaller pool for beginners, a gym, saunas and ancillary areas. The total volume is 11.500 m3 .Of special interest is the fact that the energy consumption for running this under- ground public bath and swimming pool has been cut down to 45% of what would have been necessary for a similar building on the surface.

Fig.2. Odda sports hall. Completed 1972.

Fig. 3. Gjøvik swimming pool in rock. Completed in 1975.

The experience gained, both for the sports hall and swimming pool is very good. Five such installations will be shortly described, but first some general comments.

DUAL PURPOSE INSTALLATIONS

Rock installations have a surprising resistance against weapon effects ranging from direct hits from conventional weapons to all the effects of a nuclear weapon. Thus all these installations have a dual purpose: sports halls or swimming pools etc. in peacetime, civil defence shelters in wartime. The local community is by law responsible for building the installation, but approx. 2/3 of the cost is paid for by the state government because it can also be used as a shelter. This makes it cheaper for the community to build in rock, compared to a construction in open air.

CONSTRUCTION PRINCIPLES

Normally a very thorough geological investigation will be done. Span width and direction of the caverns and tunnels will be recommended. Based on these recommendations the lay-out of the installation will be decided according to the future use. The engineering geologist will also often recommend the blasting technique, plan for excavating and the rock securing systems. The assistance given by the engineering geologist is of greatest importance for a successful job. The highest possible competence should therefore be consulted. After blasting, excavation and securing the rock, the caverns and tunnels are ready to be transformed into a sport hall or a swimming pool etc. The price for this volume varies in Norway roughly from 30-50 dollars pr. cub m. This rock volume has certain basic properties which will greatly influence the building construction to be used. The inside temperature is fairly constant, varying from 6-8°C throughout the year. In the entrance region the rock overburden will often be small. In the winter season, with low outside temperature, the rock will here be cooled down and cause condensing of water on the surface. The water and the humidity can give problems if not taken care of in the right way. The humidity in the rock volume will very often be high and there will normally be water seepage through the rock. Wind and sun, rain and snow will not influence the construction with the advantages this means. Inside the cavern there will be almost complete darkness. So all lights inside will have to be artificial.

BUILDING CONSTRUCTIONS

The building constructions in the cavern will depend on several factors, but in good rock the following three principal solutions are often chosen:

1. In good relatively dry rock: Gunniting roof and walls. Drainage of water seepage with rock-wool canals.
2. Concrete arched roof and concrete walls.
3. Horizontal concrete roof and concrete walls.

1, 2 and 3 will often be in combination.
2 and 3 are often used as cast in place construction or prefabricated.
Many other systems are in use, for instance fire proof polymer-materials in both roof and wall etc.

VENTILATION
Sport halls and/or swimming pools in rock with the dual purpose use as shelters, will have three alternatives for the ventilation :

- Ventilation through gas filters for shelter purpose.
- Normal ventilation for shelter purpose.
- Peacetime ventilation

Both the civil defence demand for the shelter and its peacetime use, must be considered To obtain an optimum result it is imperative to consider several alternatives, concerning the demand for filtered air, cooling demand, transmission and heat accumulation in the rock.
In order to save energy, heat gainers are installed both in the air intake and at the side of evacuating. In installations containing both a swimming pool and a sports hall the former has a slight under-pressure in relative to the surrounding rooms, to prevent water and humid air to cause condensation in the entrance tunnel. In the entrance tunnel a dry zone should be established by blowing dry preheated air to prevent condensation on roof and walls.
For peacetime use the whole installation should be at a slight under-pressure. To prevent gas infiltration in wartime, the installations will have a fairly high overpressure (normal and filter-ventilation) under these conditions.

ELECTRO-TECHNICAL INSTALLATIONS - ILLUMINATION
The illumination in an installation in rock can be de- signed under nearly ideal conditions, because there are no disturbing elements as sunshine and daylight. It is of vital importance to make an ideal combination of colour , lights and constructions to create the best conditions for human beings, related to the planned activities in the installation. Psychological effects as claustrophobia could by these means be avoided.

Fig. 5. Artificial “windows” with indirect light give an impression of outdoor view.

Fig. 4. A long entrance tunnel can be divided by light and colour. Notice the white tiles on the floor.

Fig. 4 and 5 show some examples of creating "sunshine and daylight" underground. The intensity of the illumination follows the same rules as for an above-ground installation. Because of the dual purpose use as shelter, an emergency power aggregate will always be installed.

EXAMPLES OF UNDERGROUND SPORT HALLS AND SWIMMING POOLS IN THE OSLO REGION

From 1978 to 1982 several huge installations will be constructed in the Oslo region. Five of these installations will be mentioned in the following. The first will be presented relatively thoroughly, the four last ones only by highlights.

The installations are :

1. Skårer: Sport hall 25 x 50 m and swimming pool 10 x 12,5 m etc. The, community of Lørenskog.
2. Holmen: Sport hall 25 x 50 m, club room, squash hall etc. The community of Asker.
3. Holmlia: Sport hall 25 x 45 m, swimming pool 20 x 40 m etc. The city of Oslo.
4. Tærud: Sport hall 25 x 50 m etc. The community of Skedsmo.
5. Vassøyholtet: Bowling hall, shooting range, gymnasium etc. The community of Skedsmo.

Responsible for complete planning and design of all installations mentioned :
FORTIFIKASJON A/S, OSLO.

SKÅRER -SPORT HALL AND SWIMMING POOL

General

The community of Lørenskog had a great need for public shelter. In addition they also needed 4 gymnasiums for two schools under construction and swimming pool for beginners. They decided to see all those needs together and built a rock installation close to the schools. The blasting and excavating started in October 1978 and all construction work was finished October 1980.

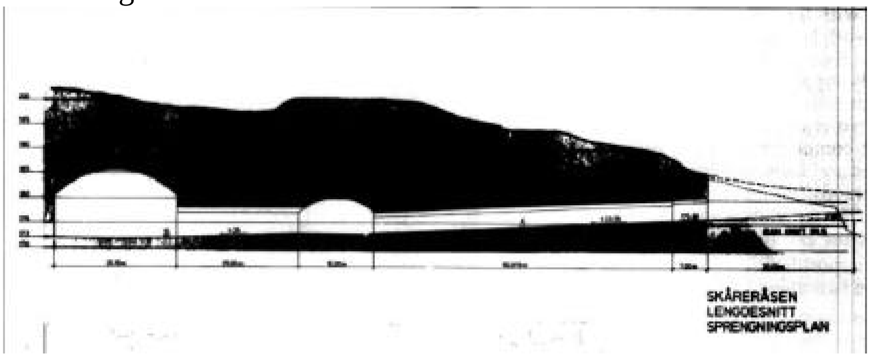

Fig. 6. Cross section. Skårer sports hall.

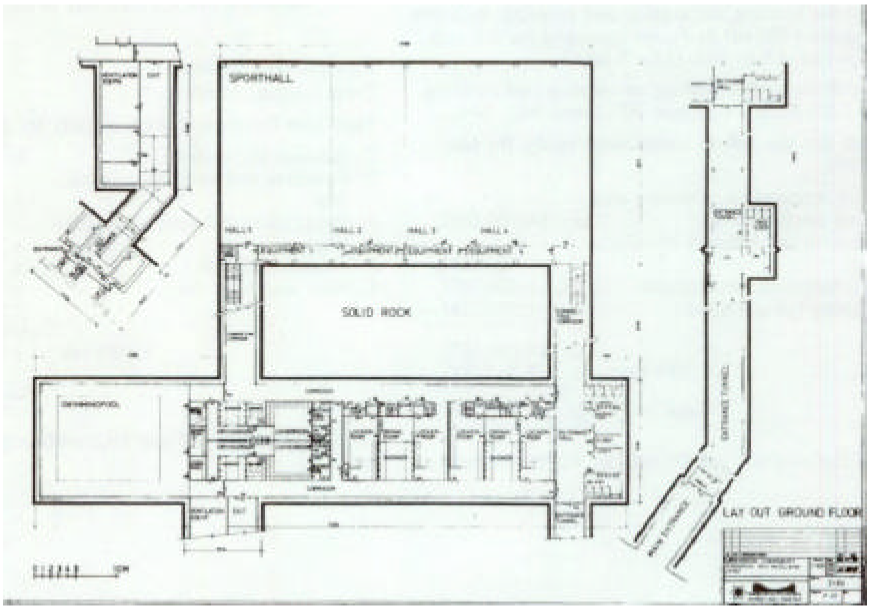

Fig. 7. Lay-out Skårer sporst hall and swimming pool.

Geology

Noteby A/S (Norwegian consulting firm) has undertaken the engineering geological investigations. The installations are situated in a small hill, Skårerasen. The rock is relatively homogeneous gneiss. Three core drillings were done. The main halls have a N-S direction which is the best in this region. Greatest span is 25 m and the least overburden is 15 m. The blasting and excavating have been carried out without any great problems. The securing of the rock in the blasting period was done mostly with rock bolting (3-4 m) and some gunniting. For the permanent securing about 400 bolts 2-4-5 m were used. The securing work can be characterized as moderate.

Lay-out

The main entrance tunnel has civil defence blast doors (open in peacetime) and leads to the section with 6 locker rooms and 4 shower rooms both for the sport hall and the swimming pool. The southern entrance with staircase is for the swimming pool and serves also as emergency exit. The locker rooms and shower section is connected with the main hall by 25 m tunnels. All construction work is cast in place concrete. Because the installation will be used as a wartime shelter, the roof is additionally supported by cast in place, reinforced concrete arches with a spacing of 5 m. Between the arches prefabricated reinforced concrete elements are placed. The whole complex is supplied with fresh, preheated, cleaned and dehumidified air from two ventilation centrals, one for the swimming pool and one for the sport hall. The circulating amount of air is constant for both peace-use and the use as shelter.

The sport hall 23.000 m^3

The swimming pool 5.000 m^3

The amount of fresh air in peacetime and normal ventilation as shelter is 19.200 m , for filtered air in wartime 9.600 m^3 .It is the use as a shelter for 4.000 persons that is the decisive factor .

Mass and cost figures

Volume: 25.600 m^3 .

Total cost for blasting, excavation and securing, inclusive transportation (500 m) exclusive taxes and general cost: (NOK. 1978) 4.936.160 NOK

Cost pr. cub meter for blasting, excavating and securing, inclusive 7,5% general cost and 20% taxes: NOK. 249,-.

Total cost for the whole installation ready for use (NOK. 1978):

1. Blasting, excavation, concrete and building constructions:	14.500.000,-
2. Ventilation and sanitary installations :	1.700.000,-
3. Electro-technical installations :	1.400.000,-
4. Consulting fee and misc. :	1.200.000,-
Total cost:	18.810.000,-
+ 20% tax	3.762.000,-
22.572.000,-	

The Civil Defence will pay 10 million NOK for the shelter use.

HOLMEN SPORT HALL

General

The community of Asker had a great need for public shelter (5.500 persons) in a special area. Holmen Sport club needed club rooms and wardrobe/shower facilities for their activities. The building of a sport hall for hand- ball-playing etc. had also been planned in the same area. Close to the existing outdoor sport stadium suitable conditions for an underground sport centre were found and it was decided to combine all these needs in one rock installation. The rock is limestone of relatively good quality. The rock material will be used for road construction. Fig. 9 and 10 show the lay-out.

Fig 8. Holmen Sport club area with football ground, jumping hills and tennis /icehockey field. The entrance to the rock installation, one on each side of the tennis field

The installation has two entrance tunnels. One (left) leads to the wardrobe section for the outdoor activities (ground floor) and to clubroom and meeting room on the first floor. The entrance to the right leads to rooms for ventilation equipment and wardrobe section for the sport hall (ground floor). The first floor will be used for playing squash.

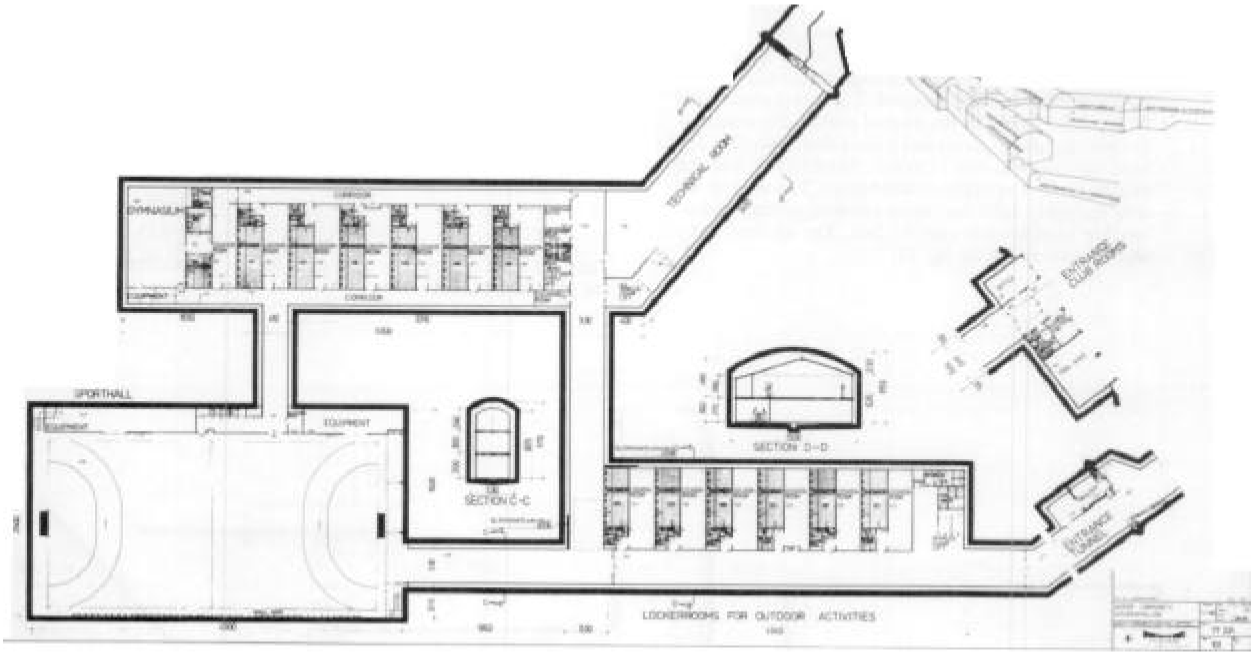

Fig. 9. Holmen sports hall. Ground floor.

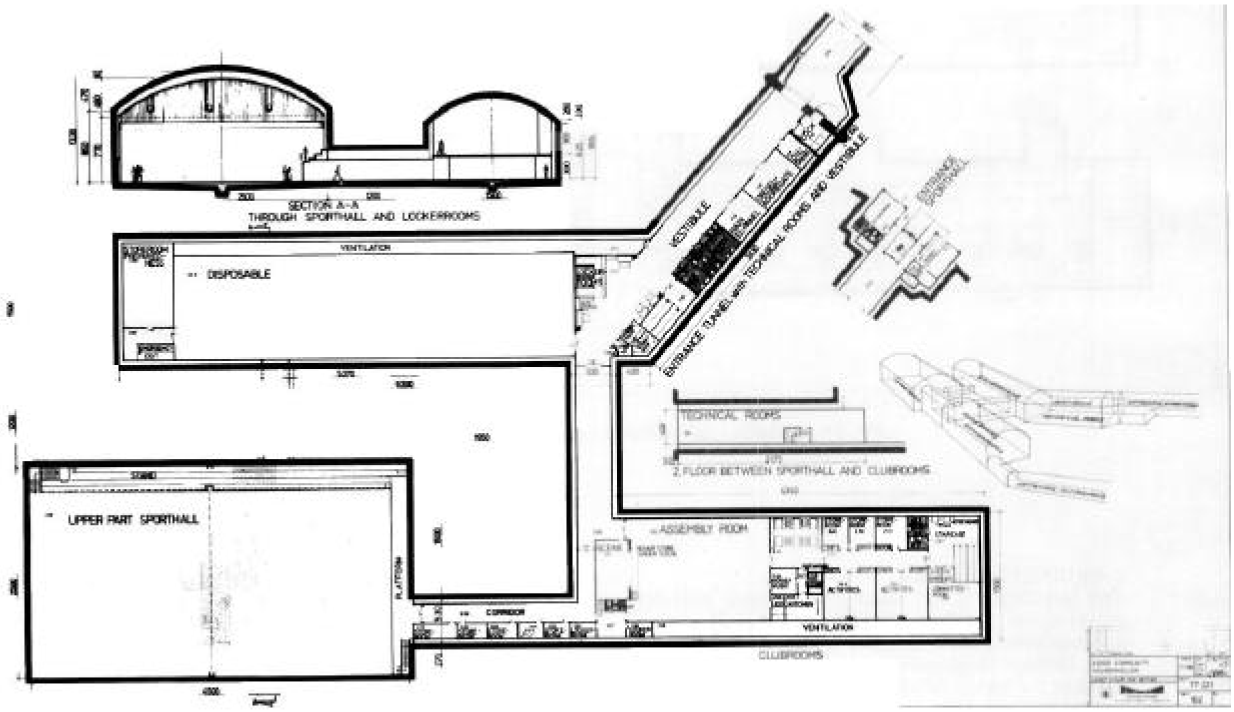

Fig. 10. Holmen sports hall. 1. Floor

Total volume: 35.600 m^3 .

Total cost for the installation ready for use (NOK. 1978)

Blasting and excavation :	5.700.000,-
Concrete and building construction:	8.600.000,-
Ventilation and sanitary installation:	2.470.000,-
Electro-technical :	2.250.000,-
Misc. and cons. fee:	2.580.000,-
Total.	21.600.000,-
+20% tax	4.320.000,-
Total cost:	25.920.000,-

The Civil Defence will pay 16,2 mill. for the shelter use.

HOLMLIA SPORT HALL AND SWIMMING POOL

The city of Oslo is expanding rapidly, at the moment in a southern direction. In the Holmlia area {southern Nordstrand) a new living area for 15.000 people is under construction. It is a rough mountainous landscape and difficult to find a suitable place for a sport hall and swimming pool needed for the population. There was also a need for a public shelter in the central part of the area. For the preparation of roads and places there was also a great need for crushed rock material. Therefore it was only natural to choose the rock alternative. The rock in the area is mostly solid and homogeneous granite and a suitable location was easy to find. The lay-out and sections are shown on fig. 11.

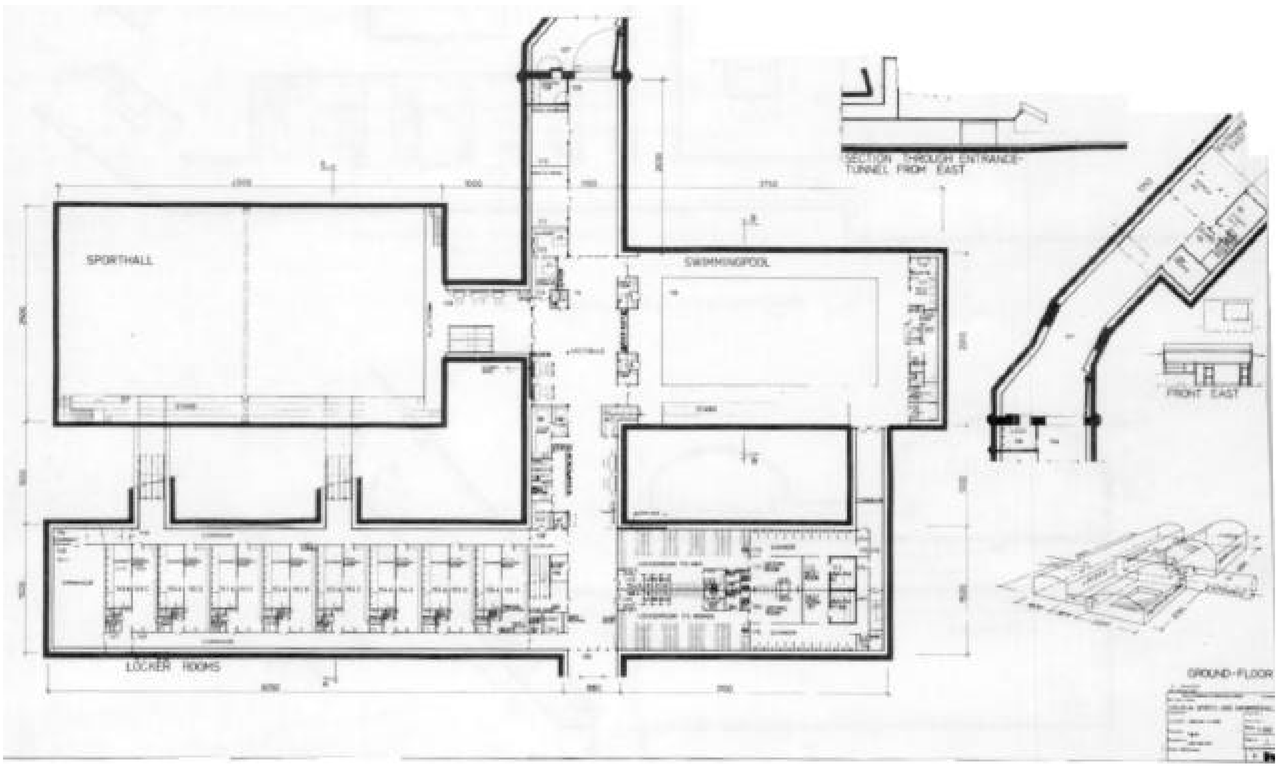

Fig. 11. Holmlia sporst hall and swimming pool. Ground floor.

Mass and cost figures

Total volume: 50.000 m^3 .Total cost of the installation ready for use (NOK. 1979):

1. Blasting and excavating etc.	6.600.000,-
2. Concrete and building construction etc.	13.700.000,-
3. Ventilation and sanitary inst.	2.338.000,-
4. Electro-tech.	2.795.000,-
5. Misc. and consultants fee	3.667.000,-
Total	30.100.000,-
+ 20% tax	6.020.000,-
Total cost	36.129.000,-

The Civil Defence will pay 20,4 mill. for the shelter use. (7.500 persons)

TÆRUD SPORT HALL

This installation is very much like the sport halls described earlier. Besides a complete sport hall 25 x 50 m and wardrobe/shower facilities, it also contains the local Civil Defence Headquarter. The installation will be located in the middle of a housing area which needs both a sport hall and shelter.

Mass and figures

Volume: 35.000 m^3. Geology: Gneiss. Total cost: 23 mill. NOK. The Civil Defence will pay 13.8 mill. NOK. for the use as a shelter for 4.000 people and 1,4 mill. NOK. for the local Civil Headquarters. Fig. 12 and 13.

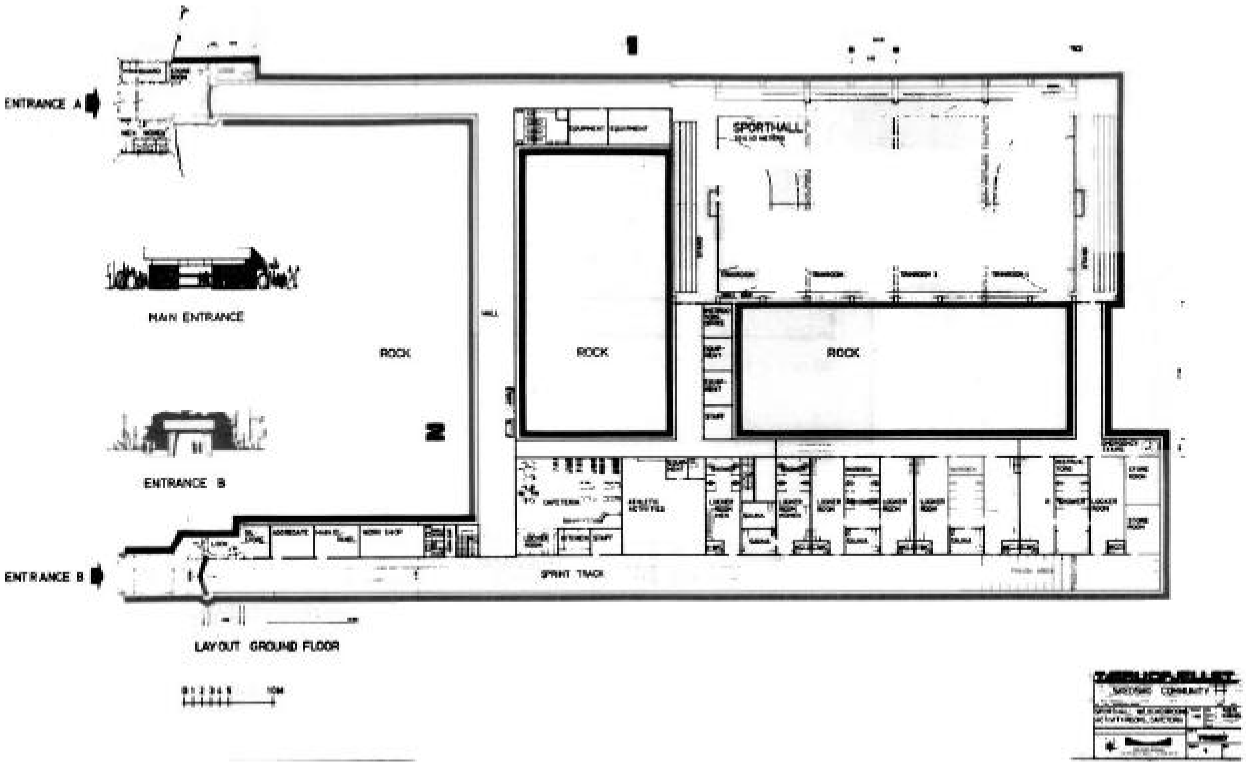

Fig 12. Tærud sporst hall. Ground floor.

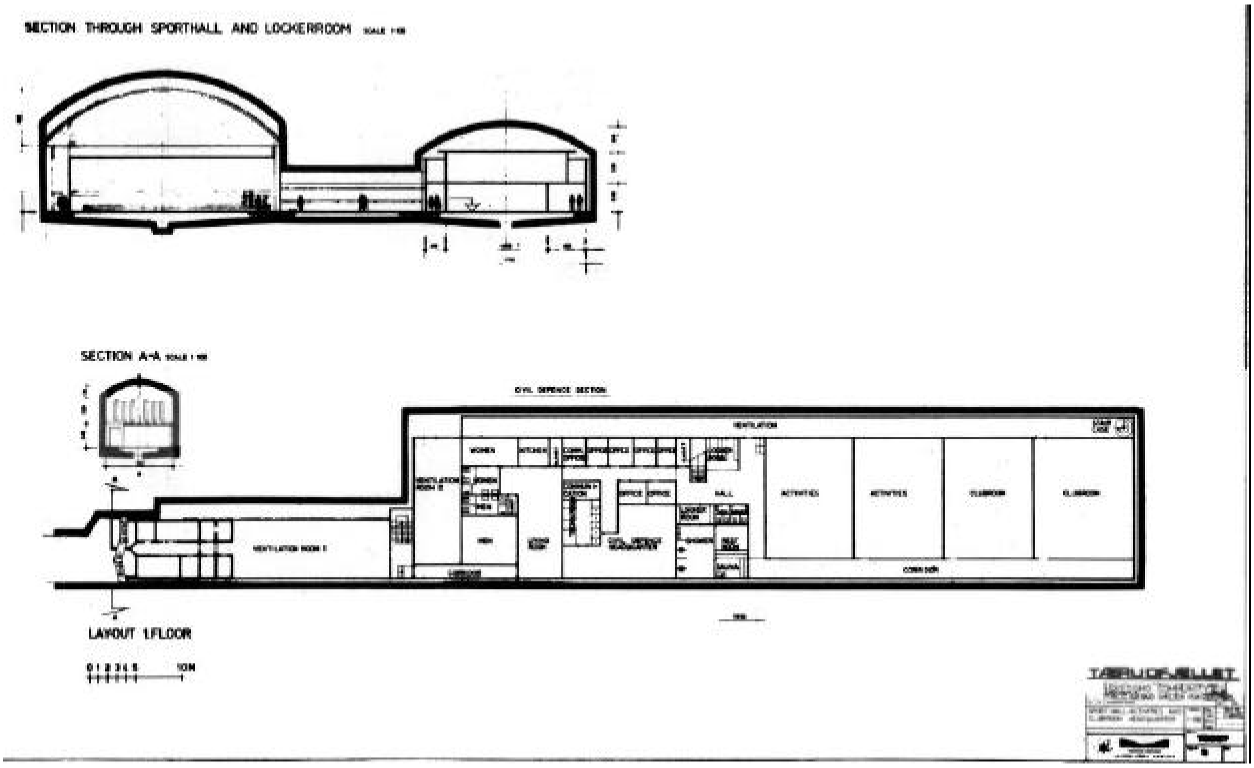

Fig. 13. Tærud sports hall. 1. floor.

VASSØYHOLTET ACTIVITY CENTRE
This is an example of a small installation containing bowling hall, shooting range, gymnasium etc.
Lay-out fig.14. : Mass and cost figures
Volume: 5.500 m^3. Geology: Granite. Total cost: 10 mill. NOK. The Civil Defence will pay 5 mill. NOK for the use as a shelter for 1635 persons.

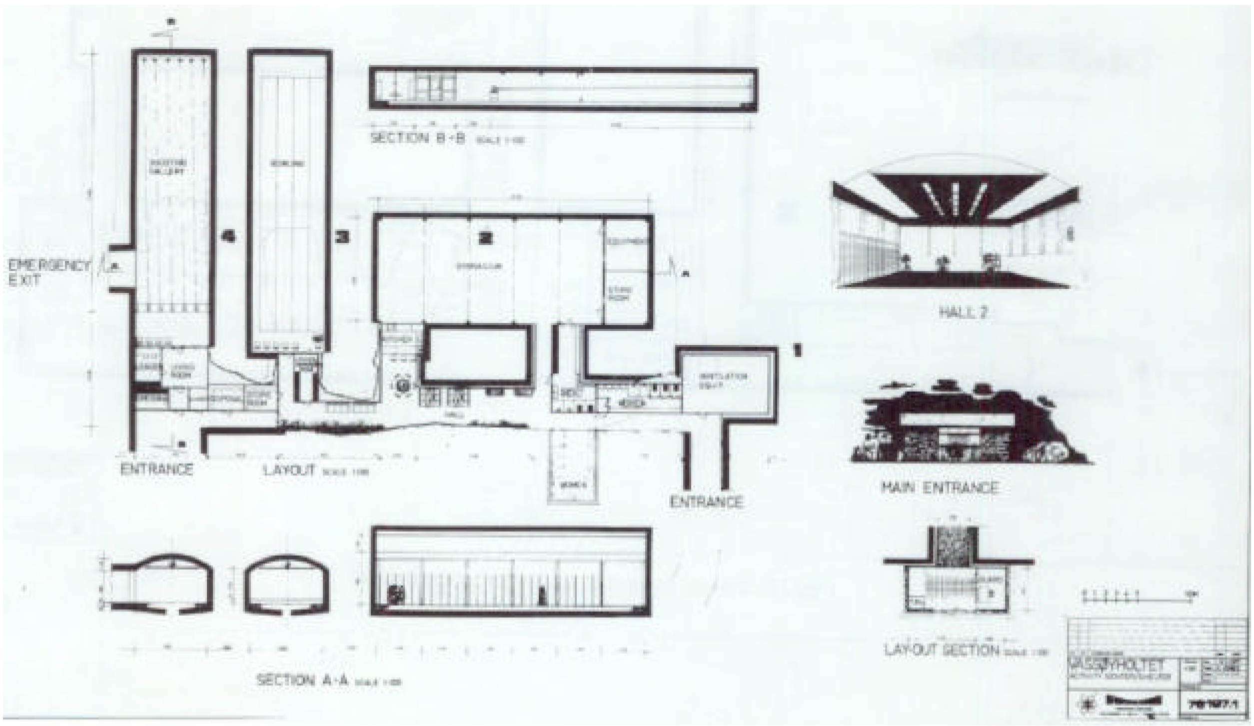

Fig 14.Vassøyholtet sports centre.

FUTURE PROJECTS

Many projects are under construction and also many future projects are under planning. The projects are ranging from huge installations for oil and gas, stores for frozen food etc. to sport halls and swimming pools. One question has been asked several times: How big can the span width be? To gain experience, planning and design of one installation has been carried out. Span- width: 60 m, length 127 m and height 20 m.

By combining an ice stadium with a swimming pool, great advantages are gained in saving energy approximately 300.000 NOK per year. In Norway we have experienced the great advantages of going underground by:

-saving valuable space in the open air

-saving energy

-easy protection

The underground installation, is therefore very often the most attractive alternative.

STORING WATER IN ROCK CAVERNS IS SAFE AND CHEAP

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ABSTRACT
In Norway a number of underground openings have been excavated for storing of drinking water. Such rock cavern tanks are safe, invisible and easy to extend and maintain. With reasonable geological conditions they are cheaper than conventional concrete and steel tanks, when the required volume extends approx. 8.000 m .A close cooperation between the consulting engineer and the engineering geologist is necessary for a successful result. The authors' experience from such cooperation are outlined and typical examples of rock cavern tanks are described.

INTRODUCTION
During the last 10-15 years a number of underground openings have been excavated in the hard bedrock of Norway either as replacement for or as alternative to open reservoirs or concrete or steel tanks for storage of drinking water. A closed water tank, as is the rock cavern tank, has several advantages compared with the traditional open reservoirs. It is above all easier to keep undesired pollutions under control with a closed tank.
In open reservoirs the drinking water is exposed to the influence of sunlight and pollutions from the air. Open reservoirs are also commonly situated in natural or artificial depressions and will therefore have a draining effect on the surrounding landscape. Especially if such reservoirs are placed close to populated areas, there is a danger that polluted surface water or groundwater may be drained into the drinking water. Today the health authorities in Norway will normally not accept open reservoirs for storage of drinking water. New drinking water systems still have to include closed tanks of some kind. Old schemes with open reservoirs will often have to be redesigned and reconstructed.

FUNCTIONS AND LOCATION OF WATER TANKS
The basic function of a water tank is to act as a storage buffer to cover the variations in the consumption and keep the water head stable. This makes it easier to operate the treatment plant and the pumps at constant capacities. It allows smaller dimensions of the main pipe lines and gives stable pressures. In addition the water tanks will act as emergency storage in case of fire or failure in the supply system. Small water tanks are normally single chamber tanks. If the total volume exceeds approx. 10.000 m^3, the tanks are often made as double or even multiple chamber tanks. This allows one chamber to be emptied for cleaning and maintenance without interrupting of the water supply. A water tank should be situated at an elevation which gives a suitable water pressure in the consumption area. It is also preferable to locate the tank as close to the consumption area as possible. This is especially important if the capacity of the tank is designed to cover the variations in the daily consumption. Most water tanks in Norway have been freestanding structures made of conventional reinforced concrete or pre-stressed concrete. When double chambers have been necessary, either two separate structures have been made or two concentric chambers in one structure. To minimize the impact such concrete structures may have on the environment, they have been dug often to some extent into the ground or tried hidden away in other ways .
One way of making water tanks invisible is to put the tank completely underground. And the most economical way of doing this, is to use the rock mass as the construction material, i.e. making caverns for the water in the bedrock. A rock cavern tank will normally consist of an access tunnel

and one or two (or more) chambers in the form of large, but short tunnels. In front of the chamber there will have to be a dam wall.

COMPARISON BETWEEN ABOVE GROUND AND UNDERGROUND WATER TANKS.

The topographical and geological conditions in an actual area for a water tank often may allow solutions for the tank both as an underground and as a surface structure. In such cases a careful evaluation of the two alternatives should be carried out.

Normally the following factors will be in favour of a rock cavern tank:

- high degree of safety, also against war hazards.
- constant and low water temperature.
- the tank is invisible (see Fig. 2) .
- good possibilities for future extensions.
- low maintenance costs.
- the excavated rock masses may be used for other purposes.
- low or no addition in price for a two chamber solution.

(The explanation for the last statement is that when a two chamber rock tank is excavated, the contractor gets two tunnel faces to work on. This means that he is using his equipment more economically, while when drilling is going on at the one face, mucking and hauling can be carried out at the other.)

The following may disfavour an underground tank.

- polluted groundwater may seep into the drinking water
- water may leak out through the rock masses.

Favouring a surface tank is the following

- no risks for pollution of the water (except in case of sabotage)
- leakages are easy to observe and normally also to repair.

The following disadvantages may be mentioned

- the tank occupies valuable building ground.
- the tank may be regarded as a foreign intruder and thus resisted by the neighbouring society (see Fig. 3) .
- the tank may endanger the surroundings in case of war activities.
- the water temperature will change with seasons.

Even though the above mentioned positive and negative factors for the two different solutions are important enough by themselves, the decisive factor for the choice of type of water tank will normally be the price. Later in this paper the authors will show that for certain conditions rock cavern tanks compete well in price with other water tanks.

PLANNING AND DESIGN PROCEDURES FOR ROCK CAVERN TANKS

As the rock cavern tank is basically a rock mass structure, a successful result of the planning and design is depending to a large extent upon the good cooperation between the consulting engineer and an engineering geologist.

One of the authors has outlined earlier the general design procedure for underground openings used by engineering geologists in Norway (Broch & Rygh, 1976, Selmer-Olsen & Broch, 1977) .This procedure is divided into four stages:

I. A location is selected which from a stability point of view shows the optimal engineering geological conditions of the actual area.
II. The length axis of the openings are oriented so as to give minimal stability problems and overbreak.
III The openings, including both caverns and tunnels, are shaped taking into account the mechanical properties and the jointing of the rock masses as well as the local stress conditions.
IV. The different parts of the total complex are dimensioned so as to give an optimal economic solution.

Mistakes in one stage will always have economical consequences. The extent of these will vary with local conditions and the type of project. Normally rock cavern tanks in Norway are unlined and have a limited overburden of rock masses. This implies that the rock masses are subjected to low stresses. The joints may thus be of a more open type than are joints at a deeper level in the rock mass.
It is therefore important that the permeability of the rock mass, or in other words the possibilities for leakages along intersecting joint sets, is taken especially into consideration when water tanks in rock are being designed.
It is a general experience that stiff rocks like granites, quartzites etc. have a tendency to give greater leakages than more deformable rocks like micaschists, phyllites etc. Carbonate rocks like limestones and marbles, and rock masses with calcite containing joints and faults, are of special interest from a leakage point of view as calcite is easily dissolved by cold water.
The most important decision to be made during the planning of a rock cavern tank is to locate the tank. It must never be forgotten that when the place for an under- ground opening is decided, the choice of material into which the opening is going to be excavated, is also made. It is therefore of the outmost importance that this crucial decision is based on the advice from an experienced engineering geologist.
To give such advice the engineering geologist will have to carry out geological and geotechnical investigations of the actual sites. At this stage it is of particular interest to get information about the rock types and the weakness zones (or faults) .Combined with information from the consulting engineer about the upper and lower water levels and approx. capacity of the tank, and with information about the topography from detailed maps and air photos, the engineering geologist will be able to delete unfavourable rock volumes. He will finally end up with limited areas at the requested levels.
Before the consulting engineer starts planning the layout of the underground scheme within these areas, he also will need to know what directions of the length axis for caverns will give the best stability and least leakages. Registration and evaluation of the joint sets in the rock mass by the engineering geologist will give an answer to this. This information is also of importance when the shape and the dimensions of the different parts of the rock caverns and the connecting tunnels are to be decided.
A particular problem for rock cavern tanks as for all other underground schemes is the entrance. This is the only part which will be visible to the public in the future. From 0.:'. excavation point of view it is one of the most difficult parts as the rock mass is generally unstable due to weathering. Great care should be taken first of all in finding the most suitable place. As for the excavation itself, restrictions should be put on the contractors work. All too often one can find ugly tunnel entrances where the rock mass has been torn up unnecessary by too hard blasting. A combination of knowledge of the rock masses and careful blasting is the only way to get a proper result. A thoroughfu1 discussion of the excavation of tunnel portals is given by Garsho1 (1979) .

THE KVERNBERGET ROCK CAVERN TANK.

Kristiansund is a fishing harbour situated on some islands on the north-west coast. The need for a new water supply system made it necessary to cross two fjords with a pipeline from an inland lake. At the town-side of the fjords a water reservoir placed at a level of 80- 100 metres above sea level was needed. Approx. 2 km from the town centre and only some hundred metres from the planned pipeline a mountain called Kvernberget rises to 200 metres above sea level. Engineering geological investigations showed that the rock masses of Kvernberget might be used for a water reservoir, the rock being a Precambrian gneiss. Favouring the decision to put the water reservoir into rock was among other things the fact that future expansion could easily be planned for the reservoir.

Figure 1 shows the layout of the water tank. Two basins of 11 x 7.5 x 120 metres give each an effective volume of 8.000 m^3 of water. The distance between the basins is 15 metres. The entrance tunnel is extended allowing a third basin to be excavated without disturbing the operation of the two existing basins.

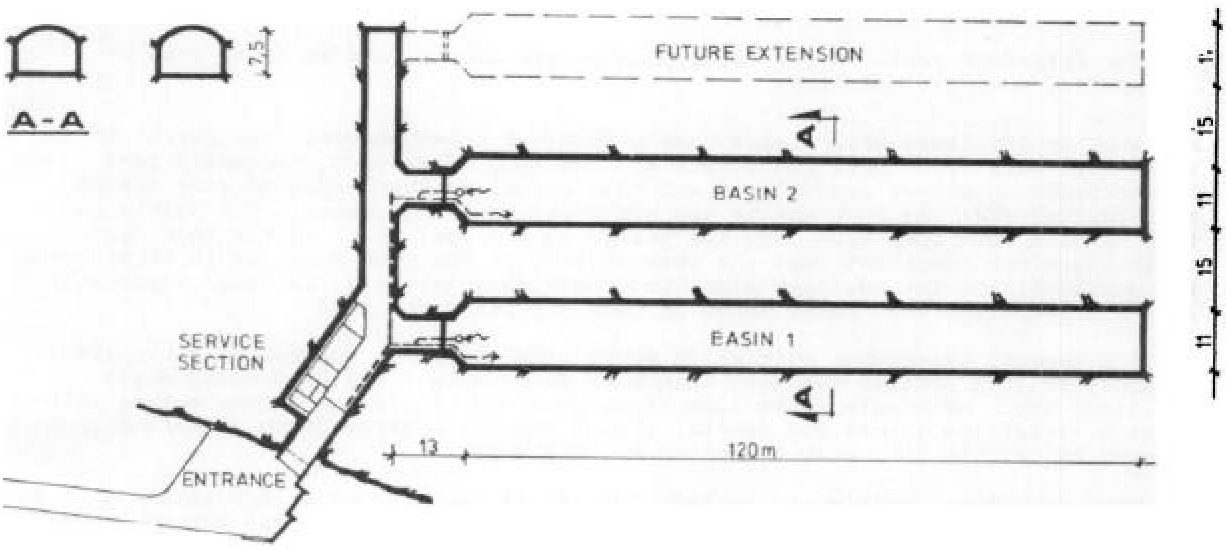

Fig. 1. Layout of the Kvernberget rock cavern tank.

Along the outer part of the entrance tunnel a service section is built containing all the equipment for the operation of the whole water supply system. To support the rock in the basins 100 m of shotcrete~ was applied and approx. loo rock bolts were installed. The rock is thoroughly cleaned and the basins have a concrete door. The outer part of the entrance tunnel is fully shotcreted, with approx. 1.200 m^3 of shotcrete being used.

No leakages were observed when the basins first were filled with water and so far, after three years in operation, no water loss has been measured.

A total volume of 21.000 m^3 of solid rock was excavated at a price of 2.95 million NOK. transportation of muck an~ supporting of the rock masses included. This gives a price of 140 NOK. per m. Total costs for the water reservoir, including the operation central, were 5.05 million NOK.

Only a year before the Kvernberget water tank in rock vas completed, a conventional "above-the-ground" water tank with exactly the same capacity, 16.000 m , was put into operation at Huseby in the city of Trondheim. Total costs for this, building site excluded, were 5.9 million NOK. All prices based on 1979-1evel.

To demonstrate that water tanks in rock are favourable also from an environmental point of view, the visible part of the Kvernberget tank, i.e. the entrance, is shown in Fig. 2 together with the water tank at Huseby in Fig. 3.

Fig. 2.The entrance to the Kvernberget rock carvern tank; storage capacity: 16 000 m^3

Fig. 3. The Huseby water tank in reinforced concrete, storage capcity: 16 000 m^3

THE STEINAN ROCK CAVERN TANK.

The newest water tank in Trondheim was put into operation in October 1979. This ii the rock cavern tank at Steinan with a capacity of 20.000 m^3 .As can be seen from Fig. 4 the layout is very much the same as for the Kvernberget tank with to basins and a service section close to the entrance. The reason for showing the Steinan tank is to demonstrate that even where the geological conditions are not so simple and favourable as at Kvernberget, a rock cavern tank is a possible and economical solution.

At Steinan the water level for hydraulic reasons would have to be situated at 190 +/- 10 m.a.s.l. Based on a study of the geological conditions and the desired size and shape of the basins a minimum rock mass overburden was defined. A line A-B, see Fig. 4, could thus be drawn along the 215 m.a.s.l. curve. This defined the outer limits in the hill side for the basins.

Two weakness zones were outcropping in the hill along the dotted lines b-c and c-d. Strikes and dips of these were measured and the zones were projected down to the level of the basins.

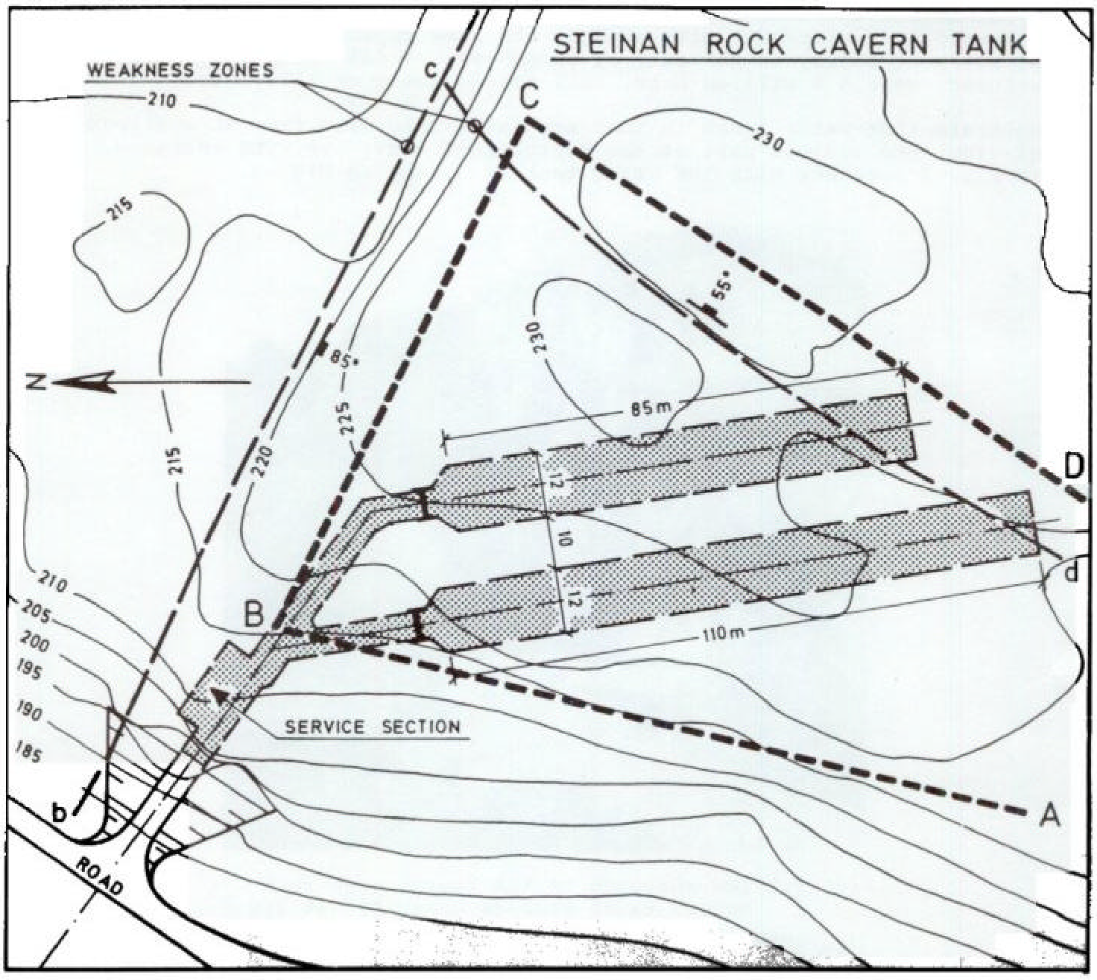

Fig. 4. Layout of the Steinen rock cavern tabk, storage capacity: 20 000 m^3.

To account for uncertainties in the measurements safety margins of 10 metres were added and the lines B-C and C-D were drawn. The area where the basins could be placed in undisturbed, stable rock masses, was now limited by the lines A-B-C-D.

To find the most stable orientation for the long caverns in the highly deformed greens tone pillow lava a thorough survey of the joint sets was carries out. The dominating steep joint sets had strikes of N 70- 80º W and N 75- 90º E. According to orientation rules described in earlier mentioned papers the length axis of the caverns was oriented along the direction N 10º W. The desired capacity of the tank was thus obtained by two basins of width 12 m, height 10 m and lengths 90 m and 115 m respectively.

The distance between the basins was made 10 m, i.e. 5 m less than at Kvernberget. During the filling of the first basin at Steinan some leakages in the empty basin was observed. As such leakages were not observed at Kvernberget, a distance of only 10 m between the basins seems to be too small if leakages have to be avoided. No leakages from the total tank has been observed after six months of operation.

THE GROHEIA ROCK CAVERN TANK IN KRISTIANSAND

When storing of water can be combined with the transportation, i.e. when the transportation system itself, or a part of it, can be used to, or easily converted to, a storage tank, especially cheap solutions can be obtained. In hilly areas pipe lines often have to be substituted by small tunnels.

When extension of the profile of a tunnel is made during the planning, the needed storage volume is easily obtained at a marginal price. As one of a number of examples parts of the planned Tronstadvann inter-municipal water supply system near Kristiansand in the southern part of Norway is shown in Fig. 5.

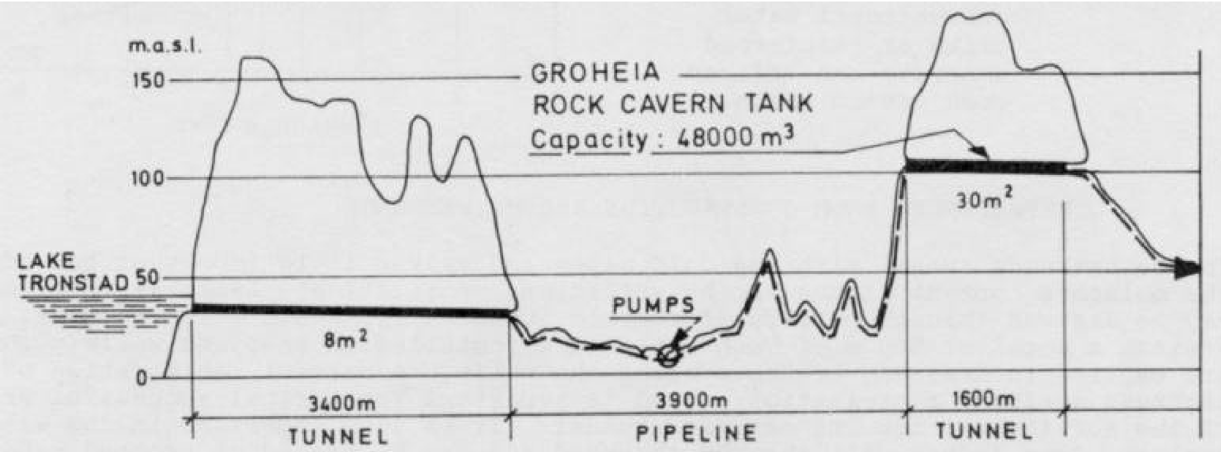

Fig. 5. The Groheia rock cavern tank as part of the Tromstadvann intermunicipal water supply system. (Consulting engineers: Andresen & Grøner A/S, Kristiandand, Norway.)

From the intake reservoir of the lake of Tronstad2the water is conducted through a 3.400 m long tunnel with a minimum profile of 8 m .At the lowest point of a 3.900 m long pipeline the water is lifted by pumps to a 1.600 m long tunnel through Groheia. This tunnel which is situated approx. 100 m.a.s.l. has a cross section of 30 m^2 and thus a storage capacity of 48.000 m^3. Both elevation and capacity are in accordance with the needs for the scheme. The rocks in the area are sound Precambrian gneisses. A few weakness zones are crossing the tunnel and some supporting measures may be needed. The tunnel will, however, like the earlier described rock tanks be unlined.

PRICES

Some prices are already given. A number of conventional water tanks of reinforced' concrete and rock cavern tanks have been recalculated at the Norwegian price level of 1979. The results are summarized in Fig. 6, where prices also are given in U.S. dollars. The prices include 13% taxes.

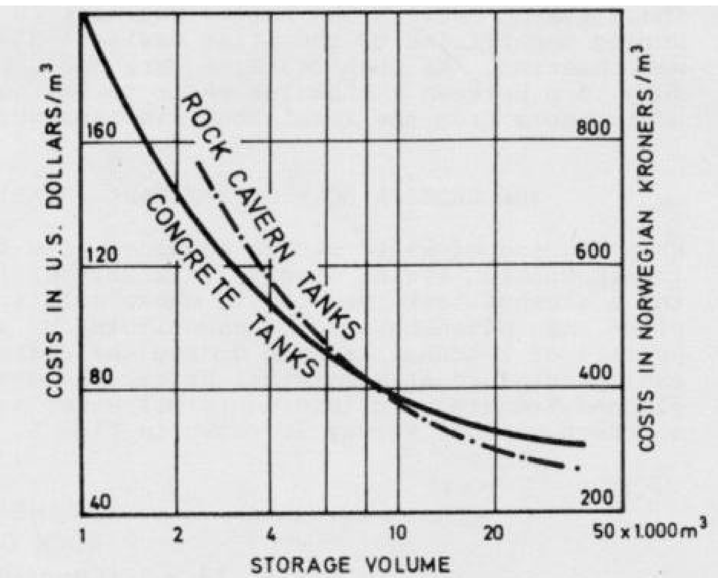

Fig. 6. Specific cost pr m^3 storage volume for conventional water tanks of reinforced concrete and unlined rock cavern tanks.

The two curves are intersecting at a storage volume of approx. 8.000 m^3 , indicating that for storage volumes exceeding this number a rock cavern tank will normally be the cheapest solution in Norway. Such rock cavern tanks have a concrete floor to facilitate cleaning, but no lining of walls or roof. The permanent supporting of the rock mass is done for a

relatively low level of safety as the outfall of minor . blocks of rock in the water filled basins will not interrupt the operation or harm anybody. In the service section and the entrance tunnel the support is made to a high level of safety, normally with a thick layer of shotcrete as the final measure.
Poor rock conditions will increase the costs of supporting to be carried out3in the unlined rock caverns and thus give an intersection point higher than 8.000 m .On the other hand, if the excavated rock masses can be sold, or if the price of land for freestanding water tanks is high, this will favour rock cavern tanks for storing volumes even less than 8.000 m .

EXPERIENCES FROM CONSTRUCTION AND MAINTENANCE
In the entrance tunnel with the cold pipes and valves it is important to reduce the moisture content in the air by sufficient ventilation. Leakages from the rock may be drained through perforated plastic tubes before shotcreting is done. At Steinan a total of 500 m of such tubes were installed in roof and walls. The tubes are emptied in draining trenches along the walls. A careful registration of all leakages during the excavation period is important for a final successful draining of the service section and entrance tunnel. It is also important in the water basins. .Open joints will then be observed and can be sealed or grouted before the basins are filled with water. Because of a draw down in the ground water table above the rock caverns, water will normally be streaming towards the basins rather than from the basins. Leakages from water tanks in rock have according to the authors' experiences never been observed, nor have seepages of polluted water into such tanks been reported. One should, however, always be aware of the possibilities.
All drinking water tanks should be regularly emptied and inspected for leakages, cleaned and disinfected. To facilitate the cleaning, tubes with valves at intervals of for instance 12 m should be installed along one of the walls in the basins. If the valves are left open when the basin is filled with water, the same tube and valve system can be used to obtain a circulation in the stored water when this is regarded as necessary. Water or compressed air may be used.

CONCLUSION
When the topographical and the geological conditions give a choice between a conventional surface water tank and a rock cavern tank, both alternatives should be seriously considered. It is shown that unlined rock cavern tanks are cheaper when the storage volume exceeds 8.000 m^3 .A number of factors may favour rock cavern tanks for considerably smaller volumes.

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SUBMARINE TUNNEL RAFNES -HERØY, SOUTHERN NORWAY

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SUMMARY
This article describes a tunnel crossing under the Frierfjord, some 200 km southwest of Oslo. The deepest part of the Frierfjord at the crossing is about loo m, and the bedrock is covered by another loo m of marine clay. The tunnel, about 3.5 km long, is inclined 1:6 from each end, and crosses the deepest part of the Frierfjord 250 m below the sea.

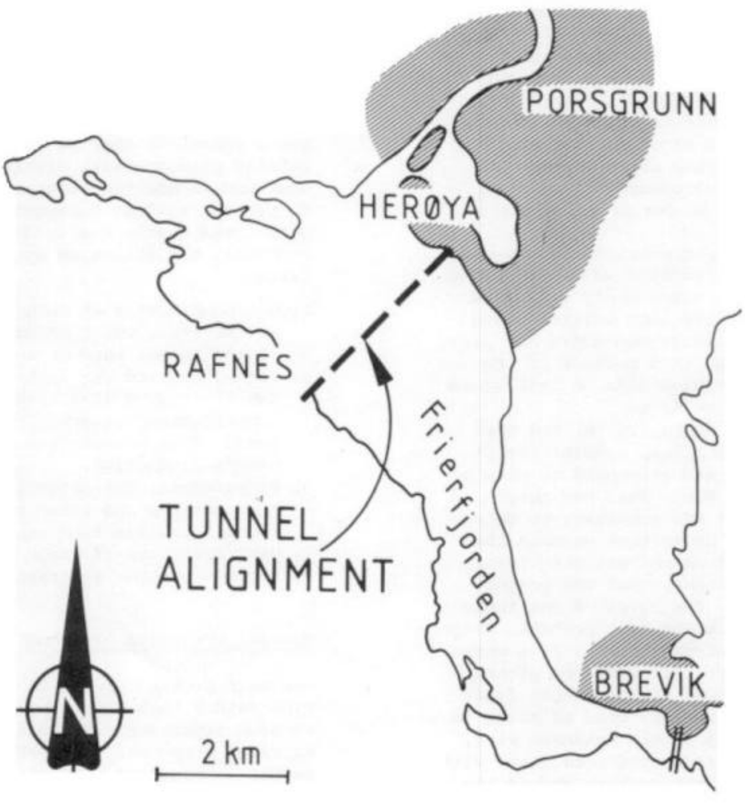

FIG. 1. OVERVIEW MAP

Preliminary investigations performed included thorough geological mapping, evaluation of the experience with previous underground projects in the area, and seismic surveys with boomer-sparker and refraction measurements. Core drilling was contemplated, but the rather good knowledge of the geology, confirmed by the seismic measurements, made the owner Norsk Hydro A/S, decide to build the tunnel without prior drilling. The drilling in this case would have been very costly and difficult to perform.
The tunnel construction, including blasting procedures, saltwater problems, injection procedures, and test drilling ahead of the tunnel face are described. Incurred costs are compared to the costs of conventional tunnelling. Quantities and types of tunnel lining and reinforcement are given.

SITE INVESTIGATION
Norsk Hydro A/S needed a connection between Rafnes and Herøya, as a part of the Rafnes project. Two alternatives were possible: a pipeline on the bottom of Frierfjorden or a tunnel in the rock

under the fjord. NGI was requested to investigate the geological conditions in the area for a possible underwater tunnel (see Fig.1) .

The geological knowledge and characteristics from previous tunnel operations in the area were first gathered. The geological information used was taken from a paper by Henningsmoen and Spjeldnæs (1960) .The head curator J.A. Dons then established a map of known large faults in the area, based on earlier Norsk Hydro jobs. He also described the expected geological conditions near the steep cliff located out in the Frierfjorden. There was thus enough information on the geological conditions to establish a probable geological profile along a potential tunnel alignment.

The question arose as to which investigations would provide the most useful and informative data on the geological profile already obtained and in particular on the location of the bedrock under the sea floor. The investigation of the latter, most decisive for design, included a general profiling with acoustic equipment (Boomer) over large areas to determine the general characteristics of the bottom topography and depth to bedrock. On Fig. 2 is shown a typical profile from this investigation.

The acoustic measurements showed mainly constant bottom characteristics in the proposed tunnel area. It was therefore possible to pursue the more detailed investigations along the shortest tunnel route. The investigations included the usual seismic profiling (refraction- seismic methods along the tunnel alignment and supplementary cross-sections, as illustrated in Fig.2.

The acoustic and seismic measurements agreed with respect to location of bedrock. The test results thus confirmed that about loo m of loose material overlay the bedrock in the Frierfjorden.

In the middle of the fjord stood a steep cliff in the bedrock topography. This cliff corresponds to the lowest layers of the cambro-silurian sequence. The same formation can be seen onshore, south of Brevik (see Fig. 1) .

The main characteristics of the geological profile were now believed well understood. There remained however an uncertainty whether a large steep dipping weakness zone existed along the cliff. The seismic measurements did not pro- vide any information about this because of the rock topography. More precise data on this would have to j)e obtained from borings.

With such large water depth (loo ml and then another loo m of loose material, special equipment had, to be used. It was attempted to hire a drilling rig used in the North Sea, but they proved too expensive. It was necessary to deter- mine the required tunnel depth that ensured that one felt confident, based on the existing information on the rock conditions, that the project could be carried through. The level of the tunnel was selected 40 m below the most probable deepest section at the foot of the cliff. If a weakness zone was located there, it would be picked up by the sounding run ahead of the tunnel front and necessary safety precautions could be taken in time. Outside from this possible weakness zone, the seismic measurements indicated some zones with low velocities; a zone in the western end of the horizontal part of the tunnel was considered as a weakness zone of significance (see geological profile on Fig.2).

The danger for a water breakout in a submarine tunnel design 250 m under sea water level represented one of the most important considerations. Control borings ahead of the working face and preparedness for injection work were clear requirements for the project. Among the appraisals carried out, the following should be emphasized:

It is rather improbable that typical surface phenomena such as rock weathering and, as a consequence, high permeability is significant under sea level. The fjord bottom is covered with clay that acts as an impervious layer and prevents larger water breakouts. In addition, experience with the water tunnel under the River "Skienselva" indicated that the main water problems occurred in the precambrian rocks and that the cambro-silurian shale were relatively impervious through to the boundary zone.

Based on these geological and geotechnical considerations, it was decided to build the tunnel. The project was granted to the engineering firm Thor Furuholmen A/s. The following paragraphs describe the work and experiences.

CONSTRUCTION
The usual technique for a 16-m2 tunnel was followed. The drilling was carried out with a three-boom rig equipped with three air-driven boring machines Gardner Denver PR 123J and Atlas Copco Cop 126. The bored blasting rounds length was 4.5 m long.
The rounds were loaded with a Caterpillar 966 shovel on wheels and transported with the usual boogie-trucks. Niches were blasted every 100 m. These were combined loading and turning niches.
For this 3.6 km tunnel, a 3-km length had a 1:6 slope (9.5º); most of the 3.6-km length had a 50 to 60-m rock cover and nonetheless lay under as much as 250 m of water. A most decisive factor in the problems that occurred was the salt in the water. The slope of 1:6 over such a long distance meant extensive wear on the equipment. Water- related problems will significantly increase the wear and impede the progress of the operations.
To protect against unexpected large water break- outs and enable one to deal with the water problems, the following special measures were taken:
1. One pump system at each working site (Rafnes and Herøya), which could debit 1500 l/min out of the tunnel through 6-inch pipes.
2. Boring carried out each Saturday, with at least two 50-to 60-m long holes in the direction of construction, slightly upward from the tunnel roof. This measure was meant to cover the next week's production.
3. Preparedness for injection. Based on results from borings and water leakages after blasting rounds, preinjection was carried out with cement/bentonite if large leakages occurred.
4. Emergency power generator for the pumps.

<u>Special operation problems</u>
The main problem proved to be the water composition rather than water quantity. The saltwater created important difficulties with all the electrical components for example, in a loading machine. Vagabond currents and damages due to corrosion occurred and the operation safety was greatly reduced. Many problems occurred with the brakes of the shovel due to submerged front wheel under loading.
Because of the electrical ignition of the blasting rounds, one needed to also use air-driven pumps at the working face. An Atlas Copco Dip 30 was used. Otherwise, the pump system consisted of Bibo 5 and Bibo 3 electrical pumps with steel pumping sumps.
Remedy for boring under high water pressure
For borings carried out with a 5l-mm diameter boring head at elevation -253, the water pressure provided about 500 kp reaction force on the boring string. In order to pullout the string under controlled conditions and to enable closing of the hole, a special de!\lice was developed.
The device was anchored to bedrock with two expansion bolts. In addition to holding the strings in place during pulling out, and closing the hole
afterwards, the device could introduce and anchor injection packs in the hole. Without such measures, the injection-rock boundary could result in uncontrollable leakage and dangerous working conditions during boring. The device was built mainly at the site by engineer Bjørn Finden.

Extent of work.
The connection tunnel work started in May 1975 and was completed in early summer 1976. The tunnel was driven from both ends, and involved at most about 100 men.
In addition to the 3630-m long tunnel, the following works was carried out;

Boring of investigation holes 10.500 m
Injected cement 31 tons Shotcrete 450 m^3
Bolts 2.500 units

Formwork 4.500 m^2
Cast concrete 4.500 m^3
Some reinforcement and wire mesh

The leakage after completion of the blasting (exclusive of bore water) stabilized at 1600 l/min, for the whole tunnel.

This means that 500.000 m^3 of water was pumped out during the construction period, corresponding to 8 times the excavated tunnel volume

Economy

If blasting costs and costs due to the pumping system, pipes and emergency generator are taken as 100%, the costs of stabilization works amount to 30% and the costs of investigation borings, injection and water problems amount to 25%.

Compared to a corresponding horizontal tunneling operation without saltwater problems, the above costs constitute nearly a doubling of the blasting costs. The operation in the sedimentary rock formation from the Herøya side did not give any saltwater problems at all. However, precambrian rocks in the horizontal part of the tunnel driven from the Herøya side, led to a good deal of leakage.

Final remarks on tunnelling

Construction and operation on such a submarine tunnel lead to special problems and the degree of difficulty and costs depend heavily on the rock conditions. One must establish a realistic balance between pump system and extent of injection, such that leakage does not exceed pumping capacity. Investigative borings must be carried out to check against larger water breakouts. Tunnel projects of this type can realistically be completed within a given economical bracket, when special measures are taken and when the rock conditions are normal.

FOLLOW-UP

After the tunnelling-work was under way, NGI compared geological measurements in the tunnel to the geological profile obtained from preliminary investigation. The investigation was also aimed at guiding the contractor and informing the owner with respect to required permanent stabilization work. As a first step and in agreement with the owner, NGI contacted K. Rønning, from the university of Bergen. From the study of fossils in the area, Rønning drew a fairly detailed profile of the cambro-silurian rock characteristics on the Herøya side. It was essential, among other tasks, to localize the boundaries of the black shales in the lower part of the cambrian sequence. The shales (called alun shales in Norwegian) lead to difficult building conditions, partly because they expand under weathering, and partly because their pore water can corrode steel and concrete. For the main part of the tunnel, in precambrian rocks, the conditions were geologically simpler, and no special investigations were undertaken except mapping of diabase dykes (see geological profile, Fig. 2) .

The geological profile gives the weakness zones encountered. On Fig. 2 is also given an overview of the lining and reinforcement recommended and as carried out. Shotcrete was used as preliminary stabilizer where the tunnel was driven in parts with low stability. Such zones were afterwards lined with cast concrete during the permanent stabilization.

As the two tunnel sections, the first driven from Herøya and the second from Rafnes, met in the middle of a large weakness zone, extensive stabilization was carried out with cast concrete at the working face. For the entire job, the stabilization work amounted to expected magnitudes.

Leakage was in general related to the precambrian rocks, while the cambro-silurian shale material gave little leakage. This represents a general experience, and as mentioned above, a similar experience was also gained with the earlier water tunnel driven under the river "Skienselva".

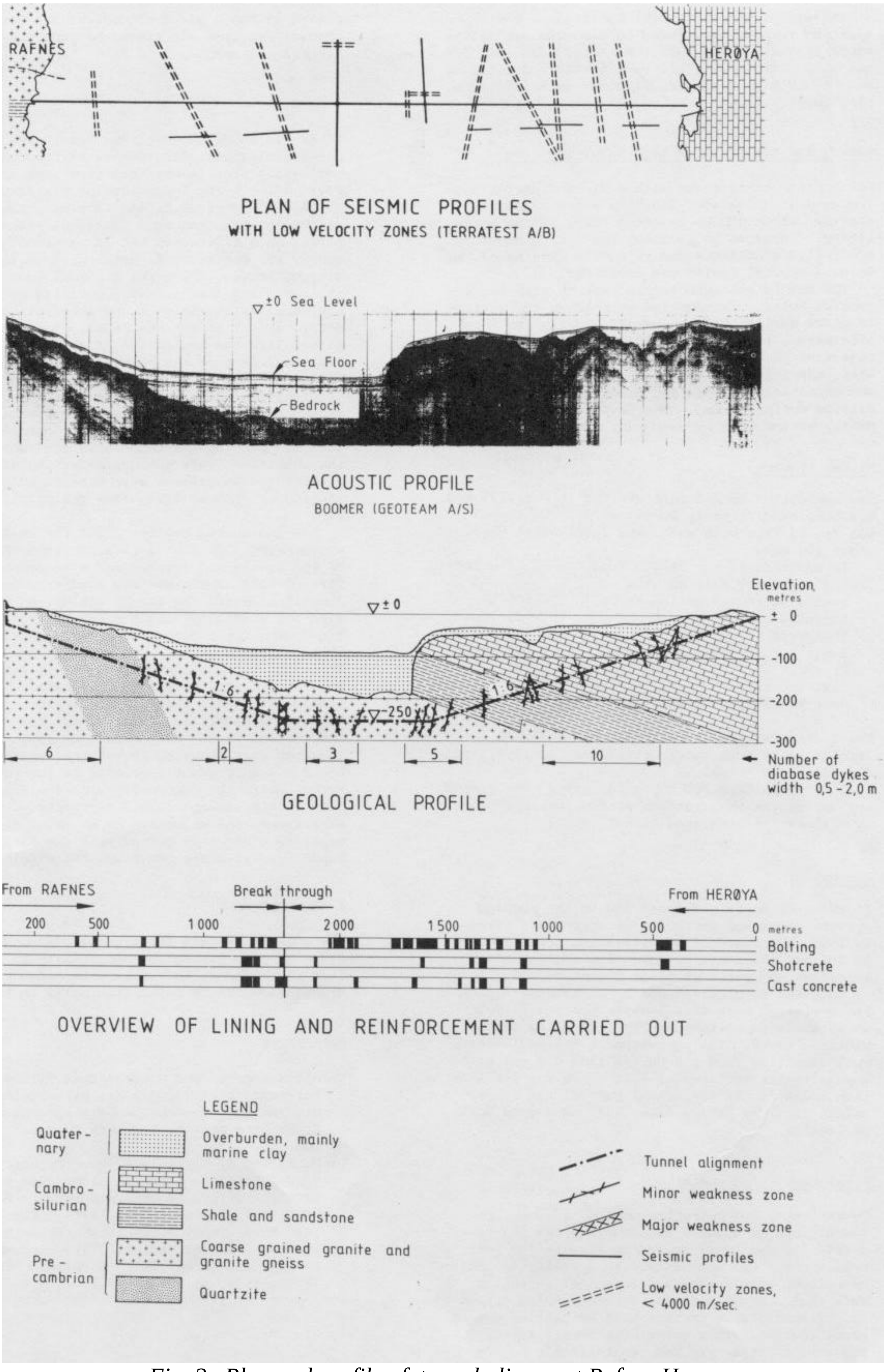

Fig. 2 . Plan and profile of tunnel alignment Rafnes-Herøya.

ACKNOWLEDGEMENT
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UNDERGROUND WATER TREATMENT PLANT AND RESERVOIRS IN OSLO

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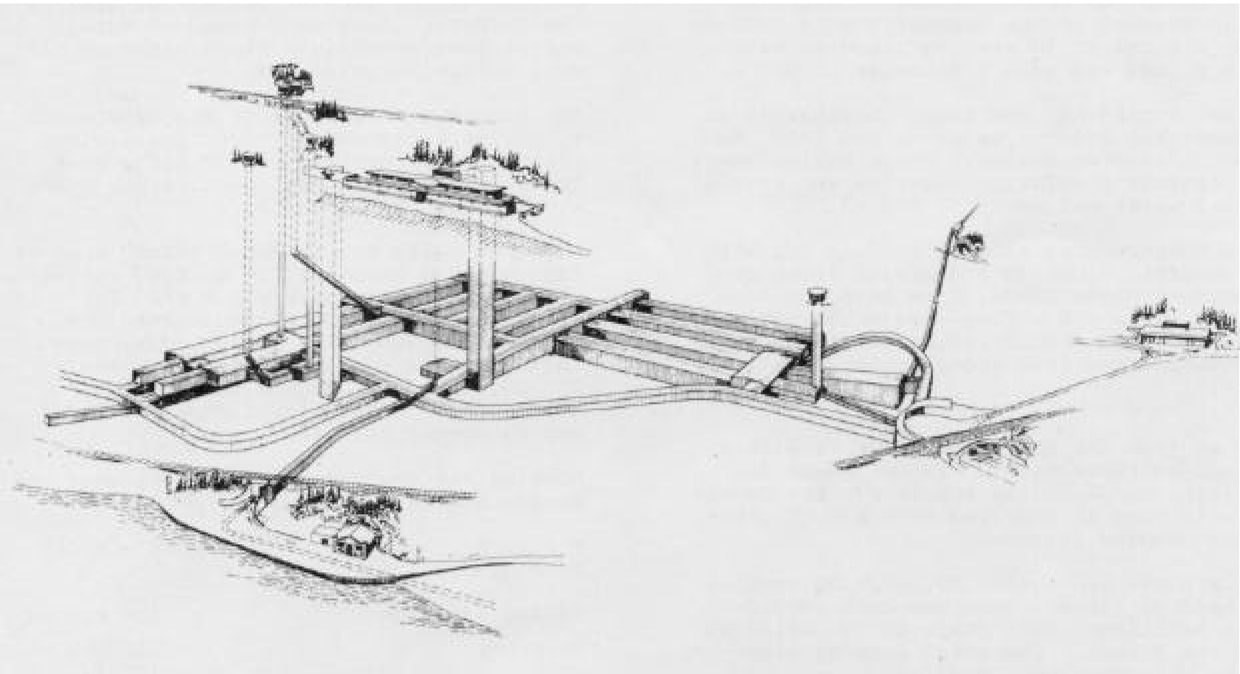
ABSTRACTS
The Maridal water shed, total area of 250 km^2, is the City of Oslo's main source of water supply and covers 75 per cent of the total consumption. Oset Water Treatment and Pumping Plant is located at the lake Maridalsvann, underground in solid alkaline syenite rock. The plant was constructed in the period 1966- 70.
The plant has a design capacity of 6 m^3/sec. It consists of a network of caverns and water carrying channels. The main caverns have a cross section area of approx. 200 m2. The paper describes shortly the water treatment from the intake and through the treatment plant. It also describes the principles of the structural design.
Part of the water from the treatment plant is distributed to a high pressure pumping station where the water is pumped to Årvoll - Åsen reservoir against a head of 115 m.
Total excavation volume was approximately 400000 m^3.



The City of Oslo water supply is based on the utilisation of surface water from the forest areas surrounding the city.
At present the city has at its disposal water sheds covering a total area of 350 km^2.
The Maridal watershed is Oslo's main source of water and supplies 75 % of the city's total consumption. Maridalsvann is the intake reservoir. The total area of the water shed is 250 km^2, and the average runoff is 6 m^3/sec.
During the late 1950's a major water quality investigation of Maridalsvann was carried out. This investigation revealed that Maridalsvann was a satisfactory raw water source for a large city.
In the course of 1963, after a number of laboratory experiments, the major questions surrounding the use of Maridalsvann had been satisfactorily resolved. Planning and design of the proposed Oset Water Treatment and Pumping Plant was initiated shortly thereafter.

The construction work for this subsurface freshwater treatment and storage plant was started in late 1966 and completed by the end of 1970.
The plant has a design capacity of 6 m^3/sec. It consists of a network of halls and water carrying channels. A large part of these channels are covered by concrete decks designed to carry vehicular traffic loads. The total floor area is close to 30 000 m^2.
The size and proportions of the halls as well as their sequence were determined mainly by their functions in the water treatment -and pumping processes. In principle, the plant has been built as two parallel units, each of which can operate independently.
The main water intake is near the deepest part of Maridalsvann, located at a depth of 30 m.
The water is clorinated at the upstream end of the 700 m long intake tunnel which has a cross section of 22 m^2.

At the downstream end of the tunnel, the water is pumped to a constant head by means of 12 pumps with a total capacity close to 12 m^3/sec. The maximum pumping head is 10 m.
The water is then aerated with compressed air to add oxygen and remove part of the free carbon dioxide. After being aerated, the water flows past clorination point 2 and into the retention basins which have a water volume of 19000 m^3. Clorinated and aerated water flows from the retention basins through a distribution channel to the microstrainers. There are a total of 30 micro-strainers, each 4,5 m long and with a diameter of 3,0 m. After straining, the third clorination is undertaken and the water is led into the four pure-water basins. These basins serve as diurnal regulation magazine and have a total water volume of 50 000 m^3.The construction at Oset is laid out with a concrete floor on a drainage layer of coarse-crushed stone, free standing concrete walls and a free span arch. This construction should reduce the risk of contamination from ground water to a minimum.

Water from the treatment plant is distributed to three mains. Two of these are gravity lines, while the third, the Oset - Årvoll main is supplied from a high pres- sure pumping station.
Four pumps are installed, working against a head of 115 m. Room has been provided for additional four pumps to be installed in the future. The total pumping capacity will then be 2,5 m^3/sec.
From the pumping station, the water is pumped through two 1000 mm diameter steel pipes to an underground valve chamber. From this valve chamber, a 1500 mm diameter steel pipe, embedded in concrete, has been laid in a tunnel up to Årvoll- Åsen reservoir. The tunnel is approximately 2400 metres long, and with a cross section of 8 m^2, it enables another 1500 mm steel pipe to be laid in the future. Årvoll - Åsen reservoir has a cross section of 100-110 m^2 and a length of 670 metres. Only the floor has been concreted at this stage, but if it becomes apparent that pollution from external sources is increasing, the walls and roof will be constructed as well. All the excavations are located in Nordmarkite, an alkaline syenite rock common in the Oslo area. Zones of weakness occur with a north-south orientation and a steep dip. The orientation of the various halls was chosen such that they cross the weak zones at a favourable angle.
Engineering geologists were employed to investigate the area before the final placing of the excavations was determined. The investigations were based on maps, aerial photographs and field work, no diamond boreholes were used. The extent of support work was determined as the work proceeded. Concrete lining was needed at certain particularly weak zones, but a large part required no other support than a few bolts.

The main halls at Oset have a free span of 13,2 m and a height of 16 m, i.e. a cross section of approximately 200 m^2. The halls were excavated in three sections. The top section was 8 m high, giving an initial cross section of approximately 100 m^2. Two 4 m benches were used to attain full depth. Årvoll - Åsen reservoir was excavated full face.
Most of the excavated rock was crushed and used by the municipal departments .

The work comprised the following approximate quantities:

Excavated volume:	400000 m^3
Formwork:	50000 m^2
Concrete:	50000 m^2

A BRIEF DESCRIPTION OF THE ULLA -FØRRE HYDROPOWER DEVELOPMENT

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GENERAL

In the south-western part of Norway the State Power System is planning and constructing the Ulla -Førre hydroelectric scheme, which, when finished, will be the country's largest hydro power system. The work in the field started in 1974 and will continue towards 1990, and the job is about half way through.

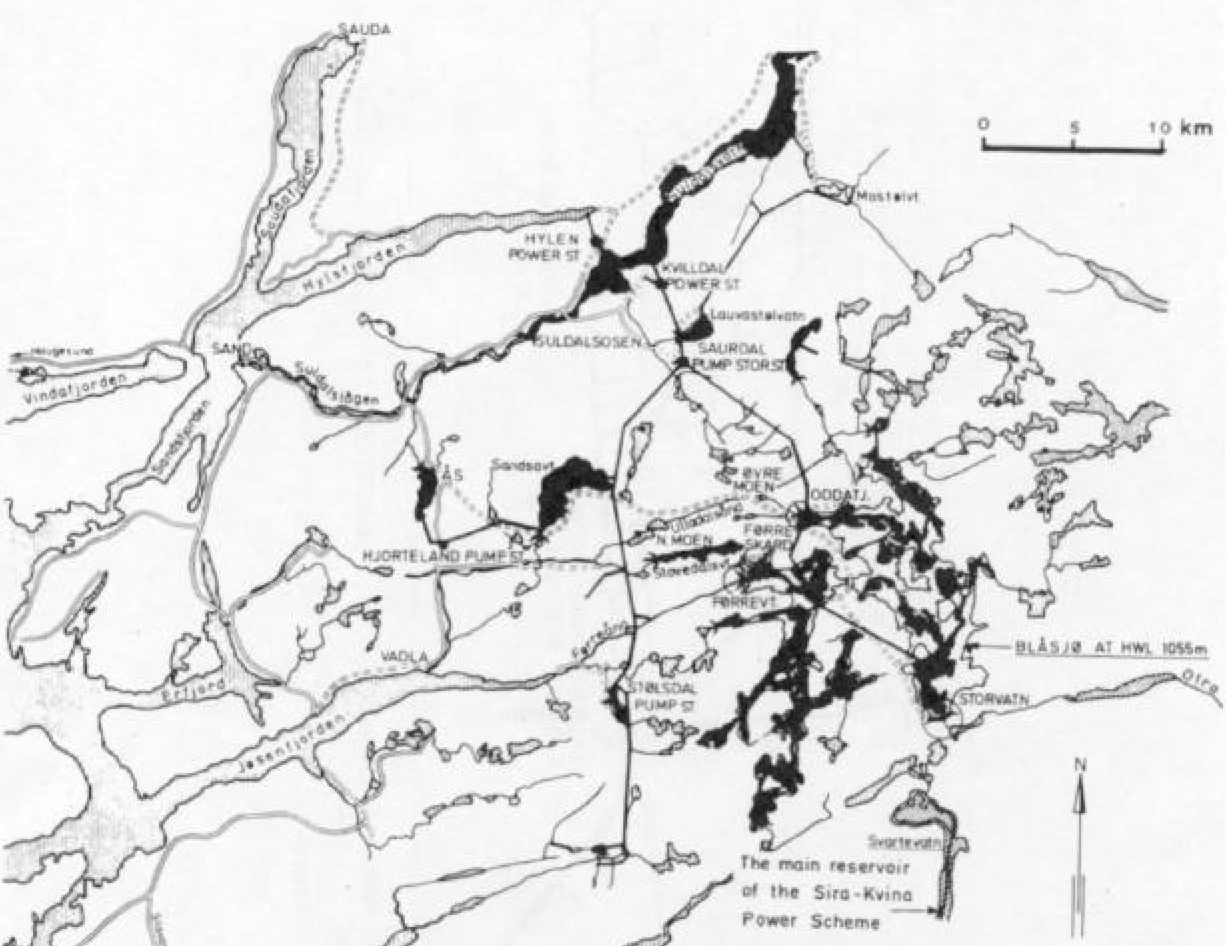
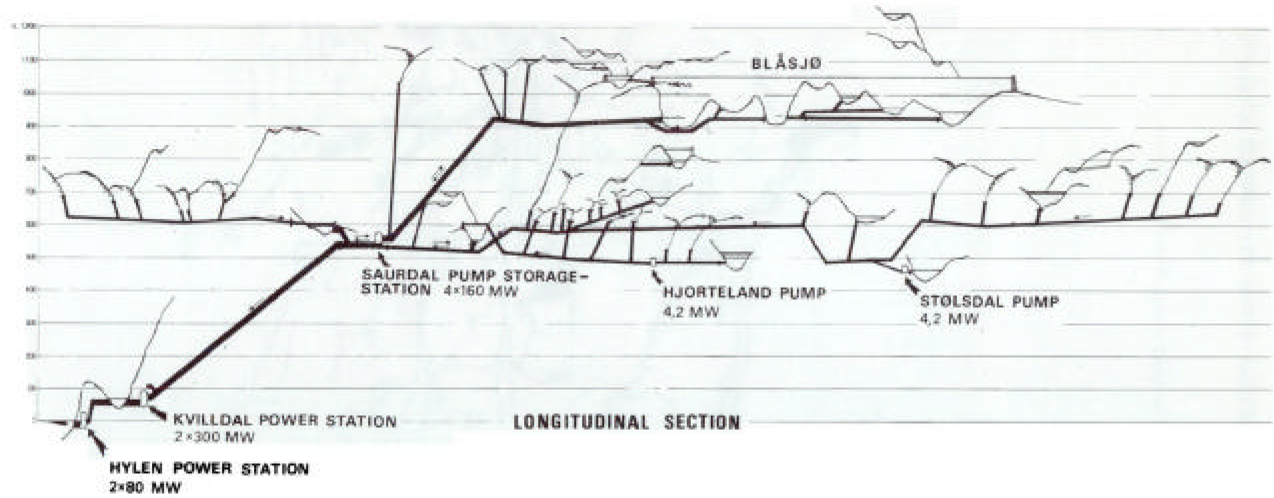


Figure 1.

The construction area is characterized by a highland plateau at about 1100 metres above sea level. The eastern part consists of hard gneisses and granites, well suited to tunnelling and rock cave blasting. The western part is covered by various rock qualities, less suitable for such activities. The catchment area covers 2000 km^2 and the precipitation is rather high, nearly 2000 mm a year, decreasing towards the east. The project utilizes the water in 3 stages down to sea level, in round

figures, metres a.s.l.: 1000-600, 600-70, 70-0. Total installation in these 3 steps will be 2040 MW and the mean annual production of the project 4350 GWh.
At elevation approximately 600 a number of rivers and streams have been connected by tunnels and shafts, thus collecting the run-off, leading it to two small reservoirs, the only ones in this area. However, a large reservoir was needed. A very suitable location was found at the elevation of approx. 1100, where an artificial lake could be established, with a surface of 82 km^2, a volume of 3100 mill. m^3, and an energy content of 7,8 TWh. This lake will serve as a back-up reservoir for the country's power system in periods of water shortage, and Ulla -Førre's contribution to the national firm power production will thus be greater than the figure mentioned above.
The reservoir will partly be filled by the run-off from its own field, partly by pumping from the 600 metre level through the Saurdal pumped storage station. Pumping will be done in periods with abundant water, i.e. in summer, in draw- down periods Saurdal acts as an ordinary generating station.
The Saurdal pumped storage station is thus the uppermost stage. The discharge from this station goes directly to the headrace tunnel of the next step, The Kvilldal Power Station, which utilizes the head from 600 to 70 metres a.s.l. From this level down to the sea Hylen Power Station comprises the last stage of the Ulla -Førre system.



THE RESERVOIR AREA
As already mentioned, a number of small lakes at the elevation of 1000-1100 have been regulated to form one large reservoir. Below natural water level the communication between the original lakes are established by a system of interconnecting tunnels. Above natural level the water is retained by a number of dams, four of them are mentioned here:

Dam Storvatn will be the greatest rockfill dam in Norway, dam volume 9,1 mill. m^3. It was originally planned as a fill dam with a moraine core, -the most common Norwegian dam type. But because of the long -and correspondingly expensive- transport of the core material, an asphaltic core was chosen, the thickness of which varies from 80 cm at the foundation to 50 cm at the top. The dam has a maximum height of 90 metres and a crest length of 1400 metres.

Dam Oddatjørn is also a rock fill dam, but in this case with a moraine core. Dam volume amounts to 5,7 mill. m^3, dam height to 140 m.

Dam Førrevatn: because of the topographical conditions the natural choice was a heavy concrete arch dam. On both sides of the gorge
the arch is connected to long gravity dams. Maximum height of the arch is 90 metres, total length of the dam crest is 1300 metres and total concrete volume is 245000 m^3.

Dam Førreskard: conventional rock fill dam with moraine core, height 80 metres, dam volume 1,5 mill. m^3

THE POWER STATIONS

A 10,5 km long tunnel of 100m^2 cross section leads from the reservoir at the elevation of 1100 to the Saurdal Power Station, located underground. Part of the tunnel is inclined 1:10 towards the four steel-lined shafts leading to the turbines. Maximum static pressure in the inclined tunnel is 400 metres. Between the tunnel and the penstocks a vertical surge shaft 450 m long and with a cross section of 90 m^2 is established.

In the central four vertical units of 160 MW each will be installed. Two of them are conventional turbine/generator sets, whereas the other two are reversible and therefore can be used either as turbine/generator or as pump/motor.

Starting procedure for the pumping operation is "back to back".

The transformers are placed in a separate hall, from here cable connections to the gas isolated switchyard above ground.

Kvilldal will be the country's largest hydro station when it is completed. The headrace tunnel is 3,3 km long with a cross section of 135 m^2. It is inclined 1:9 towards the station, and the maximum static water pressure of the inclined tunnel is 480 metres. Steel lined shafts lead to the four Francis turbines, each with a capacity of 310 MW. The measures of the machine hall are: length 130, width 20, and maximum height 43 metres, total volume 70 000 m^3.

Whereas Saurdal has a conventional surge shaft in the headrace, Kvilldal is supplied with a pressurized air chamber to take care of the oscillations in upstream system. The air chamber was chosen both for technical and economical reasons. The system is described in detail in the following article.

The first aggregate was put into operation in December 1981, the next set is due in February 1982.

Hylen Power Station utilizes a brut to head of 67 metres down to sea level. The station is equipped with 2 vertical Francis sets, each of 80 MW. The headrace tunnel has a cross section of 160 m^2, the outlet tunnel 200 m^2.

The station has been in operation since 1980.

The tunnelling works are of course far more comprehensive than described in this article. Total tunnel length when the Ulla -Førre system is finished will be 100 km with cross sections from minimum to 200 m^2. In addition about 3000 metres shafts of varying lengths.

The tunnels are mostly driven in the conventional drilling-and blasting way, but about 10 km will be handled by a full face boring machine, diameter 3,5 m. Because of the rock quality, a coarse- to middle-grained gneiss with large feldspat crystals, we feel that this represents an experiment for a tunnelling machine with 15 1/2 in. cutters.

According to the Norwegian way of tunnelling practice the tunnels are left unlined. Roof bolting, eventually combined with shotcrete is used when roof support is needed. Only when extremely bad rock is encountered a full concrete lining is used.

AIR CUSHION SURGE CHAMBER IN KVILLDAL POWER PLANT THE ULLA-FØRRE DEVELOPMENT

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The Ulla -Førre Development, which will get two power plants, one pumped storage power plant, and two pumping plants and be ready in 1987, are situated near Stavanger in south-western Norway. The owner and chief con- tractor for the construction of this project is the Norwegian Water Resources and Electricity Board, State Power System.
Kvilldal power plant, with its first generating unit in operation in December 1981, has a so-called air cushion surge chamber as the means of regulation in its head-race, with the following considerable dimensions:

Excavated volume including transport tunnels:	120000 m^3
Excavated chamber height:	17-24 m
Excavated chamber width:	16 m
Air cushion volume:	Varying between 80000 and 95000 m^3
Water surface area:	5200 m^3
Maximum air pressure, ata:	4.2 Mpa
Free air quantity:	3.2 million m^3

These dimensions must be looked at in relation to the size of the power station. When fu11y developed it will have an installation of 4 generating units with 1200 MW effect. Maximum water flow per unit will be 70 m3/s.

WORKING METHOD

As one knows, the air cushion surge chamber has the same function as the traditional surge shaft, i.e. to damp variations in the water flow and the oscillations in the supply tunnel and pressure shafts. The water surface in the surge shaft open to the atmosphere is in this case replaced by a closed underground chamber with a water surface exposed to com- pressed air. This is a relatively new principle in hydro power, and its application is partly related to the desire to construct the supply tunnel in the form of a slanting, trafficable, and unlined pressure tunnel leading from a lower, central adit in the power station area.
It would be natural to consider an air cushion surge chamber in connection with such a solution, which as a whole will enable the number of shafts to be reduced to a minimum. On the plus side one can also mention the advantage from the landscaping point of view in avoiding adits and rock tips on the hillsides.
From the purely regulating point of view, an- other considerable advantage can be obtained over the surge shaft because of the shorter distance between the free water level in the surge chamber and the turbine.

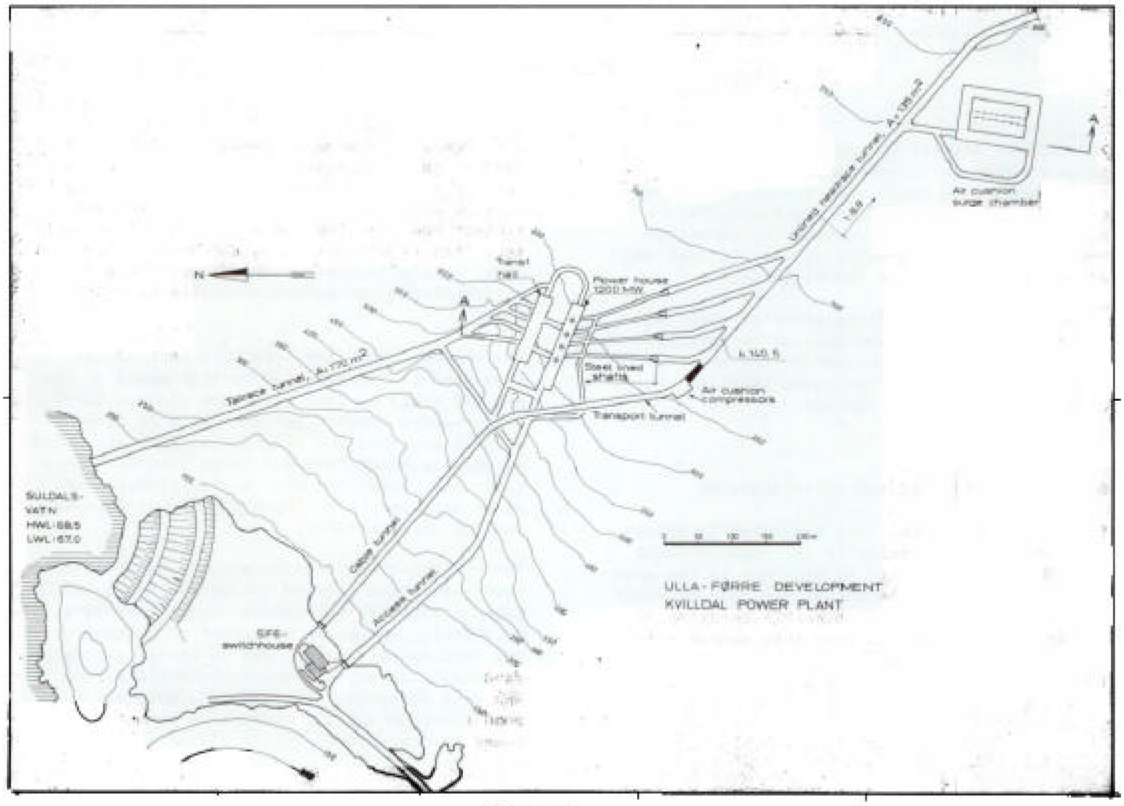


Figure 1.

PLANNING AND PRELIMINARY INVESTIGATIONS

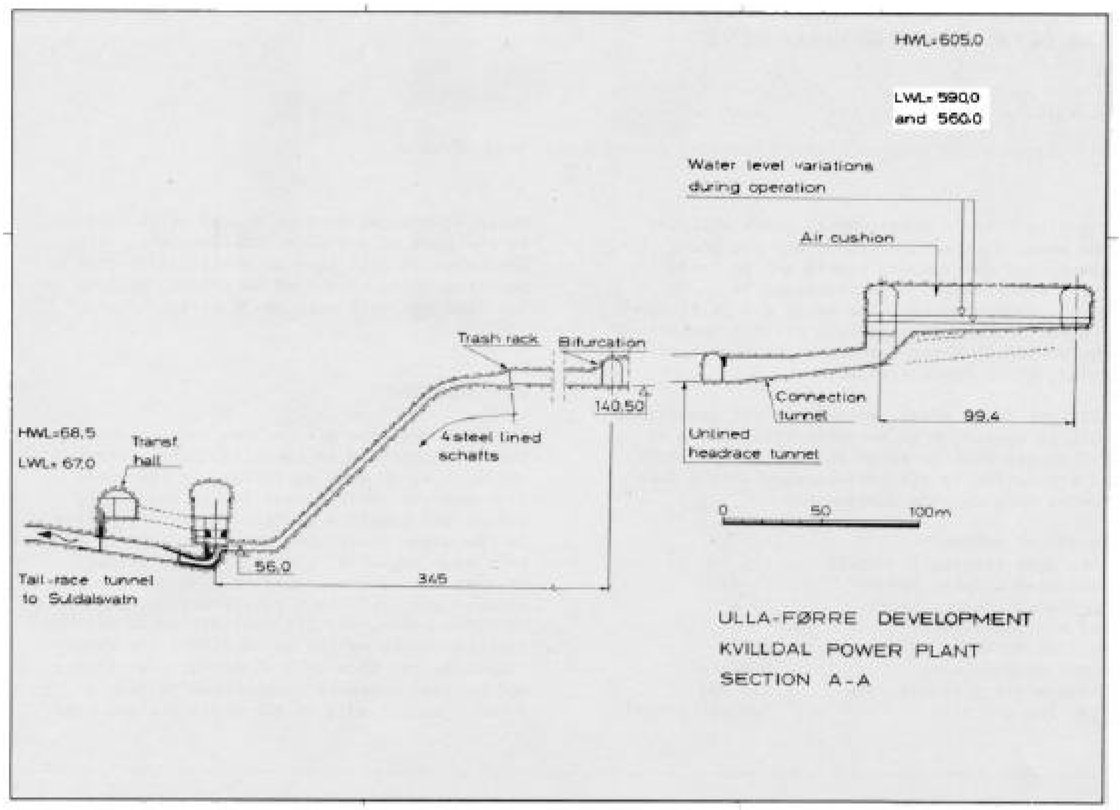
The supply tunnel had been excavated in advance, thus providing reliable information about the first-class quality of the rock in the area that ~ be suitable for an air cushion surge chamber. In addition a systematic charting of the area was undertaken from this tunnel with drilling and sample-taking -altogether approx. 1600 lm distributed between 8 to 10 holes.

The rock in the area in question consists of massive gneiss-granite crossed by several marked fault zones. Nevertheless, thanks to the careful preliminary investigations, the whole project could be located within an area completely free from fault zones and joints. This is, above all, of essential importance for ensuring air-tightness during the operation of the power station.

The chamber takes the form of a rectangular

ring tunnel with external lateral edges of approx. 78 x 115 m. This gives the project an advantageously compact and tightly knit form without any need for wide spans. When all these favourable conditions was taken into consideration, any comparison with a traditional surge shaft was clearly to the advantage of the air cushion surge chamber.

The hydraulic calculation and the model experiments have been carried out by the River and Harbour Research Laboratories at Trondheim University. Norwegian Institute of Technology. The rectangular form proved to be hydraulically favourable as well, because the oscillations and wave movements were reduced. Moreover a good location for the connection tunnel was found in one corner.



EXCAVATION

The excavating method was quite conventional as regards both the equipment used and the way it was carried out, but the requirements with regard to accuracy and careful blasting were very strict. This would have an important effect for keeping the rock surface free from fissures (tightness) and for the carrying out of any sealing- and tightening work that might be necessary. The chamber was excavated by way of a separate transport tunnel, which was sloped up to the chamber bottom and from there further up to a top heading (see figure) .After that the work sequence was as follows:

1. Full face excavation of a 7.5 m high and 16 m wide top heading. Contour holes 48 mm in diameter and at a distance of 80 cm. Light explosive charge.

2. After the top heading had been excavated along the whole length of the chamber, the next step was the mining of a bench, 10 m high, full face. This was done with vertical holes after a presplit (64 mm in diameter and at a distance of 80 cm) along the walls.

3. The bottom bench varying in height from 0-6 m down to the finished bottom. Horizontal hole-drilling.

This important project was carried out in a manner that can be characterized as very successful with regard to price, time used, and the quality of the work. The rock surfaces have been made especially even and fine. This excellent result can be attributed to the following factors:

4. Decisive importance was attached to localizing the very best area for the project. We have succeeded in doing this to such an extent that safety measures (bolts, shotcreting, etc.) have not been carried out at all. As regards air-tightness, no injection was found to be necessary either.

5. The work never faced any great difficulty as regards meeting the deadline, so that the highest priority could be given to quality the whole time. Moreover the working method used enabled and resulted in the successful use of two separate enterprises:
Drilling and blasting: The State Power System
Loading up and carting away: A private contractor

6. Regular checks and recordings during the carrying out of the blasting work.

OPERATIONAL SUPERVISION

The volume of the air cushion, the location of the water level, and thereby also the pressure are determined on the basis of the dimensioning regulation situation. In the course of operations the water level will rise and fall in step with the pressure variations in the supply tunnel, and the air cushion will thereby follow the gas laws. Part of the supervision will therefore consist of controlling that the given and necessary product of pressure and volume remain constant (Boyle-Mariotte, with corrections for temperature if necessary). Too Iowa PV product means, for example, that air has leaked out and that some will have to be pumped in.
A very precise safety check will be carried out. In principle, this will consist of ensuring that the water surface in the surge chamber does not become too low as a result of the pressure in the head-race, thus causing a dangerous situation to develop. Primarily this safety check will raise an alarm if the water levels in the surge chamber itself are low. The following system has been introduced, in connection with successively falling water levels:

1. Preliminary warning:	Rapid stopping of all generating units, i.e. controlled running down.
2. Alarm:	Instant, automatic emergency stopping of all generating units.
3.	Defined lower safety limit, with a good margin for blowout.

In addition to and independent of the abovementioned items, there is an automatic emergency stop system connected with the possibility of a dangerous development in the supply tunnel itself. This is based on precision measurement of a critically low pressure measured in the turbine area. This emergency stop will in time function a step ahead of item 1 above. Furthermore strict instructions will be given with regard to operating the gate in the head water so that any undesirable development in the supply tunnel will be eliminated.
The pipelines and pipes for air-filling and pressure measurement in the surge chamber are fitted into a concrete bracket. Beside the concrete plug close to the bifurcation tunnels, in the adit, an instrument room has been set up for registering and reading off measurement data. Here there are also three compressors for maintaining the quantity of air. The signals then proceed further to the control room and operational center of the power station.

NORWAY TAPS FOLGEFONNI GLACIER

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GENERAL

Folgefonni (fonni = glacier) , Norway's third largest glacier is situated in a mountainous region by the Hardanger fjord east of Bergen on the west coast, see fig. I.
The glacier covers an area of 225 km^2 with the top at 1650 metres above sea level. The western part of the glacier adds a substantial part to the catchment area for Folgefonni Power System, see fig. 2. These wild and inaccessible mountains made it necessary to construct the most advanced and complicated system that the Norwegian State Power System so far has completed. The main power station at 250 MW and a secondary one, a pumped storage station of 35 MW, together generate a total production of 1180 GWh in an average year.
An arm of Folgefonni, called Bondhus Glacier, drops down through a steep and narrow gorge to 450 m above sea level, see photo I. The annual runoff including glacier melt, amounts to 60-70mill. m3. To bring several streams from the glacier into the power system a 6 km long collection tunnel was established between the Mysevatn reservoir and the Bondhus Glacier. Here the only way to catch the runoff would be to construct a subglacial intake. Melting of holes from the ice surface to bed- rock indicated an ice thickness of 160-170 m.

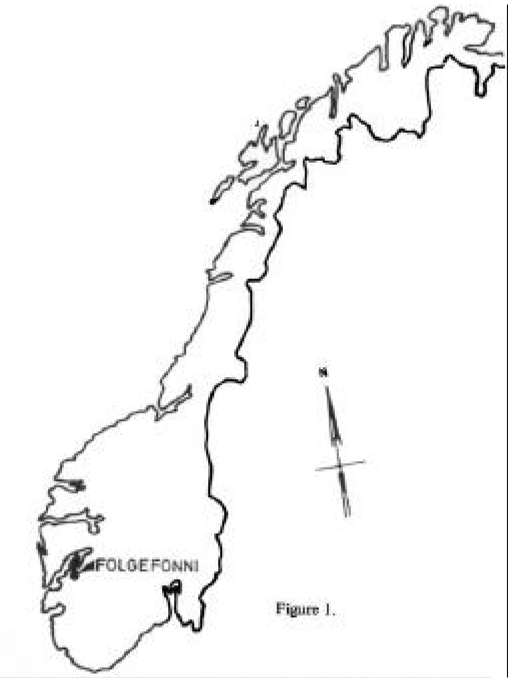

Figure 1.

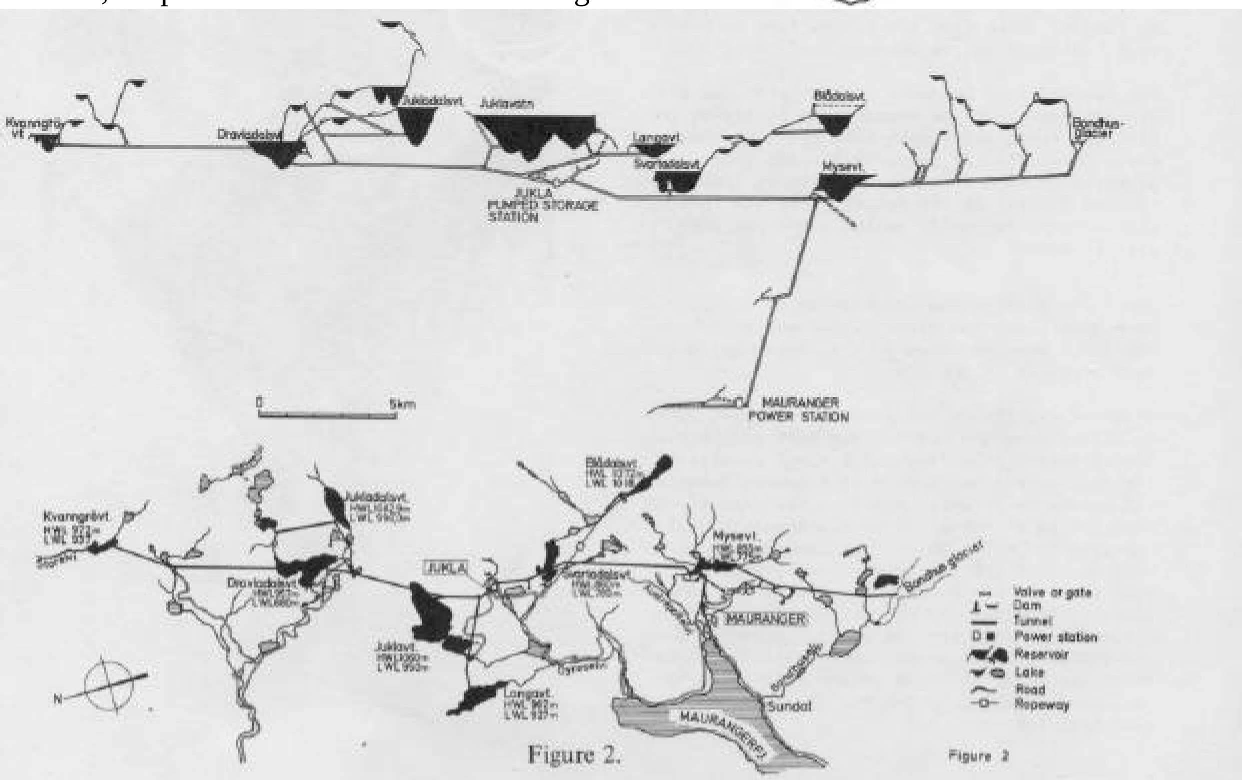

Figure 2.

Figure 2

One was aware of the many unknown problems and disappointments to be encountered in trying to find the river underneath that much ice. However, other similar projects already under planning could draw on the experience from the Bondhus Glacier in the future. This was then the decisive factor in making a bid for sub-glacial water.

The first problem was the mapping of the bedrock underneath the glacier. Seismic soundings could not be used owing to the narrow gorge and deep crevices in the ice. Previous experience had proved that holes could be melted from the ice surface with an electric heated spear. This revealed a rough profile of the surface but not enough to find the subglacial stream or streams. However, one knew enough to drive 10 m^2 tunnel 20-30 m underneath what appeared to be the bottom of the ice, and to be sure to catch all of the subglacial streams, well past the low point indicated by the found profile. From this tunnel drill holes through the rock and into the ice gave a rather good picture of the topography, but few indications as to the exact location of the water. Shafts were then driven from the tunnel through bed- rock into the ice.

Electricite de France had been working with subglacial intakes in the Alps for years and had developed a technique for making tunnels in ice, using hot water (40-50ºC) .This experience was put to use on the Bondhus Glacier and gave very exact information of the topography underneath the glacier.

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As the ice moves about 30-40 cm on the top and 0-20 cm at the bottom a day, only 30-40 meter long tunnels were practical. This is because as loon as the melting operation stops, the deformation and general movement of the ice soon close the tunnel. Up to this point, the only access to the construction site underneath the glacier had been by a 1 km long ropeway up to the adit of the 6 km long collection tunnel, see fig. 2. From this point the transport had been undertaken by muck-cars through the tunnel. In addition to the planned subglacial intake there were sever- al other conventional intakes on the tunnel. In order to use this inflow in the already finished power station, the tunnel had to be temporarily separated from the glacier intake system. Before that, however, a new access was needed. This was obtained by branching off with a 500 m long tunnel out in the vicinity of the glacier.

To this point the only transport available, was by helicopter. An enlargement of the access tunnel was made into a new workshop and storage room. Outside and above the tunnel entrance a combined helicopter-platform and living quarters were constructed.

The subglacial water carries a large amount of materials with particles ranging from clay to big boulders. From observations below the glacier the amount of materials that would follow the water through the subglacial intake were estimated at 3-5000 m^3 per year. To prevent this from clogging

the tunnel, it was decided to make a 5000 m^3 chamber where the materials could be deposited. The chamber was constructed to be emptied by loaders and muck-cars left behind before the collecting tunnel was plugged.
The task of locating the river underneath the glacier proved to be more difficult than anticipated. The ice tunnels had uncovered a rock surface full of cliffs and cavities. But by tedious and systematically work the contours of the bedrock could slowly be filled in. With the help of geophones, drilling, tracking by spraying coloured water in the glacier and new ice tunnels, finally in the winter 1977-78, after four years of searching, the main water- stream was located.
To construct the intake, a 15 m tunnel at 1:10 was driven till only 3-5 m of rock remained underneath the riverbed. From this tunnel a 10 to 15 m long horizontal tunnel was driven in both directions. From these tunnels 4 narrow slits were blasted into the riverbed, see fig. 3.
The intake has performed very well, catching close to 100% of the glacier's total runoff.
The loose material deposited in the chamber has varied in amount from 1500 to 3000 m^3 per year or somewhat less than

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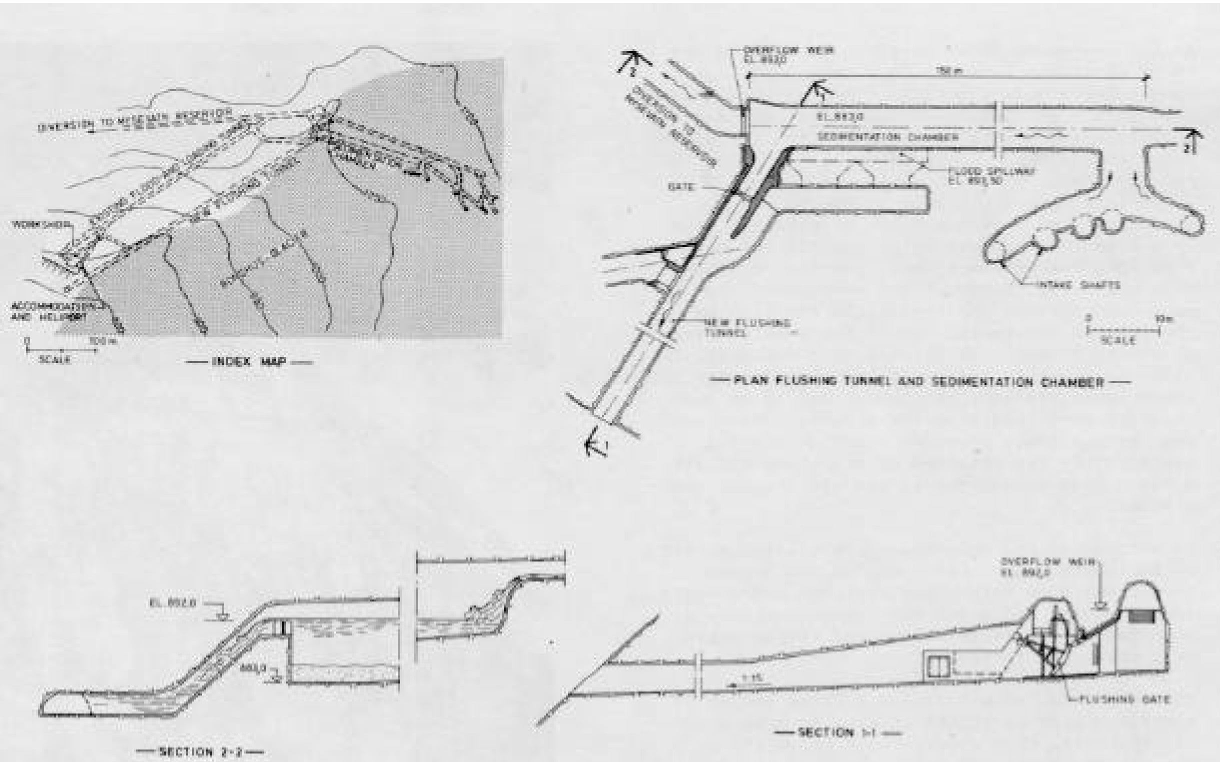

Figure 3.

INTAKE BONDHUS-GLACIER
GENERAL LAYOUT

expected. Among the glacial debris there has also been a great deal of boulders, up to 5 m^3. The largest of them have been blasted before handling. Whereas the summer runoff varies from 3 to 15 m^3;sec., January through April seldom see more than 0.5 m^3/sec. The removal of the materials in the chamber therefore has been done during the winter. However, the combination of bad weather and helicopter transport have made the work very expensive. This is because the new working

environment law allows this type of work only to be performed with good flying conditions for helicopter in case of accidents.
As the intake is performing very well, it has recently been decided to construct a system where the water itself can empty the chamber through a new tunnel at 1:15, depositing the materials on the glacier below, see fig. 3. A flushing gate in the bottom of the chamber will be opened as soon as the accumulation of sand and gravel has reached a certain depth.
Model studies at the Norwegian Hydrodynamic Laboratories in Trondheim show that 4 m^3/sec. of water will empty 70-80% of the sedimentation chamber in 1 ½ to 2 hours.
The construction work of the new arrangement will take place during 1982-83. The new tunnel for emptying the chamber will be about 410 metres long and dropping off at 1:15. For access a tunnel will be blasted at 1:8 downgrade from the elevation of the existing construction facilities below the heliport. In the gate area the work involves blasting of space for the flushing gate and its operating equipment.
There will also be an accessible passage to a platform in the sedimentation chamber where one can supervise the flushing process.
The necessary equipment, including a LHD-shovel, will be taken apart and flown in by helicopter with load limits at 2000 kg. The construction work will be carried out by three crews working two long shifts a day and with one crew always on leave. The work is expected to be finished by summer 1983.
The task of locating the subglacial river at the Bondhus Glacier and constructing the intake has been very demanding. The use of helicopters as the only means of transportation has further enlarged the difficulties. The experience from this site, however, indicates that the difficult working conditions and demanding task ties planners, workers and location staff closer together to reach the final result.
To glaciologists the subglacial intake system at the Bondhus Glacier has provided a unique opportunity to study the bed of a more than 150 m thick glacier. Owing to the good housing and workshop facilities it has been possible for the first time in the world to do scientific work underneath such a thick glacier. A lot of observations, measurements and samples have been taken in ice tunnels, and instruments for continuous records of pressure, temperature, strain and ice moving velocity are operating at the glacier bed. This has provided, and still will provide, a lot of new very interesting and much needed field data to glaciology.

References:
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The Fo1gefonna Project, Water Power, Dec. 1970

Norway drives for low cost road tunnels

***David Martin,** Editor*

Norway probably has more road tunnels per head of population than any other country. This would have been impossibly expensive if only conventional fully fitted and lined tunnels were built, but huge savings have been made by careful site selection, smooth blasting, and by avoiding the use of expensive concrete linings where possible.

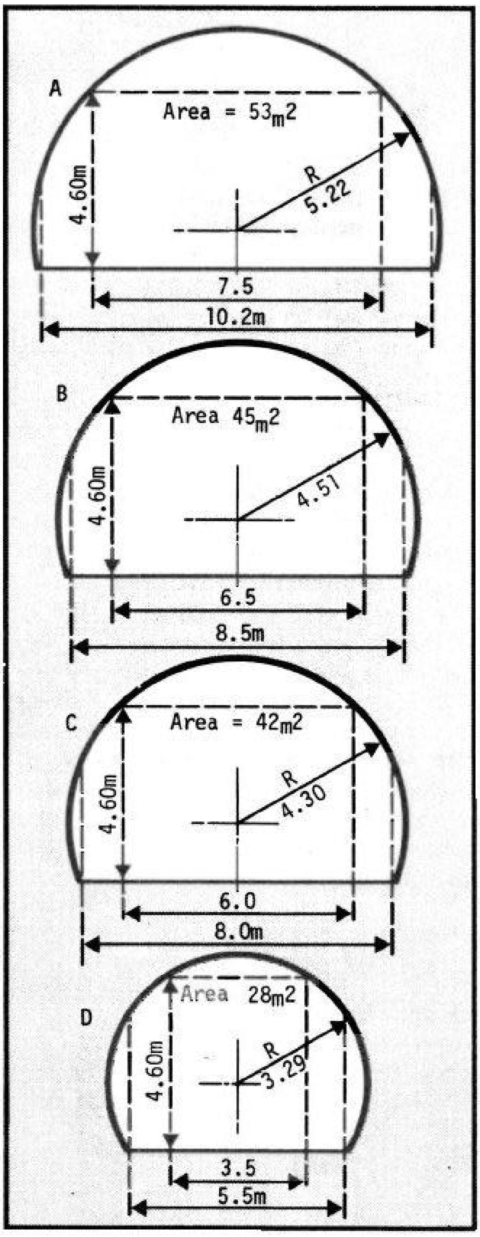


Fig 1. Standard profiles of Norwegian road tunnels, A to D top to bottom.

Norway has successfully proved that up to two thirds of the cost of constructing road tunnels in hard rock can be saved if the right techniques are used, and if expensive permanent concrete linings are largely avoided. The Norwegians have been using unlined hard rock tunnels for a very long time, but have developed the system technically into a fine art since they started road tunneling in earnest after the Second World War, and particularly since they set up a special committee to study the problem in the late 1960s.

The need for tunnels in Norway stems from the geography of the country, which has a population distributed in widely separated towns and villages often on opposite sides of high mountain ranges. The large number of islands and very deep fjords stretching many miles inland have made the need for tunnels even greater to enable a satisfactory road network to be developed and gradually to replace some of the 250 ferry links in the country.

Already there are probably more than 750 road tunnels of one type or another in Norway, some 400 of them on the primary national road network and adopted by the Public Roads Administration (PRA). The remainder are on country or secondary roads, and may be community or privately owned. The private owners are allowed to charge tolls for use of the roads and tunnels, but there is a program for the PRA to adopt the more important of these in due course. Many of these roads have been built by contractors principally for their own use in constructing large projects such as remote hydroelectric schemes. The tunnels on these roads can be just as long and useful to the traveler as those on the adopted national network.

(right)
Polyethylene for water and frost protection.

(far right)
Aluminum watershield in place.

The relatively small four million population provides a limited tax base to pay for the vastly extensive and expensive road network that is required. It has therefore been impossible for Norway to build the very expensive twin carriageway motorway road tunnels, lined from end-to-end with in-situ concrete and incorporating full lighting and
ventilation, which are common in Switzerland, Germany and Austria. More economical methods have had to be used, and frequently a virtue has been made of this necessity.

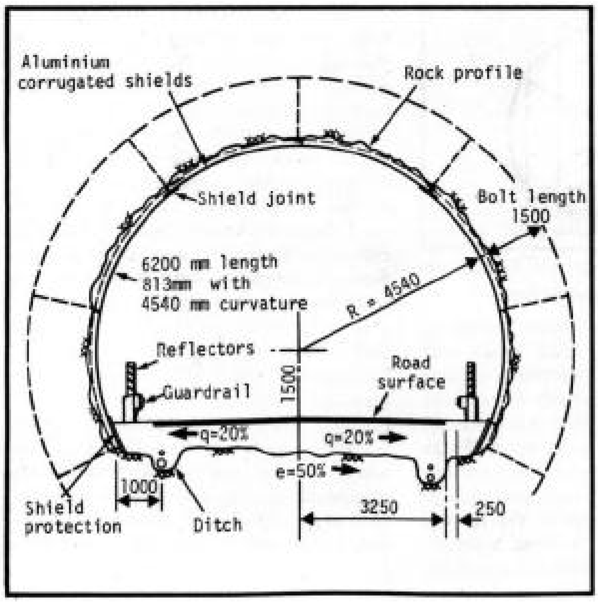


Fig 2. Cross section of tunnel showing position of aluminum corrugated shields.

However, there are many problems with unlined tunnels; fault zones and poor rock occur even in the best sites, and so most Norwegian tunnels have short sections within each tunnel where in-situ concrete linings have to be used. Water leaks through the roof of most tunnels to some extent, and this is compounded in winter because it freezes in the tunnel, making it impossible to drive through. With very cold winds blowing about the mountains, frost protection is also necessary inside some of the tunnels and particularly near the entrances. Special measures have been developed by the Norwegian Road Research Laboratory based in Oslo (part of the Public Roads Administration) to overcome all these hazards, and are now being incorporated in all new tunnels, as well as being used to up-grade older tunnels when money permits.
Conventional hard rock tunnels of the Swiss and Austrian type are lined with thick, multi-layered and expensive water- proof linings all the way through. First there is an outer layer of concrete (on the rockside); this is attached to a plastic waterproof membrane to keep the inside of the tunnel completely dry; and then there is the internal (roadside) smooth concrete lining often 25cm thick or more. The cost of this three-part lining, remembering especially the high cost of labour needed to install it, forms a large part of the total overall cost of these tunnels. This system is completely dispensed with in Norwegian low cost tunnels.

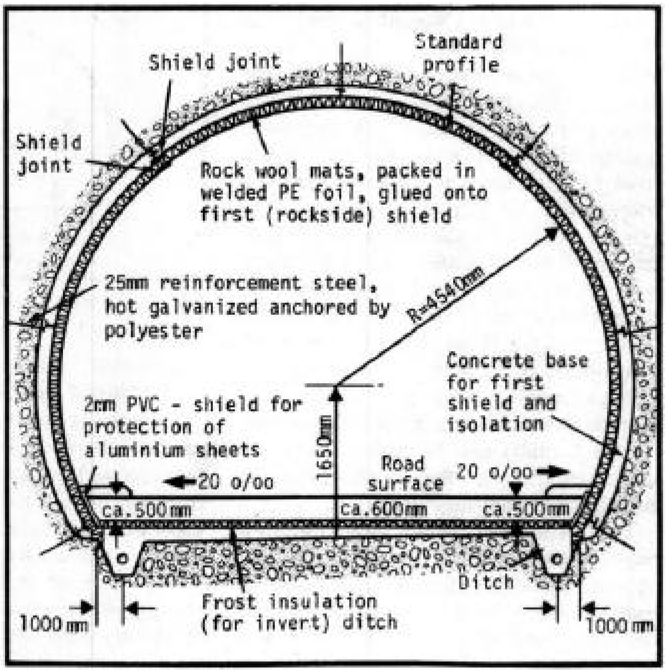


Fig. 3. Double lining of aluminum with frost insulation between.

First choose the right place

The first principle to follow when considering the construction of a low cost tunnel is to choose the best site. If sound rock can be chosen, then obviously the need for support will be less. Special attention must be paid to site investigation to ensure that the best route is chosen. With road tunnels, very often there is some flexibility of route so that the proposed line of the tunnel can be moved over somewhat if the original route is found to be fractured or to be through bad rock. But obviously this decision needs to be made before tunneling starts!

Having selected the best site and started work, special attention needs to be paid to the construction of strong portals. These are necessary to protect the road from rock falls and protect the entrances from the weather in winter. Some portals are as long as 100m at each end (Vard0 for example).

Most tunnels in Norway are built using drill and blast methods, and here the Norwegians excel with their smooth blasting technique, thus reducing the demand for rock reinforcement. If the tunnel is going to be substantially unlined when in use, smooth walls are very important, and overbreak is also reduced to a minimum. Only scaling and roof- bolting are used throughout the tunnel to prevent rock falls. Best results are accurately and by avoiding the use of too much explosive.

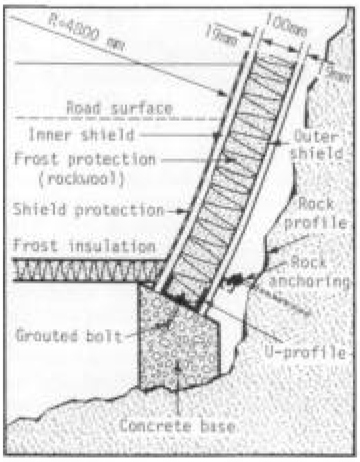
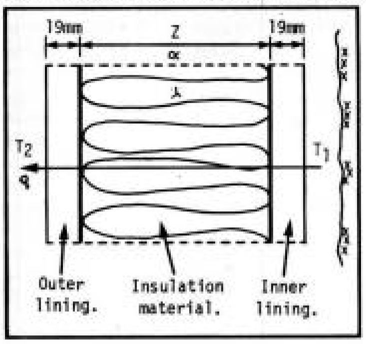
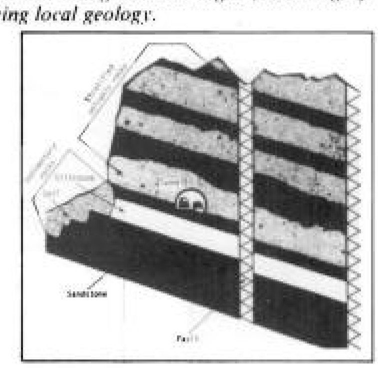


Fig 4 (left). Detail of corner where shield joins roadway. Fig 5 (below left). Loss of heat through insulated tunnel lining: T_1 temperature of water on the lining; T_2 temperature of tunnel air: T_1-T_2=ΔT: λ thermal conductivity of the insulation; z thickness of the insulation; α heat transfer coefficient between lining and air. Fig 8 (below right). Holmestrand Tunnel. Cross section showing local geology.





Head support used during construction must not interfere with any permanent support installed later. In most Norwegian hard rock, low cost tunnels there are a few areas of bad ground and the occasional fault which cannot be avoided. Concrete in-situ linings which can be up to 1 m thick are used at these special locations for relatively short stretches. But on average about 95 per cent of the length of Norwegian road tunnels in hard rock do not have this lining, its absence being the major factor in cost reduction.

Insulated panels for frost protection.
Water entering through the rock and cascading onto the road surface inside the tunnel is a problem which has been successfully solved by the Norwegian Road Research Laboratory. The conventional method of shielding which was developed in 1970 consists of corrugated sheets of aluminium fixed to the tunnel wall on brackets and joining at the top of the tunnel. Any water coming through the rock is diverted to the sides, collected in drains and transported out of the tunnel; Without this protection, not only is the tunnel unpleasant in summer with water sheeting down on the vehicles, but in winter it freezes making the road surface very hazardous and sometimes forcing the tunnel to close.

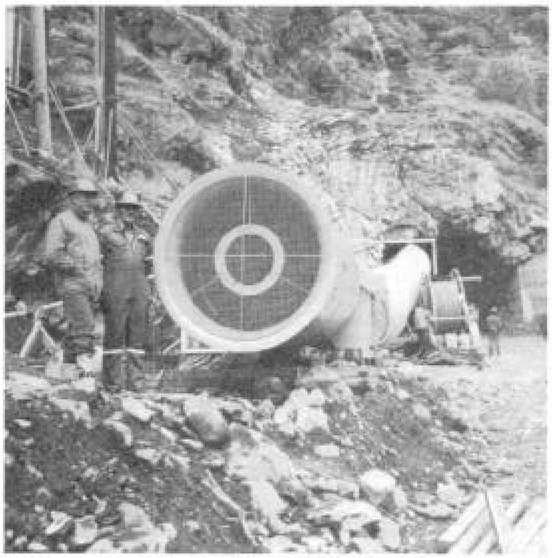
Ice in Norwegian tunnel when unprotected in winter.

In many places, insulated panels to provide frost protection are used. These frost protected shields are made of two aluminium sheets with rock wool packing between them. Various other types have been tried in an experimental tunnel at Grønvold and the best type has proved to be a sandwich structure, the outer skins made of a thin glass fiber reinforced polyester material (1mm thickness) with a thicker 5cm layer of polyurethane foam in the center. The polyester material totally encloses the polyurethane foam inside each separate panel. Each arch is made up of two elements fixed to the tunnel wall on brackets and meeting at the top. Each panel is 1.5m wide and interlocks with its neighbours so that erection time is reduced, making the linings cheap and easy to install. The space between the panel and the tunnel wall is packed with rock wool at each end of the structure so that cold air will not enter the void.

This new grp type is expected to be used for the first time commercially in the Holmestrand Tunnel (referred to later in the article). Its price and performance there will decide whether or not the road departments will take a further interest in it.

Other types of panel tried and now unlikely to be adopted used pvc on the outer skin with packings of rock wool, steel panels as the outer skin with a rock wool packing, and a type similar to the one chosen but with a less satisfactory suspension and interlocking system.

Another device used in Norway to keep the tunnels 'warm' inside is to fit large double doors at each end which are kept shut in winter, and have to be opened and reclosed automatically or by each passing vehicle. These are more common on the private roads, and those with infrequent traffic.



Tunnel entrance at Flåm showing Korfman fan.

Norwegian tunnels also save money by avoiding expensive installations. The Swiss, Austrian and German type of road tunnels are equipped with elaborate and expensive modern ventilation and lighting systems. Norwegian tunnels generally have none of these and it is very rare to find even a: few lights inside. However, tunnels are generally curved slightly near the entrances and exits so as to make it safer for drivers inside using full headlights. Ventilation is natural draught, and sometimes on a long tunnel where wind conditions do no ventilate the tunnel properly the headlights of on- coming vehicles appear in the distance to be a dirty red or brown colour due to the high proportion of nitrogen gas in the atmosphere. Three sites at which tunnels are presently under construction will be used as examples of the foregoing.

Holmestrand by-pass.

At Holmestrand on the Route E18 in the Vestfold district of Norway, a 1780m long road tunnel is being constructed to by-pass the main street of the town and keep the through traffic separate from the local traffic. The rock is a mixture of volcanic and sedimentary types with gently inclined strata consisting of basalt, agglomerate and siltstones with dykes and some faults.

A 70m long drift has been driven into the side of the mountain, and from a gallery inside, two headings are being driven outwards from the inside using drill and blast methods. One reason for this is that there is bad ground consisting of weathered and broken rock at both portal areas and this will be more easily overcome by tunneling out from inside. There is also the added advantage that only one drilling jumbo is necessary as it can be used for both headings. It can be drilling one face whilst blasting and mucking take place at the other.

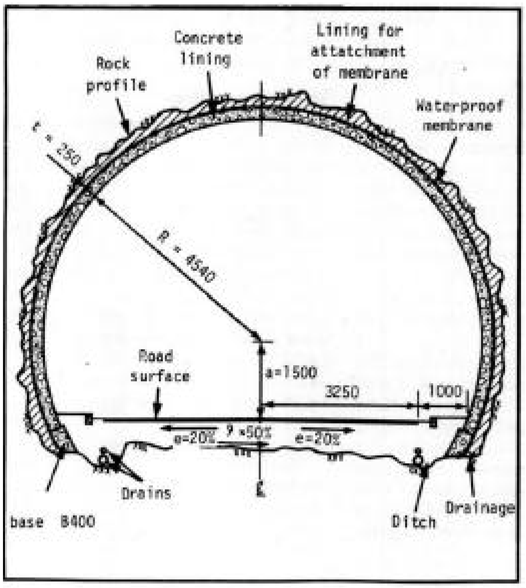


Fig. 6.Cross section showing concrete in-situ lining in road tunnel.

The rig in use here is an Atlas Copco 3-boom Promec 475. Some 120-125 blast holes and three 4in cut holes is the normal pattern to a depth of about 4m. Boart drill steels on Atlas Copco necks are used, with dynamite or ANFO as the explosive. The area of the tunnel face is 65m2 using Profile A (see Fig 1). There are nine men per shift and two shifts working per day. An average of 2.7 blasts per 24h has been achieved, and the best progress achieved in one week by the time of our visit was 72m driven in 11 shifts. The cost is said to have averaged 10 000 NKr per meter over a two month period. The

tunnel has a slight curve (r=4500m) as illustrated in Fig 9. Korfmann ventilation fans are used to supply air, and mucking out is with CAT 966 rubber tyred dump trucks. Client is the Norwegian Public Roads Administration and the contractor is the local Hordaland Road Department using the Administration's own consulting engineers and geologists as advisers.
Work started in September 1980 and completion is due in January 1982 although the tunnel is not due to be officially opened until June 1984. When *Tunnels* & *Tunneling* visited the site, work was estimated to be about 8 weeks ahead of schedule. The tunnel will be mostly unlined, but concrete will be used for small sections where there are faults, and waterproof elements or aluminium shields as described earlier will also be utilized.

Høyanger Tunnel
The second site visited by *Tunnels* & *Tunneling* was the 7465m long Høyanger Tunnel in the Sogn & Fjordane, which will provide a much needed road link on the north side of the Sogne Fjord. Its width is 8.6m with the standard C Profile (Fig I). The geology here is granitic gneiss, quartzite and amphibolite.
Drill and blast methods were used, with an Atlas Copco 3-boom hydraulic jumbo drilling at the eastern end and a Gardner-Denver 4-boom pneumatic jumbo at the western end of the tunnel.

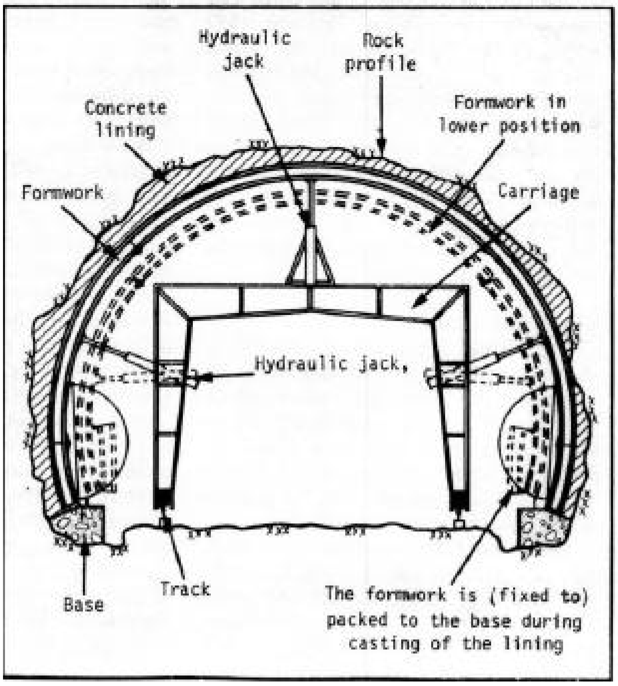


Fig. 7. Formwork carriage used for placing concrete lining.

Work started at the western end in November 1976, and at the eastern end a year later in November 1977. Hole through was achieved some four years later, in October 1980, 2700m from the eastern portal and 4000m from the western portal The tunnel is due for completion by April 1982, ready to be opened in May and integrated with the road network.
The number of holes in the drilling pattern was 83-90 plus 4 cut holes. Depth of each was 4.10 m and dynamite was used as the explosive. Two shifts a day were worked, Monday to Friday only, making a ten shift week. An average of 1.3 blasts/24h was achieved, adding up to 6 or 7 blasts/ wk. Coromant, and some Reploy and Seco drill steels were used.
Approximate costs per meter were said to be 17000 NOK.
Support in the tunnel was with rock- bolts, some 75 000 being used, as well as 4000m^2 of wire mesh. Concrete linings were dispensed with, except for one place where~ water-bearing faults made it necessary to have a 12m long section lined with in-situ concrete. A special Atlas Copco rock bolting machine was used to speed-up roof support, using resin rock bolts 2.4m long x 20mm. Aluminium water shields are to be erected at 15 different places, and special frost protection is to be provided at the eastern end of the tunnel where very strong prevailing winds blow through the tunnel causing ice formation in winter . The first 400m of the tunnel at this end, which catches the

particularly cold wind, is to have frost protection panels fitted. They will be of the conventional type already described.

Strong concrete portals extending 20m have been built at each end, and special avalanche protection has, been built at the western end. The tunnel is curved, mainly to make it easier for driving, but there was also some geological advantage. Traveling east to west, the tunnel rises slightly with a grade of 0.7% and at about the middle turns downwards with a grade of about 1.4%.

Very high horizontal stresses reaching 34 M Pa occurred in the rock in this tunnel, causing considerable rock bursts during construction. A great deal of scaling had to be carried out after blasting, and an unusually large number (75 000 as mentioned above) of rock bolts had to be used to control the situation. The site construction supervisor, Sigleik Hetle, told *Tunnels* & *Tunnelling* "if it wasn't for the spalling we could have done 600m more tunnelling each year and finished the job so much more quickly". In fact, not less than 20-50 rock bolts were used per 4m of tunnel.

During construction, ventilation was provided by the usual Korfmann fans, and after completion the permanent ventilation will consist of 2 x 14 300kW. axial fans, controlled as necessary.

The client is the Public Roads Administration, who provided its own consulting engineers and geologists. Construction was by the Sogn & Fjordane Road Department using direct labour, not by an outside contractor . Total civil work costs were said to be 127 million NKr. At the time of our visit, the work was running to schedule.

Tunnel to a village

A third tunnel site visited was at the tourist center and beauty spot of Flam, at the end of the Aurlands Fjord. It is an unusual town consisting of a large and rambling oldish hotel, now extensively modernized, a railway station and very little else. The branch railway connecting with the Oslo-Bergen main line reaches the town via a spiral tunnel and terminates on the quayside where ferries leave regularly for Aurland, Kaupanger and other parts of Norway. The fjord is also used by cruise liners, and during my visit I was pleasantly surprised to see the Cunarder *Queen Elizabeth* 2 lying in the fjord disgorging day trippers.

The tunnel itself can be viewed from the bedroom balconies of the hotel, and the blasting could be clearly heard all over town as progress had reached only 60m into the mountain during my visit. linking Flåm with the village of Undredal (population 200) which at present is cut off from the rest of Norway by road. But at some date in the future the road could form part of an all-year ferry-free road link between Oslo and Bergen, the latter city being reached by road only by using ferries.

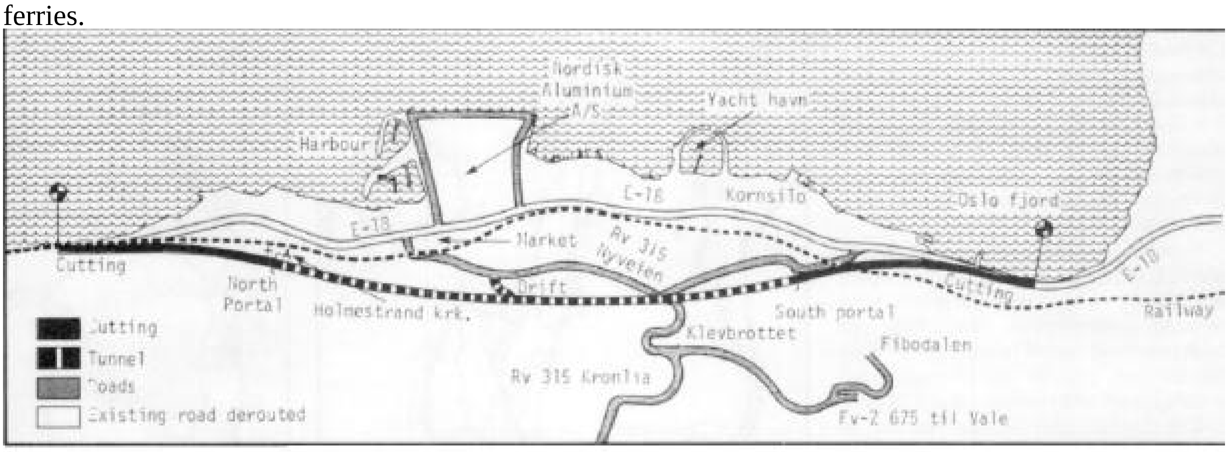


Fig. 9. Holmestrand by-pass tunnel. Plan showing route and drift.

The tunnel is being constructed with an 8.6m diameter following standard Profile C with a 44 m^2 face area. Being driven from one end only, it will be 5000m long when completed and will pass through rock formations consisting of mangeritic gneiss, amphibolite and anarthosite. Tunnelling is by drill + blast using an Atlas Copco Promec TH506 4-boom jumbo equipped with the COPI038 hydraulic drills. The jumbo is equipped with three BUTI5 booms, one BUT30C cut boom and a working platform. This C- version of the BUT30 boom for drilling the cut hole and has a centrally positioned feed and no feed dump cylinder. It is a simplified version of the BUT30, lighter in weight and intended for applications where there is no need for the dumping movement, ie no cross cuts or roof drilling. This was the first of these new units to be sold abroad and it was particularly interesting to come upon it by chance during my visit. The pattern consists of 80 blast holes plus four 3in cut holes, with a depth of cut of 4.l0m. Sandvik and Boart drill steels are in use and the rate of progress is said to be 2.5 blasts per 24h using two shifts, but during my observations from the balcony, progress appeared to be not too swift. This may be due to teething problems at the beginning of the drive.
The cycle time in this tunnel was given as: 2h for drilling; 1h for charging and then the blasting; Y2 for scaling; followed by up to 3h for mucking out. This adds up to a total cycle time of some 6 to 7h, but the site construction supervisor expected that this would speed up to 1 Y2 rounds/shift when they got properly into the job. Mucking out is with a CA T980B loader and three or more Scania dump trucks.
The tunnel will utilize aluminium water shielding and frost protection panels where needed, but it will not require much concrete lining. The tunnels start with a curve to the left of radius 300m and then will go straight into the mountain. There is a climbing grade of 6 per cent.
The geological surveys indicate that a few faults can be expected, but some spalling is likely in this rock. Roof bolts will be used extensively, but concrete lining will be at a minimum, if required at all. Aluminium water shields will be used where necessary, and some frost protection panels will be needed.
However, this 5000m tunnel, built using the low cost methods developed in Norway, will cost only a fraction of the cost of such a tunnel built using conventional methods and with a full, three-layer, concrete lining. In fact, if it were not for these methods it would be impossible for the Norwegians to build such tunnels, particularly as this one will serve such a small community.
Ventilation during construction is provided by a Korfmann 180k W fan, and when the tunnel is in use will be provided with a series of axial fans as this is rather a long tunnel.
Client is the Public Roads Administration providing its own consulting engineers and geologists, though there was some geological help from the University of Oslo on this job. Work started in May 1981,just before *Tunnels* & *Tunnelling's* visit, and had only progressed 60m, so reaching the face was no problem for once! The contractor is the road department of the Sogn & Fjordane using its own direct labour. Completion is due in 1985 ready for opening by 1987. The civil works were estimated to cost 75 million NKr. A more interesting figure given to me was the cost per meter tunnelled, which was said to be between 11000 and 15000 NKr.

Conclusions
These three tunnels -picked at random because one had holed through, one was just starting, and one was being driven with two headings from a central gallery - serve to illustrate the extent and methods used by the Norwegian road tunnellers. Many of their tunnels are very long, most have no internal lighting (to save money), and ventilation is only provided where absolutely necessary.

The major saving is in not putting in a full end-to-end three layer concrete lining, but other factors such as professional smooth blasting make a big contribution.
The main technical advances are in finding comparatively simple and cheap solutions to the problems of water, by using aluminium shielding, and frost protection, by inventing the excellent 'sandwich' of polyester and polyurethane foam.
It is true that up to two-thirds of the cost of a particular tunnel can be saved using these low cost techniques, but on average the saving works out at about half the cost. Taking all the tunnels into consideration the total length lined with concrete comes to less than 3 per cent.

There are only five tunnels with concrete lining from end-to-end. Less than 25 per cent is water shielded and about three quarters of this length is insulated.
Of course, those who live elsewhere will say that the Norwegians are very lucky in having all that nice hard rock to tunnel through. But as we have said before in these pages, the Norwegians do seem to love tunnelling almost for its own sake, and have made a virtue of necessity. I have rarely found such pleasure in making an overseas field trip, thanks to the excellent hospitality and helpfulness of all concerned, even though the weather in Norway during June seemed to consist mostly of rain and clouds!
This was a very extensive tour during which I was driven through perhaps 120 or more completed tunnels (and one or two as yet uncompleted), and I was very impressed by everything I saw. Other countries with similar problems could usefully copy the style of these 'Viking' tunnellers, and perhaps get more tunnels for their money. But if one minor observation from an outsider could be made, why don't they install a proper allocation of reflective cats eyes -of the type invented in Britain -down the center of all those very long and dark unlit tunnels? It could make driving in the dark a lot easier, and safer!

Vardø Tunnel –
An undersea unlined road tunnel Norwegian style

David Martin, Editor

Norwegians are famous for their long substantially unlined road tunnels built at relatively low cost. At Vardø they have driven one 2600m long under the sea to connect an isolated offshore community with the mainland.

In the far north of Norway, beyond the Arctic Circle where it is light for 24 hours a day in summer and virtually dark all the time in winter, lies the small offshore island of Vard0. It is inhabited by a community of some 4000 people who are mostly devoted to fishing and the fish processing industry. Situated at the entrance of the Varanger Fjord, bordering on the Barents Sea, it is the most easterly point in Norway and so far north that one can see the Soviet Union in the general area of Murmansk if one looks *southwards!*

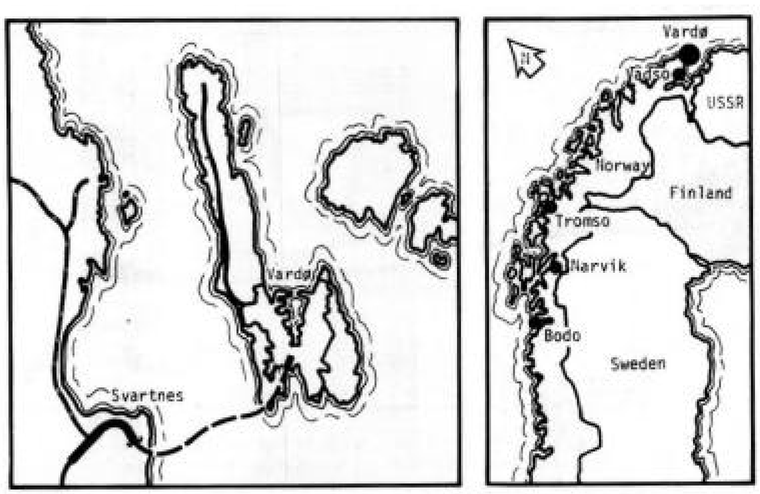
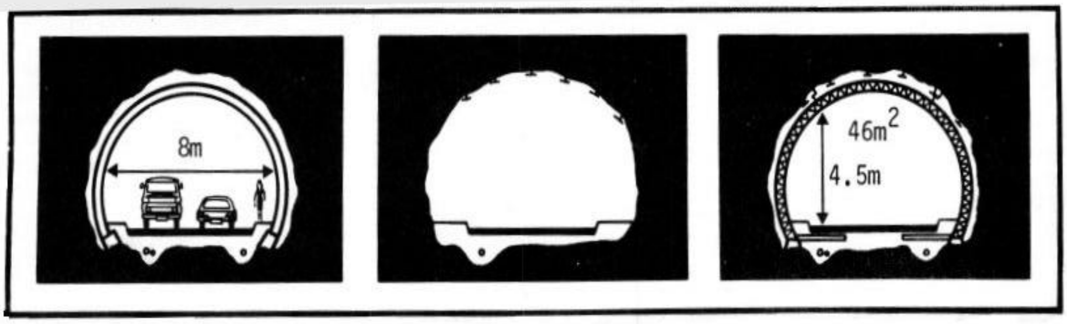


Fig. 1. (left)
Location of Vardø island.
Fig. 2. (far left)
Route of the tunnel from Svartnes to Vardø.

This isolated community, with a history documented since 1307 and evidence of habitation going back 6000 years, relies for communications with the rest of the country and the outside world entirely on an hourly ferry service from the island to Svartnes. In fact, after the 1945 War in which a major part of Vardø was burned down, it was thought to be in the best interests of everyone if the entire community moved to the mainland and relocated the town there so that it would not be so cut off. Fortunately this never happened for the island has a beauty and charm all of its own, and has now developed into a picturesque and thriving community.

Now at last the Norwegian Government, following its policy of developing permanent road communications with isolated communities, has built an undersea road tunnel to replace the ferry service and provide an all-year-round weatherproof and stormproof connection.

The tunnel, due to be opened in December 1982 (about a year from now), is a single bore carrying a two-way roadway. It is 2800m long by 8 m wide, and includes 100m long concrete portals at each end to prevent snow blocking the entrances in winter. Some 1700m of it is below sea level, the lowest point reached being 87m below sea level. The surface area of the tunnel face during excavation was 53m2 a):d the tunnel was driven from both ends using drill and blast methods. Client is the Norwegian Public Roads Administration and the main contractor is Ingenier Thor Furuholmen A/S.



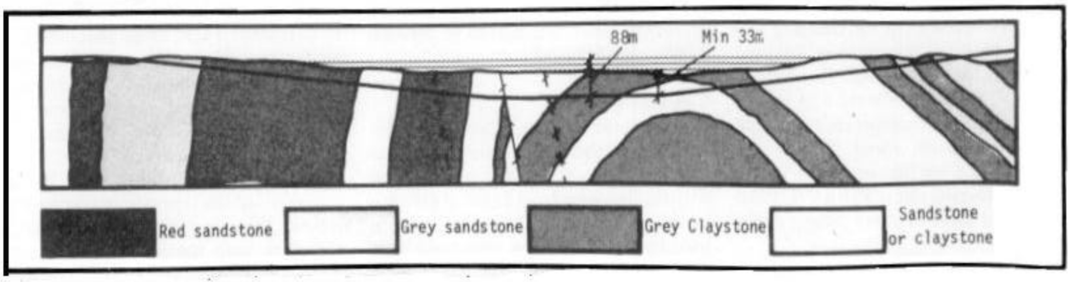
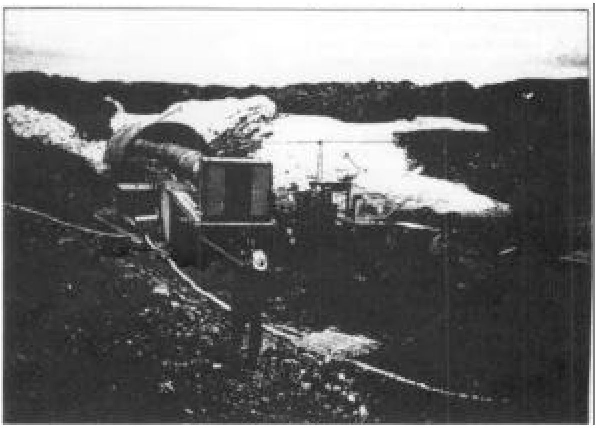


Fig. 3 (top)
Tunnel sections (left to right) lined with concrete, supported with rock bolts, and lined with panles for frost and water protection.
Fig 4. (above)
Longitudal section of the Vardø tunnel showing geology.

Work started in June 1979 at the Vardø end and early the following year (1980) from the Svartnes (mainland) end. Final breakthrough was achieved on July 24 of this year (1981). The most remarkable feature of the Vardø Tunnel is that although it passes through faulted ground under the sea only about 25 per cent-some 600m-is going to be fully lined with in-situ concrete. The rest is supported entirely by shotcrete and/or rockbolts, with only lightweight aluminium water/insulation panels for interior protection.

Bridge or tunnel?
When studies were first made in 1977 , the lowest priced alternatives for a link with the island were a road fill or a bridge founded on piles. Both were ruled out as_not feasible. That left a choice between a stronger bridge and a tunnel. Preliminary geological studies~ had revealed that the bedrock was faulted. partly soft and also permeable. A tunnel was thought to be too risky and further planning was confined to a bridge. although the tunnel alternative was not completely forgotten.
However after several similar successful tunnel projects had been reported- notably the crossing of the Frier Fjord in South Norway and the tunnel under the Tsugaru Strait in Japan- it was decided to look at the tunnel alternative again in more detail and produce an accurate cost estimate. Field studies took place step-by-step so that they could be terminated at any point if it was found that the tunnel costs were exceeding the estimate for the stronger bridge. All aspects of the project had to be studied, including geology, installations and maintenance. Seismic measurements were taken and boreholes were required.



Tunnel portal in Vardø end showing ventilation and construction plant.

Four decisive factors influenced the final cost of a tunnel. First, because tunnel depth determines the tunnel length. the position of the bedrock is of great importance. Secondly, as driving unexpectedly into an overburden filling a gap in the bedrock surface would be a disaster, much attention had to be paid to detection of possible gaps. Thirdly, rock quality has a major influence on driving progress and rock support required, and so mapping of rock quality was essential. And fourthly, because continuous water sealing to prevent inflow is expensive, especially as a permanent measure, it needed to be minimized.

Normally, the upper crust of the bedrock is weathered and permeable and the rock changes gradually with depth to be more sound and less permeable. An optimum tunnel depth can therefore be found by weighing the tunnel length factor against the estimated cost of water prevention measures.

View of part of the 100 m long insulated tunnel portal at Svartnes (mainland end)

Bedrock features

Some 9 km of seismic profiles were shot along possible alignments of the tunnel. They offered a more accurate location of the bedrock features than the acoustical measurements and also provided a distribution of the seismic velocities in the bedrock. The detection of gaps in the bedrock surface is particularly difficult with this method, and so further information had to be obtained from carefully sited boreholes. Altogether 41 vertical boreholes were drilled from a barge anchored in the sound and two inclined holes drilled from the shores.

The bedrock consisted of shales and sandstones of low metamorphic grade, and the whole formation was found to be folded in an anticline with its axis located along the middle of the sound. The bedrock is partly jointed and some distinct faults were identified. A large number of disturbance zones were indicated. Permeability tests indicated normal water leakage conditions.

It was decided that the tunnel project would be feasible, and that the tunnel could be driven using the normal hard rock tunnelling procedure, that is drill and blast. There were several practical factors in favour of a tunnel: a bridge would have increased the distance between Vardø and the new harbour area of Svartnes by 7 km; in winter, periodical Arctic storms would expose bridge traffic to hazards and the weather conditions would restrict bridge construction to the summer only. The cost of a bridge was estimated at 120 million NKr (1978 prices) whereas the tunnel cost estimate varied between 70 and 110 million NKr.

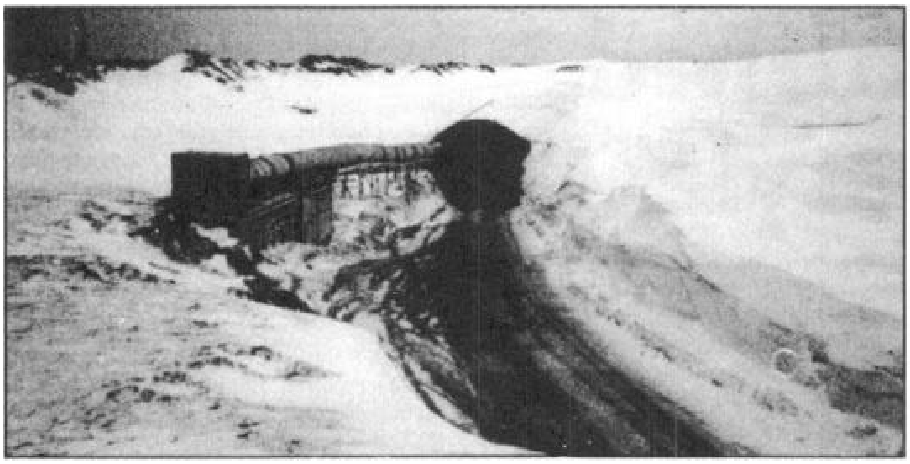
These advantages, plus a conviction that the risks involved could be overcome, led to a decision to construct a tunnel. Contractor Engineer Thor Furuholmen started the work in June 1979 and by the end of that year some 300 m of tunnel had been driven from the Vardø end, .the main part being head-supported by occasional rock bolts, and the rest by thin layers of shotcrete. Unfortunately, at about that time, a cave-in occurred in a section of poor rock and about a month and a half had to be spent on driving and lining a 30m section.

Thin overburden

At Svartnes on the mainland, tunnelling started at the beginning of 1980, time being needed to construct and line an entrance. Progress was slow because of the thin overburden, weathered rock and unfavourable stratification. Several minor cave-ins occurred and much time was spent driving and lining through the poor rock and faults. On the Vardø side some 900m were driven in 1980, half of it head- supported with occasional rock bolts and half with shotcrete. Water leaks have been small and only a couple of sections had to be grouted ahead of the face.

Drilling was done with three Furuholmen designed and built jumbos equipped with Montabert H70 hydraulic rock drills. Two were equipped with 3 booms and a basket and one with 2 booms and a basket. Although Furuholmen has designed and demonstrated computer controlled jumbos for hydraulic rock drilling *(see Tunnels* & *Tunnelling,* May 1981, p12) the jumbos used in the Vardø Tunnel were not the fully computerized versions. Explosive used was ANFO and depth of cut was 420cm.

Apart from the short lengths of in-situ concrete already mentioned, some 10000 roof bolts are being put in combined with nylon nets and steel bands to provide the final support. Scotcreting has been extensively used, both for immediate support and as part of the final support, using Schwing concrete pumps.



Svartnes portal, note the heavy snow accumulation although it is June!

The tunnel design engineers recommended that horizontal holes be driven in front of the face during the tunnelling to give extra security, and this was done to detect faults and bad ground in advance. During construction the tunnel was ventilated using 120cm diameter S0der- berg fans.

During the excavations, over 270 000m^3 of rock were removed. Mucking out was by three CAT980C loaders and seven Kokum 442 30 tonnes capacity 4-wheel dump trucks. The material excavated has been sensibly used to improve amenities at both ends of the tunnel.

Following the hole-through in July, the interior of the tunnel is being completed by inserting the rest of the rock bolts to provide final stability and by installing a further 1500-20OOm of combined aluminium water shields and insulation to take care of leakages in the unlined sections of the tunnel. A full description of these appeared in *Tunnels* & *Tunnelling* in October 1981, p35. Water leaking into the tunnel will be collected in drains and pumped out from sumps equipped with powerful water pumps. When completed the tunnel will also carry the main water supply pipe to the island.

The entrances are protected from winter snows with 100m long portals made of concrete and protected on the outside with 6cm thick plastic panels made of Styrofoam Hi (from Dow). The same type of panels are used inside to prevent the drains from freezing. Estimated overall price of the tunnel is 130 million NKr (1981), and there are no indications so far that this estimate will be exceeded. The work was said to be running some 13 weeks behind the original schedule, most of it due to careful tunnelling through faults containing loose rock 'and gouge. Total water leaks amounted to 1.5m^3/min, only about half of the estimate. No serious inbursts of water were experienced.

Client: Public Roads Administration
Contractor: Ingeni0r Thor Furuholmen A/S
Consulting Engineer: A B Berdal A/S