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USE OF THE UNDERGROUND IN NORWAY

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ABSTRACT: In the hard rocks of Norway, underground works have a long tradition. Today the subsurface has been taken into use for a wide variety of purposes, such as for instance caverns for oil, gas and drinking water, wine and liquor stores, cold stores, waste disposal and state archives. The particular use of unlined high pressure tunnels and so called air-cushions for hydropower schemes is described. The dual use of underground air-raid shelters for sports activities seems to be a special Nordic tradition. Some cost figures are given, and the advantages of using the underground is discussed. A brief description of the geology and topography of Norway is also given.

1 INTRODUCTION

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 almost 200 underground power plants. During the last three 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 100 - 150 km.

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 treatment plants, parking lots, factories, telecommunication centrals, swimming pools and sports halls are also to be found in the Norwegian underground.

A selection of examples of underground installations will be shown in this paper. Before doing so, it seems natural to give a brief description of the geological and topographical conditions in Norway. This will to some extent explain why the comprehensive use of the subsurface has taken place in this country.

2 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, greenstones and marbles as well as granites, gabbros, sandstones, shales, limestones and other unmetamorphosed rocks form the rootzone 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 limestones 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 an 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 out- crops, 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.

3 HYDRO-ELECTRIC POWER PLANTS

3.1 Introduction

In Norway more than 99% of an annual electricity production of 100 TWh comes from hydro-power. With a population of 4 millions, this gives a consumption of 25.000 kWh per person per year, which is by far the highest in the world. Fig. 1 shows schematically how the general layout of hydro-power plants has developed with time. It reflects firstly how the subsurface has been taken into use and secondly how the confidence in the rock masses themselves has grown with the increasing use of the subsurface.

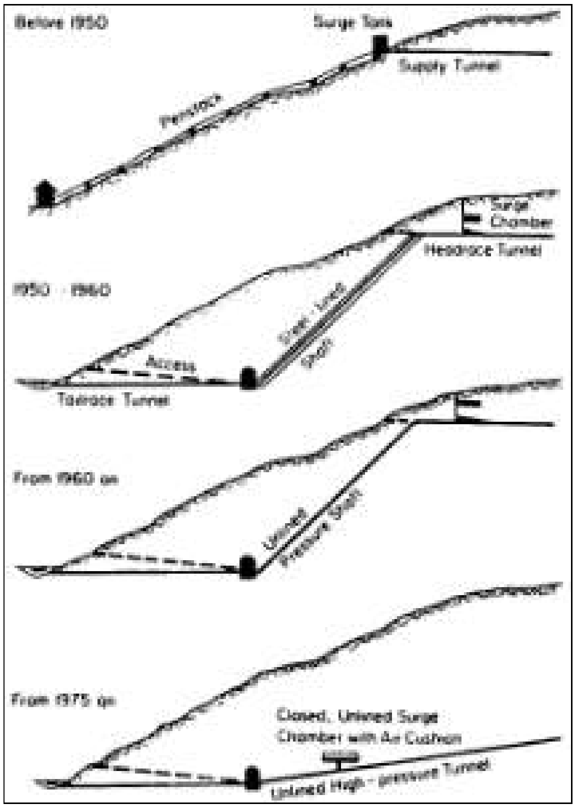

Fig. 1 The development of the general lay-out of hydro-electric plants in Norway. From Broch (1982, B).

3.2 Powerhouses

The situation today is in fact that putting the powerhouse underground is regarded as the conventional solution. In Norway one can find almost 200 underground stations out of which only 4 were built before 1950. This means that since that year an average of 5 new underground stations have been put into operation every year. Fig. 2 demonstrates how modern hydro-power development has become an underground industry.

The reason for putting the hydro-power stations underground is the same today as when it started. It gives the most economic solution. Additional benefits are that underground installations give almost perfect protection against war hazards and sabotage, and that they are almost "invisible", and thus have a very small impact on the environment.

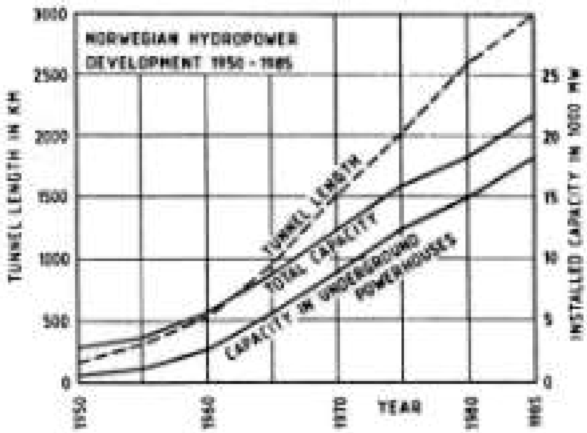

Fig. 2 The development of Norwegian hydro- electric power production capacity and the accumulated length of tunnels excavated for the period 1950- 1985.

In the early underground powerhouses there was a tendency to more or less built a house inside the rock cavern with roof and walls. Even false windows in the walls were quite common.

In modern design the uncovered rock makes the walls. This requires very precise drilling and blasting, and, of course, stable rock masses.

3.3 Unlined high pressure tunnels

As Fig. 1 shows, penstocks were first substituted by steel lined pressure shafts and then later by unlined pressure shafts. Today, more than 80 unlined shafts and tunnels with water heads higher than 150 m are in operation. Recently an unlined high-pressure tunnel with a static water head of 1.000 m was put into operation.

From Fig. 2 is interesting to see how the length of tunnels per installed unit has increased with time. Hydro-power schemes including 25- 50 km of tunnels have been quite common in recent years.

3.4 Air cushion surge chambers

For 9 hydro-power schemes the new solution with a closed, unlined surge chamber has been taken into use, see bottom Sketch in Fig. 1. These closed chambers which have volumes varying from 2.000 m to more than 100.000 m^3 are partly filled with com- pressed air. The compressed air acts as a "cushion" to reduce the waterhammer effect on the hydraulic machinery and the waterways.

Some key figures for the nine existing and the one under construction air cushions are given in Table 1. Fig. 3 shows an example of the lay-out of one of them (Goodall et al., 1988).

Name	Year Completed	Air ressure (bar)	Volume of Chamber (m^3)
Driva	1973	42	6,600
Jukla	1974	24	6,200
Oksla	1980	44	18,100
Sima	1980	48	10,500

Kvilldal	1981	41	120,000
Nye Osa	1981	19	12,000
Tafjord K5	1981	78	2,000
Brattset	1982	25	9,000
Ulset	1985	28	4,800
Torpa	1989	44	14,000

Table 1. Closed, unlined surge chambers with air cushions in Norway.

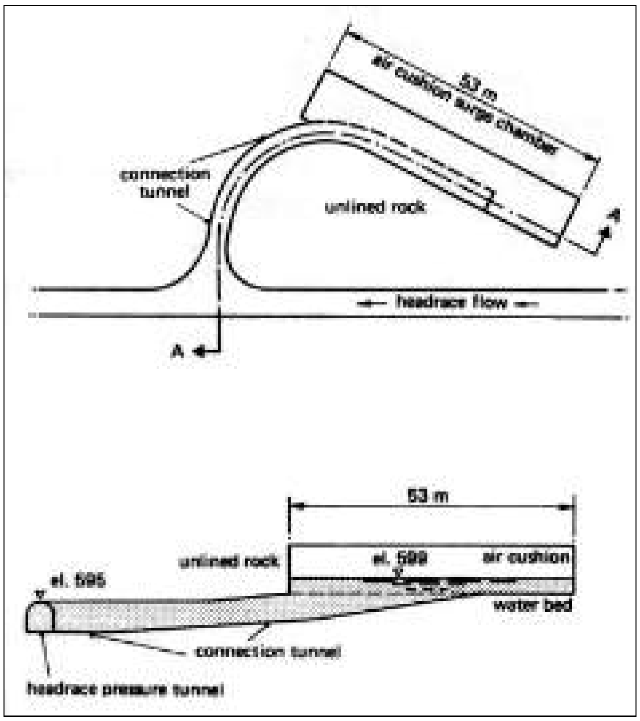

Fig. 3 Plan and profile of the Ulset air cushion surge chamber, from Goodall et al. (1988) .

These air cushions are unique to Norway and represent an unparalleled possibility to study the behaviour of compressed air (or gas) in unlined rock caverns. The results from the comprehensive research work related to these "gas caverns" which has been carried out here at the Norwegian Institute of Technology/SINTEF, will be presented in several papers at this conference. Participants are also invited to visit air cushion surge chambers during the post conference tours.

5 UNDERGROUND STORAGE

4.1 Introduction

The worldwide increase in the use of the subsurface for storing of oil and gas clearly demonstrates that people in different places with different geological conditions and different economical systems have discovered the advantages. And the major advantage is simply the costs. Fig. 4 is a compilation of cost figures from West-Germany and Scandinavia made by this author in 1980. Although prices have changed, a recent control shows that the general trend for the curves are still valid. Together with information from other parts of the world the figure indicates that when the volume of a bulk storage exceeds between 5.000 and 10.000 m^3, the construction costs for an underground storage is lower than for conventional above-ground structures like steel or concrete tanks.

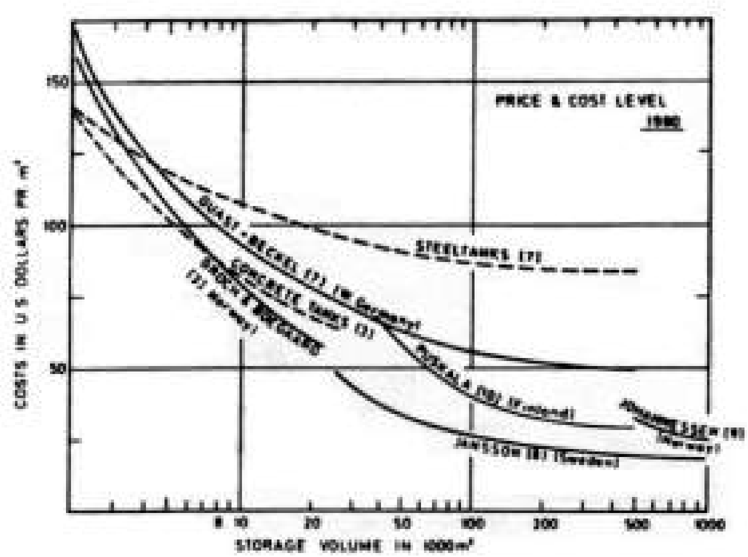

Fig. 4 Construction costs for underground bulk storages in hard rock. Costs for conventional surface steel and concrete tanks are shown with dotted lines. From Broch (1982, A).

4. 2 Oil

The underground was first time used for storage of oil in this country here in Trondheim during World Was II. Several caverns were excavated in the hard rocks near the harbour and steel tanks were put inside the caverns. Thus the rock was only used for protection. In the early 60's some steel lined caverns for oil storage were constructed. Since that time all underground storage of oil have been in large rock caverns without any lining and with the use of a minimum of rock support.

The latest installations were taken into use last year. At the Sture terminal just north of Bergen, the facility consists of five parallel caverns, four of these being planned for storage of up to 550.000 m^3 of oil and the fifth for tanker ballast water. Each cavern is about 33 m high, 19 m wide and 314 m long, se Fig. 5. To prevent gas leakage a water curtain was installed from the ground surface, see Fig.6.

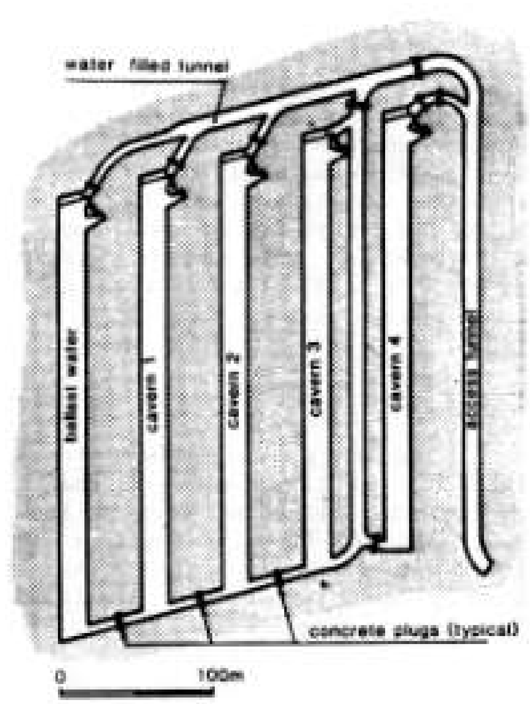

Fig. 5. Plan of Sture Oil Caverns, from Midtlien (1986).

At the Mongstad oil refinery, also north of Bergen, the storing capacity for oil was increased by 1.3 million m^3 last year when six new rock caverns with lengths of approximately 500 m were taken into use. Cavern cross-sections were similar to those at Sture.

The total volume of oil stored in underground rock caverns in Norway is now rapidly approaching 5 million m^3.

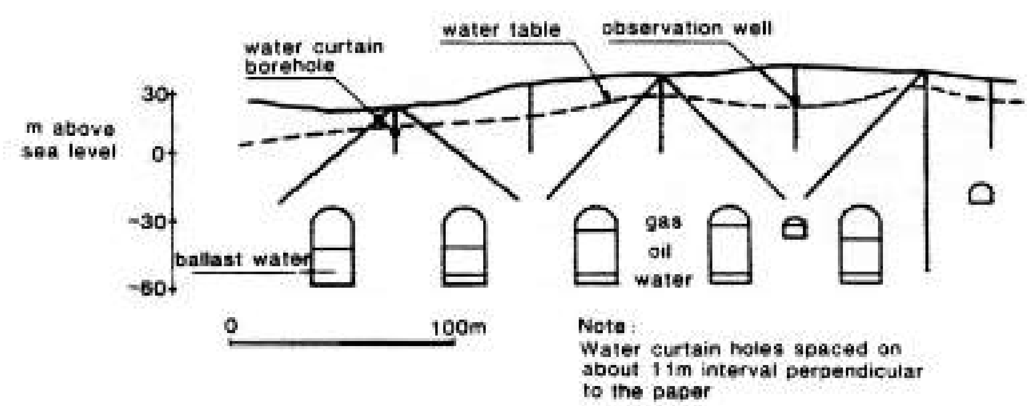

Fig. 6 Cross-section of Sture Oil Caverns with water curtains, from Midtlien (1986) .

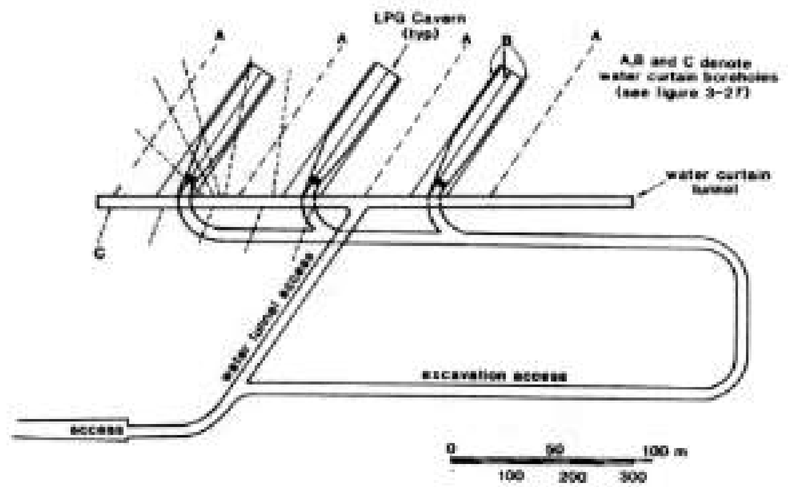

Fig. 7. Mongstad LPG cavern plan and water curtain.

4.4 Drinking water

Next to oil and gas storage the most important is the storage of drinking water. Fig. 8 shows the lay-out of one of several unlined rock cavern tanks in this country. This particular facility is in Trondheim, and will be visited by the participants during the conference. The capacity of the tank, 22.000 m^3, was obtained by the excavation of two caverns with a width of 12 m, a height of 10 m and lengths 85 m and 110 m respectively. Also the service section is put underground. It consists of a small workshop, a combined control- and lunchroom, wardrobe with lockers and washing room with shower and a toilet. The service section is well accepted by the users, but is not in daily use.

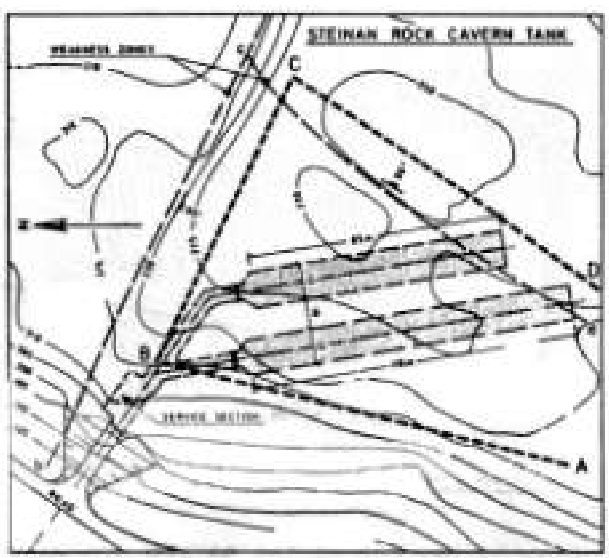

Fig. 8. The Steinan rock cavern tank in Trohdheim. Total capacity 22.000 m3. From Broch and 0degaard (1983) .

Additional benefits which favour an underground solution for drinking water tanks may be the following:

- High degree of safety, also against war hazards, sabotage and pollution.
- Constant and low water temperature.
- Low or no addition in prices for a two chamber solution.
- The rock masses may be used for other purposes.
- Low maintenance costs.
- And finally, the tank is almost invisible and causes, thus, no harm on the local environment.

4.5 Molasses

In Stavanger two silos for storage of the very viscous liquid molasses have been excavated in the rock close to the harbour. The silos which are situated close together have a total capacity of 20.000 m with diameters of 22 m and heights of 30 m. Only the upper 6 m are lined with concrete. The saving of space, reduction in costs and the minimal heat losses were the main reasons for this unconventional solution.

4.6 Industrial waste

In Odda, in the southwestern part of the country, the Norzink company, a major producer of zinc has by the Norwegian Environmental Authorities been instructed to deposit the residues from the production in a pollution-safe place.

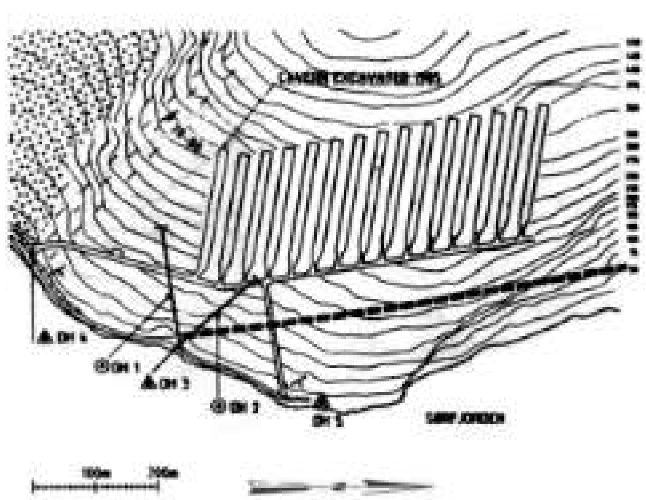

Fig. 9. Lay-out of the cavern system for storage of industrial waste at Odda. From Aarvoll et al. (1986).

The nearby steep mountains were considered to be the ideal place for the construction of rock caverns for storing of the annual production of 50- 60.000 m^3 of residues. A series of parallel caverns are planned to be excavated, one cavern each year. The first one was completed in 1985, see Fig. 9.

Planning has also recently started for a central governmental operated underground storage of dangerous industrial waste in Rana, northern Norway.

4.7 Wine, liquor and beer

So far all examples have concerned the bulk storage of liquids (or gases). As, however, is well known, people have throughout the centuries stored their bottles or casks of wine in deep cellars or even in caverns or small tunnels dug out of soft, but stable rock masses. The constant temperature in these caverns is known to be advantageous for the maturation of the wine. On a much larger scale this storage of wine and liquors in subsurface caverns, even in hard rock, has been adopted by the State-owned wine and liquor companies in the Nordic countries. Breweries, too, have taken the subsurface into use for storage purposes.

4.8 Cold stores

Favourable temperature conditions are one reason for choosing the subsurface alternative. Another reason can be the favourable insulation that rock masses around a cavern can provide. The "walls" can, in many cases, be regarded as being of infinite thickness. Thus rock caverns have for some time been used as cold stores where, for instance, fruits and vegetables have been stored at normal refrigerator temperature, + (2- 5)°C, and frozen food like fish, meat and ice cream have been stored at so-called deep freezer temperatures -(25- 30)°C.

With ground temperature of 6- 8°C the subsurface storage of food at refrigerator or deep freezer temperature can be made with favourable energy economy. The energy consumption for deep freezer storage is 75% and for refrigerator storage only 25% of similar surface stores. .The peak energy requirements, and thus the installations, are even more favourable. The deep freezer storage will need 50% and the refrigerator storages only 20% capacity of similar surface stores. Especially favourable energy economy is obtained when surface production and underground storage of frozen products like ice cream is combined. Cooling machinery is then used in the production during day time and in the store at night. The participants of this conference will have the opportunity to visit such an installation here in Trondheim.

Strongly reduced insurance rates are also favouring the underground solution for cold stores. This is due to the fact that the rock mass surrounding the storage caverns contains a big cold reservoir. In case of a breakdown in the cooling machinery, this will act as a reserve. Experience have shown that with cooling machinery out of function for a couple of weeks, an increase in the temperature of only 2- 3°C is measured.

4.9 Archives

A rather unusual type of subsurface store is that of the State-archives in Oslo. The good possibilities of obtaining the correct storage conditions for old papers were amongst the reasons for choosing the subsurface solution. A high degree of safety is, of course, also of importance.

5 RECREATIONAL FACILITIES IN ROCK

5.1 Introduction

Civil defence installations are normally to be found underground, some in deep, windowless cellars or basements, but also a number in excavated caverns in the bedrock. Such air-raid shelters have for a long time been used as car parks and for different storing purposes in peacetime. During the last two decades the public of Norway has experienced a more varied use of the underground air-raid shelters, and today the possibilities for civilian peacetime use is incorporated at an early stage in the planning. An especially interesting trend in this development is the dual use of the air-raid shelters for various sport activities. More than ten such dual purpose air-raid shelters/sports halls are now in daily use in this country (Broch & Rygh, 1988) .Three examples will be described in this paper.

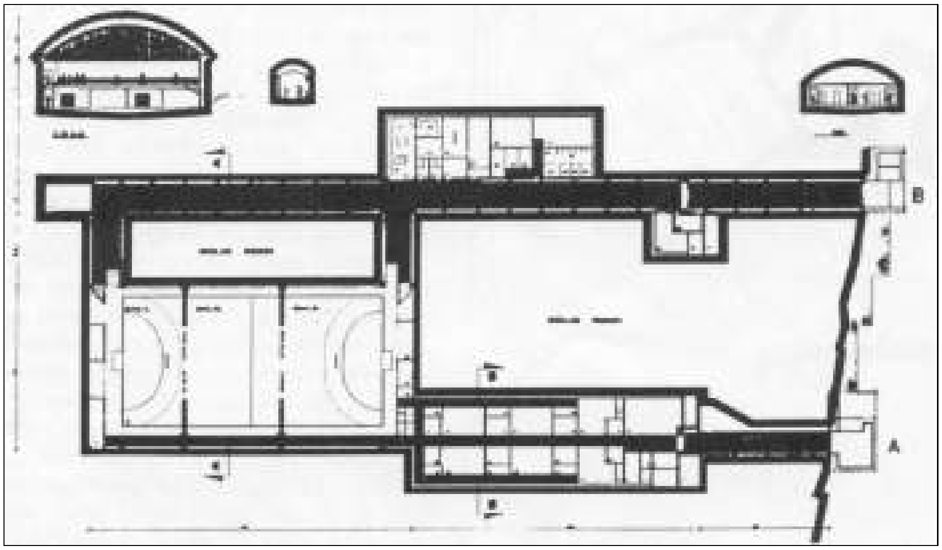

Fig. 10. Lay-out of the Odda Sports Center. From Broch & Rygh (1976)

5.2 Odda sports center

In the small industrial town of Odda suitable conditions for the construction of a rock cavern were found close to the outdoor sports stadium and the junior college. Here the first underground sports center in rock was completed in 1972.

Fig. 10 shows the general layout with two entrances, A and B. A leads to four wardrobe/shower sections, while B leads to a 100 m sprint track and a jumping ground. This long tunnel is also used for shooting.

The main hall is 25 x 60 m which makes it adequate for international handball games. It can be divided into three gymnasiums by curtain walls. The hall has a gallery stand for 500 persons. 25.000 m^3 of solid rock was excavated, giving an available floor area of 2.700 m^2.

5.3 Gjøvik swimming pool

In 1975 the first underground swimming pool with international standards was opened in Gjøvik, Norway. This swimming pool makes part of an underground scheme which also includes a telecommunication center and head-quarters for the local civil defence. As Fig. 11 shows, the entrances to these subsurface installations are close to the main street of Gjøvik.

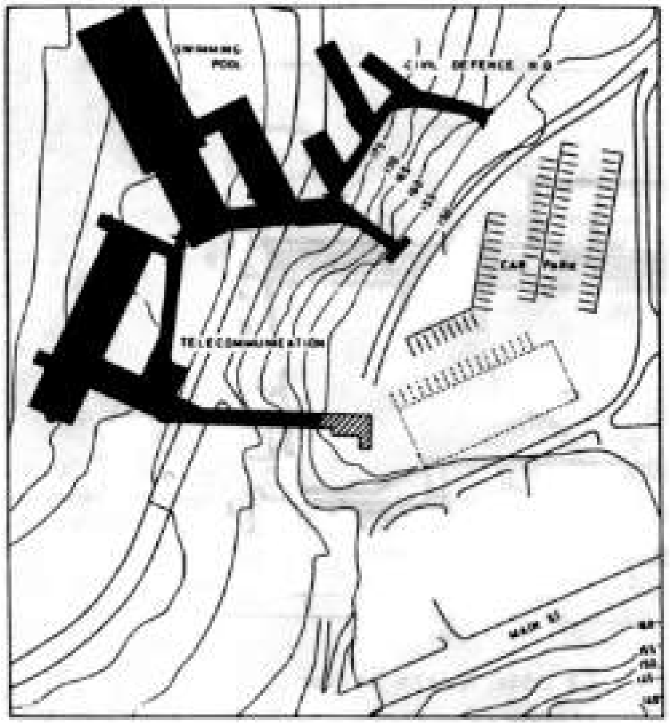

Fig. 11. Lay-out of the subsurface installations in Gjøvik. Broch & Rygh (1976) .

From the entrance lobby with ticket office and cloakrooms, traffic is divided into two wardrobe/shower sections for men and women respectively. Toilets and saunas make parts of these sections.

In the main hall with a span of 20 m are the swimming pool with 6 lanes of 25 m and children's playing pool of 4 x 8 m. Separated by a glass wall is a small gymnasium (also used as a meeting room) .

Of special interest is the fact that the energy consumption for running this underground public bath and swimming pool has been cut down to approximately 50% of what would have been necessary for a similar building on the surface. Both this and the fact that there were limited areas for building in the center of the town, were important factors when the decision to put the swimming pool underground was made. 11.000 m of solid rock was excavated and transported to a nearby marina under construction.

5.4 Holmlia sportshall and swimming pool

In 1983 a combined underground sports hall and swimming pool was taken into use at Holmlia, a new suburban area in Oslo. During the planning of the area it was decided that a modern center for varied sports activities should be built. In accordance with Norwegian civil defence regulations, blast and gas tight shelters for approximately 7000 people were needed near the center of the new development area. Furthermore, large amounts of rock material were needed for the construction of roads and parking lots in the area.

Within easy walking distance from the Holmlia railway station and a shopping center, a small hill of gneissic rock was the obvious place for an underground sports center. The rock cover was rather small, in certain places only 20 m above the roof of the sportshall, but was accepted by the Civil Defence Authocities.

Fig. 12 shows the general lay-out of the Holmlia sports hall and swimming pool as well as the main dimensions. The sports hall is 25 x 45 m and is equipped for different ball games. The swimming pool has 6 lanes of 25 m. 53.000 m^3 of rock was excavated and the total floor area, including the swimming pool, stands, gallery and first floor above entrances, is 7.550 m^2.

When completed in 1983 the total costs (including civil defence installations) added up to 54 million NOK. 8% was design, supervision and administration, 67% civil engineering works, and 25% heating, sanitary, ventilation and electrical installations.

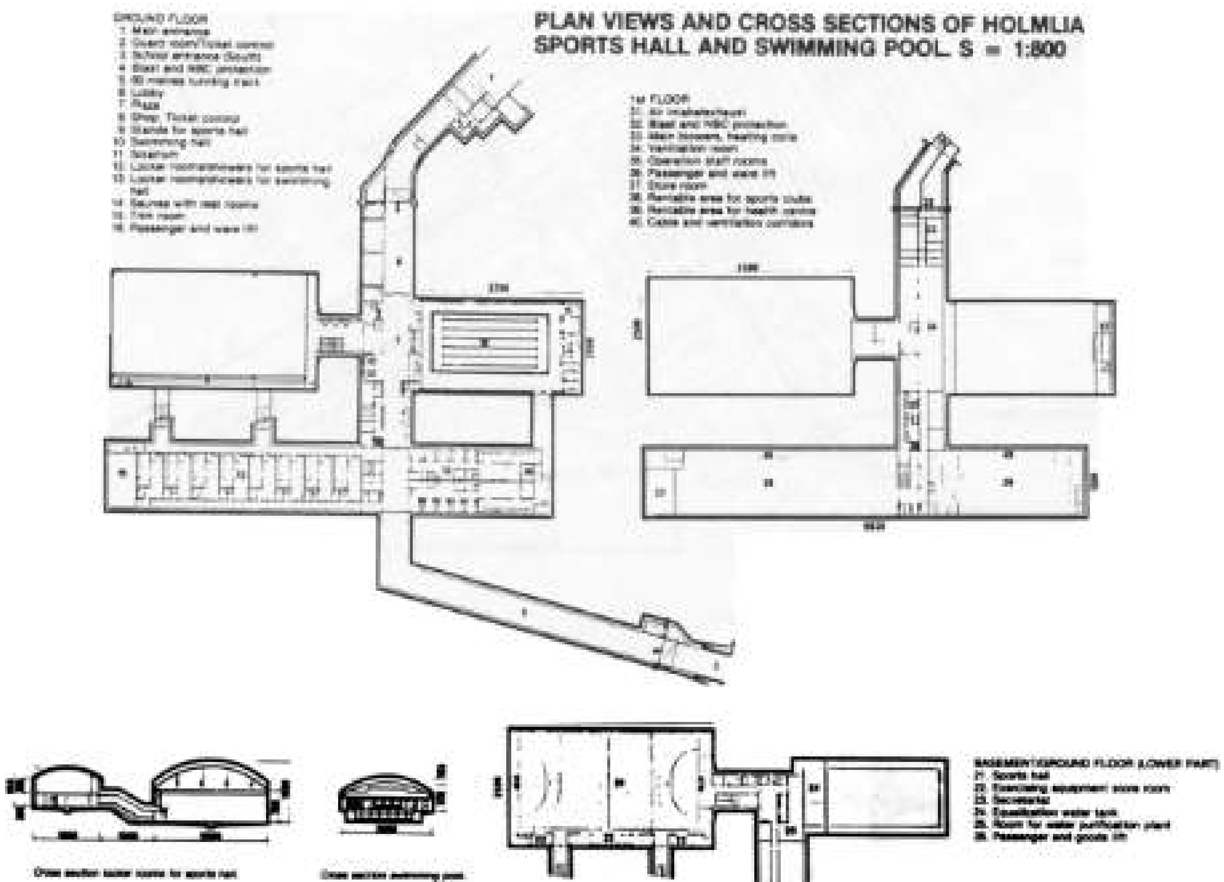

Fig. 12. Plan views and cross-sections of Holmlia sportshall and swimming pool.

6 OTHER UNDERGROUND INSTALLATIONS

6.1 Telecommunication center

It is already mentioned that there is an underground telecommunication center in Gjøvik. This is not the only one in Norway. Such centers are really the nerve centers of modern society, and should therefore be protected as good as possible. The use of the subsurface is thus the evident solution. Excellent possibilities for climate conditioning are also favouring this solution.

6.2 Water treatment plants
In 1970 Oslo took in use a new plant for treatment of drinking water with a capacity of supplying half a million per- sons. The entire plant is situated under- ground close to the shore of the lake Maridalsvannet 5 km north of the city center at an elevation of 150 m above sea level. Fig. 13 and 14 show the general lay-out and a vertical cross-section through the Oset Water Treatment and Pumping Plant.

The nominal capacity of the plant is 6 m^3/sec and the retention time for the water is approximately 5 hours. 350.000 m of solid rock (syenite) was excavated for the different basins and tunnels. 2 This gave a total floor area of 30.000 m^3 of which one half is paved and one half is wet.

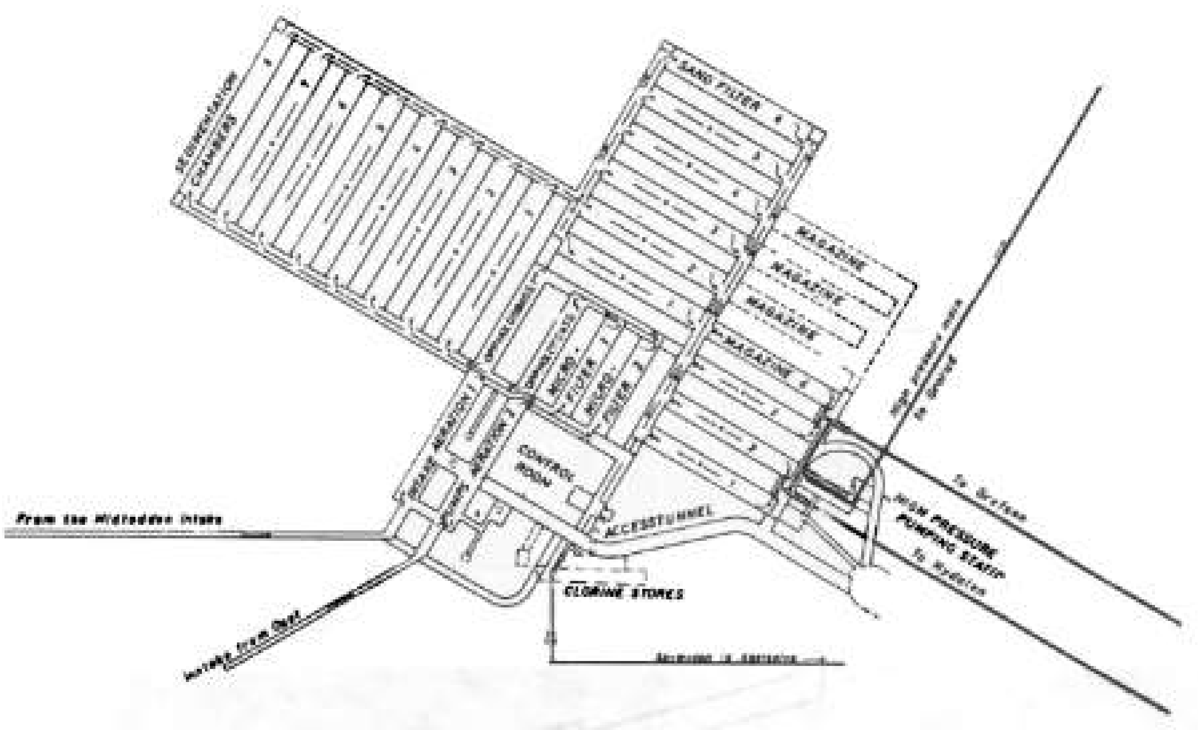

Fig. 13. Lay-out of the Oset Water Treatment and Pumping Plant.

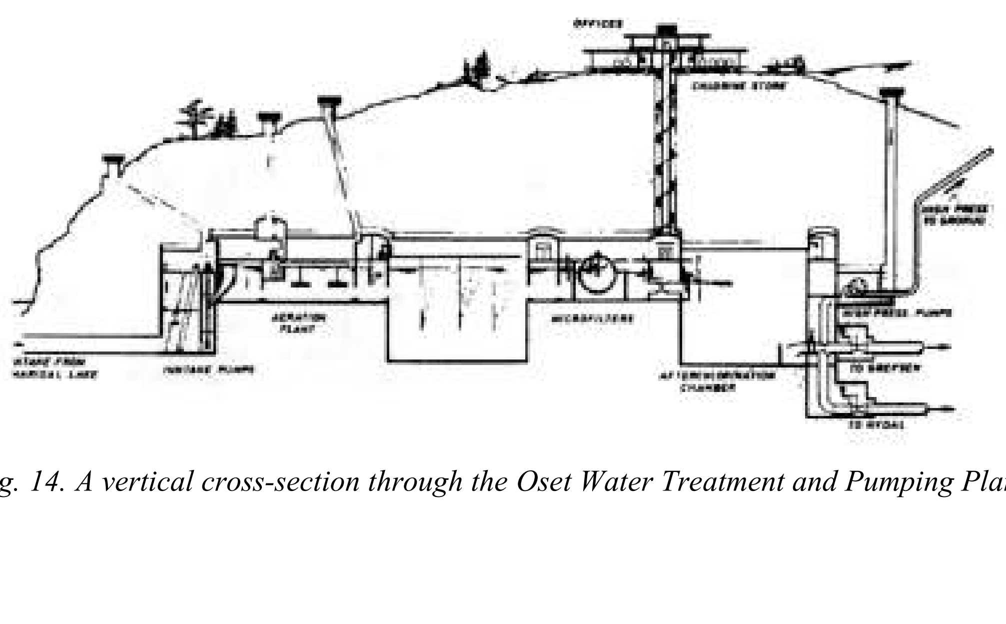

Fig. 14. A vertical cross-section through the Oset Water Treatment and Pumping Plant.

6.3 Sewage tunnels and treatment plants

Tunnels for transportation of sewage have been excavated in many of the cities and towns in Norway. A tunnel boring machine TBM, was for the first time successfully used in the hard rocks of Scandinavia to bore a sewage tunnel in Trondheim in 1972. And in the early 80's six TBM's were operating simultaneously in the bedrocks of Oslo to excavate the 42 km long sewage tunnel system.

Also the sewage treatment plants have been put underground in several of our cities and towns, among them Trondheim. The conference members will have the opportunity to visit this plant.

Even though cost estimations may have shown that construction costs for underground treatment plants are higher than for similar on-the-ground plants, the underground solutions have been chosen. Favouring this choice is first of all the wish to avoid the impact such big installations may have on the environment. For the drinking water treatment plant the safety aspect has also been taken into consideration. A valuable additional benefit is the produced rock masses which there always is a need for in an urban area.

6.4 Industrial facilities

As early as in 1954 an ammunition factory at Raufoss in Norway took into use nine big underground production halls with a total flow area of 25.000 m^2.An underground locomotive repair shop has also been used for a long time in Oslo.

7 CONCLUDING REMARKS

It was mentioned in the introduction that railway tunnels were the first underground works in this country. That is, of course, outside the mining industry. And in fact we have approximately 750 railway tunnels, the longest 10 km and several tunnels longer than 5 km.

In recent years there has been a considerable increase in the construction of tunnels for roads and highways. Participants in our post conference tour to Bergen will drive through a large number of tunnels along the steep fjords on the West Coast. An interesting new development is the subsea road tunnels crossing under fjords and straits. Outside Ålesund two tunnels with a total length of 7.730 m are already in operation while a third tunnel is under construction. Maximum depth is 140 m below sea level. Other subsea tunnels are recently completed, are under construction or are being planned.

Another very challenging highway tunnel presently under construction is the Oslo tunnel which runs in front of the City Hall. It comprises two parallel three lane tubes running 1520 m through bedrock and 700 m as cut-and-cover. Ground conditions are in places extremely difficult.

Hopefully this paper has shown some of the wide variety in the use of the underground in Norway. The author has concentrated on examples of unconventional use. Other examples are therefore deliberately omitted, while some, of course, may have been forgotten. There is no doubt in this author's mind that the underground is a valuable resource which we can not afford to leave unutilized in the future

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EXPERIENCES FROM PLANNING AND BUILDING OF LARGE STORAGE FACILITIES IN ROCK

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ABSTRACT: Norway's long experience in hard rock underground engineering has been utilized for defence purposes of various kinds. Among those, building of large storage facilities for storing of fuel, ammunition and military equipment compose up by far the greatest proportion. The latest finished rock storage facilities are in the Trøndelag area, built for prepositioning of ammunition and equipment for a US Marine Infantry Brigade.

In spite of a somewhat higher cost for rock storage, compared to building above ground, this alternative was chosen mainly due to factors concerning military aspects, such as vulnerability and safety, but also for environmental reasons. Operating costs are, on the other hand significantly lower for rock storage than for above ground storage.

1 INTRODUCTION

Based on the comprehensive and long experience of Norway in constructing installations of various types in rock, it was only natural that the Norwegian Defence over many years has been planning and building defence installations for various purposes in rock. The largest installations were for storing of ammunition, fuel and equipment.

The latest rock installations built were facilities for prepositioning of ammunition and equipment for a US Marine Expeditionary Brigade (US MEB) earmarked for deployment to Trøndelag in Central Norway.

At the outset, the intention of NATO was to build these storage facilities as large warehouses in the open.

The consequences would then have been:

- Acquisition of large areas of valuable real estate.
- Costly systems for alarm, guard and operation.
- Vulnerability to sabotage and attack in war.
- Unstable storing conditions due to variations of temperature.

Norway insisted on building the storage facilities in rock, and the Chief of Defence decided in 1983 that all detailed planning should be concentrated on this alternative.

The consequences of this decision were:

- Cost of construction would be higher than the construction of open air storages. (Extra cost to be paid by Norway).
- Large areas would not be affected and consequences for the environment negligible.
- Considerably less vulnerable to sabotage.
- Considerably less vulnerable to attack in war.
- Ideal storing conditions.

2 PLANNING

In order to secure the best basis for planning of the underground facilities, a construction firm with extensive experience in planning and building rock installations was selected to undertake the job. Necessary geotechnical and geological engineering expertise was engaged at an early stage in the planning process.

After extensive reconnaissance and thorough investigations three sites for the equipment storage facilities were selected. These were Bjugn in South- Trøndelag, Frigård and Tromsdal in North- Trøndelag. The ammunition storage-sites were to be built at Kalvå in South-Trøndelag and Hammerkammen and Hammernesodden in North- Trøndelag. Fig. 1 refers.

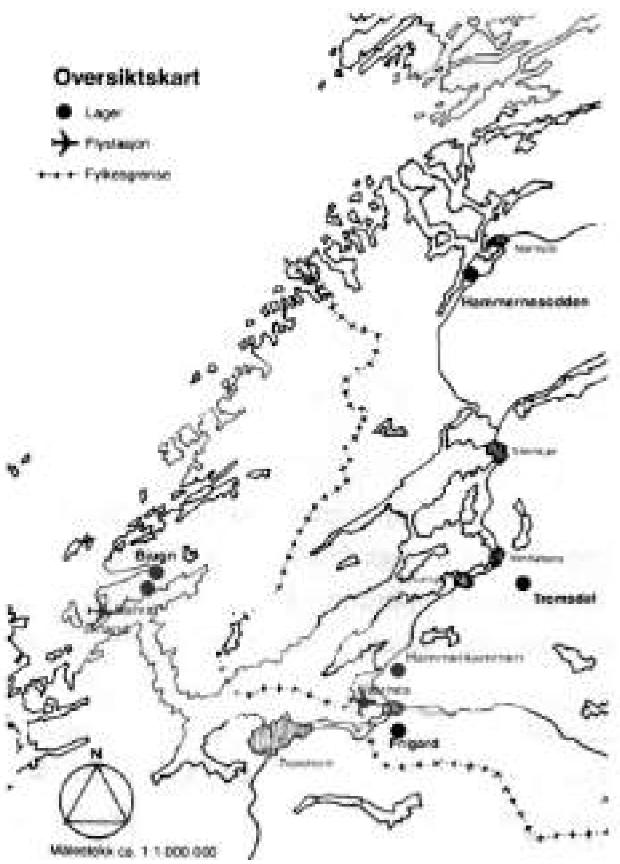

Figure 1

The types of equipments and materiel to be prepositioned and the results of the geotechnical and geological engineering surveys determined the layout of the different facilities. To gain the maximum repetitive effect during the planning process, the same basic model was selected for the three equipment storages. The ammunition storages were roughly similar. Fig.2 refers.

Characteristic data are as follows:

Width of the hall	20,5 meters
Height of the hall	Wall/center
Shelf storage	5,25/9,30 m
Floor storage	3,80/7,85 m
Vehicle storage	3,80/7,85 m
Container storage, Hospital section	9,00/13,50
Total floor space, all three facilities	75.500,00 m^2
Total excavated rock, all three facilities	510.000,00 m^3

3 GEOLOGICAL ENGINEERING AND GEOTECNICAL SURVEYS

Detailed geological engineering mapping, two core drillings and refraction seismic investigations were carried out at each of the sites. In addition, resitivity measurements were done at two sites which gave a good opportunity to test this comparatively new method to detect weak zones in the rock mass.

These investigations resulted in only minor adjustments of the positioning and orientation of the caverns and tunnels. The investigations, however, made it possible to foresee and describe the extent of rock securing and the best methods to be used to cope with this problem.

The different types of rock were investigated in a laboratory .The mechanical properties were determined in order to decide how the excavated waste rock could be used in embankments or as top layer on roads or parking areas.

Topographical surveys and geotechnical investigations were carried out to decide the best sites and areas for facilities in the open and for rock waste areas. These investigations were test excavations, penetration tests (soundings) auger drillings supplemented by other geological information (maps).

These investigations resulted in evaluation of ground stability and the recommended foundations for buildings and installations.

The project team used data-assisted construction (AUTOCAD) for the production of drawings. This proved to be a useful method from the point of view of standardization. The same main basic design was laid down for all facilities, and changes were easily implemented through the automated system. Changes to the original plans had to be made due to the geological conditions encountered during actual construction of the facilities, and also as a consequence of the decision to expand one of the facilities to house prepositioning of special containers for a 500 bed field hospital unit. This field hospital was to be established in the facility when all the prepositioned equipment for the Marine Infantry Brigade were removed.

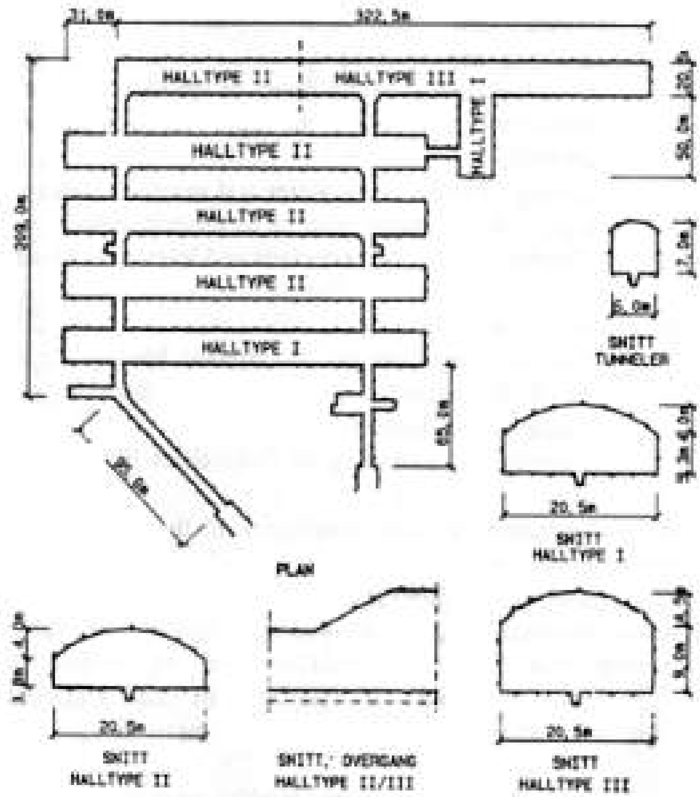

Figure 2

4 CLIMATIC CONDITIONS

The user requirement for climatic conditions in the facilities was that the relative humidity should be less than 50%.

The minimum temperature requirement was established at + 5° C. The configuration and dimension of the ventilation system were based on that fact that the storing was passive and no maintenance work would take place in the facility, as well as securing a safe climatic condition in the facility when a great number of vehicles were driven out producing considerable exhaust gases. There were no difficulties in meeting these requirements.

To meet the requirement of relative humidity of maximum 50% a plant for dehumidification with heat recovery was selected. About 80% of the effect would be recovered which would result in a temperature in the facility of +11-12°C in the summer and + 8-9°C in the winter.

To maintain required humidity controlled conditions, different methods of closing the halls and tunnels were evaluated. Finally a system based on covering the entire rock surface in the storage areas in a tent-like manner with fire resistant armoured PVC-sheet lining was selected. This was the best method from a cost-effectiveness point of view.

In addition, to solve the problem of water leaks from the rock, the white PVC-sheet fabric had considerable advantages when lighting was concerned. The number of lighting points could be reduced considerably and savings in electric consumption were achieved. Fig. 3 refers.

The producer of the PVC fabric was involved at an early stage in the planning process in order that a feasible mounting system could be incorporated in the construction rock cavern

through a system of bolts and wires. All seams were welded so that the fabric presented a water-tight
surface leading water to a draining system on both sides of the hall or tunnel. The PVC fabric is non-flammable and accepted by the Fire Prevention Authority. (Statens brann- inspeksjon).

The tear and stretch properties of the PVC fabric complies with the requirements set out in DIN 53356 and 53354. In practical terms this means that the fabric can withstand the weight of a man inspecting the roof of the cavern above the fabric. Small spalling of rocks on to the fabric would not cause problems.

Figure 3

5 EFFORTS TO REDUCE FIRE-HAZARDS

Due to the great areas and the high value of the stored equipment, it was necessary at an early stage in the planning to decide on different methods to reduce fire-hazards. Public Regulations in force for fire prevention were the basis for the solutions selected for sectioning of the storage halls, the fire alert systems and the escape routes.

The mounting of the PVC fabric "tent" in order to secure a dry climate was thoroughly investigated also from a fire-technical point of view. The PVC fabric selected was non- flammable. could not be set on fire or would not actively contribute to spreading a fire. The central Fire Prevention Authority accepted the use of this fabric based on the fact that storing of equipment was passive and that only a small number of persons would be involved in the maintenance of the equipment.

Non-flammable gates and doors were installed all through. Automatic closing was assured at fire alert through the fire alarm system. This system was based on optical sensors as well as the traditional smoke and heat sensors to give the earliest warning for evacuation and fire fighting. A wide- spread network of fire extinguishing points would make fire fighting possible at a very early stage in case of fire.

To facilitate an effective planning process the Defence Construction Service selected a group of planning engineers with sufficient data processing capability .Specialists in engineering geology, fire prevention and fire alert systems were engaged in the planning at the correct time, as was the supplier of the PVC fabric.

Through this planning process the actual construction was rationalized and the costs of construction differed to a very little degree from the calculations arrived at early in the planning process. This contributed, in addition to the other advantages of building the storages in rock, to the acceptance by NATO of the rock-alternative.

6 THE CONSTRUCTION

Two stages of construction of the two largest facilities was introduced in order to achieve the earliest start and the shortest construction time possible.

The first stage mainly comprised:

- Blasting and securing.
- Rough planning of the floors in the rock facilities and of the areas where out-door buildings were to be build.
- Laying of drain pipes, water and sewage pipes and pipes for the accommodation of electric cables.
- Construction of the accessory and internal road net.

The second stage comprised

- Concrete constructions of different kinds
- Humidity control and ventilation systems
- Electric installations
- Rigging and mounting of PVC-sheet lining.

Due to the magnitude and complexity of the work, pre-selected tenders were invited to compete for the civil works. For the two biggest facilities, the contractor who won the contract for stage one, was allowed to negotiate the contract for stage two. For facility number three no division of stages were introduced. If time had not been a critical factor, none of the contracts would have been divided.

For all three equipment storages tenders were asked for:

- Civil works.
- Tubes and pipes.
- Ventilation system.
- Electrical works including power supply and communications systems.

Contracts were closed directly by the Defence Construction Service with the contractors for each of these categories, each being responsible as a side contractor. The form of contracts selected placed the responsibility for coordinating the activities between the different categories of work directly with the Defence Construction Service. At each site a local construction office responsible for management and control was established under the leadership of the Trøndelag Regional Division of the Defence Construction Service. Qualified engineer geological and geotechnical expertise were available to these local offices for routine control and inspections. This again called for a high level of local coordination at the local construction office. The experiences from this arrangements, however, were positive both from the point of view of progress of work and economy.

The layout and configuration selected for the different facilities made possible a rational and effective utilization of construction equipment such as drilling rigs. No major unforeseen problems were encountered which to a noticeable degree could hamper the progress of work. One of the reasons being that sufficient engineer geological investigations were carried out during the preliminary engineering phase and followed up during actual construction. Conditions concerning the mechanical properties and jointing of the rock masses which called for special securing, such as grouted bolts and reinforced gunite, were discovered in time.

At one of the facilities it was necessary to extend the hall 25 m. in order to compensate for a shortening at the other end due to discovery of a weak zone not found during the preliminary engineering phase.

At two of the facilities the rigging of the internal "tent" of PVC-sheet lining was done by the supplier of the fabric. At one facility the rigging was done by the civil works contractor. Noticeable difference in rigging time or in the quality of the work was not registered. Even if the internal "tent" is easy to maintain and pose no problems as to inspection of the cavern roof, extensive securing against spalling in the form of gunniting was done. This, however, has to be seen as an assurance to avoid surprises during future maintenance work

It was clear at an early stage during planning that the local communities would have a great requirement for waste rock for different purposes.

These requirements turned out to be:

Tromdal:

- 200.000 m^3 waste rock for building a pier and an area for industrial use in the Verdal harbour area.
- - 3,5 km of road to be opened for public use.

Frigard:

- 135.000 m^3 waste rock delivered to the airport authorities at Værnes airfield.
- - 25.000 m^3 waste rock to a recreation area.
- - 5 km forestry road.
- - 1 km public road.

Bjugn:

- 120.000 m^3 waste rock for Bjugn industry area.
- - 10.000 m^3 waste rock for main public road.
- - 700 m embankment filling.

These figures illustrate clearly the positive effect to the public community of the military construction activity.

7 CONSTRUCTION TIME

The facilities were handed over to the user in the period May -August 1988. The total construction times were 18 -26 and 28 months.

8 COSTS

The average cost for the three equipment storage facilities was about US $ 477,- per m^2, VAT excluded. The average cost for the ammunition storage facilities was about US $ 743,- per m^2, VAT excluded.

The estimated prize for above ground climatic controlled warehouse type storage facilities would have been about US $ 428,- per m^2, VAT excluded.

The cost for the procurement of the real estate necessary for building warehouse type storage facilities would have been considerably higher than what was paid for the rock installations. Operating and maintenance costs for rock facilities are estimated to be well below the cost of above ground installations, thus the extra investments involved in the rock facilities would be compensated for within 10 -12 years.

7 THE ENVIRONMENT

The siting of the three equipment storages in Trøndelag were such that there were no special requirements needed to take care of the mud from the drilling. At one of the ammunition sites, mud disposal of mud caused problems for a near by salmon hatching farm. The Defence Construction Service has as a consequence of this, introduced a number of precautionary measures in the preliminary engineering of a facility in North Norway.

In the series of successfully completed underground installations in Norway, the prepositioning storages for the US Expeditionary Brigade in Trøndelag are among the best. Undoubtedly the Norwegian Defence will also in the future draw on the engineering expertise gained through the vast number of different underground installation build in Norway over the years, in order to utilize rock masses for cost effective defence installations.

In this connection, a very important element should be borne in mind, the minimal negative consequences for the environment which underground installations cause, combined with the different benefits which are gained by the civilian communities involved.

DESIGN OF CRUDE OIL STORAGE IN ROCK CAVERNS AT STURE, NORWAY

M.Sc. Nils O. Midtlien, Berdal Strømme a.s.

SUMMARY

The four storage caverns at the Sture terminal have been designed for a capacity of 800,000 m^3 of crude oil, and each cavern being independently operated. A fifth cavern is designed to receive ballast-, oil contaminated leakage and surface water .

The caverns are unlined and the dimensions are: width 19 meters, height 33 meters and length 314 meters , having ceiling elevations set at - 25 meters below sea level.

The oil is stored floating on a fixed waterbed, with the internal gas pressure above the oil, dependent on the degree of filling.

A sufficient overburden of water-saturated rock will prevent gas leakages to the surface.

An artificial water infiltration system facilitates control of the ground water level.

Owner:	Norsk Hydro Produksjon a.s., representing the partners of Oseberg Transport System
Contractor:	Selmer Anlegg A/S
Consultant:	Norconsult A/S
Value of contract:	330 mill. NOK (Civil works)

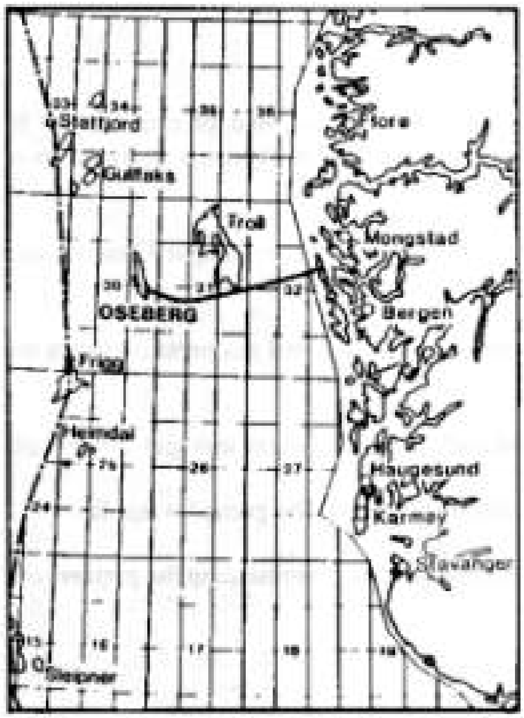

Figure 1 Oseberg Field with Pipeline to Sture

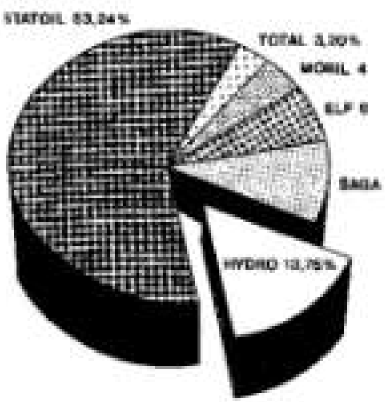

Figure 2 Partners of Oseberg

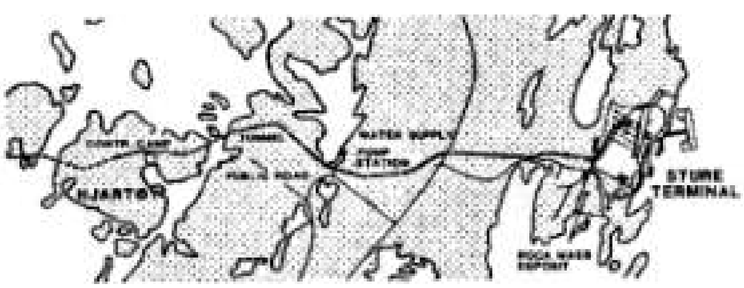

Figure 3 Pipeline from Hjartøy to Sture

1 THE OSEBERG OIL FIELD

In the spring 1984 the plans were approved by Norwegian Authorities for the development of the Oseberg Oil Field, Blocks 30/6 and 30/9.

The overall project consisted of four separate facilities developments, Process- and Accommodation-Platform, Production Platform, Transportation System including the on- shore Terminal (T -project) and the Subsea System consisting of two subsea satellite wells.

The total development cost was approximately 42 billion NOK (1987).

T -project which comprised the pipeline from the field, the shore approach, landpipe and the crude oil terminal was developed for approximately 3.6 billion NOK (1987).

The 28" diameter pipeline was the first subsea pipeline constructed from an offshore oil field in the North Sea, to the main land of Norway, a distance of approximately 109 km, at a maximum depth of 360 meters.

2 THE TERMINAL

The terminal and the storage caverns are located at Sture in Øygarden municipality, west of Bergen, on the west coast of southern Norway.

Optimization of the following factors had to be made before location selection:

- distance to the oil field
- shore approach feasibilities
- harbour facilities for tankers up to 300,000 d w .t.
- space and access for terminal and storage caverns
- possibilities for future expansions of caverns and harbour facilities.

3 LANDPIPE FROM SHORE APPROACH AT HJARTØY TO STURE TERMINAL

The main objective of the terminal is to act as a buffer storage facility for stabilized crude oil from the Oseberg Field. There are no process facilities established at the terminal.

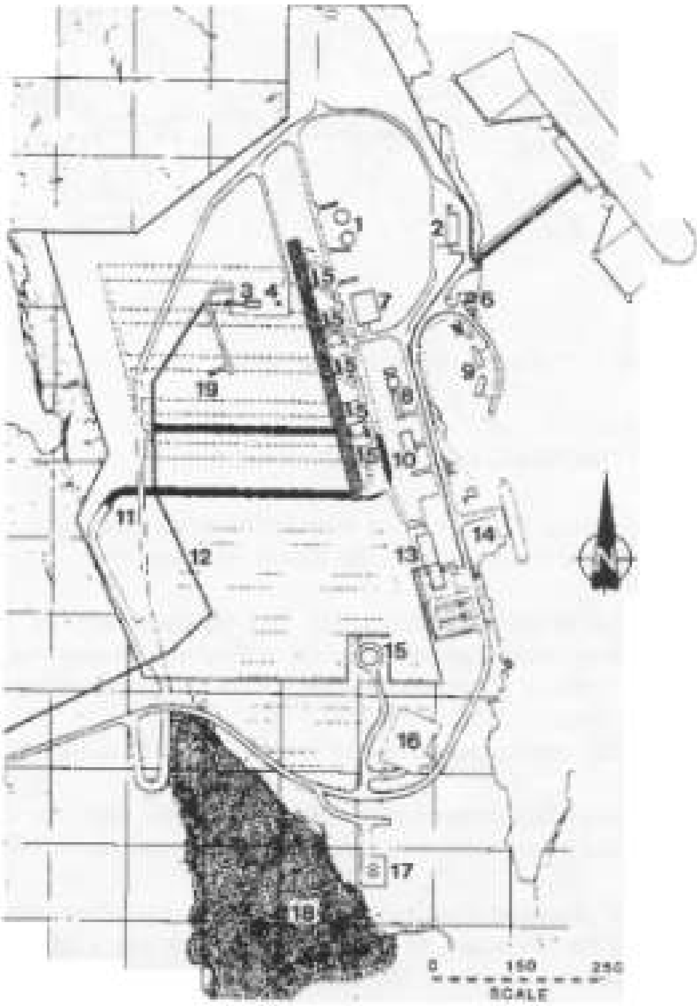

Figure 4 Sture Terminal

4 VOLUME OF STORAGE

The available crude oil storage of the excavated caverns is 800,000 m^3. The alternative method reviewed was utilizing above ground steel storage tanks. However, considerations and analysis of construction and operating costs, safety and the effective use of land proved the below ground storage cavern to be the favoured option.

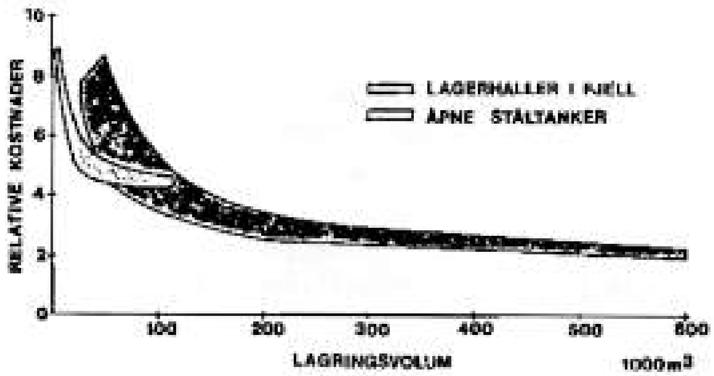

Figure 5 Relative Costs for Large Steel Tanks and Rock Caverns

Four rock caverns, each 19 meters in width, 33 meters in height and having a length of 314 meters provide the volume needed for the crude oil storage, the waterbed and a gas volume of 12% above the oil. A fifth cavern with equal dimensions was also excavated, for the purpose of receiving ballast water from oil tankers. The caverns are separated by concrete barriers, 2.4 to 5 meters thick depending on the cross section of the respective tunnels, in order that each cavern can be operated independently.

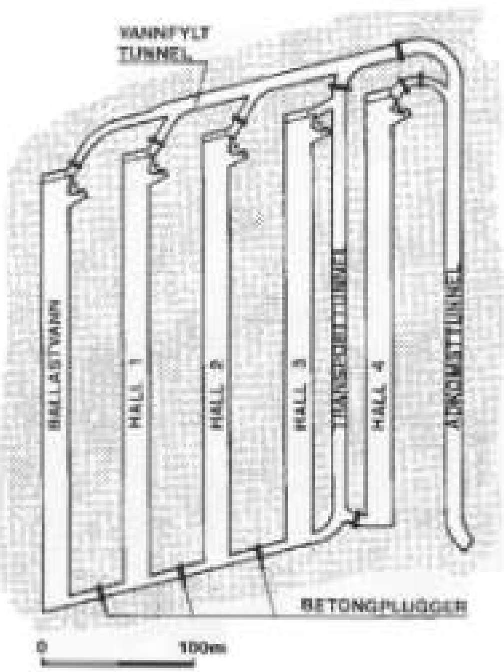

Figure 6 Rock Caverns, Plan view,

5 METHOD OF STORAGE

The crude oil floats on a fixed water bed having a minimum depth of 0,75 meters. Storage is operated as a "closed" system, to ensure that the gas above the crude oil remains in the caverns and does not mix with free air .

In the case of an "open" system, the hydrocarbon gases have to be released when pumping oil into the storage. These gases would probably be flared off.

Therefore, the advantages of the "closed" system are that loss of product is avoided, a flare tower is not necessary, and any permanent system for injection of inert gas into the storage when exporting product is not required.

The gas pressure in the caverns varies according to the quantity of stored crude. When all of the caverns are full, the gas pressure will be 2.5 bara (or 1.5 bar over pressure). When all caverns are empty , the internal pressure will be 0,5 bara (or 0.5 bar under pressure). The gas above the crude oil will be saturated with hydrocarbon gases from the crude oil, such as methan and ethan.

By locating the storage below the ground water table, all leakages will be water into the caverns and not crude oil into the rock masses.

To be able to keep the gases permanently in the cavern system, the pore pressure in the rock masses must be higher than the gas pressure in the caverns. All cracks, fissures and pores in the rock must be saturated with water .

The safety margin dictated by "Direktoratet for Brann- og Eksplosjonsvern" is that the hydrostatic pore pressure in the rock must be at last two bars higher than the maximum gas pressure. With an overpressure of 1.5 bar for a full crude oil storage, the requirement to the pore pressure in the rock above the cavern roof is 3,5 bar , corresponding to 35 meters waterhead.

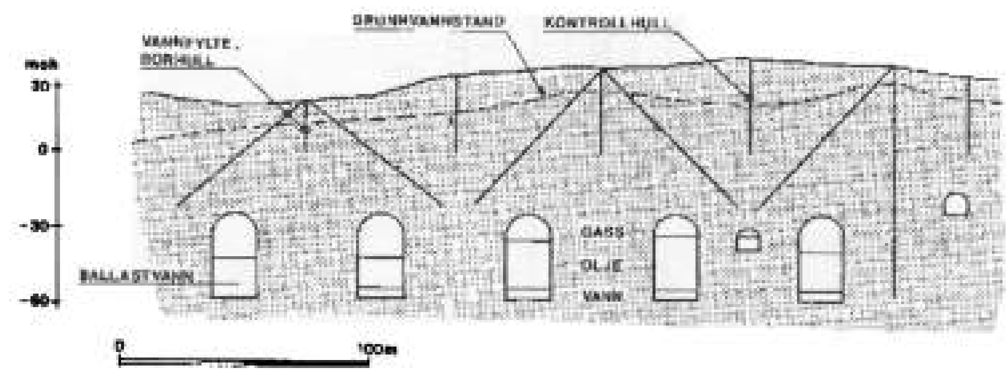

Figure 7 Cross Section of Rock Caverns

6 ROCK CAVERNS

The rock mass consists of moderate to fair cracked gneisses. Information of the rock mass ahead of the cavern excavation was based on mapping in the field, core drilling and pumping tests to evaluate the permeability of the rock.

The orientation of the caverns was chosen to achieve the best possible stability of the rock masses with respect to foliation and the systems of fissures and cracks. The choice gave an unfavourable orientation of some of the smaller service tunnels.

In the longitudinal direction the caverns are located between limitations in the terrain, i.e. lack of rock cover and major weakness zones. The distance between the limitations is fully used, and this gave the cavern length of 314 meters.

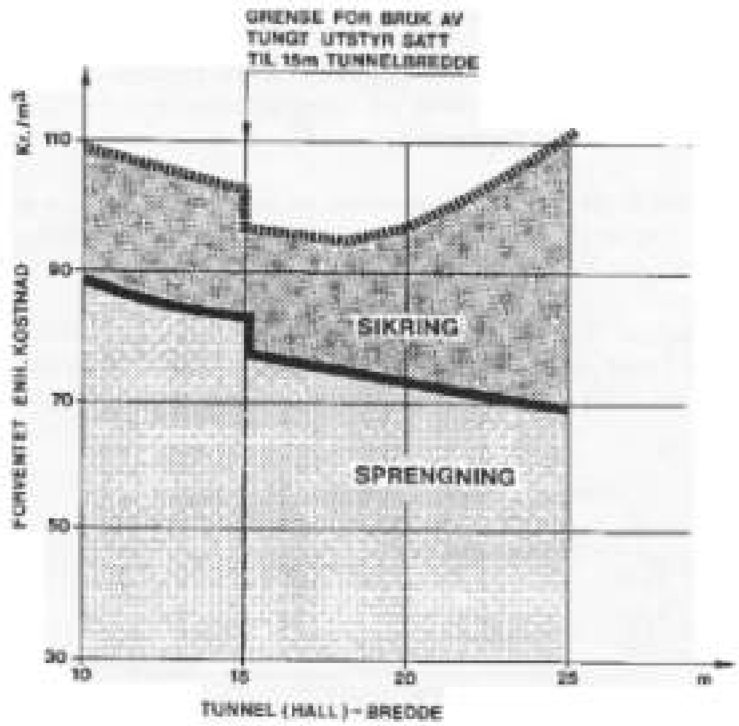

Figure 8 Longitudinal Section of a Cavern

During the conceptual design phase an optimisation of the cavern cross section for rock support and blasting gave a minimum cost for a cavern width in the range of 18 -20 meters, and a height of 32 -34 meters. The chosen dimensions were 19 meters and 33 meters respectively.

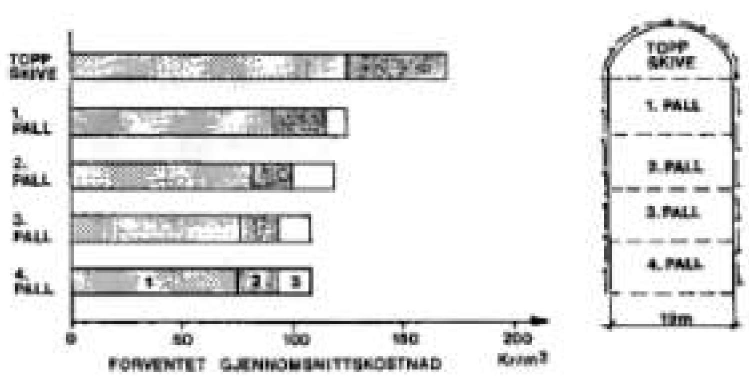

Figure 10 Cost for Blasting, Rock Support and Pumping Product

1. Blasting
2. Rock Support
3. Capitalized Costs for Pumping

The width of the pillar between each cavern is 39 meters, chosen in order to avoid high concentration of stresses/deformations, which would have a negative effect on the high walls in the longitudinal direction of the caverns. During construction the maximum vibration velocity in the neighbouring cavern was set to 200 millimetres per second.

The design of the access tunnel allows future expansion of the storage volume, if required, during storage operation.

The cross section of the access tunnel is 75 m^2 and the width is 10 meters. Branch tunnels to the caverns have a cross section of 41 m^2 and a width of 6 meters. The dimensions of the tunnels were based on the construction equipment characteristics and capabilities.

The rock excavation and tunnel system proposed by Selmer Anlegg A.S. was selected as being the preferred alternative with respect to both construction and economic evaluations.

The advantage of the Selmer Anlegg proposal was the elimination of internal ramps by excavating a tunnel parallel and between two caverns extending from the cavern roof to the cavern floor level, offering an added 60,000 m^3 storage volume.

All tunnels are closed by means of concrete barriers. During operation the only accesses to the caverns are through the drilled shafts.

There are nine shafts to each cavern, -one in the centre and eight at the eastern end. The use of the shafts are:

Instrumentation:	1 pcs. Ø 2100 mm
	2 pcs. Ø 1000 mm
	1 pcs. Ø 600 mm
Oil, export:	4 pcs. Ø 2100 mm, (one spare)
Leakage water, out:	1 pcs. Ø 2100 mm
Oil, import:	1 pcs. Ø 1200 mm

The shaft for the pipeline into the cavern and the shaft for instrumentation in the centre of the cavern are both approximately 65 meters long. All other shafts have a length of approximately 35 meters.

The shaft for the pipeline into the cavern is drilled from the shaft top area, outside of the cavern wall, into a branch tunnel at elevation of the cavern floor. From this tunnel the pipeline continues in a trench, and rises to above floor level at the opposite end of the cavern. This is to ensure circulation of the product stored in the cavern. All shafts were bored by means of raise-boring machines from the surface.

Beneath tile shafts in tile eastern end of tile cavern a pump pit was excavated, 15 meters deep, 15 meters wide and 5 meters long.

Along the top edge of the pit and towards the main part of the cavern, a concrete weir was constructed to a height of 0.75 meters water bed above final elevation of the floor in the excavated cavern.

In addition to pumping out product, the pit also serves for collection and pumping out leakage water from the complete cavern. The leakage water is pumped to a water treatment plant at the terminal.

7 ARTIFICIAL GROUND WATER INFILTRATION

To avoid gas leakages a sufficient pore pressure must be permanently maintained above the caverns. By ensuring correct elevation of the ground water level, the elevation of the caverns can be optimised .

With a pore pressure of 35 meters water head in the rock just above the cavern roof, with the roof elevation -25 meters, the ground water table must be maintained at elevation + 10 meters or higher.

During construction, it is important to avoid or reduce drop of the ground water level so that air pockets are not created in the cracks and fissures. Local air pockets in the rock mass will give a low pore pressure and an increased possibility of leakage of gas, which will be very difficult to stop later by means of injection of grout.

There are two main alternatives for artificial infiltration of water.

- Tunnels filled with water and eventually boreholes drilled from the tunnels.
- Boreholes drilled from the surface and pressurized by means of a water pipeline system above ground.

At Sture, a system with long, water pressurized boreholes was chosen primarily to reduce the depth to the storage area, which has influence on both the construction and operation costs (pumping). Bore holes were drilled at 11 metres spacing along the entire length of the caverns. See Figure 7 They were drilled at an incline, pre-determined from the substrata survey, in order to maximise on water injection into the rock mass. The minimum distance to caverns and tunnels is 15 meters.

After drilling, the loss of water from each bore hole was measured at various depths in order to assertion the depth from the surface at which water loss to the surface was unacceptable. This then established the length of the pipe insert to be grouted into the borehole to ensure water containment.

In addition to selecting the most techno-economically favourable system, the tight schedule for construction was also a prime consideration.

Based on orientation of cracks, rock cover and distance to the water pressurized bore holes it was required to have the water injection system established at last 50 meters ahead of the tunnel face during excavation. This was to ensure that the water infiltration could maintain the water table, should leakages into the cavern system occur during probe drilling, drilling or blasting.

The elevation of the ground water table was observed weekly at 50 locations above and outside the cavern area. Ten of the holes for these observations were established as early as possible, ahead of the start of excavation.

8 LEAKAGE WATER

An important thing for a society located on an island is the amount of fresh water. Also i Øygarden municipaty the total amount of fresh water is limited. To improve the situation, Norsk Hydro had to build a supply pumping station and a water tower. The volume of the 35 meters high tower is 1,800 m^3.

The available amount of fresh water for construction was estimated to be 600 m^3/day. Together with the pumping and treatment costs, the total leakage into the cavern system should be less than 300 litres per minute. To be able to be within this upper limit, all leakages and increase in water consumption during rock excavation had to be observed.

During excavation a systematic probe drilling was performed and, for leakages observed above a certain limit, grouting was performed, followed by a new round of probe drilling.

If leakages were detected during probe drilling, tests were performed to check that there was no direct connection to the drillholes in the ground water control system, in order to avoid filling up the infiltration system with grout. In general there was a restrictation set to the amount of grout to be pumped into the rock mass.

9 SAFETY

Norsk Hydro give safety a high priority .The contractor, Selmer Anlegg A/S had to show an implemented and documented QA-system before the contract was awarded.

For the underground works, Norsk Hydro established an open dialogue with the authorities with respect to safety and working conditions for the workers. As usual for tunnelling works in Norway the contractor is responsible for the rock support during construction.

For the lifetime of the storage, 40 years, a permanent rock support system was added, which was the responsibility of Norsk Hydro.

The cavern roofs were shotcreted and grouted rock bolts were installed systematically. The walls surrounding the pump pits were shotcreted and on some walls, reinforcing bars and rock anchors were also installed.

Rock support for the tunnels that were not an integral part of the permanent access system were evaluated on site.

CRUDE OIL CAVERNS AT MONGSTAD, NORWAY

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ABSTRACT

The article describes the philosophy behind and the construction of six crude oil caverns (storage volume 1.300.000 m^3) situated at Mongstad in the western part of Norway.

From an engineering point of view the main challenge was not unstable rock, but to take care of the ground water during construction and at the same time enable a short construction period (from March 1985 to end of January 1988). A water curtain system drilled from above ground was established.

To minimize costs another important engineering matter was to make a lay-out with low Ratio between total excavated rock volume and storage volume (Ratio achieved 1,39).

From a construction point of view the main challenge was the short construction period. Maximum rock production necessary, is approximately 60.000 asm^3/week.

1 INTRODUCTION

Situated at Mongstad in the western part of Norway, *Statoil* is building 6 crude oil caverns with a total storage volume of 1.300.000 m^3, the biggest ever built in Norway. Main Contractor is *Astrup Høyer A/S.*

From a jetty, crude oil is being pumped through a 2 km long pipeline into the caverns. Pipes, pumps, instruments etc. are installed from the surface down to the bottom of the caverns through concrete filled shafts with a cross section of 47,6 m^2.

High capacity pumps (5000 m^3/h), 2 in each cavern will pump the crude oil to the jetty.

2 GEOLOGY

The Mongstad area is within the "Bergen buene" and consist mostly of various gneises with alternating light and dark layers and foliation mainly with strike N 160g and dip 45-60°SW. Cavern axes are N 280g E.

Up to 1 m thick layers of pegmatite and amphibolites occur occasionally. Some zones contains minor amounts of swelling clay. Drillability, Drilling Rate Index (DRI) is (for the gneis) 47.

3 GROUND WATER MAINTENANCE

3.1 Introduction

Main tasks of the ground water above the caverns are to:

- withstand pressure from gas above the crude oil (Design pressure used is 2,5 bar a) to prevent gas-leakage from the caverns.
- ensure leakage of water into the caverns to pre- vent crude oil to leak out.

Ground water is not allowed to be lower than elevation 0.

To ensure maintenance of the ground water level during both construction and operating periods, the following actions were executed:

a) *A water curtain* system drilled from above ground was established previous to blasting.

b) *Probe drilling* from the faces was always executed previous to blasting of tunnels and top sections of the caverns. For the benches only a few parties were probe drilled.

c) All leakages > 2,0 J/min. from a 30 m long probehole with diameter 2 inch. were *grouted.*

3.2 Water Curtain System

Fig. 3 shows in principle how the water curtain system was established.

The total amount of water used for the system seems to be, after a short testing period, approximately 15-25 m^3/hr, at a pressure 6-7 kp/cm^2.

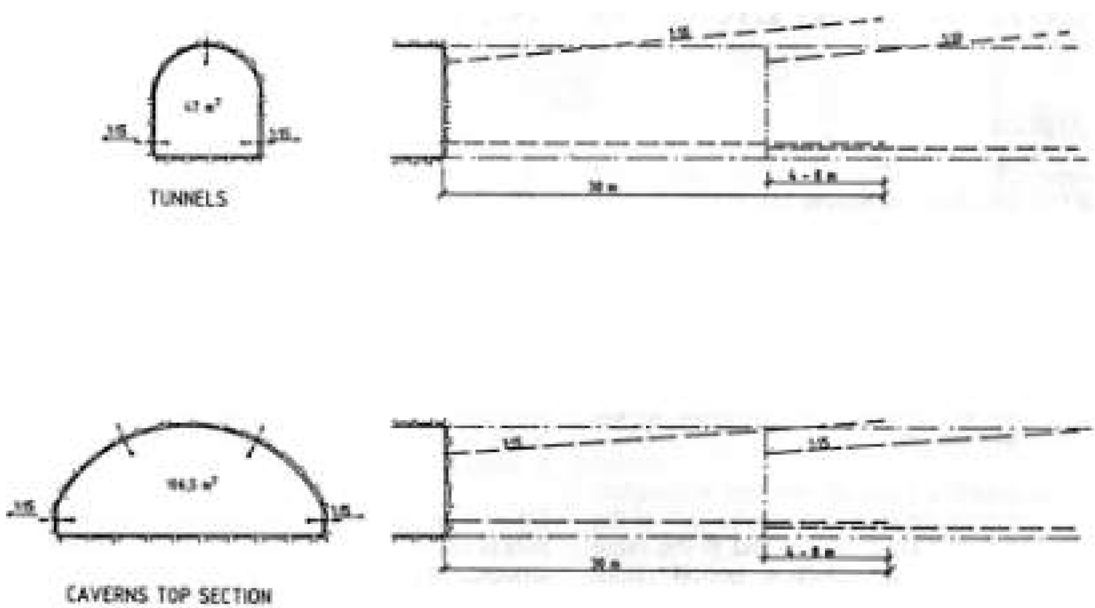

Fig. 1 Probe drilling

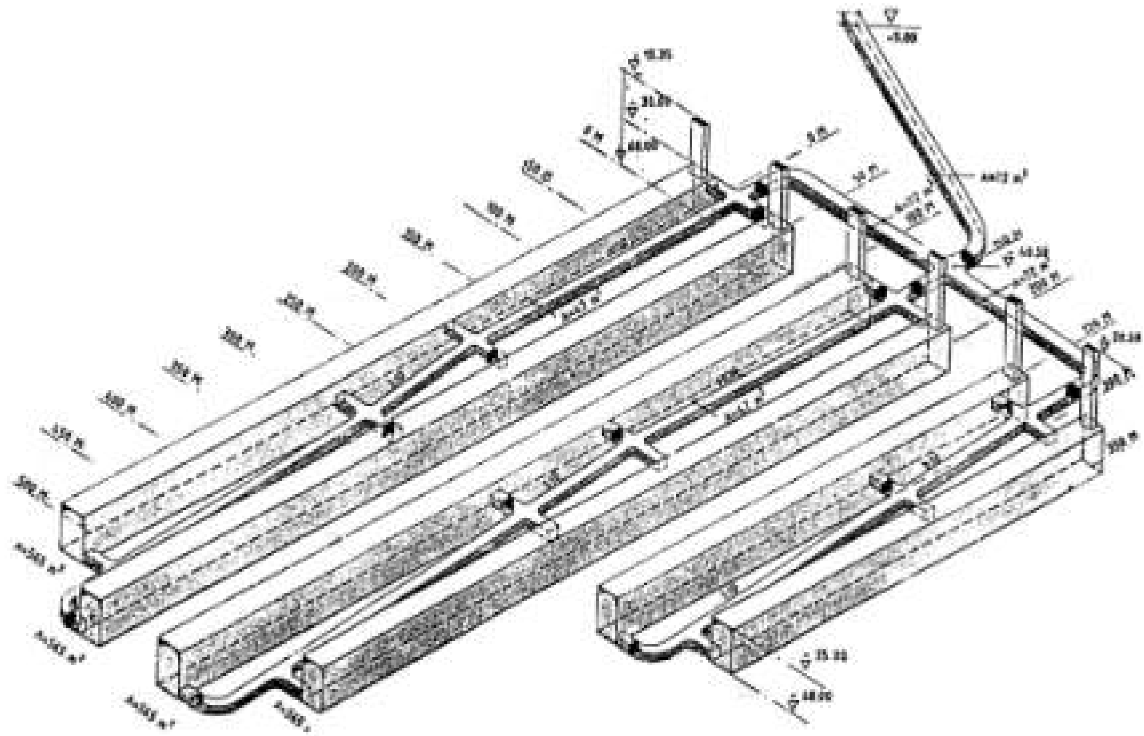

Fig. 2 Crude oil caverns 1,300,000 m³ storage volume

Experiences so far, shows that the established water curtain is operating very satisfactory. In fact the amount of grouting at the face would probably have been reduced if the effect of the water curtain had been known.

The water curtain system probably will be in function during the operating period as well. Final decision will be taken after a testing period of a couple of years.

3.3 Probe Drilling

Fig. 1 shows the pattern used for probe drilling and also necessary overlap for tunnels and cavern tops.

Systematically probe drilling for tunnels and cavern tops were executed to discover:

a) water leakages, so that the rock if necessary could be grouted previous to blasting (leakage ≥ 2,01/min.).
b) weakness zones that could cause stability problems.

For benches probe drilling will be used only in parties where the biggest leakages were discovered during excavation of the cavern tops.

3.4 Grouting

Approximately 10 % of the cavern length's were grouted from cavern top faces previous to blasting. The same areas were also grouted from the cavern top floor previous to blasting of bench I. The same procedure will also be used for bench II.

Minor leakages were grouted after the blasting was finished, but all grouting in cavern tops was completed before blasting of bench I started. The same procedure will also be used for bench II.

Grouting previous to blasting was in general carried out using Cemsil. (Cemsil is a fast hardening cement based material.) Grouting-pressure used was 10 bars higher than the ground water pressure.

Rapid cement was used for grouting carried out after blasting was completed. (10 bars overpressure was used.)

4 ROCK SUPPORT

4.1 Introduction

Good rock quality made it possible to take care of all rock support (also weakness zones) using bolts. steel bands and shotcrete.

4.2 Caverns

4.2.1 Ceilings

All cavern ceilings were systematically supported using 4 m long grouted rock bolts and a 5-8 cm thick layer of shotcrete.

Generally, bolts were placed after a 3 x 3 m pattern

Weakness zones and other parties which required more support were taken care of separately. Also these parties were supported using rock bolts, shotcrete and for a few weakness zones also steel bands.

For the ceilings a total of 12.500 bolts and 4.600 m^3 of shotcrete were used.

4.2.2 Walls

Cavern walls will not be systematically supported.

Approximately 20-30 % of the cavern length will be supported using bolts. For weakness zones and crushed parties shotcrete (5-20 cm) and even steel bands will be used.

Walls will be systematically scaled.

5 LAY-OUT

5.1 Introduction

The perspective sketch shown in fig. 2 includes a total of 2840 m- 565,5 m^2 caverns, 2010 m -47 m^2 tunnels and 200 m -47 ,6 m^2 vertical shafts.

From the surface (elevation + 15 to + 20) pipes, pumps, instruments etc. are installed down to the bottom of the caverns (elevation -68) through concrete filled vertical shafts. The crude oil flows in Ø 1200 mm concrete pipelines along the cavern floors to the end of the caverns (the opposite side of the pump-pit).

In the 13,5 m deep pump-pits (bottom at elevation -81,5) two similar high performance submersible oil pumps (capacity 5.000 m^3/hr. each) are installed. The crude oil will be pumped to the jetty (smaller quantities even to the Mongstad Refinery) through a 2 km long pipeline.

The crude oil will be stored on a fixed water bed on elevation -67,5. (Top of the caverns at elevation - 35, and maximum crude oil level on elevation appr. -43.)

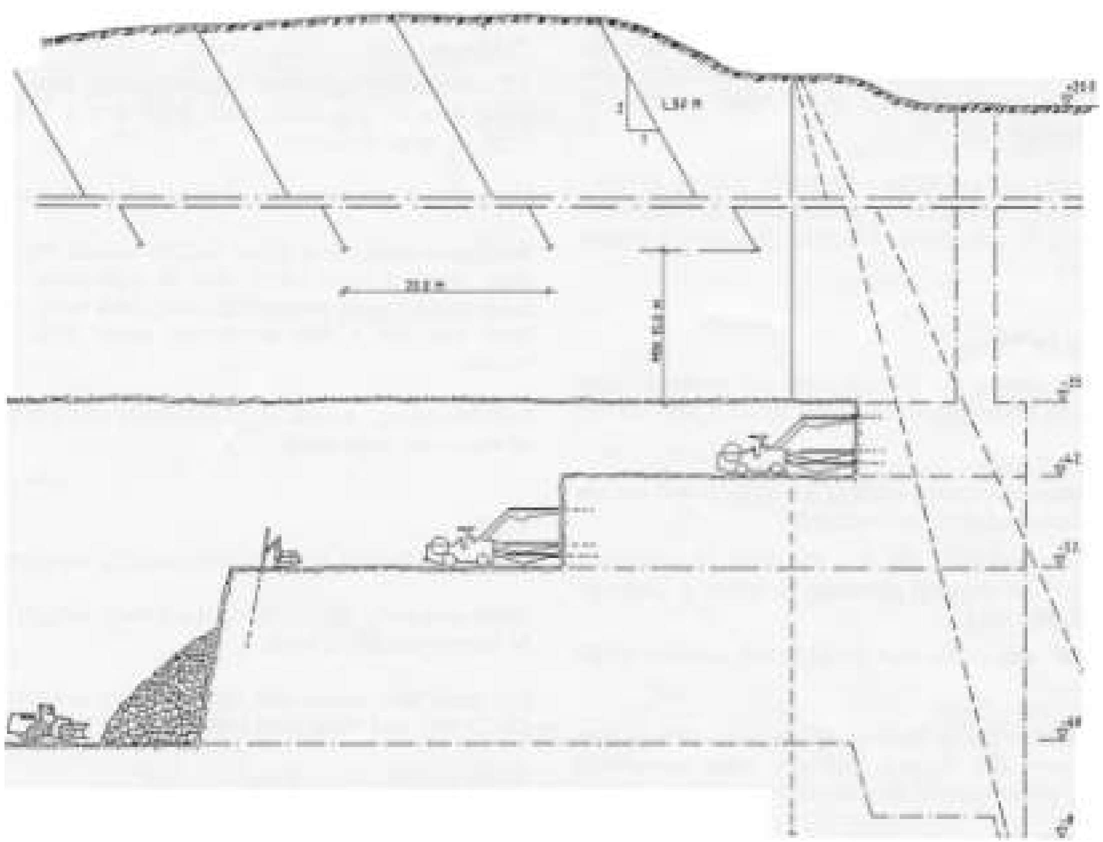

Fig. 3 Principle of rock excavation and water curtains system

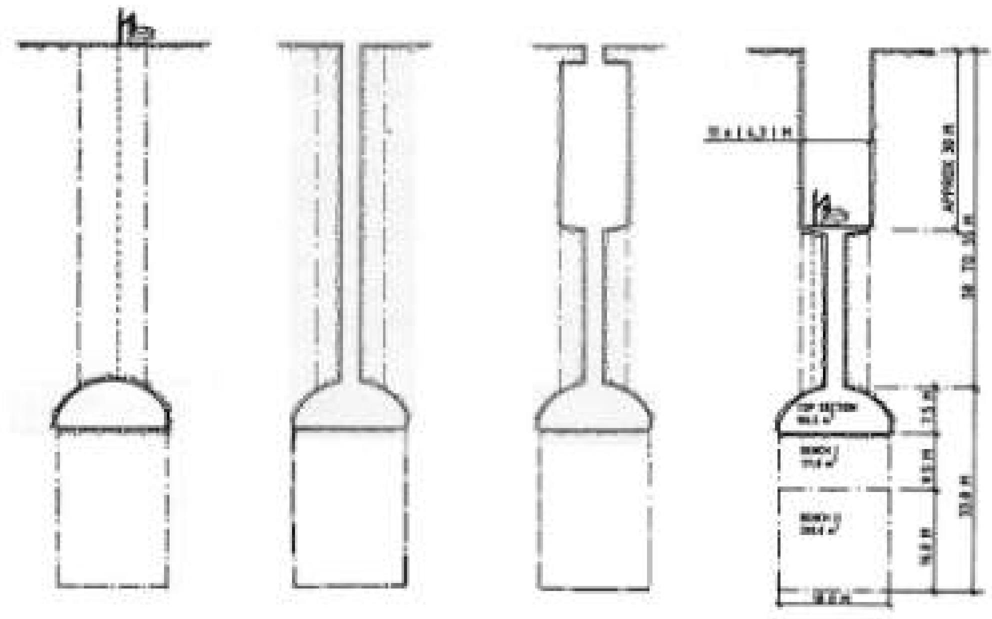

Fig. 4 Rock excavation shafts

5.2 Main Principles

The main principles behind the chosen lay-out were:

A) to maintain a sufficient ground water level during both construction and operating periods.
B) minimize the ratio between total excavated rock volume and crude oil storage volume.
C) enable blasting at as many faces as possible at the same time to minimize the construction period.

A. Some kind of *water curtain system* seemed to be necessary.

The two main principles discussed were:

a) a system drilled from above ground.
b) a system drilled from tunnels over and/or besides the caverns.

Both because of:

- low costs
- construction possible without disturbing rock excavation in the caverns and at the same be able to establish the water curtain previous to blasting
- the technical quality of system

a water curtain drilled from above ground was chosen.

B. The achieved *ratio between total excavated rock volume and storage volume is 1,39*.

The total volume of the six caverns and the access- tunnels between, 1.690.000 m^3, includes a 390.000 m^3 volume for gas on top of the crude oil. (Gas pressure 1,5 bar.) Accesstunnels between the caverns are also used for crude storage.

Twin caverns are separated with concrete plugs (fig. 5.1).

C. During excavation of the top section and even for the first bench (bench 1 drilled with tunnel jumbos, fig. 6.1) a lot of rock support, grouting etc. have to be done at the same time as blasting. It is than of great importance to have as many faces as possible to work at, to enable a short construction period.

5.3 Cross Sections

Cross sections for caverns are given in fig. 6.2.

The main accesstunnels are 10 m wide and 7 ,5 m high which gives 72 m^2. Those dimensions are chosen to enable two way traffic with 32 tons trucks. Maximum traffic jam will be approximately 2 vehicles a minute.

Tunnel in between the caverns are 7 m wide and 7 m high which gives 47,1 m^2, This enables only one way traffic, and therefore a few meeting niches have been blasted. Maximum trafficrate will be approximately 1 vehicle a minute.

Caverns are 18 m wide and 33 m high, a total of 565,5 m^2. The width have been chosen because of the rock quality. (They might even have been chosen 20 m wide without any increase in the rock support volume.)

The ratio between the height of the ceiling bow and the width of the caverns is 0.31, which seems more than sufficient in this rock. The cavern bights, 33 m, is approximately maximum if increased rock excavation and rock support costs should be avoided.

6 CONSTRUCTION METHODS, PERFORMANCES

6.1 Introduction

Rock excavation has been carried out according to conventional tunnelling methods.

Fig. 3 shows the rock excavation of the caverns in 3 steps, i.e. top section, bench I and bench II.

6.2 Tunnels and Caverns

6.2.1 Equipment

Drilling

For *rock excavation* of top section and bench I, 3 Atlas Copco -Promec TH 170 jumbos with 3 drilling machines (Cop 1238 LP) were used. Drilled length: 4,3 m, diameter 45 mm. Drilling capacity: 150-200 dm/hr.

The 4th and 5th jumbos were used for bolting and probe drilling/drilling for grouting respectively.

Charging
3 separate jumbos with ANFO -equipment on were used for charging.

Loading, Transportation
Caverns and Access-tunnels were loaded by using 3 CAT 988B wheelloaders (39 tons). For transportation a total No. of 20 Kockum 442B (loading capacity 32 tons) will be used. Loading capacity: 150-200 asm^3/hr (asm^3 -actual solid cubic metres).

For tunnels between the caverns (47 m^2), one CAT 980C wheelloader and trucks with 19 tons loading- capacity were used for loading and transportation.

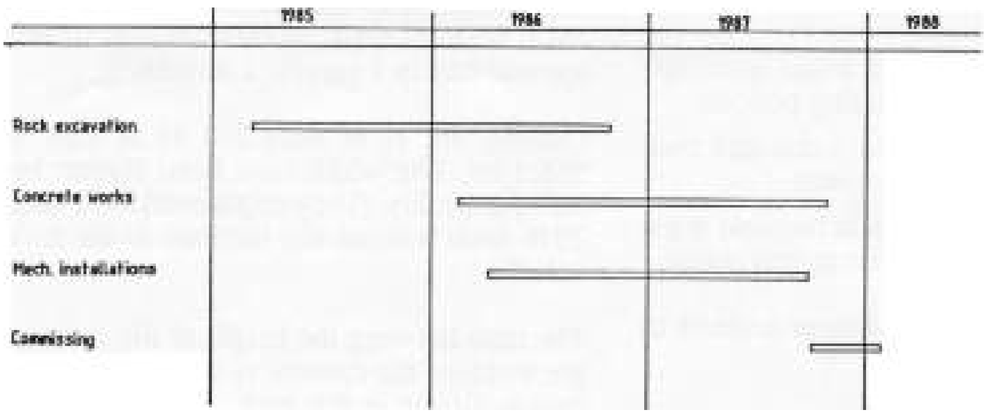

Fig. 5 Overall time schedule

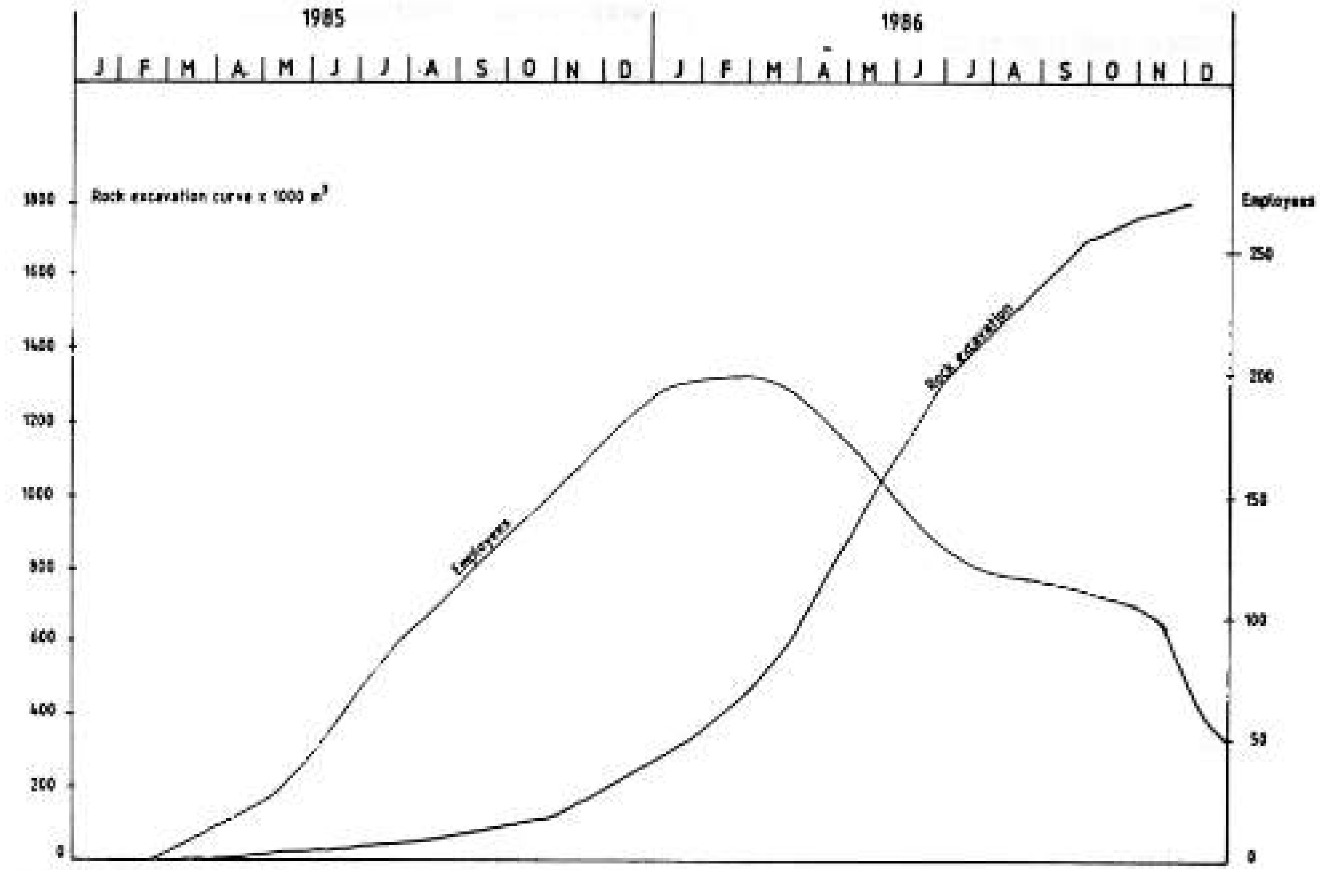

Fig. 6 Rock production and number of employees

Scaling

For both tunnels and caverns (top section and bench I) one smaller excavator on wheels with a special scaling bucket was used for faces and ceilings.

Before drilling scaling by hand was carried out.

Grouting

A special jumbo was used.

Rock support

For the shotcrete a special wet-mix spraying jumbo made by *Robocon A/S* was used. Capacity 5-8 m^3/hr, waste of material approximately 5 %.

6.2.2 Shafts

Fig. 4 shows the rock excavation of the vertical shafts (47,6 m^2) in four steps.

6.3 Specific Drilling and Charging

Specific drilling and charging are given in the following table:

	Cross section	Borhole diameter	Specific drilling	Specific charging
Access-tunnel	72 m^2	45 mm	1,36	1,41
Tunnels between caverns	47 m^2	45 mm	1,57	1,61
Top section	106,5 m^2	45 mm	1,0	1,15
Bench 1	171 m^2	45 mm	0,44	0,55
Bench 2	288 m^2	2 inch	0,3	0,47

7 MANPOWER

Manpower curve is given in fig. 6.

8 TIMESCHEDULE

Fig. 5 shows the overall time schedule.

To enable a very short construction period, high rock excavation performances was necessary.

Fig. 6 shows the rock production curve. Maximum production needed is 60,000 asm^3/week. i.e. 6,000 loadings pr. week.

Fig. 1. Example of an unlined cavern for water storage at Steinan, Trondheim, Norway, viewed from the dam wall. The concrete floor slopes toward the trench, which in turn slopes toward the dam. These features and the permanently installed Pipeline along the right wall facilitate cleaning of the cavern.

STORING WATER IN ROCK CAVERNS

E. Broch
L. Ødegaard

During the past 15 years a number of underground openings have been excavated in the hard bedrock of Norway as replacements for or alternatives to open reservoirs, concrete tanks, or steel tanks for the storage of drinking water. A closed water tank-which is what a rock cavern tank is-has several advantages over traditional open reservoirs. Above all, it is easier to keep pollution under control with a closed tank.

In open reservoirs the drinking water is exposed to the influence of sunlight and pollution from the air. Moreover , open reservoirs are commonly situated in natural or artificial depressions and will therefore tend to collect drainage from the surrounding landscape. If such reservoirs are located close to populated areas, there is a danger that polluted surface water or ground water may be drained into the drinking water.

Today open reservoirs for the storage of drinking water are not normally acceptable to the health authorities in Norway. They have to include closed tanks of some kind and old schemes with open reservoirs often have to be redesigned and reconstructed.

FUNCTION AND LOCATION OF WATER TANKS

The basic function of a water tank is to act as a storage buffer to meet variations in consumption and keep the water head stable. This makes it easier to operate the treatment plant and the pumps at constant capacities and it al- lows smaller dimensions of the main pipe lines. The water tank also acts as emergency storage in case of fire or failure in the supply system.

Small water tanks are normally single-chamber tanks. For volumes exceeding approximately 10,000 m^3, double- or even multiple-chamber tanks are often used. This allows one chamber to be emptied for cleaning and maintenance without interrupting 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, especially if the capacity of the tank is designed to cover the variations in daily consumption.

Most water tanks in Norway have been freestanding structures made of conventional reinforced concrete or prestressed concrete. When double chambers were necessary, either two separate structures were constructed or two concentric chambers were built in one structure. To minimize the impact of such concrete structures on the environment, they have often been dug partially into the ground or hidden in other ways.

One way of making water tanks in- visible, of course, 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., excavating caverns for water in bedrock. A rock cavern tank is normally comprised of an access tunnel and one or more chambers in the form of large, short tunnels; in front of the chamber there is a dam wall (Figs. 1- 3).

Fig. 2 The Steinan cavern in Fig. 1, viewed from opposite the dam wall

COMPARISON BETWEEN ABOVE- GROUND AND UNDERGROUND WATER TANKS

The topographical and geological conditions in an area may be such that either an underground or a surface structure could be constructed; In this case a careful evaluation of the two alternatives for water storage should be carried out.

Factors that would favour a rock cavern tank are:

- a high degree of safety (even from war hazards);
- constant,- and low-water temperature;
- replacement of a surface eyesore;
- good possibilities for future extension;
- low maintenance costs; .multiple uses;
- little or no additional cost for a two-chamber facility.

Factors unfavourable to an underground cavern tank are that:

- polluted ground water may seep into the drinking water;
- water may leak out through the rock mass.

Factors favouring a surface tank, on the other hand, include:

- elimination of the risk of polluting the water (except in the case of sabotage);
- ease of leakage detection and repair.

The following disadvantages with a surface facility may be mentioned:

- the surface tank occupies building ground;
- the tank may be regarded as an eyesore and thus resisted by the neighbourhood;
- the tank may endanger the surrounding area in case of war;
- the water temperature will change with seasons.

Even though the above-mentioned positive and negative factors are important enough in themselves, the decisive factor for choosing the type of water tank will normally be the cost. Later in this paper the authors will show that for certain conditions rock cavern tanks are competitive with surface water tanks.

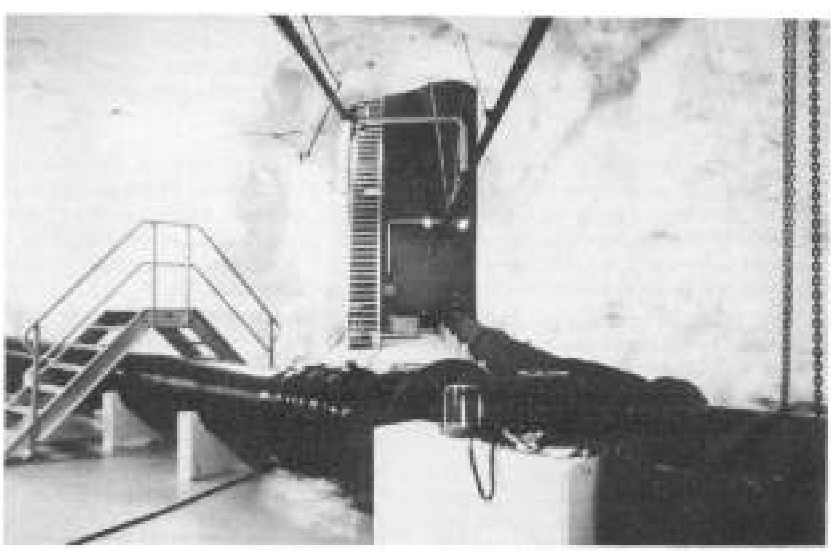

Fig. 3 The entrance tunnel at Steinan, Trondheim. The walls are shotcreted and spraypainted white.

PLANNING AND DESIGN

As the rock cavern tank is basically a rock mass structure, the success of the planning and design depends to a large extent upon cooperation between the consulting engineer and the engineering geologist. The general design procedure for tunnels and caverns in Norway is divided into four stages (Broch and Rygh 1976; Selmer-Olson and Broch 1977):

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

Mistakes in anyone stage will result in overall economic consequences, the extent of which will vary with local conditions and the type of project. Normally rock cavern tanks in Norway are unlined and have a limited overburden. This implies that the rock mass is subject 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, i.e., the possibility of leakage along intersecting joint sets, be taken into consideration when water tanks in rock are designed. Generally, stiff rocks like granites, quartzites, etc., have a tendency to greater leakage than more deformable rocks like micaschists, phyllites, etc. Carbonate rocks (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 the location of the tank. It should not be forgotten that when the location of an underground opening is decided, the choice of material into which the opening is going to be excavated is also made. It is therefore of the utmost importance that this crucial decision be based on the advice of an experienced engineering geologist.

To give this advice the engineering geologist will have to carry out geological and geotechnical investigations of the sites. At this stage it is of particular interest to get information about rock types and weakness zones (or faults). Combined with information from the consulting engineer about the upper and lower water levels and the approximate capacity of the tank, and with information about the topography from detailed maps and air photos, the engineering geologist will be able to eliminate unfavourable rock volumes. He will finally end up with a limited number of possible sites.

Before the consulting engineer starts planning possible layouts of the underground facility within these areas, he will also need to know what placement of the length axis for caverns will yield the best stability and least leakage. Evaluations of the joint sets in the rock mass by the engineering geologist will provide the answer. 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 (and for all other under- ground designs) is the entrance. This is the only part that will be visible to the public. From an excavation point of view it is one of the most difficult parts as the rock mass is generally un- stable due to weathering. Great care should be taken, first of all, to find the most suitable site. As for the excavation itself, restrictions should be put on the contractor's work. All too often one can find ugly tunnel entrances where the rock mass has been torn up unnecessarily by too hard blasting. A combination of knowledge of the rock mass and careful blasting is the only way to get a proper result. A thorough discussion of the excavation of tunnel portals is given by Garshol (1979).

THE KVERNBERGET ROCK CAVERN TANK

Kristiansund is a fishing harbour located on an island off the northwest coast of Norway. 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 situated 80-100 m above sea level was needed.. About 2 km from the town center, and only some. hundred meters from the planned pipeline, a mountain called Kvernberget rises to 200 m above sea level. Geological investigations showed that the rock mass of Kvernberget might be used for a water reservoir, the rock being a Precambrian gneiss. Favouring the decision to situate the water reservoir in rock was (among other things) the fact that future expansion could easily be planned for the reservoir.

Figure 4 shows the layout of the Kvernberget water tank. Two basins of 11 m by 7.5 m by 120 m give an effective volume of 8,000 m^3 of water each. The distance between the basins is 15 m. By extending the entrance tunnel, a third basin could be excavated without disturbing the operation of the two existing basins.

Along the outer part of the entrance tunnel a service section contains all the equipment for operation of the whole water supply system. To support the rock in the basins, 100 m^2 of shotcrete was applied and about l00 rock bolts were installed. The rock is thoroughly cleaned and the basins have a concrete floor. The outer part of the entrance tunnel is fully shotcreted with 1 ,200 m^2 of shotcrete.

No leakage was observed when the basins were first filled with water and, after three years in operation, no water loss has been observed.

A total volume of 21,000 m^3 of solid rock was excavated at a price of 2.95 million Norwegian kroner (Nkr), transportation of muck and support of the rock mass included. This gives a price of 140 Nkr per m^3. Total costs for this tank, building site excluded, were 5.9 million Nkr (all prices based on 1979 levels).

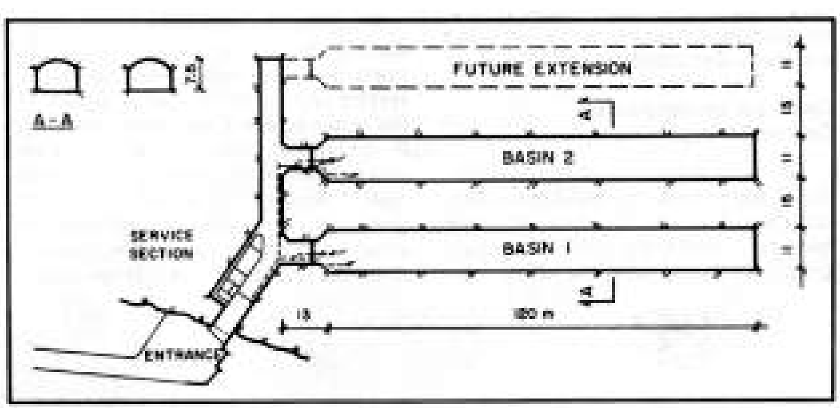

Fig. 4 Layout of the Kvernberget rock cavern tank.

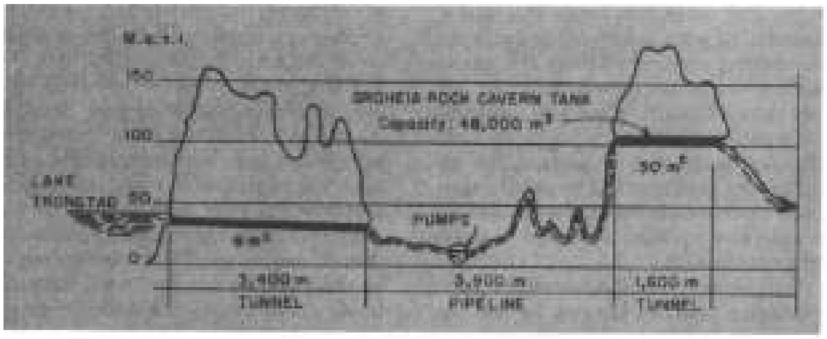

Fig. 5 The Groheia rock tank, part of the Tronstadvann intermunicipal water supply system.

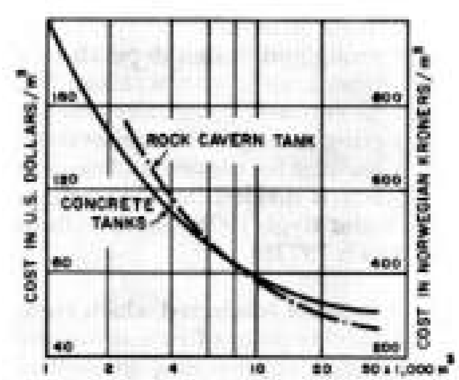

Fig. 6 Specific costs per cubic meter of storage volume for conventional water storage tanks of reinforced concrete and for unlined rock cavern tanks.

THE GROHEIA ROCK CAVERN TANK IN KRISTIANSAND

When storing water can be combined with its transportation, i.e., when the transportation system itself or a part of it can be used as a storage tank, especially cheap installations are possible. In hilly areas pipelines often have to be replaced by small tunnels. When extension of the profile of a tunnel is made during planning, the needed storage volume is easily obtained at a marginal cost. One of a number of examples, the planned Tronstadvann intermunicipal water supply system near Kristiansand in southern Norway, is shown in Figure 5.

From the intake reservoir of the lake of Tronstad the water is conducted through a 3,400-m-long tunnel with a minimum profile of 8 m^2. 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 approximately 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 of the scheme. The rocks in the area are sound Precambrian gneisses. A few weakness zones cross the tunnel and some support measures may be needed. The tunnel will, however, be unlined like the above-described rock tanks.

COSTS FOR UNDERGROUND CAVERN STORAGE

The costs for a number of conventional water tanks of reinforced concrete and for rock cavern tanks have been converted to the 1979 Norwegian price level for the purpose of comparison. The results are summarized in Figure 6 (prices are also given in U .S. dollars). The prices include 13%

in taxes. The two curves intersect at a storage volume of about 8,000 m^3, indicating that for storage volumes exceeding this a rock cavern tank will normally be the cheaper solution in Norway.

Poor rock conditions will increase the costs of support in the unlined rock caverns and thus show an intersection point higher than 8,000 m^3. On the other hand, if the excavated rock can be sold or if the price of land for freestanding water tanks is high, this will favour the choice of rock cavern tanks for storage volumes of even less than 8,000 m^3.

CONSTRUCTION AND MAINTENANCE EXPERIENCE

In the entrance tunnel with its cold pipes and valves it is important to reduce the moisture content in the air by sufficient ventilation. Leakage from the rock may be drained through perforated plastic tubes before shotcreting is done. In a cavern storage facility at Steinan a total of 500 m of such tubes were installed in the roof and walls. The tubes emptied into draining trenches along the walls.

A careful registration of all leakage during the excavation period is important for successful drainage in the service and entrance tunnel, as also 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 drawdown in the ground water table above the rock caverns, water will normally flow towards the basins rather than from them. Leakage from water tanks in rock has never been observed, nor has seepage of polluted water into these tanks been reported. One should always be aware of these possibilities, however .

All drinking water tanks should be regularly emptied, inspected for leakage, cleaned, and disinfected. To facilitate the cleaning, tubes with valves at intervals of about 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 create circulation in the stored water when this is necessary.

CONCLUSION

Where topographical and geological conditions allow a choice between a conventional surface water tank and a rock cavern tank, both alternatives should be seriously considered. Generally, unlined rock cavern tanks are cheaper when the storage volume exceeds 8,000 mg. A number of factors may also favour rock cavern tanks for considerably smaller storage volumes.

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THE RAFNES PROPANE STORAGE CAVERN – 12 YEARS OF SUCCESSFUL OPERATION

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ABSTRACT: The unlined propane storage cavern at Rafnes, with a total volume of 100,000 m^3, is situated in precambrian granite rocks and was put into operation in 1977. The propane gas pressure is varying between 0.50- 0.65 MPa (abs) -depending on the butane content, and the cavern is situated at a depth of 90 meters below sea level. In addition to the gradient put up by the "natural" ground water a water curtain was installed during the excavation. The paper comments on the monitoring results of pore pressures, salt water intrusion and the volume of inleaking water.

1 INTRODUCTION

The Rafnes high-pressure propane storage cavern is situated some 200 km south-west of Oslo at the western shore of the Frierfjord. The cavern is designed for a propane volume of 100,000 m^3 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 principally unlined, except for bolting and mesh reinforced shotcrete in the roof.

The design criteria for the Rafnes propane storage cavern was that the hydraulic gradient towards the cavern should be greater than one. We recognize that this criteria of $i>1$ is much on the safe side. The condition for gas escaping the cavern is not only dependent on the gradient, but also on the joint width, the orientation and roughness of the joint and the capillary forces. The requirements set up by the authorities in Norway for unlined caverns, is based upon the general ground water level above the cavern. The ground water level must be equal or above the level corresponding to the internal maximum over-pressure of the gas plus 20 m head of water. For the Rafnes propane cavern, which has a design maximum propane gas pressure of 0.79 MPa (abs.) the ground water level must be minimum at level -1.

2 THE GROUND WATER LEVEL WAS KEPT REASONABLY HIGH DURING THE EXCAVATION BY GROUTING WORKS AND THE INSTALLATION OF A WATER INFILTRATION SYSTEM

During the excavation of the access tunnel the already installed observation wells showed a marked decrease in the ground water level. At this stage it was obvious that grouting and watering of the surface was not sufficient to keep the ground water above the minimum design level. It was decided to install a water infiltration system from four niches in the access tunnel. A total of 26 infiltration boreholes were bored. The ground water level now rose to its initial level and when another 8 boreholes were added to the infiltration system during the final stages of the excavation, the ground water level was still about the same as before the construction period. The total length of the boreholes is 2,110 m and the water consumption was 450 l/min in 1977 (corresponding to an average permeability of 0.5 Lugeon-units), see Fig. 3.

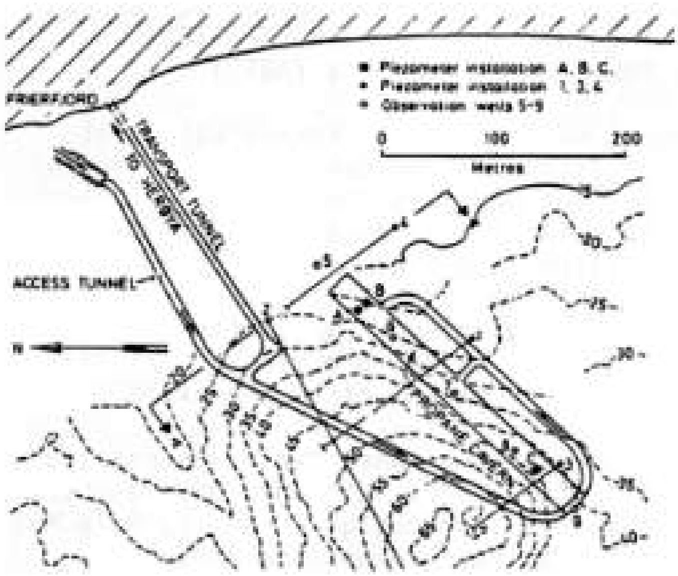

Fig. 1 Plan view of propane cavern with observation wells and piezometer installation.

3 TO MONITOR THE GROUND WATER PRESSURE ABOVE THE CAVERN, FIVE OBSERVATION WELLS AND THREE PIEZOMETER HOLES WERE DRILLED BEFORE AND DURING THE EXCAVATION

Five observation wells were drilled before the excavation started to observe the round water level. Late 1976 three vertical boreholes (A, B and C) were drilled directly above the cavern down to approximately elevation -85 (5 m above the cavern roof). In each of these bore- holes three piezometers were installed at elevations -71, -77 and -84 to monitor the pressure distribution above the cavern roof.

4 EXTRA GROUTING WAS CARRIED OUT BEFORE THE CAVERN WAS PUT UNDER A TEST PRESSURE OF 0.79 MPA (ABS.)

About half of the inleaking water into the cavern was concentrated at the outer end (roof and northern wall) where the piezometer boreholes A and B had been installed. To minimize the volume of inleaking water and to ensure a high hydraulic gradient towards the cavern, it was decided to do more grouting at this location. The effect of the extra grouting can be seen in Fig. 5.

The testing of the cavern was carried out in June 1977. Nitrogen gas was used, and the maximum design pressure, 0.79 MPa (abs), was reached on June 4th.

The pore pressures in A, B and C all rose between 0.2- 0.4 MPa during the test period, but the ground water level did not change appreciably.

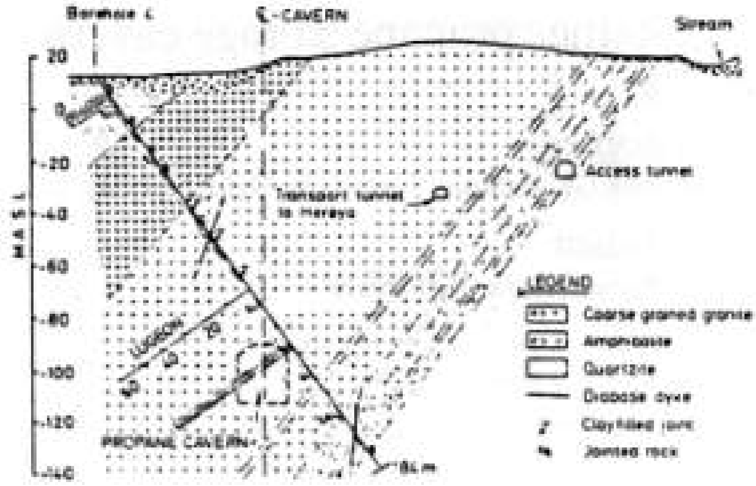

Fig. 2 Cross-section A-A, through bore- hole No.4.

It seems that the ground water 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 ± 0 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.

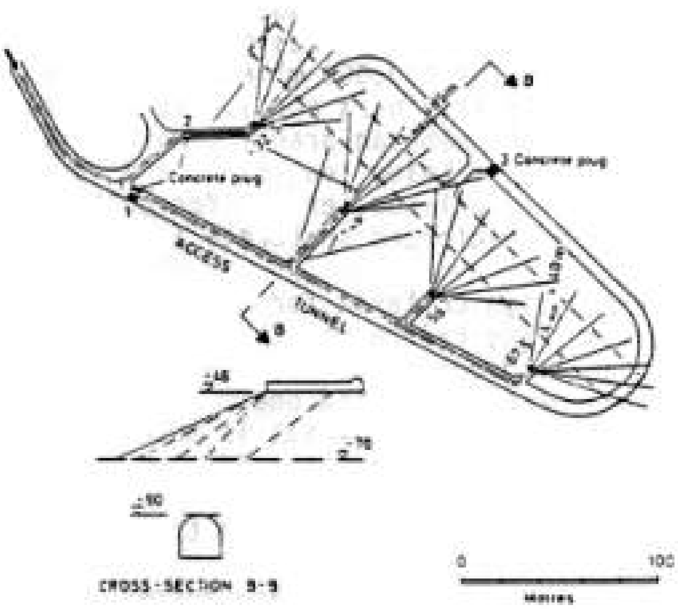

Fig. 3 Plan view of water infiltration holes 36 holes, diameter 2!t2". Total length 2,110 m.

5 DURING MORE THAN 10 YEARS IN OPERATION, THE INFLOW OF WATER HAS BEEN CONSTANT WHILE THE VOLUME OF WATER PUMPED INTO THE WATER INFILTRATION SYSTEM HAS BEEN REDUCED

The inflow of water into the cavern has been relatively constant, approximately 200- 300 l/min, since 1977. The variation of inflow from one day to another seems to be influenced by the variation in the gas pressure which may vary due to loading or unloading of gas. The gradient in the rock do not respond to this variation as quickly as the pressure drops or rise in the cavern.
Thus the inflow of water may vary up to 50% from one day to another while the pressure variation may be 0.05 MPa (10%).

The reduction of the volume of water pumped into the infiltration system has been reduced from approximately 450 l/min (1977) to 300 l/min (1987). Most of this reduction was observed to take place during the first 2-3 years of operation - the injected water volume has been nearly constant since 1981.

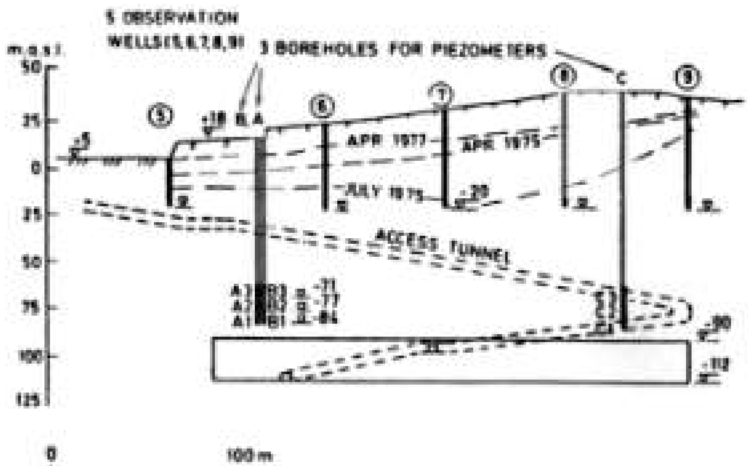

Fig. 4 Longitudinal section through cavern with piezometers and obser- vation wells

The main reason for this reduction may be a clogging effect in the boreholes due to impurities in the water. This effect is also observed at other projects where water infiltration systems are in use. The result of this clogging effect is a reduced permeability around the boreholes which again reduces the effectiveness of the infiltration system. This reduced effectiveness seems, however, to have stopped and the gradient from piezometer holes A and B are still very much on the safe side, while the gradients from piezometers in hole C and the watertable level seems not to be affected at all (see Fig. 6.)

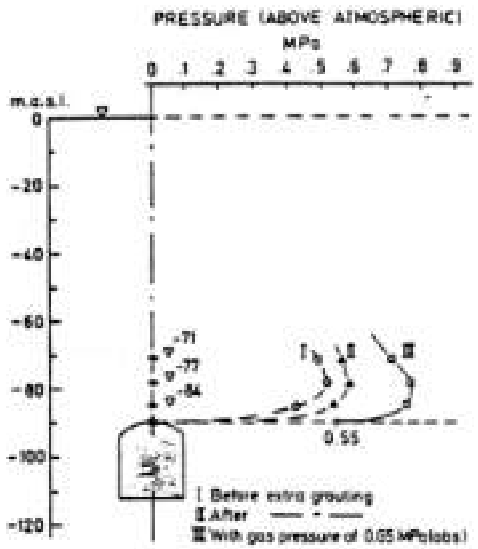

Fig. 5 Pressure distribution in borehole
I Before extra grouting
II After extra grouting
III With gas pressure of 0.65 MPa (abs.)

6 THE SALT CONTENT OF THE INLEAKING WATER SEEMS TO BE DEPENDENT ON THE GAS PRESSURE IN THE CAVERN AND THE ELEVATION OF LIQUEFIED PROPANE

Before the cavern was put into operation in 1977 several samples of inleaking water was analyzed on chloride content. The highest chloride concentration was found in a sample from the Frier Fjord end of the cavern, 520 ppm. This is approximately the same as the average concentration of the inleaking water in 1985.

A reduction of the gas pressure and a decrease in the liquefied propane level seems to increase the chloride content. The concentration may vary between 200 and 800 ppm chloride.

7 THE MAINTENANCE COSTS FOR THE WATER INFILTRATION SYSTEM ARE SMALL

The water infiltration system is connected to the fire extinguisher system for the Rafnes plant. This system has a water pressure of 0.65 MPa and is connected to a water tunnel which has an ample capacity for all needs of water to the plant.

The maintenance costs for the system are small. The pressure and volume of water are checked once per day and an alarm is set up for low pressure in the system. A reserve system will be connected if the water tunnel or other parts of the ordinary system fails. but this reserve system has not been used since the plant was opened in 1977.

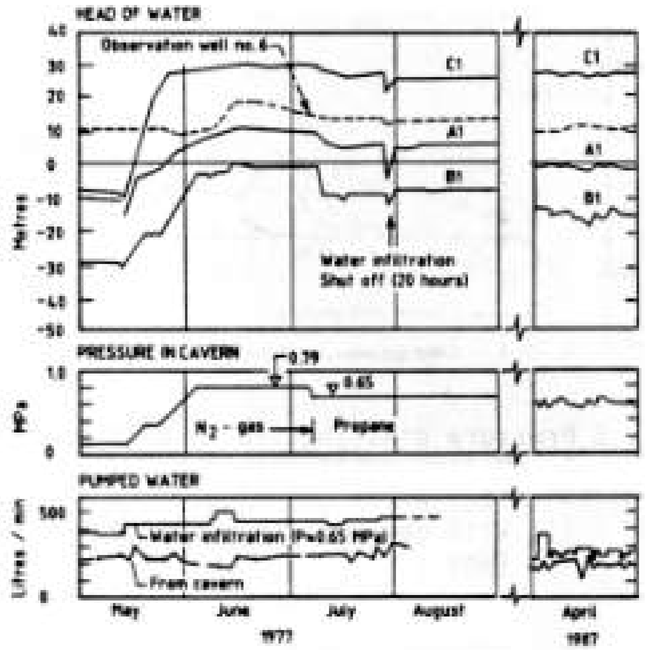

Fig. 6 Data from 1977 and 1987.

8 THE COSTS FOR PUMPING OUT THE INLEAKING WATER IS FIRST OF ALL RELATED TO OVERHAULING/REPLACING ONE PUMP EVERY 3 YEARS AND "STRIPPING" THE PROPANE/BUTANE CONTENT IN THE WATER

There are 2 pumps pumping approximately 200 l/min of inleaking water. An overhaul or replacement of these pumps take place on average every third year and the cost for this operation is NOK 4-600,000. Nitrogen is used to "strip" the content of propane/butane in the water. The cost for this process is stipulated to approximately NOK 1-200,000 per year.

The water also contains a small amount of methanol. At present this methanol is not taken out, but in the future this will probably be done using a biological process.

In 1981, having a low ground water level, a low level of liquefied propane in the cavern and high amount of butane giving a lower gas pressure, the chloride content (Cl^-) rose to above 900 ppm in the water phase. The chloride content in the propane/butane was in this short period so high that some of the equipment in the processing plant had to be replaced due to corrosion.

In conclusion the propane storage cavern at Rafnes has been operating well and the maintenance costs have been low.

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GROUND WATER MAINTENANCE AND LEAKAGE CONTROL DURING CONSTRUCTION OF UNLINED ROCK CAVERNS FOR PRESSURIZED GAS STORAGE, MONGSTAD

B.Nermoen & O. T.Blindheim
O.T .Blindheim, Norway

ABSTRACT: Ground water is used to balance the gas pressure in 3 unlined rock caverns in gneissic rocks, for LPG gas storage.

Before excavation a water curtain was established above and between the caverns to maintain the groundwater level during construction and operation.

Experience during excavation proved that the rock mass had a very varying permeability. The ground water was mostly connected to irregular pipe-shaped channels of a limited volume along some few weakness zones. One of the weakness zones intersected the entrance of the caverns and made a revision of the design necessary.

Pregrouting was used on basis of observations of inleakages from probe holes. The contour of the caverns were also grouted after excavation in order to improve build up of hydrostatic pressure in the joints as close to the caverns as possible.

1 INTRODUCTION

During upgrading of the Mongstad Oil Refinery in western Norway 3 unlined rock caverns for LPG storage have been constructed.

Statoil a.s is owner of the plant and main contractor was Astrup Høyer A/S, now Aker Contractor. Geotechnical pre-investigations for detailed localization of the caverns were done by Norconsult A/S and Viking Engineering AB. The pre-engineering civil and system design was done by Fuelstore, and the design criteria approved by the Directorate of fire- and explosion protection.

The detailed cavern design was done by the main contractor in co-operation with the owner. Ing. Chr. F. Grøner A/S was engaged by the contractor for specifications of concreting and the quality assurance program, and Dr.ing. O. T. Blindheim for the geological engineering follow-up during construction.

2 PRE-INVESTIGATIONS

2.1 Methods

The pre-investigations were done in several steps. Ordinary methods were used such as air-photo studies, surface geological mapping, core-drilling with water injection (Lugeon tests), and well drilling with short-time test pumping.

2.2 Main conslusions

The pre-investigtions concluded that the rock mass in the area consists of various moderately jointed gneisses with strike about E-W and dip 450-600 SW. A few SW-dipping weakness zones were detected, see fig. 1. These zones were expected to have a high rock frequency, and limited thicknesses of clay seems.

The groundwater level observed in the test holes was depending on the topography, and typical variations over the caverns were from 5 to 15 m above sea level. The rock mass was generally characterized of a low permability, but a more or less continuous groundwater magazin was expected.

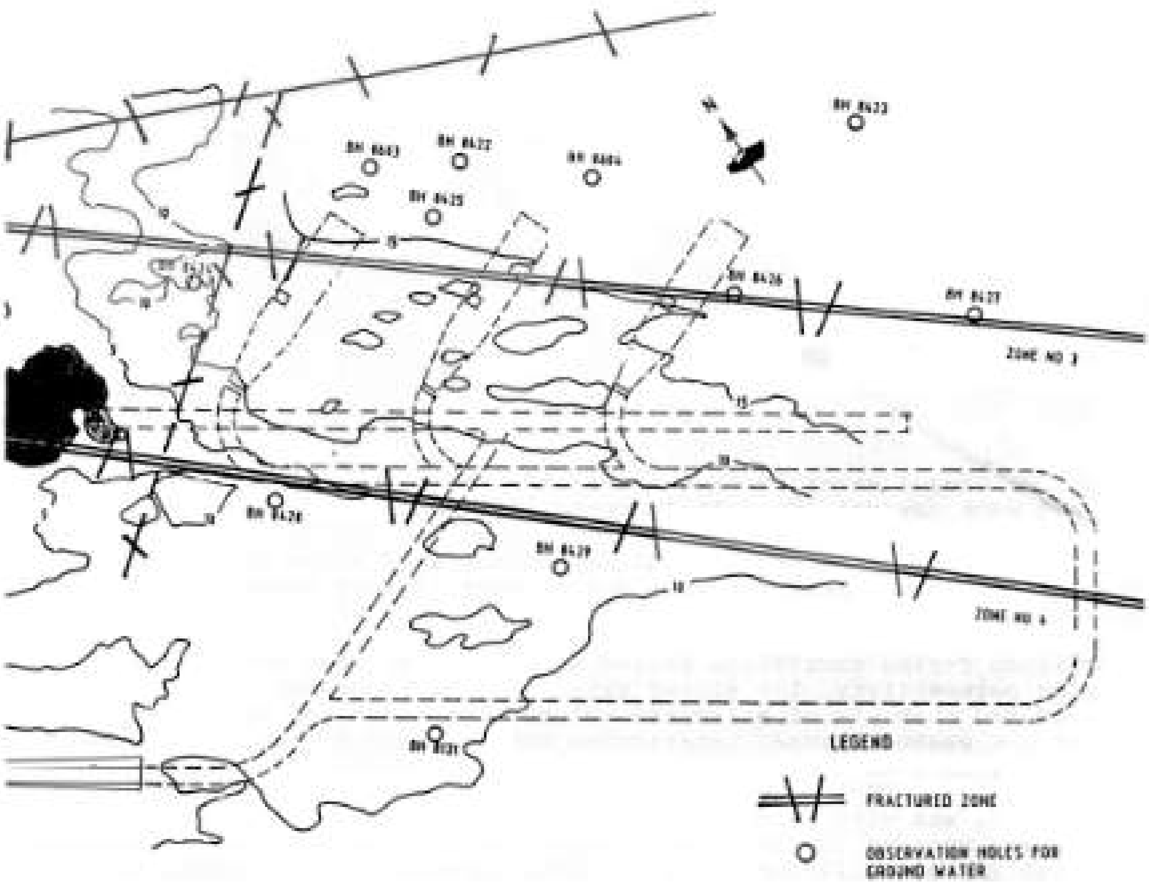

Fig. 1 Situation map from the cavern area showing the groundwater observation holes and the main fracture zones

3 DESIGN

3.1 Detailed localization

On basic of the pre-investigations, the chosen layout of the shaft top area, the design storage pressure and temperature of the different gases, the volume with the best suited rock mass properties was chosen for excavation of the caverns. The 3 caverns were located on different depths because of different storage pressure for each of the gas types. Fig. 2 shows a general layout of the plant.

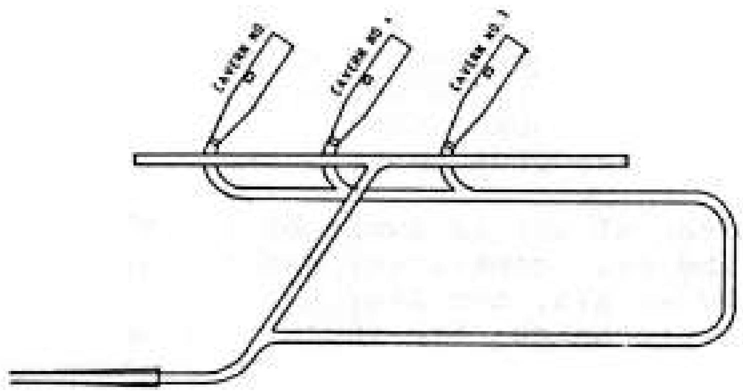

Fig. 2 General layout of the plant, with water curtain gallery above the caverns. Cavern 1 is deepest.

3.2 Principle of controlling the gas pressure and the ground water

The design principle was balancing the gas pressure in the caverns with the hydrostatic pressure from the ground water. A certain inleakage of ground water should prevent gas to enter joints in the rock mass. The rock material in itself is impermeable.

Because of expected problems with re-establishing of the pore pressure in a drained rock mass, the groundwater level during the construction was not to be allowed to drop lower than sea level. This corresponds to the gas pressure in the caverns plus a safety margin of 20 m water pressure.

In order to keep the groundwater level at elevation *to* or higher during construction and operation, a gallery at elevation -20, or about 40-55 m above the cavern roofs was planned. From this gallery a water curtain could be drilled.

Excavation of the lower part of the adit tunnel was therefore not allowed before the water curtain was in operation.

Inleakage of ground water in the entrance, the gallery and the caverns should also be minimized by pregrouting of the rock mass, to help avoidance of a drop in the groundwater level during the construction period.

3.3 Observation of ground water

For observation and monitoring of the groundwater level the boreholes from the preinvestigation and some additional percussive drillholes, a total number of 12, were monitored 1 or 3 times a week, depending on the stage of excavation and rock conditions at the tunnel face.

3.4 Design of the water curtain

The water curtain was made of 3 parallel horisontal drillholes above each cavern, and a vertical fan of 5 drillholes besides and between each cavern, see fig. 3. The drillholes were planned to be connected to the main fresh- water supply by hoses and grouting packers.

3.5 Closing the caverns

The caverns should be connected to the surface by vertical shafts, completely concreted after installation of pipes. After finishing all works in the caverns, they should be closed by short concrete bulkheads and the entrance filled up with water.

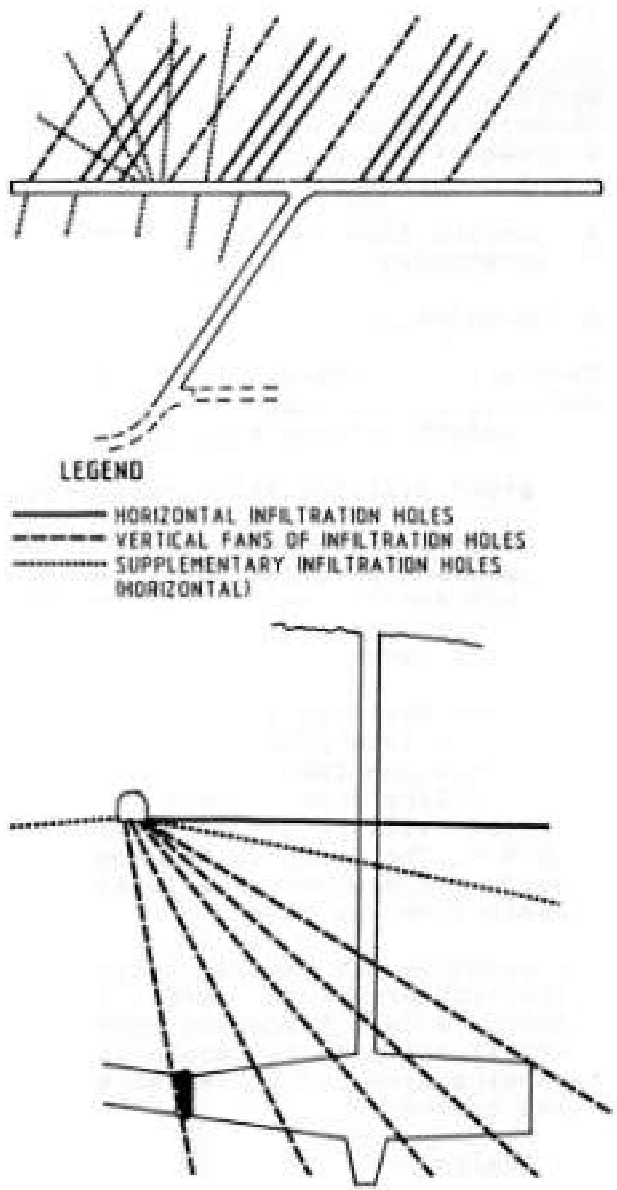

Fig. 3 Design of the water curtain system shown in plane view and vertical section. (Not in scale)

In addition to the wanted reduction of permeability near the contour, the inleakage of ground water in the finished caverns should if necessary be reduced by grouting, because of the cost of pumping.

4 CONSTRUCTION PROCEDURES AND TECHNIQUES

4.1 Excavation

Excavation of entrances and caverns was done by drilling and blasting, with smooth contour blasting.

4.2 Probe drilling and pregrouting

To reduce inleakage of ground water to a minimum pregrouting of the rock mass was decided on information from probe holes drilled ahead of the advancing faces.

The probe drilling was executed with three 30 m long percussive drill- holes from the face with the ordinary drilling jumbo. Every section of probe-drilling had an overlap of 4 to 8 m. The criteria for starting pregrouting was set at a leakage of 2 l/min from one hole.

The grouting was usually carried out using ordinary rapid cement. In the entrance a fast hardening cement (Cemsil) was used to reduce the hardening time and the total excavation cycles.

4.3 Shafts

The shafts were at first raisedrilled, but a revised design of the technical installations ordered wider shafts. They were therefore enlarged by drilling and blasting. This operation was executed from a drilling platform lowered down with a winch.

To keep the ground water out of the shafts, the rock mass was pregrouted from long parallel holes drilled in a circular pattern. The grouting was done in two or three steps depending on the results from water injection tests after each round of grouting.

5 OBSERVATIONS AND EXPERIENCES FROM FIRST PART OF THE CONSTRUCTION WORKS

5.1 Access tunnel

The first observations from the access tunnel confirmed in general the very stable rock conditions and a low frequency of fissures and waterbearing joints. A fractured zone (No 4) was crossed in the gallery adit, with appearance like a schistous part of the rock mass. The center of the zone had small openings with well grown calcite crystals in irregular pipe shaped subvertical channels.

The zone also contained clay or calcite on joints and fissures, and only limited quantities of water. Most of this water was drained out in some few hours, and some of the observation holes showed a not wanted drop in water level. The zone was grouted with 15 t cement and reinforced with rock-bolts and shotcrete.

5.2 Pregrouting of shafts

Later on, during the drilling of the grouting holes in the shaft areas drilling problems were experienced when crossing a similar fractured zone (No 3). The grouting started from NW in the area for shaft No 1. After some minutes grey coloured water flowed out of the low level testhole BH 8424. This hole was plugged before the grouting continued. The ground- water level in the other observation holes was then controlled several times a day. After more grouting at shaft No 1, the grouting holes for shaft No 2 and No 3 overflowed. A rise in the groundwater level was registrated in test hole BH 8428.

Totally 63 t cement were used for grouting in shaft area No 1, 28 t in area No 2 and 8 t in area No 3

5.3 Preliminary experiences with rock conditions and ground water

So far the experiences and observations indicated these conclusion about the prevailing conditions in the area:

- the rock mass did not have a continuous and well corresponding groundwater magazine
- the ground water followed the main fractured zones parallel the foliation, and was of limited quantity
- most of the ground water was connected to pipe-shaped channels, that seldom seemed to be continuous for more than some few metres
- fractured zone No 3 was unexpectedly the most permeable one and would cross the caverns at their entrance. The preinvestigations had concluded that zone No 4 was most permeable and the one that should be avoided
- response in BH 8428 could indicate at least some communication between zone No 3 and 4

5.4 Establishing of the water curtain

Next step was the drilling of water infiltration holes from the gallery. This work was delayed in time to avoid plugging from the pregrouting of the shaft areas. The total length of infiltration holes was about 2200 m, the longest hole 105 m, all drilled with tophammer drilling equipment and guide-bits. The holes were not logged after drilling.

The infiltration holes were set under water pressure one by one as soon as they were finished, to avoid drainage of the rock mass. When the infiltration holes were finished, a higher and more stable groundwater level in the area above the caverns as achieved. It was obvious that most of the infiltrated water flowed out at the surface to some small creeks and in the lower parts of the terrain to the NW.

6 FURTHER EXCAVATION – REVISION OF DESIGN

6.1 The access tunnel

Further down the access tunnel only a minor leakage occurred when fracture zone No 4 was met. This leakage was grouted at the tunnel-face.

6.2 Entrance of the caverns

In the entrance to Cavern No 3 fractured zone No 3 was identified by probe drilling just outside the planned location for the bulkhead. The zone was extensively pregrouted until the probe drilling gave acceptable results.

The same procedure was used for the entrance to Cavern No 2, but the pregrouting was not finished with the same success as for Cavern No 3. After excavation there were still in-leakages from some small, but open pipe-shaped channels near the centre of the zone.

The water-curtain had still enough capacity to keep the groundwater level at the same level as before the first crossing of zone No 3. In general zone No 3 proved to be very like zone No 4 as crossed in the entrance to the gallery, but zone No 3 was somewhat wider and contained more water, just as expected after the pregrouting of the shaft areas.

Zone No 3 crossed the entrance to Cavern No 2 in the centre of the planned bulkhead. This lead to an extent ion of the entrance in order to place the bulkhead inside the zone.

The same procedure was used for Cavern No 1.

Because of the locked position of the shafts and the possibility of another fractured zone further to the NE of the inner end of the caverns, it was not possible to move the caverns. Instead the caverns were made shorter and wider.

To reduce the possibility of leakage through the jointed rock around the bulkheads, they were made 1 or 2 m longer than planned. Extra grouting in the rock mass around the bulkhead and in the contact between concrete and rock were also recommended and performed.

6.3 Groundwater control

During the excavation of the entrance to Cavern No 1 the groundwater level dropped several meters. Primarily this was connected to the leakage into the tunnel and possibly grouting of some of the water infiltration holes, through the permeable zone No. 3.

The actions taken to re-establish the wanted high groundwater level were more grouting, of zone No 3 outside cavern No 1 and 2, to reduce the leakage. Then a system of supplementary infiltration holes were drilled above Caverns No 1 and 2, and also a system of infiltration holes penetrated zone No 4. The latter because of the response in one observation hole penetrating zone No 4, while the shaft that crossed zone No 3 was grouted. All holes were drilled from the gallery and like the others connected to the freshwater supply system. The groundwater level was then restored.

6.4 Changed design criteria

At this time the owner decided to increase the storing temperature of the gas to avoid the possibility of accumulation of hydrates. The gas pressure had to be increased and the safety margin in the construction concept reduced. This led to an intensive grouting of all leakages in the roof and walls of Caverns No 1 and 2 to permit a build-up of hydrostatic water pressure in joints as close to the contour as possible, and restoring the 20 m water head safety margin.

This grouting was done with micro cement and polyurethane.

7 GENERAL EXPERIENCES

7.1 Re-establishing and maintenance of ground water

From experience with operation of air cushion chambers at hydropower plants, it does not seem to be any problems with re-establishing the groundwater level in a drained rock mass with intersecting joints. As an example, groundwater level has been re-established several times after drainage of the rock mass during excavation of Kvildal Hydropower plant. However, at Mongstad the owner did not want to take the risk connected to draining and re-establishing ground water in the varying gneisses.

The experiences from the construction show that it is possible to maintain a ground water table above rock caverns during construction by water infiltration and control by a net of observation holes, even if the permability in the rock mass varies much. The minor drops in groundwater level in the construction period were re-established seemingly without problems.

A practical problem with the observation holes was damages caused by the intense construction works on the ground surface. Such conflicts have to be reduced by protecting the holes with heavy concrete blocks and lot of red paint!

The water supply to the infiltration system was discontinued a couple of times due to pipe breakage in the construction area, or by misoperation of the valves.

The new refinery will be put in operation in the late spring 1989 The final conclusions about gas tightness can not be drawn until then.

8 ACKNOWLEDGEMENT

The authors want to thank Statoil a.s and Astrup Høyer A/S for cooperation and the permission to publish the experiences front the construction period.

THE WATER CURTAIN – A SUCCESSFUL MEANS OF PREVENTING GAS LEAKAGE FROM HIGH-PRESSURE, UNLINED ROCK CAVERNS

H. Kjørholt and E. Broch

Abstract: In Norway, high-pressure air is stored in ten unlined rock caverns, called air cushion surge chambers. These surge chambers are characterized by pressures up to 7.7 MPa and volumes up to 110,000 m^3. This paper describes the successful use of water curtains to prevent air leakage from three such caverns, even when the storage pressure head is twice the thickness of the overburden.

Résumé: En Norvège, de l'air sous haute pression est stocké dans dix caverns non revêtues, appelées chambres de compensation par coussin d'air. Ces chambres de compensation se caractérisent par des pressions pouvant atteindre 7,7 MPa et des volume allant jusqu'à 110,000 m^3. Le texte décrit les bons résultats obtenus avec l'utilisation de rideaux d'eau pour éviter les fuites d'air dans trois de ces cavernes, même lorsque la différence de pression de ockage atteint le double de la couverture.

1 INTRODUCTION

Thus far, no hard rock storage has been developed specifically for storage of natural gas or as a CAES (Compressed Air Energy Storage), although much work has been carried out to establish the economic and technical basis for such a storage. The main concerns with regard to the acceptance of the hard rock concept are prevention of gas leakage through the fractured rock mass and storage economy.

At present, hydrocarbon gases such as propane and butane are routinely stored in hard rock caverns, but at a much lower pressure than will be required for natural gas storages and CAES. The only large-scale, high-pressure storage experience in hard rock caverns described in the literature comes from ten Norwegian air cushion surge chambers.

An air cushion surge chamber is a pressurized air-filled cavern, the function of which is to dampen transients in the headrace tunnel of hydro power plants (Goodall et al. 1988). Figure 1 shows the design principle for a hydro power plant equipped with an air cushion surge chamber.

The surge chambers are hydraulically connected to the headrace tunnel by a short (<100-m-long) tunnel. The pressure in an air cushion is consequently dictated by the reservoir elevation. The surge chamber has a water bed below the air cushion. Compressors are used to fill and maintain the air cushion.

Figure 2 provides an overview of pressure and volumes for the surge chambers in chronological order. The first air cushion surge chamber was constructed at the Driva power plant, and commissioned in 1973 (Rathe 1975); the last chamber began operating at Torpa power plant in 1989.

Figure 2 shows that as many as six of the air cushions have pressures that exceed 4 MPa. The highest pressure is reached at Tafjord, where the maximum operating pressure is 7.7 MPa. The cavern volumes are generally less than 20,000 m^3; an exception is the Kvilldal surge chamber, which has a volume of 110,000 m^3.

Three of the surge chambers (at Kvilldal, Tafjord and Torpa) are equipped with so-called water curtains to restrict the air leakage through the rock. These water curtains are arrays of boreholes, with typical hole spacing of 5 to 20 m, drilled above the rock chamber. Water at a pressure slightly higher than the air pressure in the cavern is fed into the holes. Thus, an artificially high groundwater pressure is established around the cavern. This high pressure prevents air from leaking through the surrounding rock mass.

This paper describes the design and construction of such water curtains, and also discusses experience from ordinary operation and special tests performed at the three surge chamber sites.

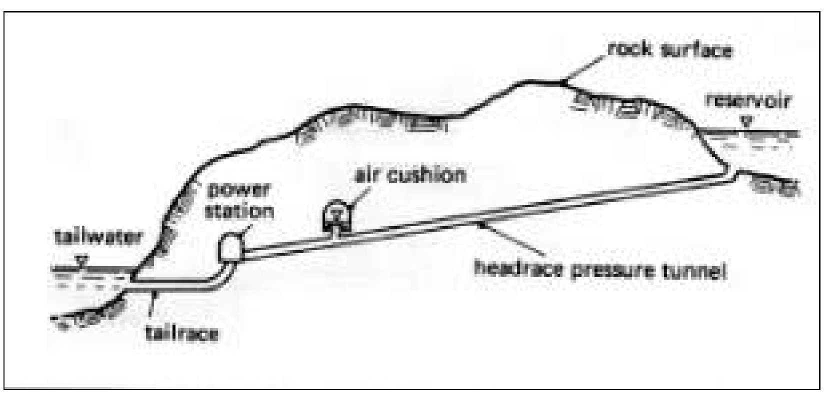

Fig. 1 Concept of a power plant with an air cushion surge chamber.

2 METHODS TO LIMIT OR ELIMINATE GAS LEAKAGE FROM A GAS STORAGE

Figure 3 suggests different methods for limiting or eliminating leakage from an underground gas storage. These methods are based on two main principles-permeability control and groundwater control.

Permeability control means that the leakage is eliminated or kept at an acceptable level by ensuring that the rock mass around the storage has a sufficiently low permeability.

No general permeability-controlling technique for non-leaking storages is currently available for full-scale commercial use. The most developed alternative is the steel-lined storage, which is more or less ready for prototype testing. The authors believe that the frozen storage concept will prove to be a realistic and favourable alternative. However, so far this concept has suffered from low research activity. It is further believed that cold storages equipped with a water curtain outside the frozen zone could be considered, in order to provide a double barrier against leakage. Permeability control is, how- ever, beyond the scope of this paper.

The principle of groundwater control is based on the fact that the presence of groundwater reduces gas leakage. The leakage reduction, or degree of ground- water control, depends on the magnitude of the groundwater pressure as compared to the storage pressure.

Leakage prevention by groundwater control offers two possibilities: control based on either (1) the natural groundwater pressure; or (2) groundwater pressure that is artificially enhanced by use of a water curtain. The sealing effect of the curtain is conditional, depending on a somewhat higher water pressure in the boreholes than in the storage. In this way, an inward hydraulic gradient, high enough to prevent outward gas migration, is established. The water curtain should cover at least the crown of the storage. Under extreme conditions, a water curtain that completely surrounds the storage may be necessary.

To completely avoid leakage by groundwater control, the groundwater pressure in all potential leakage paths, directed upward from the storage, must exceed the storage pressure over at least a small (infinitesimal) distance.

Complete gas tightness based on natural groundwater is, in general, not an economical alternative for high-pressure storages because of the requirement that the allowable storage pressure must be low in relation to the thickness of the overburden. Therefore, a water curtain should be used to increase the groundwater pressure artificially. This type of arrangement will allow a higher ratio between storage pressure and depth, and will increase the operational flexibility. Experience shows that water curtains have been used successfully to avoid gas leakage at storages with pressure up to twice the hydrostatic groundwater head.

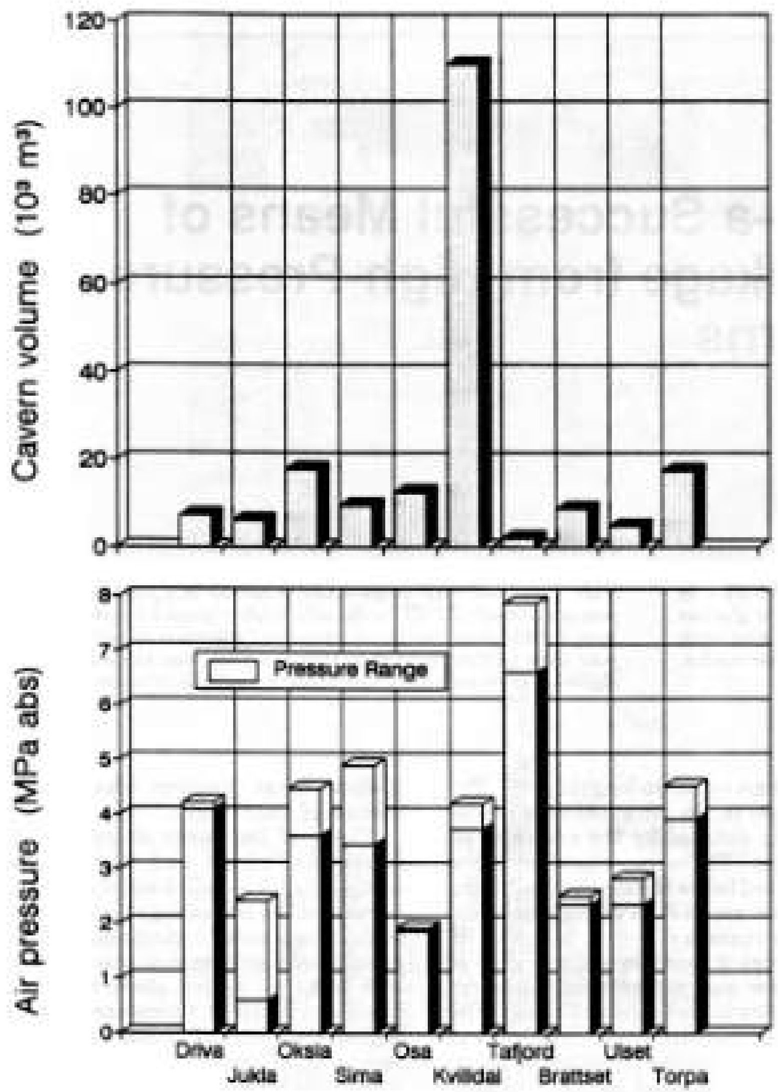

Fig. 2 Volume and pressure of the air cushion surge chamber.

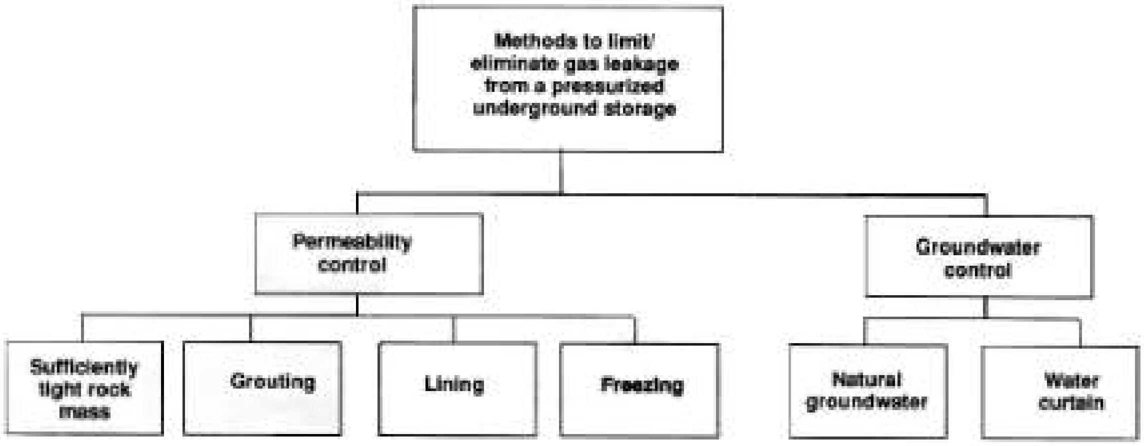

Fig. 3 Methods to limit or eliminate gas leakage from a pressurized underground storage.

3 EXPERIENCE FROM THREE AIR CUSHION SURGE CHAMBERS WITH WATER CURTAINS

Water curtains have been installed at three air cushion surge chambers: Kvilldal, Torpa and Tafjord. Only at Torpa was the water curtain included in the original design. The two other water curtains were constructed as a consequence of unacceptable air leak- ages. The geometry of the three caverns and water curtains is provided in Figures 4, 7 and 10.

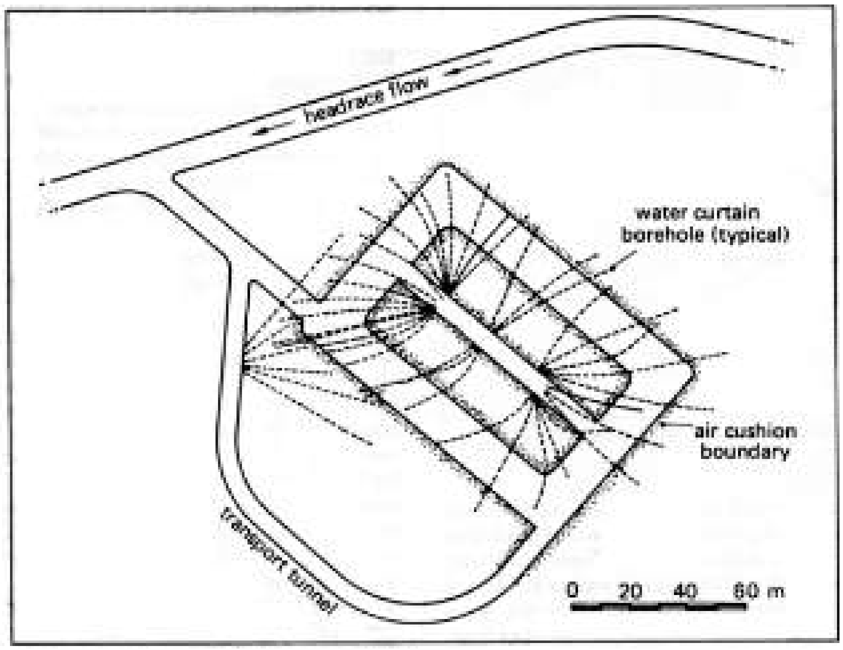

Fig. 4 Plan of Kvilldal air cushion surge chamber with water curtain.

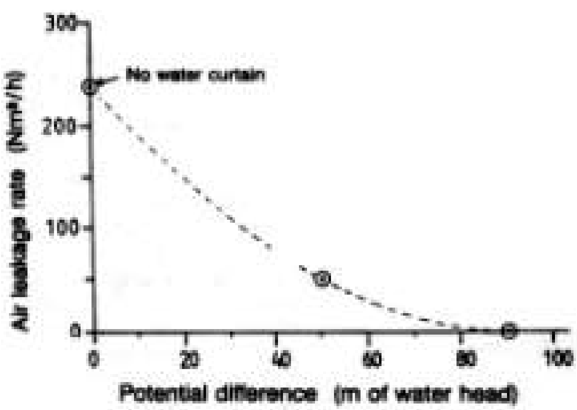

Fig. 5 Air leakage at Kvilldal air cushion surge chamber as a function of the difference between the water curtain potential and the potential at the cavern roof.

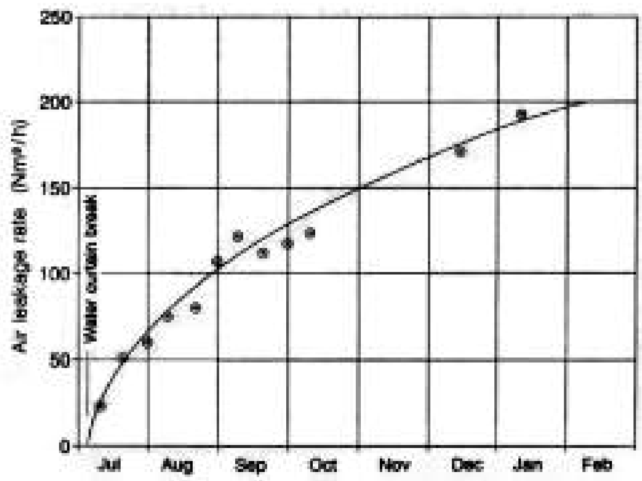

Fig. 6 Air leakage development at Kvilldal air cushion surge chamber after water curtain break.

3.1 Kvilldal

The Kvilldal air cushion operates at a pressure around 4 MPa, with a minimum rock overburden of 520 m in a steeply sloping terrain. The cavern was originally constructed without a water curtain, but experienced an air leakage of 240 Nm^3/h after commissioning in 1981. In an attempt to reduce (but not necessarily eliminate) this leakage, an overlying water curtain, consisting of 47 percussion-drilled boreholes (diameter of 51 mm), was installed in 1983. The geometry of the water curtain is shown in Figure 4. As can be seen, the water curtain is very irregular, with borehole spacing of up to more than 20 m in certain areas.

This water curtain has completely eliminated the air leakage through the rock. Tests have resulted in the relationship between water curtain overpressure and air leakage shown in Figure 5. As indicated in the figure, any leakage through the rock is eliminated at Kvilldal if the potential in the water curtain exceeds the potential in the air cushion (measured at cavern roof level) by 90 m of water head.

In 1986, a water curtain supply pipe broke and put the water curtain at Kvilldal out of commission. As can be seen in Figure 6, this action resulted in an increasing air leakage, approaching the level experienced before the water curtain was installed (240 Nm^3/h). However, the leakage developed quite slowly; in two months, only 50% of this initial value had been reached. After the water curtain was repaired in 1987, the leakage through the rock mass was eliminated again.

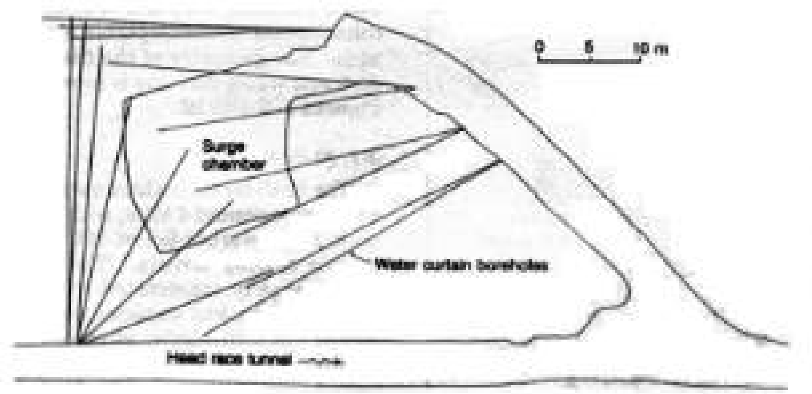

Fig. 7 Plan of the Tafjord air cushion surge chamber with water curtain.

3.2 Tafjord

The air cushion surge chamber at Tafjord (Fig. 7) was constructed in 1982. The air cushion operates at a pressure between 6.5 and 7.7 MPa, while the minimum rock overburden is only 440 m (steeply sloping terrain).

Like the Kvilldal facility , the Tafjord surge chamber was originally constructed without a water curtain. Al- though the leakage at this site was somewhat less than that at Kvilldal, the compressors installed to maintain the air cushion did not have sufficient capacity. The surge chamber at Tafjord was therefore out of operation from 1982 to 1990 (i.e., the cavern was completely filled with water). Attempts to grout the surrounding rock did not improve the leakage condition.

In 1990, a water curtain was installed at Tafjord, partly as a research project. The curtain consists of 16 core drilled holes (diameter of 56 mm), which cover both the roof of the cavern and the upper part of the cavern walls.

Results from a water curtain test at the Tafjord air cushion are shown in Figure 8. The upper curve represents the water curtain overpressure (difference between the water curtain potential and the potential in the air cushion at cavern roof level). The lower curve represents the air mass in the air cushion. The air cushion pressure was 7.6 MPa during this test.

At the start of the test, on February 15, the potential difference between the water curtain and the air cushion was 32 m of water head. Under this condition, no change in air mass was recorded, which means that no air was leaking from the air cushion. On February 18; the water curtain pump was stopped and remained shut off for two days. The air leakage (reduction in air mass) started immediately and reached a constant value (125 Nm^3/h) within a few hours. When the water curtain was restarted, the air leakage simultaneously disappeared. A subsequent reduction in the potential difference to 2 m of water head produced a new air leakage of approximately 60 Nm^3/h. A subsequent increase in the potential difference to 8 m reduced the leakage to less than 5 Nm^3/h.

Based on a number of such tests, it has been possible to draw the two leakage curves shown in Figure 9. The upper curve shows the air leakage at Tafjord as a function of the potential difference for an air cushion pressure of 7.6MPa. The lower curve shows the same data for a pressure of 6.5 MPa. Two effects should be noted:

1. The necessary potential differ- ence to avoid leakage increases with increasing storage pressure.
2. The leakage is significantly higher for a storage pressure of 7.6 MPa than for 6.5 MPa (as expected).

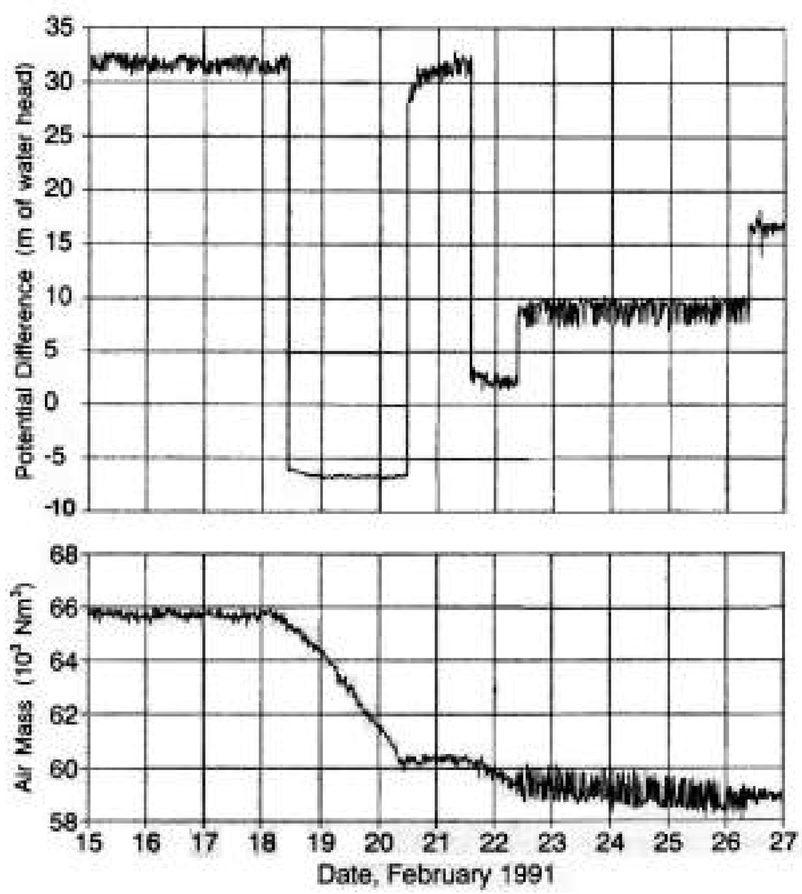

Fig. 8 Air leakage at the Tafjord air cushion surge chamber in response to changes in potential difference between water curtain and air cushion.

3.3 Torpa

At Torpa, the maximum air cushion pressure is 4.4 MPa, while the cavern overburden is only 220 m (see Fig. 10). The pressure situation for the water curtain is even more extreme.

The routine operational pressure in the water curtain is 4.6 MPa, with a minimum overburden thickness of only 207 m. Torpa is the only air cushion surge chamber where a water curtain was included in the original design. The water curtain consists of 36 percussion-drilled boreholes (diameter of 64 mm), drilled from an excavated gallery 10 m above the cavern roof. Access to the gallery is through a vertical shaft extending from the cavern roof. The gallery is hydraulically separated from the surge chamber by a concrete plug in the shaft.

Tests carried out at Torpa show that the leakage without the water curtain in operation becomes 400 Nm^3/h, and that no leakage is registered as soon as the potential in the water curtain exceeds the potential in the upper part of the air cushion by 20 m of water head.

In all three water curtains discussed herein, the water consumption is somewhat less than 1.0 l/s. The water is untreated, and is of drinking quality.

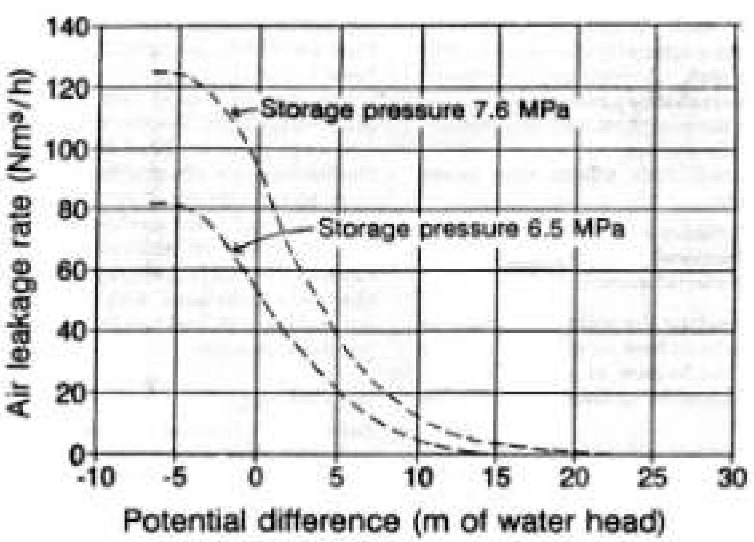

Fig. 9 Air leakage at Tafjord air cushion surge chamber as a function of the potential difference between the water curtain and the air cushion.

4 WATER CURTAIN DESIGN

A water curtain is characterized by:

- The spacing of the boreholes.
- The distance between the boreholes and the storage cavern.
- The extent of the water curtain.
- The pressure (or potential) in the water curtain relative to the storage pressure (potential).

The main factors governing the design of a water curtain are:

- The storage pressure versus groundwater pressure and overburden.
- The storage geometry.
- The tightness requirement.

Other factors that should be considered are:

- Rock jointing.
- Access for drilling the water curtain boreholes.
- The upper pressure limit for the water curtain due to the risk of hydraulic jacking.
- Rock stress situation near the storage.
- Economic considerations regarding construction and operation.
- Any restrictions on water consumption or inflow to the storage.
- Maximum borehole length.
- Expected borehole deviation as a function of borehole length.

Water curtain design is not an exact discipline. Even though the criterion of complete tightness is a simple one, its practical application to a fractured rock mass involves difficulties related to the irregular nature of the rock fractures. Therefore, practical design should be based on a combination of experience from existing storages, theoretical calculations, and hydraulic testing at the site.

The following general statements give an idea of the typical magnitude of the key design parameters:

- The distance between the storage and the water curtain should not be less than 10 m for small caverns, increasing to 30 m for large caverns.
- The practical boreholes spacing is in the range of 5 m to 20 m.
- The water curtain should at least cover the roof of the cavern. As the ratio between cavern pressure (in m of water head) and overburden (in m) approaches 2.0, it is also necessary to cover the sides of the storage, as is the case at Tafjord and Torpa (see Fig. 7 and Fig. 10).
- The necessary pressure in a properly designed water curtain normally need not exceed the storage pressure by more than 0.5 Mpa.

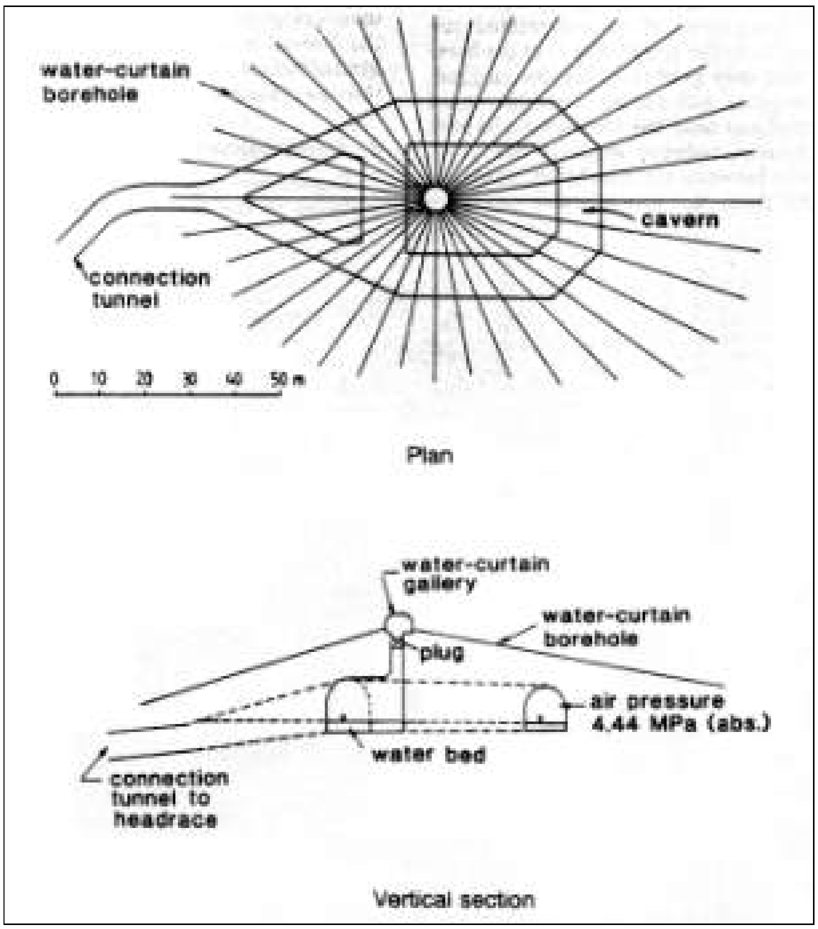

Fig. 10 Geometry of the Torpa air cushion surge chamber with water curtain.

More detailed guidelines for design and construction of water curtains are presented in a doctoral thesis by Kjørholt (1991), and will also be presented in Kjørholt and Broch (1992).

5 SAFETY ASPECTS

The typical safety concern for a gas storage facility is related to leakages that can cause financial losses, fire, explosion; or which may be harmful to people and the environment in other ways.

The possible ways that leakage may occur at a gas storage facility isolated by a water curtain can be divided into three categories:

1. Improper water curtain design or construction.
2. Long-term effects.
3. Operational problems.

Improper water curtain design or construction may cause minor leakages between water curtain holes, or outside the extension of the boreholes. Thorough hydraulic testing of the water curtain during construction, in addition to theoretical analyses, will minimize this risk. Still, if a leakage is experienced after commissioning, an increase in the water curtain pressure or a reduction in the storage pressure may be used to eliminate the leakage.

Long-term effects are related primarily to the possibility that the bore- holes may gradually become clogged. Clogging will result in an increased head loss near the borehole walls and, thereby, reduced groundwater pres- sure between the boreholes. If this is the case, the groundwater pressure will approach its critical value over time, and eventually the storage will start to leak. A clogging phenomena can be revealed by a reduction in water curtain consumption and reduced in-flow to the storage.

At least three effects may cause clogging:

1. Particles in the water supply.
2. Chemical precipitation.
3. Bacterial growth.

By treating the water, it should be possible to reduce or eliminate these effects; Andersson et al. (1989) discusses possible actions that can be taken.

If a critical reduction in water curtain efficiency occurs, immediate action should be taken to increase the water curtain pressure or to restrict the maximum storage pressure.

In the case of bacterial clogging, it has been found possible to re-establish the water curtain by high-pressure flushing of the individual holes (Barbo and Danielsen 1980).

Operational problems, the third possibility for a water curtain failure, include all possible problems in keeping the water curtain pressure at the desired level. Typical problems in this category are insufficient supply of water or power , and failure in pumps, pipelines and monitoring systems. It is believed that the desired level of safety against this kind of failure can be obtained through the use of back-up systems.

6 CONCLUSIONS

Experience from the use of water curtains at the three Norwegian air storages discussed herein, at pressures from 4 to 8 MPa, is encouraging. It has been found that a properly designed water curtain totally eliminates any gas leakage from the storage, even for a storage pressure head that is twice the thickness of the overburden.

A water curtain may provide not only a cost-effective method to restrict gas leakage from unlined hard rock caverns; currently it also appears to be the only practical way of totally preventing gas leakage from a high- pressure storage.

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GEOTECHNICAL DESIGN OF AIR CUSHION SURGE CHAMBERS

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ABSTRACT: An air cushion surge chamber is an alternative to the traditional open surge shaft in hydro power plants, and has been used in Norway since 1973. The surge chambers have proven to satisfy the hydraulic demands, and have also shown to constitute an economic alternative that gives substantial freedom in the lay-out of the tunnel system, and the siting of the plant. Air leakage prevention is the major challenge when designing and constructing an air cushion. The paper shows how it is possible to handle this and other geotechnical aspects in an efficient way.

1 INTRODUCTION

Air cushion surge chambers are used as an economic alternative to the traditional open surge shaft for damping of headrace tunnel transients from changes in powerplant loading. As illustrated in Figure 1, the air cushion surge chamber is a rock cavern excavated adjacent to the headrace tunnel, in which an air pocket is trapped. The surge chamber is hydraulically connected to the headrace tunnel by a short (< 100 m) tunnel.

The air cushion concept was originally introduced to improve the economy of a hydro-powerplant where a traditional open surge shaft would be an expensive solution due to topographical reasons (Rathe 1975). The air cushion solution also gives substantial freedom in the lay-out of the tunnel system, and the siting of the plant. It is no longer necessary to maintain shallow, nearly horizontal, headrace tunnels for surge shaft economy (Figure 2a). Schemes which have used air cushions have tended to slope the headrace tunnel directly from the reservoir towards the power station as indicated in Figures 1 and 2b.

Air cushions are also favoured where the hydrautic head of the headrace is above ground surface. In such cases, construction of an open surge shaft would require erection of a surge tower, which may be expensive and environmentally undesirable in comparison with an air cushion.

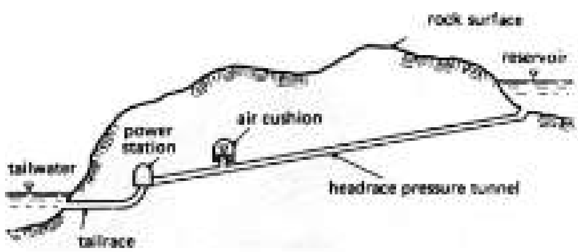

Fig. 1 Concept of powerplant with air cushion surge chamber

2 HYDRAULIC REQUIREMENTS

The hydraulic design of air cushions follows the same principles as design of traditional open surge shafts. Pressure surges in an open surge shaft system is according to the changes in water level in the shaft. In a system with an air cushion, pressure surges gives compression and expansion of the cushion according to ideal gas law. One should note that for normal surge periods (periods less than 5 minutes) the air cushion responds adiabatically.

As the hydraulic design of a surge shaft is a question of finding the necessary water surface area in the shaft, the necessary air volume is the key factor for an air cushion. Traditional Norwegian design practice for high head plants, that allows pressure surges up to 10 to 15% above static, is also adopted for the air cushion sites.

To avoid air-escape from the surge chambers, they are located a few meters above the roof of the near-by headrace tunnel as shown in Figure 3. The volume of the water bed should be such that there is a high degree of safety against air escape from the chamber during unfavourable combinations of down- surge and surface wave action in the chamber. However, a major blow-out may be caused by possible failure of control equipment, improper gate operation or accidents (upstream blocking of the headrace tunnel). The mechanisms of such an event should be studied, and measures should be taken to ensure that the consequences of a possible blow-out would be tolerable. No blow-out has ever happened in Norwegian air cushion surge chambers.

Normally the total surge chamber volume needed is 50% higher than the necessary air volume. The shape of a surge chamber is not a crucial point, except that the water bed response to surges may be problematic for very long caverns. Long caverns can be avoided by for instance giving the cavern a ring- shape. More details about the hydraulic design of air cushions are provided in Goodall et al (1988).

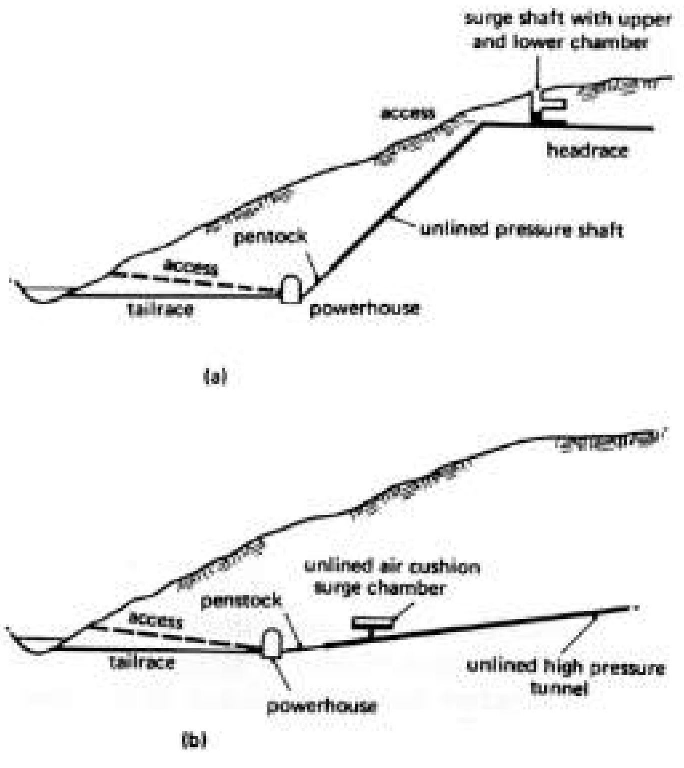

Fig. 2 Comparison of layouts for underground plant using: (a) open surge shaft; and, (b) closed air cushion surge chamber.

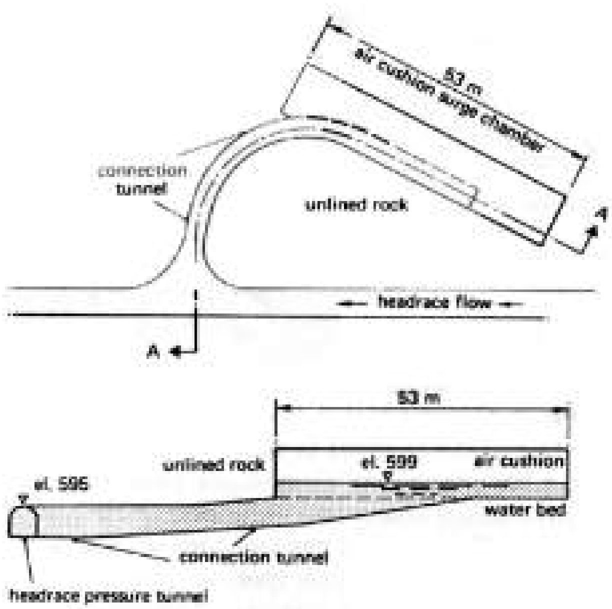

Fig. 3 Plan and profile of the Ulseth air cushion surge chamber.

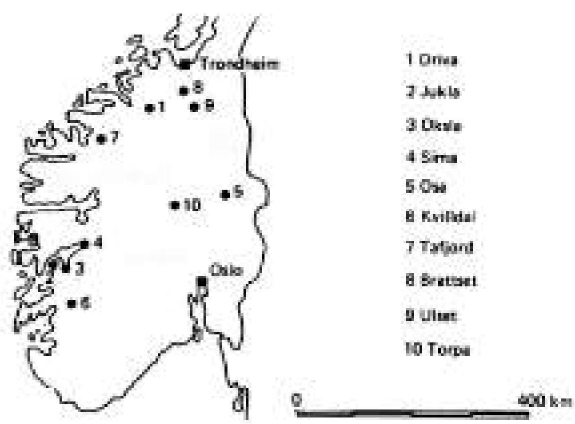

Fig. 4 Location of powerplants with air cushion surge chambers.

3 GENERAL FEATURES AND LAYOUT OF EXISTING AIR CUSHION SURGE CHAMBERS

A total of ten air cushions have been commissioned to date, all of them situated in southern Norway, as shown in Figure 4. General features of the air cushion sites are listed in Table 1. The diagrams in Figure 5 show the cavern volumes and internal pressures.

The first air cushion was commissioned at the Driva power plant in 1973, the latest one at the Torpa plant in 1989. As indicated in Table 1, air cushions have been used for power plants with capacity from less than 50 MW to more than 1200 MW.

An air cushion must be located within a limited distance from the turbine due to hydraulic reasons. In practice, distances up to more than 1000 m are found acceptable, at least at some of the sites.

Figure 6 shows two different layouts of the tunnel system in the area from the surge chamber to the power station. Usually the surge chamber consists of one single cavern, but doughnut shaped caverns around a centre pillar have been used at the Kvilldal and Torpa sites (see Figures 7 and 9). The vertical cross-section of the caverns ranges from approximately 90 to 370 m^2.

The ratio between maximum air cushion head and the minimum rock cover varies extensively from one site to an other. At the first air cushion, Driva, the minimum overburden is twice the air cushion head (in m of water column). At the most extreme site, Torpa, the overburden is only half the air cushion head.

The cavern volumes are generally less than 20,000 m^3, except for the Kvilldal surge chamber that has a volume of 110,000 m^3. As many as six of the air cushions have pressures exceeding 4 MPa. The highest pressure is reached at Tafjord where the maximum operating pressure is 7.7 MPa, equalizing a water head of 780 m .The air cushion itself occupies typically from 40 to 80% of the cavern volume, which corresponds to a water bed thickness in the surge chamber of 2 to 5 m.

Table 2 contains information about the rock type at the air cushion sites. Eight of the sites are located in various types of gneisses and granites, and the other two in phyllite and meta siltstone respectively. All caverns are essentially unlined, although some rock reinforcement, mainly in the form of rock bolts and shotcrete, have been used at a few sites.

Each air cushion is connected to one or more compressors which are located in the access tunnels (Figure 6), or as at one site (Osa), at the ground surface above the air cushion. A system of pipes and cables connects the air cushion to the compressor(s) and monitoring equipment. The compressors serve two purposes: First, to establish the air cushion before commissioning of the plant. Second, to compensate for air loss during operation.

A minimum and maximum air cushion volume is defined as a part of the hydraulic design. The air cushion volume is monitored by measuring the water bed level. All air cushions are equipped with at least two separate water level monitoring devices to safeguard against possible instrument malfunction. Air cushion pressure and temperature are measured directly only at a couple of the sites. The static air cushion pressure can be computed from the height difference between the reservoir and the cavern. The temperature is found to vary only by approximately 5° C on a seasonal base due to changes in water bed temperature (which reflects seasonal temperature fluctuations in the reservoir). More details about air cushion monitoring can be found in Goodall et al (1988) and Kjørholt (1991).

Site	**Commissioning date**	**Power-plant capacity (MW)**	**Distance to Turbine (m)**	**Conect. tunnel length (m)**	**Vertical cavern cross-sect. (Nm^3/h)**	**Installed compressor capacity (Nm^3/h)**	**Ratio between max. air cushion head and min. rock cover (m/m)**
Driva	1973	140	1300	20	111	425	0,5
Jukla	1974	35	680	40	129	180	0,7
Oksla	1980	206	350	60	235	290	1,0
Sima	1980	500	1300	70	173	270	1,1
Osa	1981	90	1050	80	176	2320	1,3
Kvilldal	1981	1240	600	70	260-370	500	0,8
Tafjord	1982	82	150	50	130	260	1,8
Brattset	1982	80	400	25	89	700	1,6
Ulset	1985	37	360	40	92	360	1,1
Torpa	1989	150	350	70	95	940	2,0

Table 1. Features of air cushion sites.

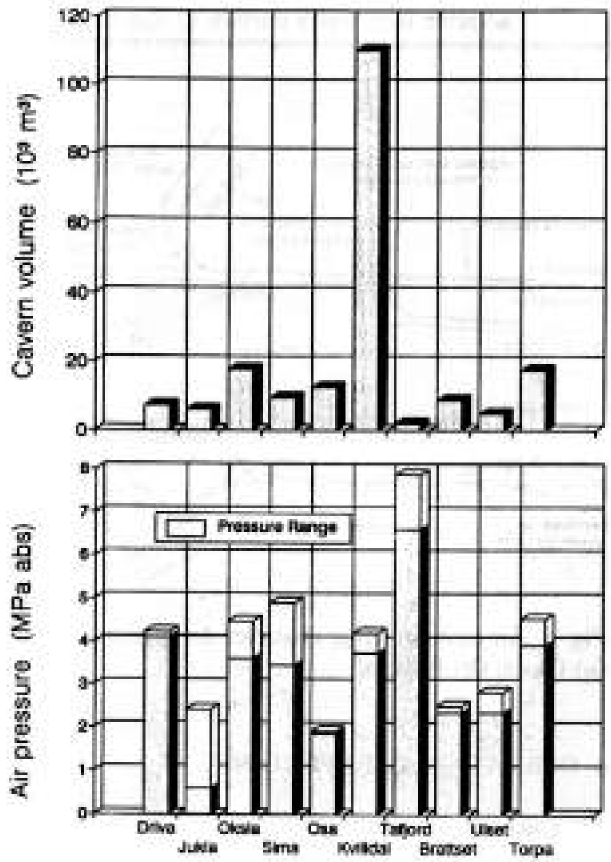

Fig. 5 Volumes and pressures of the air cushion surge chambers.

Site	Rock type	Natural rock permeability* (m^2)	Ratio between air cushion pressure and natural ground water pressure	Air leakage (Nm^3/h)	Air leakage (%/day)
Driva	banded gneiss	no data	0.6 - 0.7	0	0
Jukla	granitic gneiss	$1 \cdot 10^{-17}$	0.2 - 0.7	0	0
Oksla	granitic gneiss	$3 \cdot 10^{-18}$	1.0 - 1.2	< 5	< 0.01
Sima	granitic gneiss	$3 \cdot 10^{-18}$	0.8 - 1.2	< 2	< 0.01
Osa	gneissic granite	$5 \cdot 10^{-15}$	1.3	900/70**	11/1.0
Kvilldal	migmatitic gneiss	$2 \cdot 10^{-16}$	> 1.0	240/0***	0.2/0
Tafjord	banded gneiss	$3 \cdot 10^{-16}$	1.8 - 2.1	150/0***	5/0
Brattset	phyllite	$2 \cdot 10^{-17}$	1.5 - 1.6	11	0.2
Ulset	mica gneiss	no data	1.0 - 1.2	0	0
Torpa	meta siltstone	$5 \cdot 10^{-16}$	1.7 - 2.0	400/0***	2.0/0

* to obtain hydraulic conductivity in m/s, multiply by $\approx 10^7$
** before/after grouting
*** without/with water curtain in operation

Table 2. Air cushion factures.

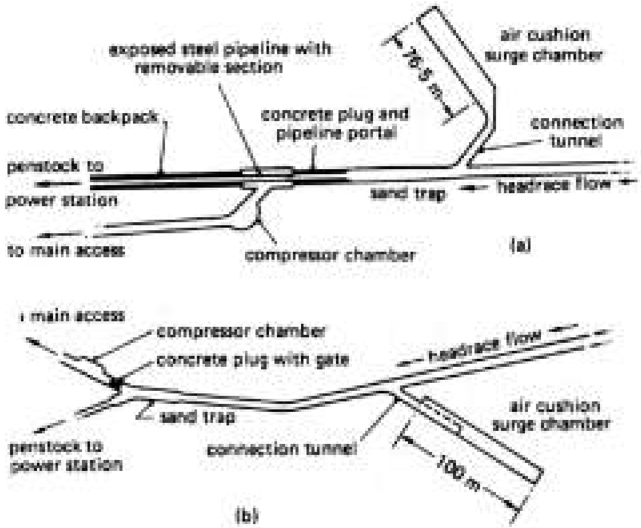

Fig. 6 Air cushion surge chamber arrangements. (a) Oksla, (b) Brattset.

4 OPERATIONAL EXPERIENCE

4.1 General

A record of operational experience includes:

- Dynamic response of the air cushion as compared to computed behaviour
- Functioning of monitoring equipment
- Compressor operation
- Rock mass stability
- Air loss and air loss prevention

Dynamic response, monitoring equipment and compressor operation are discussed to some extent in Goodall et al (1988). In this article it shall be mentioned that none of these factors have turned out to be major challenges, neither for air cushion design nor operation. It is further important to note that no specific problems related to rock stability have been recorded at the air cushion sites.

The air loss from an air cushion may be due to both air dissolution in the water bed and leakage through the rock mass. The dissolution loss per year ranges from 3 to 10% of the compressed air (depending on surge chamber geometry and pressure). This loss has no practical implication for the plant operation other than a need for supplementary air filling once or twice a year.

The measured air leakage through the rock is listed in Table 2. Six of the air cushions have a natural air leakage rate that is acceptable. Three air cushions have no air leakage at all through the rock mass. At four air cushions (Osa, Kvilldal, Tafjord and Torpa), the natural leakage rate was too high for a comfortable or economic operation. At these sites remedial work has been carried out to bring the leakage down to an acceptable level, see Table 2. One should also note from Table 2 that these four sites are located in the most permeable rock masses of all ten air cushions. Experience from the leakage prevention work at these sites are discussed below.

4.2 Leakage prevention work at Osa

Osa air cushion is located in the most permeable rock mass of all the ten air cushions, more than thousand times more permeable than the least permeable (Table 2). Although a significant cement and chemical grouting program was undertaken at the time of construction, the air leakage was measured to 900 Nm^3/h shortly after startup. After eight months of operation the plant was shut down for further grouting.

The grouting was completed within three months. A total of 36 tons of cement and 5500 1 of chemical grout were injected. This brought the leakage down to 70 Nm^3/h, which is comfortably managed by the compressor plant, even though the leakage is the highest of all the air cushions.

4.3 Leakage prevention work at Kvilldal

At Kvilldal a major weakness zone passes within 50 m of the cavern periphery and is probably responsible for a low natural ground water pressure in the surge chamber area, and thereby a higher air leakage rate than for a more homogeneous rock mass. Monitoring of the air cushion during the first year of operation showed an air leakage rate of 240 Nm^3/h.

A water curtain consisting of 47 boreholes of 51 mm diameter was adopted to reduce the air leakage. These boreholes are kept pressurized with water at a pressure of 1.0 MPa above the air cushion pressure. A plan view of the surge chamber and the over-lying water curtain is shown in Figure 7. The intention of this first water curtain used at an air cushion was to limit the leakage only. However, the water curtain showed to be able to totally eliminate the air leakage through the rock mass.

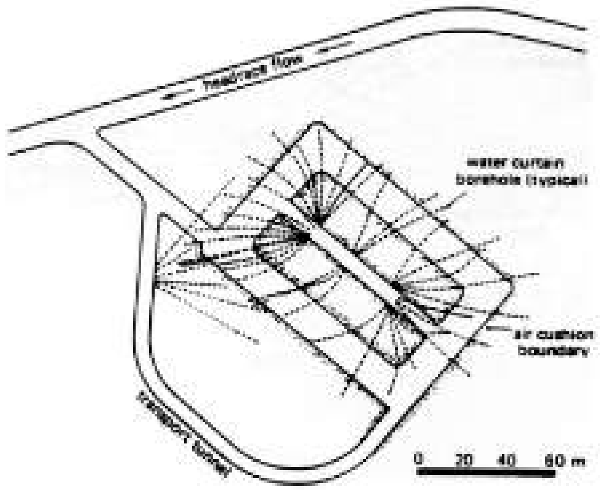

Fig. 7 Plan of Kvilldal air cushion surge chamber with water curtain.

4.4 Leakage prevention work at Tafjord

As at Kvilldal, the Tafjord air cushion was first commissioned without any leakage preventing measures undertaken. But, even though the leakage at this site was somewhat less than at Kvilldal, the compressors installed to supply the air cushion did not have the sufficient capacity. The surge chamber at Tafjord was therefore out of operation from 1982 to 1990 (i.e the cavern was completely water filled). An attempt to grout a major fracture intersecting the cavern did not improve the leakage condition. Fortunately, the Tafjord air cushion is not crucial to the operation of the Pelton system to which it is connected. The power plant has consequently been able to operate without a surge facility.

A water curtain was installed at Tafjord in 1990, partly as a research project. The curtain consists of 16 core drilled holes (diameter 56 mm) which covers both the roof and the upper part of the cavern walls as illustrated in the plan view in Figure 8. Also at this site the air leakage disappeared when the water curtain was put in operation at the design pressure (0.3 MPa above the air cushion pressure).

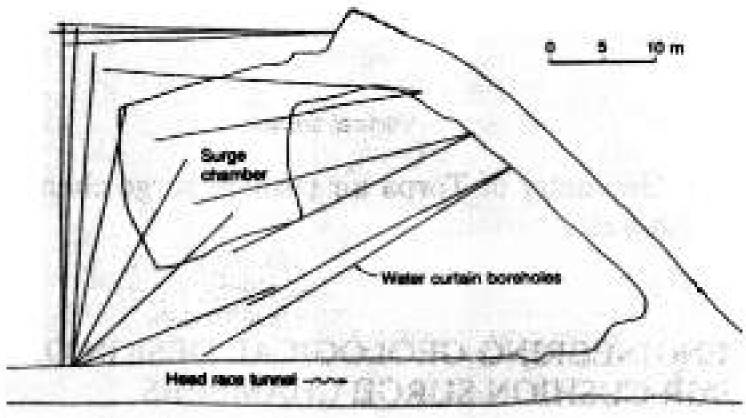

Fig. 8 Plan of Tafjord air cushion surge chamber with water curtain.

4.5 Leakage prevention work at Torpa

The Torpa air cushion is the only one where a water curtain was included in the original design. The water curtain consists of 36 boreholes (64 mm diameter), drilled from an excavated gallery 10 m above the cavern roof (see Figure 9). In addition to the water curtain, grouting was undertaken during construction to improve the rock condition.

As for the two other water curtains, no air leakage has been registered from the air cushion when the water curtain is in operation at design pressure (0.3 MPa above the air cushion pressure). To get an idea of the air leakage potential at Torpa, the water curtain was turned off for two days. This resulted in an "immediate" leakage rate of 400 Nm^3/h. The leakage ceased as soon as the water curtain again was put in operation. The measured leakage rate corresponds very well with results from theoretical calculations.

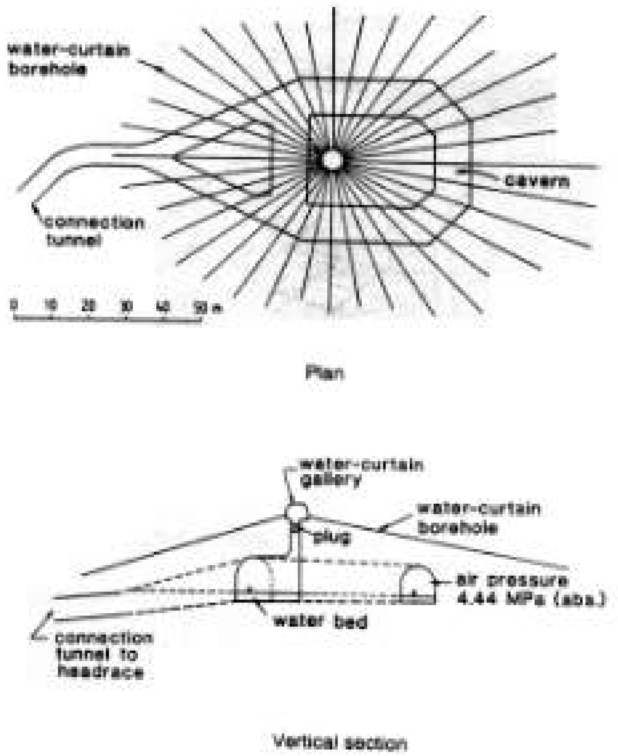

Fig. 9 Geometry of Torpa air cushion surge chamber with water curtain.

5 ENGINEERING GEOLOGICAL DESIGN OF AIR CUSHION SURGE CHAMBERS

5.1 General

The location of a surge chamber is limited by hydraulic constrains to be within approximately one km from the turbine. The challenge for the engineering geologist is to find the best location within this area. The most important factors are avoidance of rock masses with poor stability and high permeability. It is of course also essential to ensure that the rock mass has the sufficient capacity to withstand the internal pressure, in the same way as for the nearby unlined headrace tunnel.

The final location of the surge chambers in Norway is based on mapping and tests carried out from the headrace tunnel. The tests in question are permeability measurements and hydraulic jacking. Core drilling is done to verify the rock quality of a selected site.

Engineering geological mapping includes first of all mapping of:

- Rock type
- Strike, dip and frequency of rock joints, fracture zones and other discontinuities
- Rock mass permeability on the basis of water inflow
- Fracture roughness and infilling

To obtain the best rock stability possible, the long axis of the cavern are normally oriented so that it bisects the angle between the strike of the principal joint sets, More details about the engineering geology related to location and orientation of underground caverns can be found in Broch (1988).

5.2 Air leakage

Air leakage through the rock mass has shown to be the most critical factor for a successful air cushion. The air leakage can be evaluated by use of the following equation presented in Tokheim and Janbu (1982):

$$Q_{gw} = \Psi \frac{\pi K L P_o}{\propto_g G} \left[\left(\frac{P_g}{P_o} \right)^2 - \left(\frac{P_e}{P_o} \right)^2 \right]$$

where Q_{gw} is gas leakage through the rock mass (m^3/s at pressure P_0), K is rock mass permeability (m^2), L is characteristic length of storage, μ_g is dynamic viscosity of gas, G is geometry factor, P_0 is reference pressure, (normally atmospheric pressure, Pa abs), P_g is storage pressure (Pa abs), P_e is pressure at a plane isobar away from the surge chamber (normally ground surface with atmospheric pressure), ψ is a factor describing the relative leakage reduction due to the presence of groundwater. The factor ψ varies between one for "dry" rock, and zero if there is a positive groundwater pressure gradient towards the air cushion.

The above equation shows that for a given surge chamber geometry and pressure, the air leakage depends on the surrounding ground water pressure and the rock mass permeability.

To obtain a non-leaking air cushion without introducing leakage preventing measures it is necessary that the natural groundwater pressure at the air cushion site (to be interpreted as the ground water pressure before construction of the surge chamber) is significantly higher than the air cushion pressure. In this way there will be a positive ground water pressure gradient (note pressure gradient) towards the air cushion during operation ($\psi = 0$), which is the criterion to avoid an outward air leakage flow. This is discussed in detail in Kjørholt (1991).

If the natural ground water pressure is insufficient to totally prevent air leakage, an air leakage according to the above equation should be expected. For ratios of air pressure to natural ground water pressure less than unity, both the permeability and the ground water pressure will playa significant role for the leakage rate. In cases where the air pressure exceeds the natural ground water pressure, the permeability will be the dominating factor for the leakage rate.

5.2 Air leakage prevention

If the estimated or experienced leakage from an air cushion exceeds the acceptable level, remedial actions can bring the leakage down below this limit. There are mainly two actions in question: Grouting and installation of a water curtain. Our experience is that the cost benefit ratio for a water curtain is considerable more predictable than for grouting.

By grouting, the permeability of the surrounding rock mass will be reduced, and the air leakage will decrease correspondingly. Experience shows that grouting can only reduce the permeability to a certain extent, and can not be used to totally eliminate the leakage. Practical experience indicates a leakage reduction by maximum one order of magnitude if the grouting takes place ahead of excavation. Grouting after excavation is generally less effective than pre-grouting.

The working principle of a water curtain is to increase the ground water pressure around the air cushion artificially. A water curtain consists of an array of boreholes above, and sometimes also along the sides of the air cushion. These holes are connected to a water pump that maintains a permanent water pressure in all these holes, Figures 7, 8 and 9 show the water curtain design at the three air cushions which have such an installation. If the water curtain pressure is high enough to establish a pressure gradient towards the air cushion in all potential leakage paths, no leakage will take place at all, as stated above. Guidelines for design of water curtains to obtain complete air-"tightness" are provided in Kjørholt (1991).

Water curtain operation is a matter of keeping the water pressure in the boreholes at a given level. To obtain this, it is necessary that the water curtain pump operates continuously. Two types of pumps have been used, triple-plunger pumps and centrifugal pumps. Plunger pumps are used if the water has to be pumped from low pressure, while centrifugal pumps have been used in the cases that the headrace have been used as water supply.

At Kvilldal both types of pumps have been used. A plunger pump was used for the first three years until a failure occurred in the supply line for the water curtain. Since it was believed that this failure may have been caused by vibrations introduced by the pump, the plunger pump was replaced by a centrifugal pump. Torpa also uses a centrifugal pump, while Tafjord has a plunger pump mainly because of the high pressure level.

Experience has shown that if the pump stops, an air leakage will develop. How fast the leakage develops can vary significantly, and is a question of overburden, air cushion pressure and the structure of the rock joints. At Kvilldal it takes several months to approach the stationary leakage level experienced before the water curtain was installed. Tests at both Tafjord and Torpa showed that full leakage was reached within few hours. At all sites the leakage stops "immediately" after the water curtain has been put in operation again.

6 CONCLUSIONS

Air cushions have proven to be an economic alternative to the traditional open surge shaft for a number of hydro power plants. Experience shows that the hydraulic design should follow the same principles as for an open surge shaft. The geotechnical design of the air cushion cavern should follow the same basic rules as for other rock caverns.

Air leakage through the rock masses is the major challenge when designing and constructing an air cushion. A certain leakage may for economical reasons be accepted. If, however, the leakage exceeds a given limit, both grouting and the use of water curtains are possible actions. Experience has shown that grouting will reduce the leakage to a certain extent, while a water curtain is able to eliminate the leakage through the rock.

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STEEL LINED ROCK CAVERNS

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Abstract: The article describes three alternative concepts for the construction of natural gas storages and gives an evaluation of the alternatives concerning cost, construction time and safety. The three concepts are unlined rock caverns, lined rock caverns and insulated spheres for LNG at temperature -162°C. It is found that the steel lined concept is advantageous compared to the other two concepts. The storing of gas in salt domes or aquifers are mentioned, but not compared because of their dependence on geological conditions.

1 INTRODUCTION

In the future the production and use of Natural gases will be important to Norway. Natural gases produced from wells in the North Sea and even from fields further north will find domestic use as fuel for general consumption, EL-energy production, and as raw material for various industries.

The consumption is dependent on a more or less continuous delivery of gas. This necessitates for a storage in the delivery system. Traditionally natural gases have been stored as LNG in insulated spheres of steel, aluminium and concrete, or under ground by using excavated salt mines, porous fields or aquifers.

Liquid hydrocarbons and some hydrocarbon gases have been stored in rock caverns, using ground water pressure or freezing to prevent leakage.

Investigations have been made to find a storage concept with low sensitivity to ground conditions and gas properties.

The presented steel lined storage may conquer other systems in cost, safety and flexibility.

2 GAS SPECIFICATIONS AND STORAGE VOLUMES

Natural gases from the Norwegian continental shelf contains between 70 and 100% of methan (CH_4). in the North Sea normally more than 90%. In addition there are some impurities and higher hydrocarbons such as C_2H_6 to C_9H_{20}. This study is based on typical specifications for European gas for general consumption in calculations. These are:

Combustion value	38-43 mJ/Sm^3
Molecular weight	16-20 kg/kmo1
Rel density (air = 1)	0,55-0,70
Water content (max)	50 mg/Sm^3
Hydro carbon (vapor point)	-10°C at 50 bar
CO_2 content (max)	2,5 %

As a rule storage capacity should be 3-8% of the annual consumption i.e. 1 to 4 weeks delivery. This means:

1. A power plant producing 10 TWh per year would consume l x 10^8 standard cubic meters {Sm^3) natural gas in 3 weeks.
2. A storage for sale from the Troll- and Sleipner fields to Europe would require a storage volume of about l x l0^9 Sm^3 to save 3 weeks delivery.

In the following gas storage in the range of 1x10^8 Sm^3, (above 1), is discussed.

3 MAIN STORAGE ALTERNATIVES

The following alternative storage methods are in use or studied.

- Unlined rock caverns

- Lined rock caverns
- Liquefied natural gas (LNG) storage
- Storage in salt formations
- Storage in porous formations (aquifers).

Salt domes and aquifers methods can be used in special geological formations only, and is not further described.

The other three types of storages are evaluated in the following chapters.

3.1 Unlined rock caverns

This type of storage is built according to the same principles as storage for liquid hydrocarbons. Leakage of stored products are prevented by ensuring the ground water pressure around the caverns to be higher than the gas pressure. This is obtained by placing the caverns at a sufficient depth or by establishing an "umbrella" of high water pressure. For some heavy gases it is possible to have a storage at low temperatures without unacceptable leakages. In this case the gas is liquefied. In unlined rock the temperature limit is found to be around -40°C, far from temperatures for liquefied natural gases.

A gas pressure of 100 bar is used for the unlined type of storage. This means that the storage must be located ca. 1100 m below GW-level to keep a 10% safety for overpressure of ground water compared to gas pressure. At this depth ground temperature will be about +40°C. Physical volume of caverns will for the actual gas storage of 1×10^8 Sm^3 be 940.000 m^3.

The leakage water has to be pumped out of the caverns continuously. The high storage pressure will give a high content of dissolved gas in this water, which has to be taken care of at the surface.

When using low temperatures in unlined storage, the contact water/gas may form gas hydrates (ice) that may disturb tubes and mechanical equipment at the distribution net.

3.2 Lined rock caverns

At this stage we shall limit the concept to the use of steel lining. Other materials have been evaluated, but so far steel has proved to be the material that gives the best lining at lowest cost.

This storage method utilizes the rock formations to take care of the pressure and the steel lining to prevent leakage. Steel lined storage may also be used when rock conditions do not allow unlined caverns.

Steel lined caverns may be operated in a wide range of storage conditions, in regard to temperature as well as pressure. Special steel alloys are necessary when low temperature is used.

For this type of storage gas pressure is 150 bar. A low temperature design of -60°C has also been considered. This temperature is approximately twenty degrees above the critical temperature of the actual gas.

Under these conditions the actual gas volume of 1 x 108 Sm^3 needs a physical storage volume of 240.000 m^3.

The minimum depth for the caverns is designed to be 120 m below surface. Compared to the unlined storage this alternative means much smaller rock caverns and less excavated rock, also because of its shallow position.

The steel lining needs a drainage system to prevent lining from outside water pressure. Smaller volume and easier transport to and from the surface will give a lower cost for this alternative than for the unlined cavern, even if the steel lining and drainage system makes a high price pr. cubic meter physical cavern volume. The steel lined concept is described more in details in chapter 4.

3.3 LNG – storage

Most natural gases are transported and stored as liquid today (LNG}.

The LNG-storages are normally surface structures. The tanks are constructed as spheres from Ni-steel or aluminium, insulated and with a reinforced concrete structure outside.

This type of tanks takes no pressure from the gas. Liquefying the LNG will depend on the extreme low temperature of -162°C.

The tank volume needed to store LNG for the actual gas volume of 1 x 10^8 Sm^3 will be 160.000 m^3.

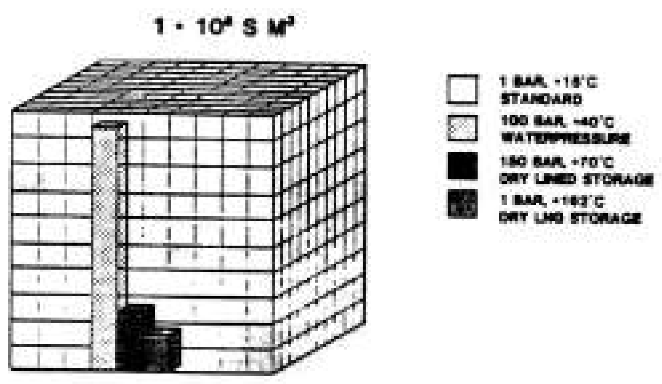
This type of storage is very expensive, and gives little protection against accidents, sabotage or warfare. Lined shallow rock caverns built with extra insulation would provide a safer storage of LNG, and may be a special design of lined storage. LNG storage will need more energy in the operating phase to bring the produced gas to storing conditions and back to the wanted conditions of the users.

3.4 Comparison

Main figures for storing gas necessary to run a 10 TWh power plant for 3 weeks in the described alternative types of storages are as follows:

Type storage	P bar	t °C	m^3	Min. rock cover
Unlined rock cavern	100	+40	940	1100
Steel lined rock cavern	150	-60	240	120
LNG storage	1	-162	160	-

The volume of storages are shown below, compared with the stored gas volume in standard conditions.



4 LOCATION, ROCK COVER AND DEFORMATIONS

This evaluation is based on ground conditions which gives minor problems for the construction of the storage caverns.

The location of the storage system depends on other aspects than the geological conditions. It is believed that the steel lined caverns are significantly less influenced by the rock conditions than the unlined rock caverns would be.

This site is supposed to be in precambrian gneiss with the following rock specifications:

Compressive strength		150 Mpa
Elasticity	E	50 Gpa

Poissons ratio	V	0,2
Specific gravity		26 KN/m^3
Permability	K	10^{-7} cm/sek
Shear angle	Ø°	35-65°
Attraction	a	1,5-5,0 x 10^5 KN/m^2

By using these data's the calculated depth will be in the range of 90 to 120 m below surface.

Computer modelling has given similar results, 100-130 m, of coverage to take care of an internal pressure of 200 bar.

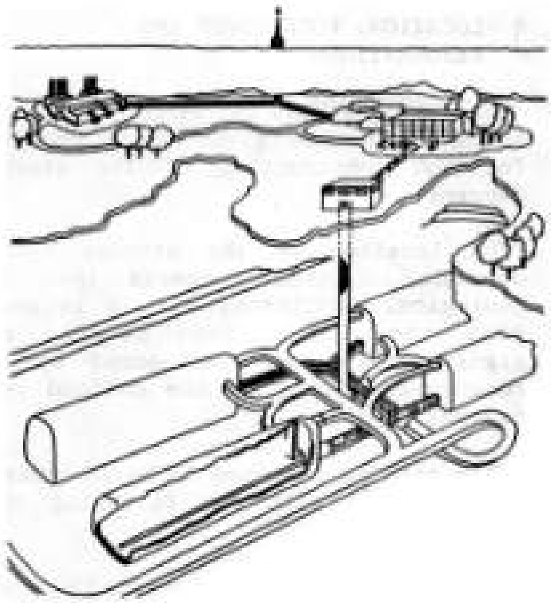
The same computer modelling indicated deformations at the cavern walls up to 10-15 mm. Shrinkage from temperature variations is limited to less than 5 mm.

These deformations give tensions in the steel lining that may be acceptable for the described construction.

Similar calculations done by the Norwegian Geotechnical Institute (NGI) have shown similar results.

5 CONSTRUCTION DETAILS

Lined rock caverns of 240.000 m^3 storage volume, may be designed in many ways. The design shown below is used for the evaluation of cost and construction time. Four separate caverns around a central tunnel and a shaft system makes the total volume. Each cavern is closed by concrete plugs. During construction, access to the storage caverns from the surface is through a descending tunnel. For operation gas pipes are installed in a shaft.



5.1 Excavation

Drill and blast techniques are used in the excavation of the tunnel and caverns . while the shaft is drilled by reaming techniques. The access tunnel is wide enough for two trucks to pass each other. It is constructed as a spiral around the store area with access both over and under the caverns. A drainage system is necessary to prevent outer pressure on the lining. It will also be used to take care of possible gas leakages. The dimensions of the caverns are designed to keep the rock support at a low rate during construction. An 18 m span and a total height of 27 m are used. This area is excavated in three steps .top heading and two benches . The length of each of the four caverns for the design volume will be approximately 150 m.

5.2 Steel-lining and concrete

Primarily the steel lining keeps the storage volume gas tight. It also serves as part of the framework for the casting of concrete between the lining and the rock surface.

The construction of the steel lining is similar to that of a ship's port hole with the ribs situated outside the plates. The ribs are fastened to the rock by bolts.

The installation of steel lining is carried out using elements of the largest possible size. The steel quality depends on the storage conditions. For a storage operating down to -60°C -a steel alloy with 1,5-4,0 % Ni - will give a suitable quality. Corrosion during the operation is calculated to be a minor problem.

Weldings are made in such a way that each section can be pressurized for testing.

The lining is installed as follows:

- A mobile rig is established on rails fixed to the cavern floor.
- The ribs are bolted to the rock walls.
- Steel plates are installed in heights of 2-4 m around the cavern.
- Before a new height is installed, grouting between the steel lining and the rock is done behind the branched plates.
- This continues up to and around the ceiling.
- Special attention is given to the injection behind the ceiling plates .
- After dissambling the rig - the floor will be covered by plates and concreted stepwise and finally injected. Tests on all welding seams are taken during the construction time, to ensure no leakage.
- Finally a concrete plug including pipes completes the storage cavern.

6 COST ESTIMATE AND CONSTRUCTION TIME

6.1 Cost estimate

For the three types of storage facilities mentioned- a rough cost estimate is made. For the surface LNG storage cost is based on prices from Chicago Bridge -for 2 spheres each of a volume of 80.000 m^3, and new information from Moss Rosenberg Shipyard. This size is about the largest ever built for this type.

The unlined rock cavern cost is based on a project carried out by "Nord-Trøndelag Power company" and The Foundation for Scientific and Industrial Research at the Norwegian Institute of Technology (SINTEF) for a storage of ca. 940.000 m^3. This concept implies one shaft to the depth of 1.100 m depth to remove excavated rock masses.

The cost of the steel lined rock cavern is based on the described design presented above.

Price reference winter 1989.

LNG storage spheres	NOK	650 mill
Unlined rock storage	NOK	700 mill
Steel lined rock storage	NOK	470 mill

Very little, if any, increase in cost for this type of work is indicated in Norway since 1986.

6.2 Construction time

The construction period is estimated on a 16 hour workday, 2 shifts, 5 days a week basis. Investigations and design time is not included.

This gives the following construction time:

LNG storage spheres	2 years or less
Unlined rock storage	5 years
Lined rock storage	4 years

7 CONCLUSION

From the above given figures of cost and construction time, and also from a safety point of view the "Steel lined rock caverns" is a very interesting concept for the storage of natural gases. It may also be suitable for other gas storage installations.

Advantages compared to the LNG-spheres:

- Low construction cost.
- Safer.
- Less running cost.
- May fit different storing conditions.

Advantages compared to the unlined caverns:

- Small physical volume.
- Limited depth.
- Less cost.
- No contact gas/water.

- Shorter construction time.

STORAGE OF INDUSTRIAL WASTE IN ROCK CAVERNS AT NORZINK, ODDA

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ABSTRACT: Waste products, jarosite residue, from the zink production at Norzink, Odda, have previously been pumped directly into the sea.

Besides other industrial activity, this has created severe pollution problems in the fjord, and the State Pollution Control Authority has required alternative disposal of the residue.

Today, the residue is stored in rock caverns located 2 km north of the zink production plant.

The caverns have a total volume of approx. 69.000 m^3. except from cavern no.7. which will have double storing capacity.

A minimum of rock support has been required during excavation, and the total cost per cavern is approx. 1.7 mill. USD {volume 69,000 m^3), and 2.7 mill. USD {volume 139,000 m^3).

Operation cost for the waste disposal plant, including transport, pumping and cleaning of excess water, is approx. 0.14 mill. USD per year.

Risk analysis of potential leakages of polluted water from the caverns give a possible leakage of 0.008-1.4 gram zink per cavern per day, which is far below the total permitted pollution of 38.25 kg zink per day.

1 INTRODUCTION

Surrounded by high mountains and glaciers, Norzink AS is located in the southern end of Sørfjorden, a side branch of the Hardanger Fjord in southwest Norway.

The company was established in 1924 under the name of "Det Norske Zink- kompani AS". The factory has produced electrolytical zink since 1929. In 1976, the name was changed to Norzink AS.

Besides zink production, sulphure acid, aluminum fluoride, cadmium, copper residue and lead/tinresidue are products from the process.

From the metal extraction, a consider- able quantity of heavy metals contaminated leach residues called jarosite are produced as waste material. The annual waste products from the production are approx. 65,000-70,000 tons of jarosite residue, representing a volume of 65,000 m^3 per year.

Previously, the waste was suspended in water to a dry substance content of 10%, and discharged to the fjord as a suspension. In addition to other industrial waste products from the area being discharged to the fjord, this has created severe pollution problems in the inner part of Sørfjorden.

Due to the pollution problems, Norzink AS was asked by the State Pollution Control Authority to develop alternative methods of waste disposal. After 1 July 1986, no discharge of residue to the sea was permitted.

Studies for alternative disposal of waste from the zink production started in 1975. Among several alternatives as solidification and chemical treatments, deposits in rock caverns was found to be the most attractive alternative.

2 PROJECT DESIGN

Location and design of the rock caverns have been assessed from geological, hydrogeological and topographical aspects. Furthermore, possibilities for long distance pumping of jarosite slurry mixes were studied.

From several alternative locations, the rock formation Mulen, approx. 2 km north of the factory, was selected. Altogether, three alternative cavern designs have been evaluated:

1. Rectangular shaped rock caverns with double access tunnels for excavation of top heading and benches.
2. Horizontal, bottle-shaped rock caverns with single access tunnel to the outer end.
3. Circular or cylindrical shafts excavated from the bottom, a design which is common in mines.

From these alternatives, alternative 2), horizontal bottle shaped caverns, was found to be most feasible.

For utilisation of the Mulen rock formation, plans were made for constructing rock caverns in two elevations; 15 caverns at each level, minimum vertical distance between the upper and lower level of 30 m, and horizontal distance between each cavern of 20 m.

For excavation of the first cavern, an access tunnel of 300 m length was necessary.

Lay-out of the rock caverns is shown in Figure 1.

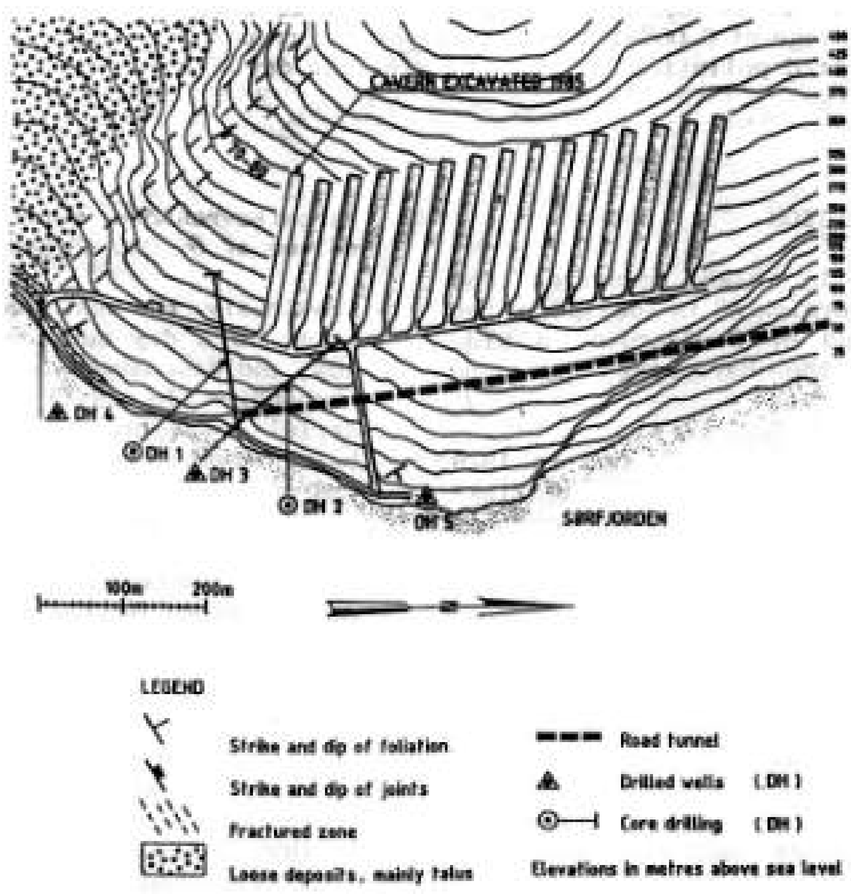

Fig. 1 Lay-out and geological information

The rock caverns no.1 to no.6 have a total length varying from 211 m to 225 m. width of 17.5 m and maximum height of 23.5 m. Total volume of the caverns varies from approx. 65.000 m^3 to 69.000 m^3. Cavern no.7. which is now under construction, is designed with double capacity, total length of 404 m, width 17.5 m, and maximum height 23.5 m, which corresponds to a volume of approx. 139,000 m^3. Design of the caverns are shown in Figure 2.

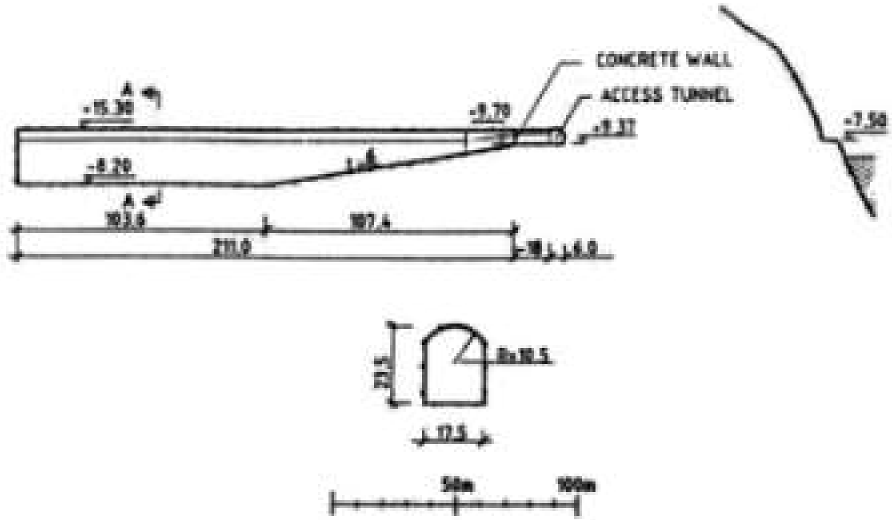

Fig. 2 Cavern design

Saline water is not permitted in the process, and leakages from the sea into the caverns must be avoided.

In order to prevent leakages of saline water into the caverns, and keep the risk of leakages of polluted water to the sea at a minimum, the caverns are located with floor elevations 7-8 m below sea level and water elevation 15-18 m above sea level.

The caverns are closed with a concrete wall at the outer end.

Tunnel spoil from the excavation has been utilised for different purposes in the factory areas and within Odda municipality. However, most of the rock spoil has been dumped in the fjord. For that purpose, a separate transport tunnel has been excavated.

3 GEOLOGY AND HYDROGEOLOGY

3.1 Ground investigations

In order to find the optimum location, cavern design, and assess the risk of leakages of polluted water from the caverns. geological and hydrogeological investigations. rock mechanical and pollution risk analysis have been performed.

Geological and rock mechanical studies include studies of maps and aerial photos, geological survey, core drilling and stress measurements, laboratory tests of rock samples, samples of clay and disintegrated rock material from faults and fracture zones. For design, theoretical models and stress analysis have been used. The excavation works are followed up by geological mapping and survey.

Furthermore, pump tests and in situ permeability tests in selected boreholes as well as chemical analysis of the ground water have been made. Chemical analysis of unpolluted ground water form the basis to assess pollution of ground water seepages from the caverns.

3.2 Geology

The bedrock is mainly medium to coarse grained granitic gneiss with occasional weins of pegmatite, amphibolite and granite. The foliation of the rock strikes in NV direction dipping steep towards NE.

Besides foliation joints, a major joint system occurs with strike in NE direction and near vertical dip. The different joint systems are presented in Figure 3.

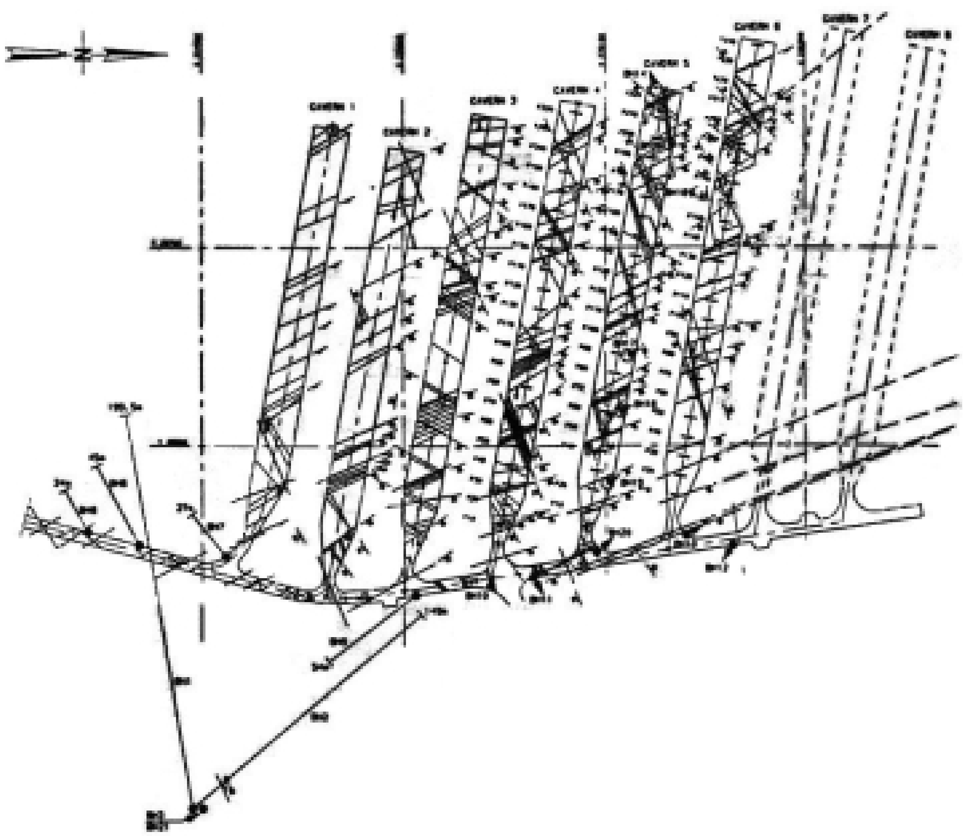

Fig. 3 Location of pump wells, fissures and fracture zones

Only minor fracture zones occur within the recommended rock formation. Clay samples of des integrated rock material show only moderate swell activity.

Rock samples tested in laboratory have an average density of 2710 kg/m^3, uniaxial strength is 145 MPa, tensile strength (point load test) is 16 MPa, E-modulus 25 GPa, and poissons ratio 0,10.

3.3 Rock stresses

Rock overburden varies from approx. 200 m towards 450 m above the caverns. Behind the caverns, the mountains rise with an inclination of 40° up to an elevation of 1000-1500 m above sea level.

Tectonic stresses have been experienced in similar rock in the vicinity. Thus, rock stresses due to overburden and/or tectonics were expected.

However, from topographical and geological features and forms, there was a certain possibility for stress releaf in the outer part of the rock formation where the caverns are located. This assumption has been proven during excavation of the first seven caverns where only minor rock burst phenomenon has been experienced.

Three-dimensional rock stress measurements have been performed by SINTEF, Rock Mechanical Division. The measurements were performed in the first rock cavern after excavation. The principal stresses, which are of gravitational origin, are $\sigma 1$ = 6.7 MPa, $\sigma\, 2$ = 1.7 Mpa and $\sigma\, 3$ = 0.6 MPa. The maximum principal stress has an orientation parallel to the valley side, the medium principal stress approx. perpendicular to the southern valley side of Mulen and the minimum principal stress is approx. horizontal and parallel to the valley side. Tectonical stresses have not been proven.

Reorientation and concentration after excavation of the caverns were investigated by measures in borehole depths 6 m and 7.5 m from the cavern wall. The results are shown in Table 1 and stereoplot Figure 4.

Depth m	σ_1	φ	θ	σ_2	φ	θ	σ_3	φ	θ
6,0	15,2	145	62	6,2	358	24	3,0	262	14
7,5	9,9	128	64	5,5	24	7	2,5	291	25
In situ	6,7	128	53	1,7	320	36	0,6	225	6

Table 1 Principal stresses in situ and at depths 6.0 and 7.5 m in the borehole.

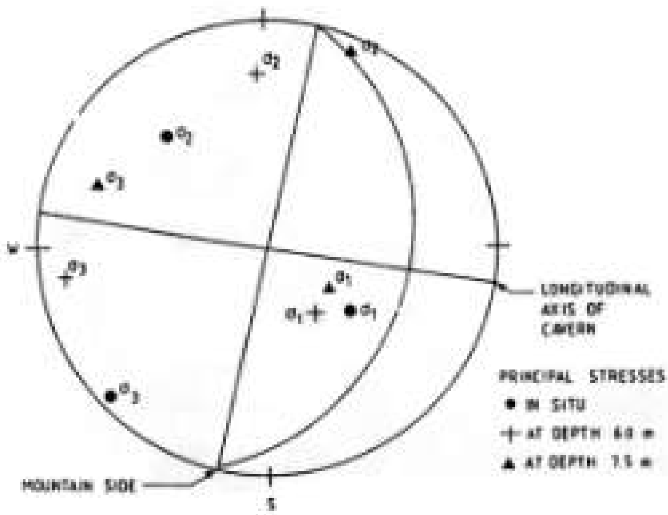

Fig. 4 Stereogram showing the principal stresses.

Rock mechanical analysis are performed to determine rock stresses, stress distribution and stability. Furthermore, the results are used to find optimal cavern design, minimum width of rock pillars between the caverns, and the minimum vertical distance between the cavern levels.

Rock stress models and variation of vertical rock stresses with depth of rock are shown in Figures 5 and 6.

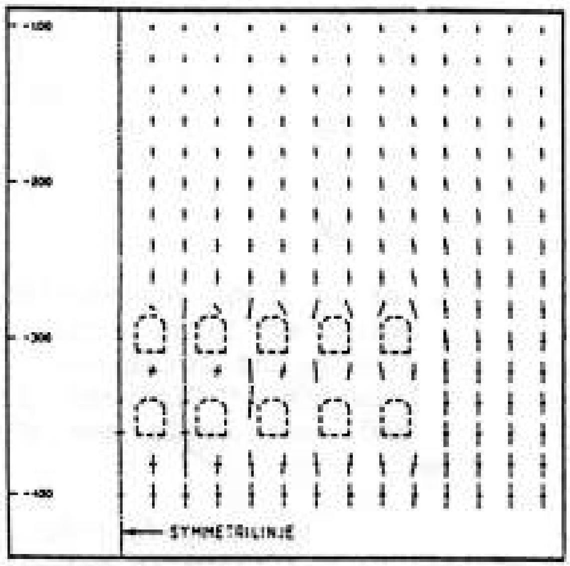

Fig. 5 Rock stress distribution 20 caverns with horizontal distance 20 m and vertical distance 30 m.

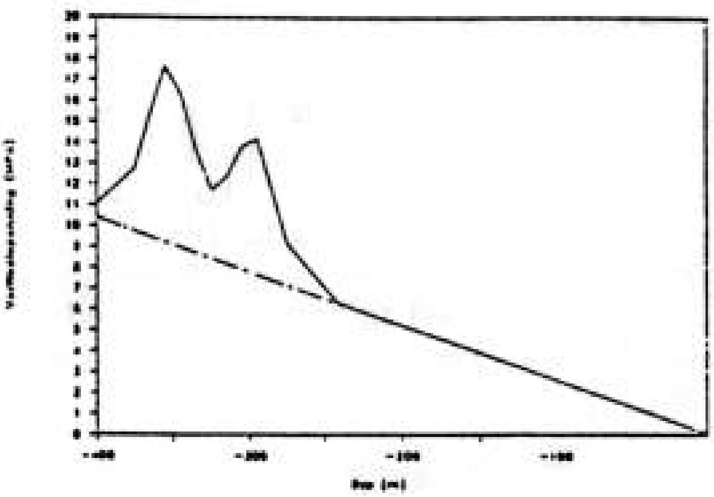

Fig. 6 Vertical stress distribution along centreline of caverns.

Stress analysis have confirmed the preliminar design assumptions with minimum distance of 20 m between caverns, and minimum vertical distance between cavern levels of 30 m.

New stress measurements and re-evaluation of design criterias will be done in rock cavern no.7. which has double length.

3.4 Permability and hydrogeology

A set of pump wells are drilled in front of the caverns in order to monitor the pollution by sampling and chemical analysis of water from the wells. Pump tests from the wells are performed to evaluate the rock mass permeability.

During excavation, registrations and measurements of joint sets, fracture zones faults and local leakages are done. Location of fissures, fracture zones and pump wells are shown in Figure 3.

Pollution risk analysis from the caverns to the fjord is performed. As part of the risk study, pump tests from drillholes, chemical analysis of watersamples from the control wells, and grain size distribution tests, permeability and consolidation tests of jarosite are done.

From hydrogeological models and use of flownet, possible seepage of polluted water and consentration of zink pollutions are calculated. The study is based on three possible ways of transport to the sea:

- Polluted water is pressed out of the jarosite residue, into rock joints or up to the surface during consolidation.
- Polluted water is pressed through the residue by ground water towards the sea.
- The decantation water is polluted by turbulence in settled jarosite created by water leaking from the cavern roof.

Rock permeability is calculated from trial pumping from the control water wells.

From these calculations, a possible seepage of 0.008-1.4 g/day of zink per cavern is found for the short-term situation, and a possible seepage of 0.008-0.8 gram zink per day per cavern is found for the long term situation.

Flow-nets for short-term and long-term situations are shown in Figures 7 and 8.

Besides seepage through the rock masses, water samples are taken from the decantation water. These samples show a concentration of 7.3-12.4 9 zink per liter of water.

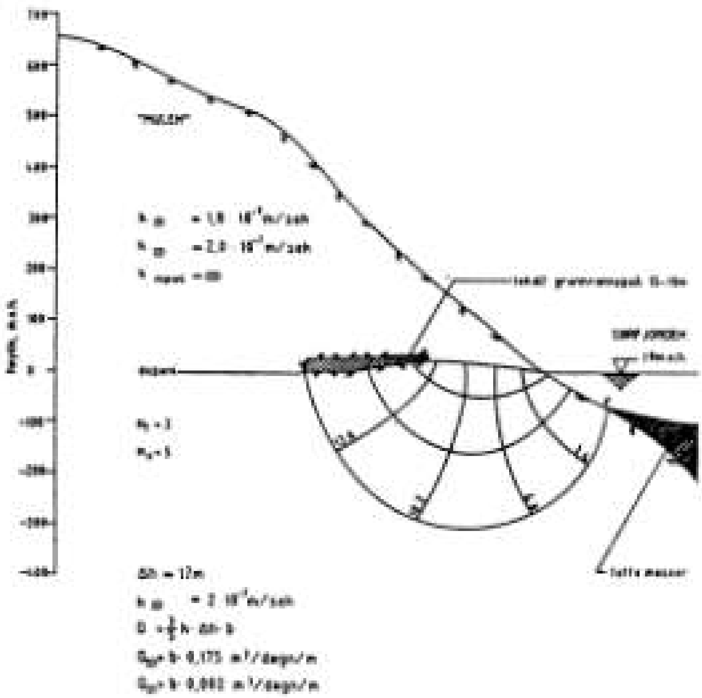

Fig. 7 Flow-net short-term.

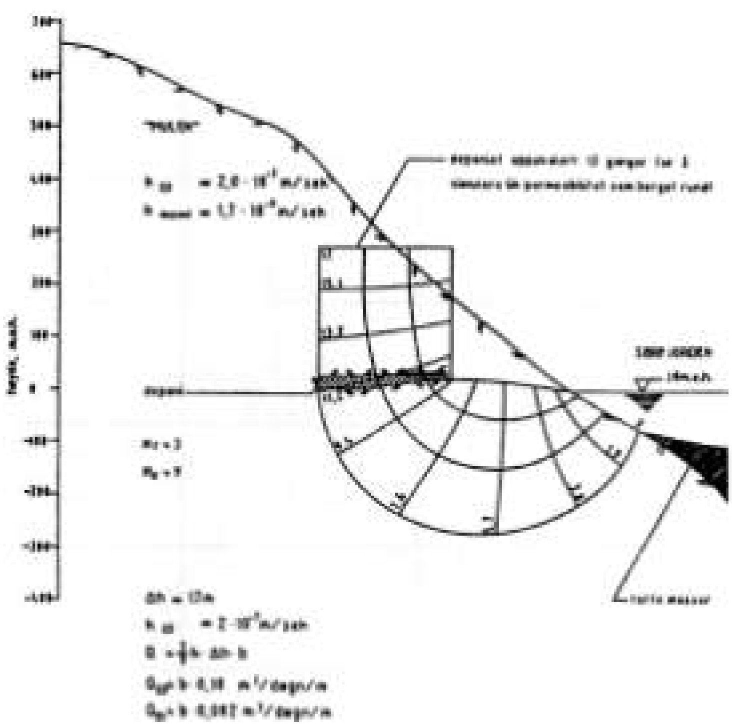

Fig. 8 Flow-net long-term.

Excess water from leakages into the caverns are cleaned in a central water treatment plant before pumped into the fjord. Water leakages into the rock caverns are in the order of 70 m^3 per day per cavern.

Possibilities of plugging and sealing the caverns have been discussed in order to avoid further problems of excess water. This possibility is not recommended for two main reasons; 1) the caverns are refilled after the first settlement period, thus they must be accessible and 2) if the caverns are plugged, the water pressure will raise to ground water pressure and the gradient will increase considerably with increased risk of pollution.

Total permitted amount of zink pollution from the plant is 38.25 kg zink per day, which is far above the calculated concentrations.

4 CONSTRUCTION EXPERIENCE

By today, 6 caverns are completed, and the seventh is under construction.

Access tunnel and the first rock cavern were constructed in 1985. All caverns are constructed by the company H. Eeg-Henriksen AS, except from cavern no.2, which was constructed by F. Selmer AS.

Tunnel and rock cavern excavations are performed by conventional drill and blast. Normally, the caverns are excavated by top heading of 7-11 m height, and two benches of 6-8 m height. For the first rock cavern, the top heading was divided in three, with central pilot tunnel and side benching.

There are not any specific requirements for the contours in the roof or walls of the caverns, and rock support requirements correspond to what is considered as a minimum support during excavation. Rock support must be sufficient to prevent block falls during filling of the caverns.

Only spot bolting and shotcrete to some extent has been required. Amount and type of rock support for the different caverns are shown in Table 2.

Total cost for the first stage, including transport pipes, access tunnels etc., was approx. 30 mill. NOK (4.3 mill. USD). Out of these, the cost for access tunnel and cavern was approx. 15 mill. NOK (2.2 mill. USD).

Construction cost per cavern is approx. 12 mill. NOK (1.7 mill. USD) which is approx. 115 NOK (16.6 USD) per produced ton of zink. Excavation costs including rock support is approx. 150 NOK (21.7 USD) per m^3.

Minor rock bursts have been experienced during excavation of the top and bottom bench in the inner part of the seventh cavern.

So far, sealing or grouting of joints or fracture zones has not been necessary. Total measured leakages into the caverns are approx. 70 m^3 per day.

CAVERN NO.	SECTION	VOLUME m^3	ROCK BOLTS pcs	SHOTCRETE m^3	MESH m^2
1	Top heading	(20,000)	125	10,8	
	Bench	(44,000)	0	23,2	
	Total	69,769	125	34,0	
2	Top heading	(20,323)	752	61,2	
	Bench	(48,313)	163	0	
	Total	72,800	915	61,2	
3	Top heading	23,000	336	43,2	
	Bench	55,950	1130	19,7	
	Total	78,950	1433	62,9	
4	Top heading	(30,900)			80
	Bench	(48,471)			
	Total	76,194	826		
5	Top heading	33,000			
	Bench	43,364			
	Total	76,364	973		
6	Top heading	30,600	1473		
	Bench	45,244	155		
	Total	75,844	1628		

(Numbers in parentheses are approximate)

Table 2 Excavated volumes and rock support. Caverns 1-6.

5 OPERATION, SUPERVISION AND CONTROL

The rock caverns have three major functions:

- Storing of residues from the production;
- Storing of other types of waste from the plant;
- Spare capacity for the process water treatment.

Besides. the caverns may be used as spare capacity for storing in emergency situations. So far. this has not been necessary.

The operation of the residue disposal plant is based on transport of the spoil from the zink production as a slurry mix of water and 20% dry content. The slurry is transported in 125 mm diameter pipelines from the zink factory to the caverns.

The pipelines are installed in an insulated concrete culvert. Frost is avoided by the temperature of the process water, which has an initial temperature of 30°C.

The slurry is pumped to the inner part of the cavern until the inner part is full. Afterwards, the slurry is pumped to the middle part and finally to the outer part of the caverns. The slurry is separated from the water by gravity and settles, while separated water flows through pipes in the concrete wall at the front of the caverns, and is pumped back to the production plant for new mixing and transport of residue.

Special waste products are stored in some of the caverns in agreement with the State Pollution Control Authority, and according to instructions made by them. Such kind of waste is stored in small caves excavated in the caverns for this purpose. The caves have been plugged and sealed by impervious membranes and concrete to avoid leakages of the waste.

The plant is being continuously supervised and monitored 24 hrs per day by skilled operators from Norzink AS. Besides volume control for monitoring and leakage detection of pumps and transportation pipes, cameras, automatic samplers, level regulators and slurry density measures are installed. The transport pipe system is monitored and supervised by volume control at both ends to detect leakages. Only one of the two pipelines are used at the same time, the other one being in reserve. Double set reserve pump capacity is installed.

Total operation cost is approx. 1 mill. NOK (0.14 mill. USD), including power and electricity, work and labourers, pumps (must be changed every second year) , cleaning of excess water, cleaning and maintenance of transport system, etc.

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COLD STORAGE PLANT IN ROCK CAVERN

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ABSTRACT A cold storage plant with an 11,000 m^3 cavern as store room was completed in Bergen in 1974. Temperature recordings in the surrounding rock have been carried out since 1977, and after 18 years in operation, the last temperature readings were carried out in 1992. The 0°C-isotherm was then at approx. 20 m distance from the cavern. The rock stability has been good, without any rockfall. Both construction costs and energy costs are in favour of the rock cavern solution compared to a surface cold store. Safety against destruction of stored products in case of power failure or other stops are good. During a maintenance period, the temperature raised only 10°C after 8 weeks.

1 DESCRIPTION OF THE PLANT

A cold storage plant mainly for meat and meat pro- ducts, situated in Bergen, Norway was completed in 1974.

The plant consists of a 12 m long access tunnel and a storage cavern 57 m long, 20 m wide, and 10,8 m high, with a total volume of 11,000 m^3. The plant is outlined in Fig.1.

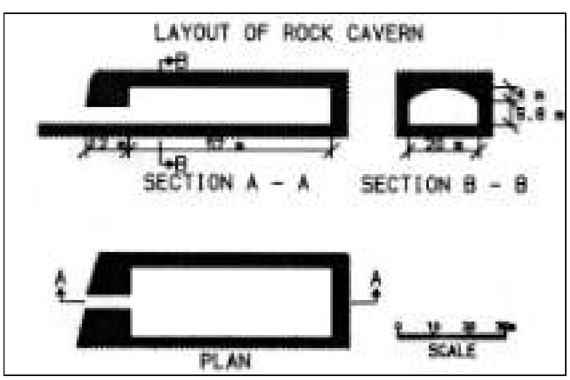

Fig. 1

A machine room is situated inside the cavern. Outside, there are office-and service buildings, also containing some ventilation equipment for the cavern.

The operating temperature has been varying between -22 and -28°C, and is now -25°C.

In the access tunnel a concrete wa11 with gates is installed.

The cavern has a concrete floor with rails for the storage racks, and the racks can be moved for maximum utilization of the storage volume.

The rock type is a precambrian granitic gneiss, and rock is exposed in the cavern roof and walls. Rock bolts are the only supporting means used.

2 EXPERIENCE FROM THE CONSTRUCTION PERIOD

Design and orientation of the cavern was based on common engineering geological methods, and the blasting and supporting works were supervised by experienced engineering geologists.

The rock stability was good. and no unexpected or serious stability problems influenced the excavation.

Grouted, 3 m long bolts were used as rock support. In the roof a total of 305 bolts were installed (1 bolt/5 m^2). In the walls, only 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 a period of about 3 weeks there was a busy time picking and removing ice. To collect the water at concentrated spots, some holes were drilled, and fans blowing cold air were placed up to these spots. The water then froze rather quickly, and no water seepage into the cavern has later occurred. Air temperature in the cavern was then -14°C. A time/temperature diagram in the cavern from the cooling period is shown on Fig.2.

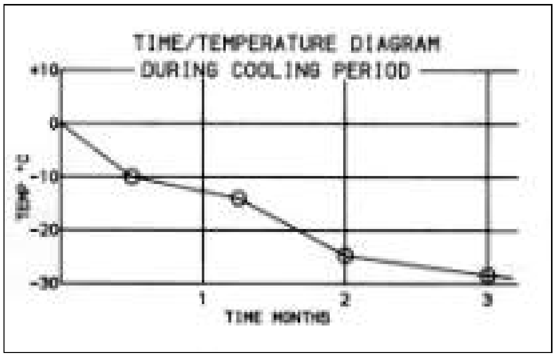

Fig. 2

3 EXPERIENCE FROM THE PLANT IN OPERATION

The temperature in the surrounding rock has been measured by means of 11 sensors installed and grouted in a 20 m long borehole.

Recordings are made at various intervals throughout the operating period from 1977 to 1992, and 5 readings are shown on Fig.3. The 0°C isotherm was in 1992 at approx. 20 m distance from the cavern.

The temperature in the rock has also been recorded some 100 m from the cavern, showing constant +4.9°C at 6-12 m depth from the terrain surface.

There has been no need for maintenance of the cavern walls and roof. and they are stable and dry .

Both construction costs and operating costs related to maintenance and energy consumption are very low.

At the time of construction the construction costs were approx. 35% lower than a similar store constructed as a surface building erected in the close vicinity of the rock store.

The energy consumption after 3 years of operation was approx. 50% lower than for a similar surface store.

Safety against destruction of stored products in case of machinery breakdown, power supply failure, or maintenance stop is in favour of underground store rooms. Cooling machinery may be switched off for weeks without any critical raise in temperature.

During a maintenance period, the cooling machinery was stopped for 8 weeks, and the temperature then raised from -24 to -14°C. When the cooling started again after that period, the temperature was back to - 24°C after 2 days.

The Owner of the plant found this type of cold storage very advantageous, and expanded the storage capacity by another cavern with a volume of 10,400 m^3, completed in 1982.

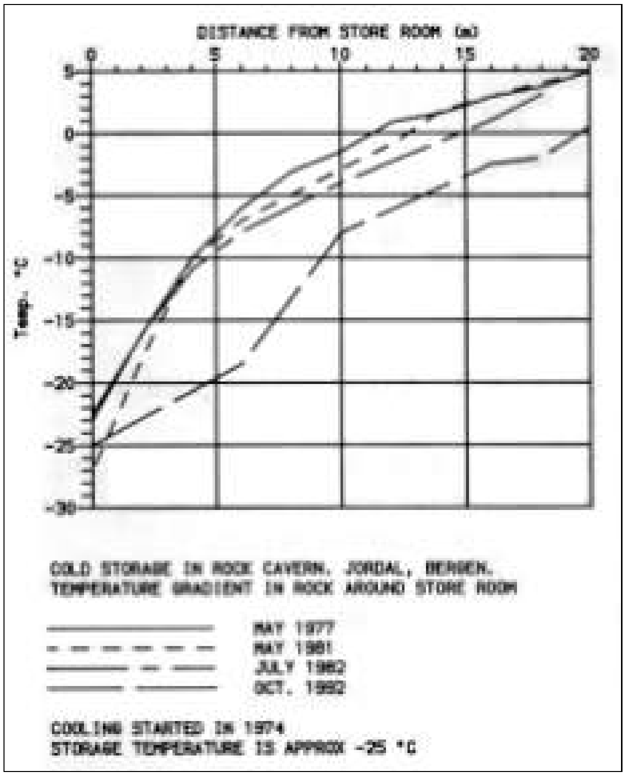

Fig. 3

Storage of Gases in Rock Caverns, Nilsen & Olsen (eds)
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BACK ANALYSIS OF HEAT LOADS ON SELECTED THERMAL STORAGES

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ABSTRACT: Heat loads computed from current operational data at two chilled storages in Norway are compared with computations based on laboratory testing of large block specimens at low temperature. One of the sites is a 60,000 m^3 liquid ammonia storage at -33°C and at a depth of 60 m in granitic gneiss. The other site is a deep freeze food storage which operates at about -25°C and is located at an average depth of cover of about 37 m in quartz-rich greenstone.

1 INTRODUCTION

Provided that thermal cracking does not occur, the heat load on a chilled storage depends directly on the thermal conductivity of the rock mass surrounding the cavern. Computation of thermal conductivity on the basis of observed heat loads on chilled storages therefore is a means of checking that thermal cracking has not occurred; it is also a means of checking that the in situ thermal conductivity of a fractured, nonhomogeneous and anisotropic rock mass is predictable on the basis of laboratory tests. Such evaluations are necessary for the prediction of heat load and/or boil-off operating costs at a chilled gas storage.

A listing of underground cold storages in rock averns in Norway is provided on Table I. with exception of the Glomfjord liquid ammonia storage, these caverns are used as deep-freeze storages for food at temperatures ranging from about -22°C to -30°C. A few of the storages were built in limestone quarries but the majority were deliberately designed and excavated at convenient locations. The Ekeberg plant is believed to have been the first specifically designed and thermally calculated cold storage in a rock cavern (Lorentzen, 1959; Frivik,1986). This study focuses on the 1974 Jordalen and 1978 Kastbrekka food storage and on the Glomfjord liquid ammonia storage which was commissioned in 1986.

Site	**Width x height x length (m)** **Volume (m^3)**	**Rock** **Type**	**Storage** **Temperature (*C)**	**Commission** **Date**
Ekeberg, * Oslo	14 x 8 x 33	Granitic Gneiss	- 25	1956
Brevik **	irregular 14,000	Limestone	- 25	1962
Gjelleråsen, Oslo	- 10,000	Syenite	- 27	1967
Hamar	- 10,000	unknown	- 22	1967
Jordalen, Bergen	20 x 9 x 57 11,000	Gneiss	- 26	1974
Kastbrekka, Trondheim	15 x 8,5 x 8 10,000	Greenstone	- 25	1978
Stathelle **	irregular 20,000	Limestone	- 25	1978
Stathelle **	irregular	Limestone	- 25	1982
Jordalen, Bergen	25 x 9 x 50 11,000	Gneiss	- 30	1982
Glomfjord	16 x 25 x 170 60,000	Granitic gneiss	- 33	1986

* Converted for commercial reasons in 1986 to unfrozen food storage.

** Established in quarry excavation.

Table 1 Cold storages in Norway (modified from Frivik, 1986).

2 THERMAL CONDUCTIVITY TESTING

As part of a continuing study of the low temperature behaviour of thermal conductivity, three 0.4 x 0.4 x 1.0 m rock slabs were tested in the high precision apparatus which was developed by Brendeng and Frivik (1974). In detail, the apparatus is very complex with peripheral shields to ensure constant thermal gradients and one dimensional heat flow. The principle and layout of the apparatus is given on Figure 1. The upper and lower faces of the rock slab are maintained at constant temperature over the full area of the specimen (0.4 m x 0.4 m) but the precise measurement of heat flux is undertaken only over the 0.2 x 0.2 m central core of the specimen. Temperatures as low as -160°C are achieved by circulating liquid propane; detailed control of the temperature distributions is achieved using a network array of electronically controlled heating elements and thermistors.

Two of the three rock specimens which are reported here were taken from the Glomfjord and Kastbrekka sites discussed in this paper. The third was a mica gneiss which was expected to provide further appreciation of .the fundamentals of thermal conductivity behaviour with low temperature.

Results of the thermal conductivity testing are summarized on Figure 2. These results are obtained for specimens which had been oven-dried, evacuated, water-saturated and frozen. It was not possible to test the specimens in a thawed condition without separate thawing and drying due to concerns for short circuiting the test apparatus as the result of possible moisture release upon thaw. A summary of selected properties of the tested rock specimens is provided on Table 2.

Property	**Glomfjord Granitic gneiss**	**Kastbrekka Quartzitic greenstone**	**Mica Gneiss**
Quartz content, %	21	31	28
Density g/cm^3	2,67	2,66	2,73
Porosity, %	0,75	1,16	0,81
Point Load Tensile Strength, Mpa (average)			
perp. to fol.	6,5	6,0	3,9
paral. to fol.	7,5	9,0	8,3

Table 2. Summary of properties of thermal conductivity specimens (directly from Utheim, 1988).

The test results (see Figure 2) indicate the following:

- quartz content has a direct influence on the magnitude of thermal conductivity as suggested by Frivik (1981);

- quartz content appears to exert a significant influence on the rate of increase of thermal conductivity with decreasing temperature;

The dependence of thermal conductivity on quartz content seems inconsistent when one compares the results on Figure 2. Quartz content alone would suggest that the results for mica gneiss should be closer to that of the Kastbrekka quartzitic greenstone than to that of the Glomfjord gneiss. This anomoly appears to be due to layering of the mica gneiss specimen which results in lower overall thermal conductivity perpendicular to the layers.

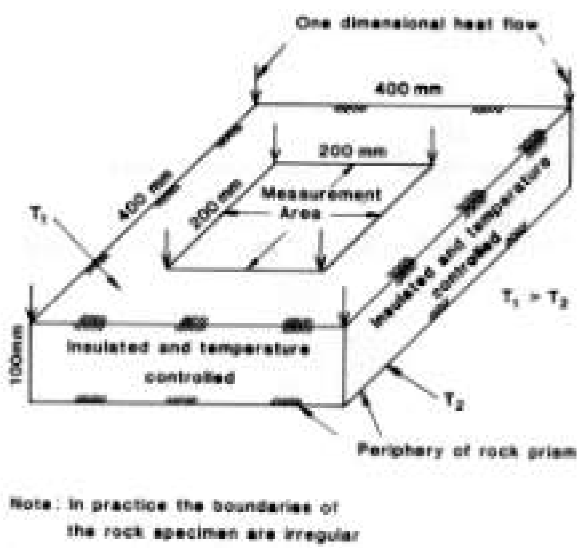

Figure 1. Schematic of apparatus for measurement of thermal conductivity of rock slabs at low temperature.

3 GLOMFJORD AMMONIA STORAGE

In 1984, Norsk Hydro initiated construction of a chilled ammonia cavern at their Glomfjord plant (see Figure 3 for location) to increase storage capacity and avoid stress corrosion problems which they had experienced with pressurized spherical tanks on surface. A refrigerated cavern storage of liquid ammonia at a temperature of -33°C and essentially atmospheric pressure was selected for economic and safety reasons (Nordmo, 1988). The 60,000 m^3 cavern was commissioned in March, 1986 and has functioned to the complete satisfaction of the owner.

The cavern serves as a buffer storage for ammonia which is imported by boat as illustrated on Figure 4. A typical boat delivery consists of 10,000 to 20,000 tonnes of liquid ammonia; the annual throughput of the storage is about 80,000 tonnes. The ammonia is pumped from the storage at a rate of about 17,000 tonnes per month but about 65 percent of this is immediately recirculated back to the cavern. About 540 tonnes per year returns from the fertilizer plant at a temperature of about 15°C. The temperature in the storage is held constant by condensing ammonia gas from the storage with two screw compressors on surface, each with a capacity of 300 kW.

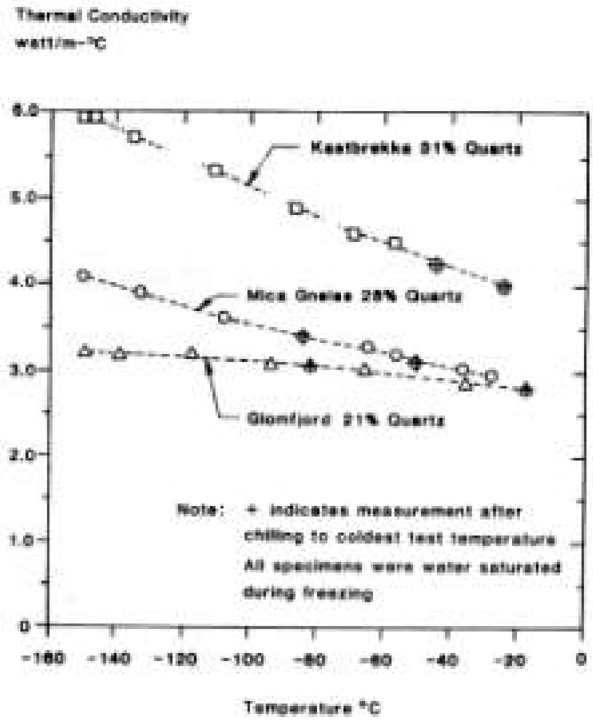

Fig. 2 Results of thermal conductivity testing.

Fig. 3 Site Locations.

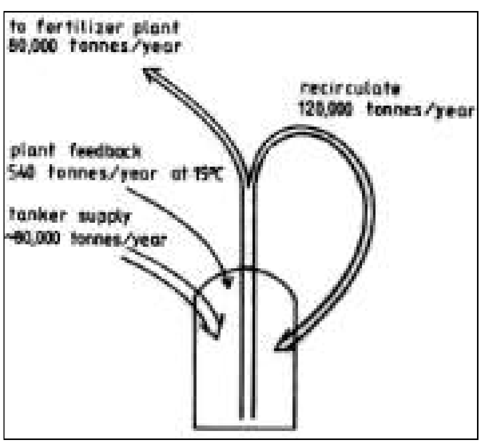

Fig. 4 Ammonia flow to and from the cavern.

3.1 Cavern Geometry

The cavern is shaped as an hexagonal doughnut (see Figure 5) with the roof of the cavern at El.-50. The cross-section is horse shoe-shaped and has a maximum height of 25m (see Figures 6 and 7). The cross-section width varies: in the two parallel, main caverns the width is 16m whereas in the end caverns it is 10 m. The floor of the caverns is blanketed with concrete and slopes towards the two pump shafts (see Figure 5).

A 540 m long access tunnel which was used for construction ramps down from ground surface to cavern level at a slope of 1 vertical to 7 horizontal (see Figures 5, 6, and 7). Ground surface varies from El.5 to about El.50 in the cavern vicinity. A water curtain was drilled above the cavern from a drift off the main access tunnel at El.-20 and was commissioned before cavern excavation. The water curtain boreholes are spaced at about 5 m and are about 35 m long; they slope from El.-20 at the access tunnel to El.-25 at their end point. The water curtain boreholes are supplied with heated, fresh water at about 15°C and a total potential of about +5m with respect to sea level. The water level in the access tunnel is maintained at about El.-29 by the same water supply system.

The access tunnel was isolated from the storage caverns by concrete plugs in each of the entrance tunnels at cavern level (see Figures 5 and 6). A double plug was also specially constructed at El.-51 after cool-down difficulties due to water leakage were experienced with the plugs at the cavern. Two vertical shafts at one end of the cavern contain all pipelines and . monitoring communication lines for operation of the storage; the shafts are identical such that one serves as a reserve fur the other.

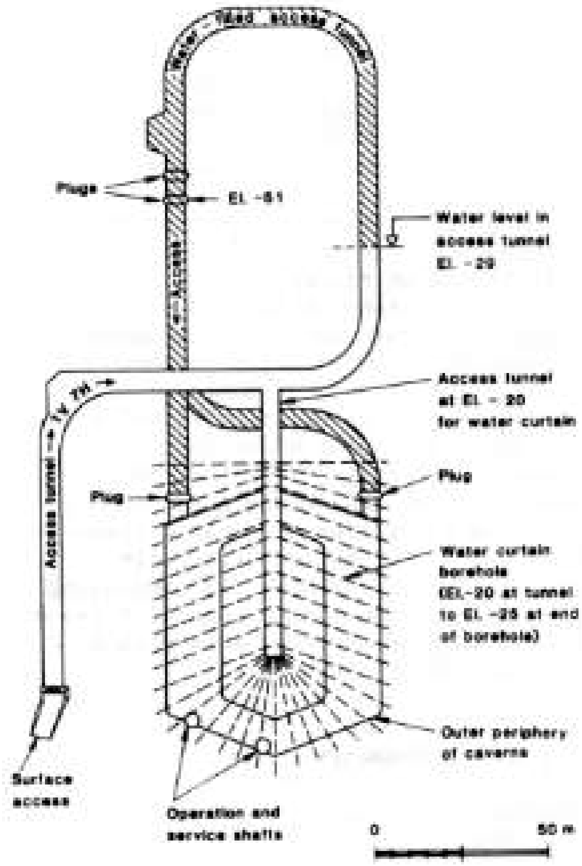

Fig. 5 Plan of Glomfjord ammonia storage.

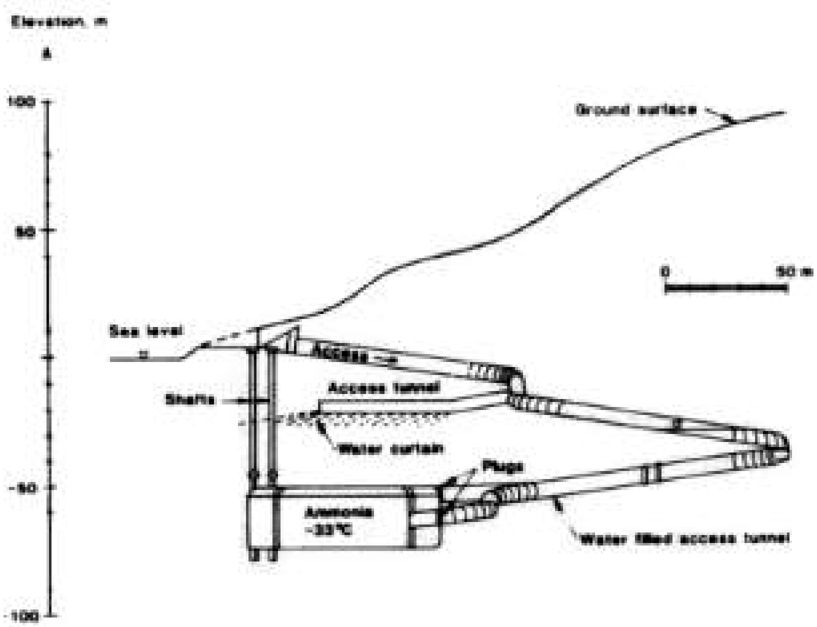

Fig. 6 Cross-section of Glomfjord ammonia storage.

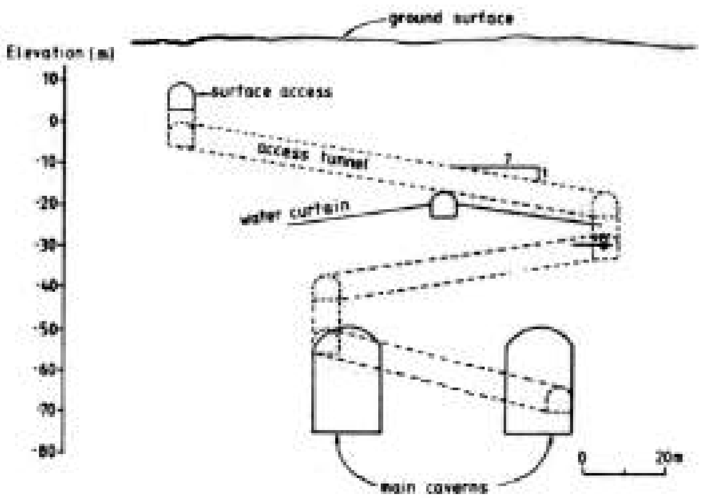

Fig. 7 Cross-section of main caverns

3.2 Geology and Rock Reinforcement

The rock consists of granitic gneiss with quartz layering parallel to the foliation (Nilsen. 1981).On average the quartz layering amounts to about one to two percent of the thickness perpendicular to the foliation planes. Analysis of drill cores indicates the mineral distribution given on Table 3.

Potassium feldspar	40-50 %
Plagioclase feldspar	20-30 %
Quartz	15-25 %
Biotite	5-8 %
Epidote	2-3 %
Amphibole	1-2 %

Table 3. Mineralogy of Glomfjord granitic gneiss (Nilsen, 1981).

Three fracture systems have been identified. One follows the shallow dipping (200-25°) foliation planes and the other two are steeply dipping. one of the steeply dipping sets is quite smooth with some calcite and clay infillings. Overall, however, the rock is only sparsely jointed. the volumetric joint count being 0.2 to 0.5 fractures/m (Lervik, 1988) .No significant weakness zones were intersected by the cavern and more than half of the borehole water loss tests indicated Lugeon values less than 0.01 ($\approx 10^{-16}$ m^2).

Four hundred 3 m long, 20mm diameter rock holts were installed in the roof of the cavern as a precautionary measure when the top heading was excavated. Plans for the use of shotcrete were dropped due to the good quality of the rock. Rock bolts were also installed in the shafts' vicinity for the added safety of the pumping facilities. Extensive cement grouting was conducted in the vicinity of the shafts to stop water inflow and at the double plug in the access tunnel (see Figure 5) to ensure gas tightness of the plug.

3.3 Commissioning and Operating Experience (Nordmo, 1988)

Cool-down of the cavern to -20°C was successfully conducted over a 4 month period beginning in early April, 1985. Air was then displaced from the cavern and further cooling to -33°C was initiated using liquid ammonia. Within two days of the start of the ammonia cooling, the liquid level, temperature and pressure of the cavern began to increase. Subsequent investigations confirmed that water was leaking into the cavern from the access tunnel with indirect evidence that the leakage could be as high as 5 m^3/h. Prior to the ammonia chilling there had been no water inflow whatever.

The ammonia cooling difficulties are thought to be caused by dissolution of ice by ammonia. The water inflow problem was resolved for the most part and it was at this time that the double plug at El.-51 (see Figure 5) was adopted as a safety measure to prevent any possibility of ammonia gas leakage up the access tunnel. Whether the access tunnel is completely full of water or not between the double plug and the plugs at the cavern is unknown.

Operating experience since commissioning of the cavern in March, 1986 has been very satisfactory (Nordmo, 1988). Less than 25 percent of the refrigeration capacity is normally required to maintain the temperature of the caverns at -33°C. Measurements of the water content of ammonia which is delivered to and extracted from the storage do indicate, however, that some water continues to leak into the cavern. On the basis of these measurements, the water inflow is about 0.014 m^3/h.

3.4 Thermal Monitoring

Twenty-one thermistors were installed in three vertical boreholes at the time of construction. These are labelled T1, T2 and T3 on Figure 8. The thermistors are installed above the cavern and below the water curtain between El.-25 and El.-50. Eight thermistors are also installed within the cavern itself on a vertical line in the vicinity of the shafts.

Recent measurements of the thermistors in the rock mass above the cavern are reported on Figure 9. Note that the horizontal positions of the thermistors on Figure 9 are plotted according to the minimum distance to the nearest cavern centreline. The thermistors in boreholes T1 and T3 show a definite frozen zone below El.-40 and the temperatures in the vicinity of the storage are currently decreasing at about 2 to 3°C per year. This is in contrast to observations at borehole T2 where no freezing seems to have occurred although there is a steady decrease in the temperatures from March, 1988 to February, 1989 of about 0.3°C. The observations at borehole T2 are surprising but consistent with water leakage into the cavern around the access tunnel plug.

The thermal profile measurements within the cavern indicate slightly cooler temperatures at the cavern roof (Lervik, 1988). The temperature in the liquid near floor level is about -32°C; in the vapour phase near the roof it is about -33.2°C.

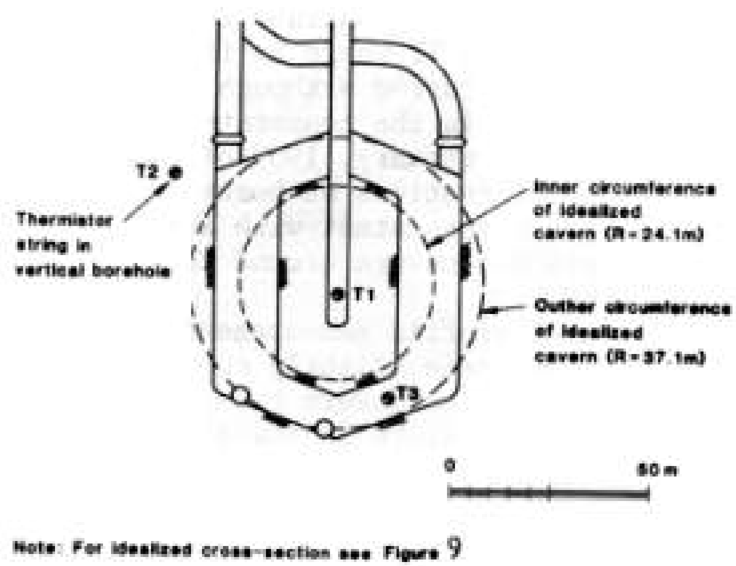

Fig. 8 Model geometry and location of thermistors at Glomfjord

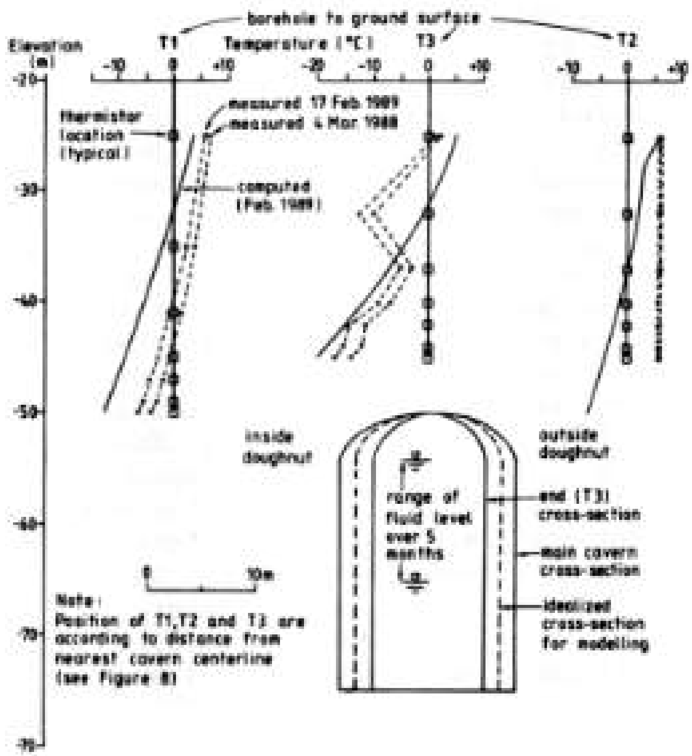

Fig. 9 Comparison of computed and measured temperatures at Glomfjord

3.5 Preliminary Modelling and Analysis

The storage was modelled as a doughnut as illustrated on Figure 8. The idealized cavern cross-section was assumed to have the average excavated width (13 m, see Figure 9) and the radius of the doughnut was selected such that the volume was equal to that of the actual storage (60,000 m^3). The modelling was conducted using the axisymmetric finite element method with a uniform initial rock temperature of 6°C and with the cavern temperature varying according to records of operation including cool-down. The temperature in the water curtain vicinity was assumed to be held constant at 5°C. The thermal conductivity of the rock mass was assumed to be according to that reported on Figure 2. The heat capacity of the rock mass was assumed to vary linearly from 753 J/kg-°C at 20°C to 544 J/kg-°C at -100°C. Assumed rock mass density was 2700 kg/m .

Computed temperature distributions for February, 1989 are reported on Figure 9 and the computed heat load on the storage from the surrounding rock mass is given on Figure 10. In general the computed temperatures are colder than those which have been measured, suggesting that the rock mass actually has a lower thermal diffusivity than assumed due to a lower thermal conductivity in reality and/or a larger heat capacity and density. Exceptions to this generalization occur in boreholes T3 and T2. In T3 the observed temperatures are colder than expected above El. -35; this cannot be explained on the basis of variation in cavern temperature nor access tunnel influences. It also seems unlikely that the nearby vertical shaft (see Figure 8) in combination with a quartz-rich foliation bed could cause such a deviation (the foliation planes dip at about 20° approximately in the direction T2-T1-T3).

Fig. 10 Computed heat load on Glomfjord storage

The deviations between observed and measured temperatures in borehole T2 are possibly related to water flow in the rock mass, a factor which the modelling has not considered.

The computed heat load from the rock mass (see Figure 10) as of 1 February, 1989 is 99.8 kW and continues to decrease slightly with time. The current disposition of energy for the Glomfjord storage is presented on Table 4.

Heat removed by compressors:	-162 kW
Recirculated ammonia:	
(i) heat gain	+ undermined
(ii) pump input	+ 3 kW
Water inflow to cavern	+ 3,5 kW
Back flow from plant	+ 3,8 kW
Computed heat flow from rock	+ 99,8 kW
Unaccounted	+ 51,9 kW

Table 4 Heat budget for Glomfjord cavern

The major unknown heat source in the Glomfjord heat budget is that due to recirculation of ammonia which is pumped to ground surface and returned to the storage. The amount of heat transferred to the recirculated ammonia is dependent on the -performance of the insulation which shields the recirculation pipelines. Computations indicate that, if the recirculating ammonia temperature increases by only 2.7°C, the rate of energy delivery to the storage will be 51.9 kW. Such a temperature increase is thought to be realistic.

An assessment of the heat gain by the recirculated ammonia is recommended to complete the heat budget for the Glomfjord cavern. The preliminary results which are reported here, however, indicate that there is no excessive heat load due to storage of liquid ammonia in an unlined rock cavern and that the heat load can be predicted on the basis of laboratory testing and theoretical modelling to reasonable accuracy.

4 KASTBREKKA ICE CREAM STORAGE

Jokeris operates a 10,000 m^3 storage for ice cream at Kastbrekka which is about 8 km south of Trondheim city centre (see Figure 3). Excavation of the cavern commenced in late 1976, cool-down was initiated about a year later and the first storage of goods occurred in February, 1978. Full commissioning did not occur until the summer of 1978.

A profile of the cavern is given on Figure 11. The storage was constructed by excavating a 15 m high cut into a bedrock ridge. Seismic studies indicated the competent rock profile which is illustrated on Figure 11. A 35 m long access tunnel was excavated to the competent rock where the full cavern cross section, 15 m wide and 8.5 m high (see Figure 11), was excavated for a total length of 85 m. A concrete slab was poured on the cavern floor to facilitate handling of goods.

The cavern is founded in a greenstone formation which contains quartz rich layers. It appears that at least the upper half of the storage is intersected by one of these quartz-rich layers which was also the source of the 31 percent quartz specimen for which the thermal conductivity results on Figure 2 were obtained.

The rock mass is extensively foliated by sub.horizontal foliation planes. The foliation planes are smooth but tight. Two other steeply dipping fracture sets also occur yielding a relatively heavily fractured rock mass with 10 to 20 steeply dipping fractures per m^3 (Lervik, 1988). As a result of this heavy near vertical fracturing, the rock mass permeability is quite high. Water inflow during cool-down greatly reduced the effectiveness of the refrigeration plant and it was necessary to partition the cavern in order to achieve the operating temperature within a reasonable time (Frivik, 1978). As the cavern became colder the water leakage concentrated itself as a single stream which finally froze when the air temperature was -13°C.

Extensive reinforcement of the rock mass around the access tunnel was required. In the competent cavern host rock, however, a total of only 10 to 15 rock bolts was used in spite of the high fracturing intensity. This small amount of rock reinforcement was possible because no significant weakness zones were encountered and because it was expected that the long term stability would be enhanced by freezing. No significant instabilities have been experienced (Lervik, 1988).

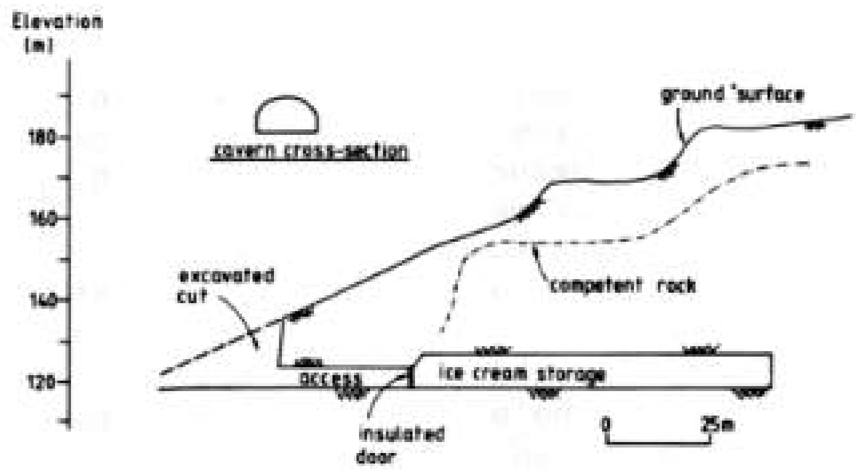

Fig. 11 Profile of the Kastbrekka storage

4.1 Preliminary Analysis

A neat budget for the Kastbrekka storage is given a Table 5 on the basis of current operations which are practically steady state. The estimated heat load on Table 5 is computed using the thermal conductivity data which is given for the Kastbrekka specimen on Figure 2. A constant thermal conductivity of 4 W/m°C was assumed. The Tokheim and Janbu (1982) steady state solution for a cylindrical cavern with spherical ends was used to estimate the heat load with an assumed average outside temperature of +5°C and a storage temperature of -25°C.

Heat removed by compressors:	- 59,2 kW
Loss due to door operations:	+ 4,3 kW
Loss due to fans in storage:	+ 6,7 kW

Loss due to lights in storage:	+ 2,5 kW
Loss due to defrosting:	+ 2,8 kW
Loss through end wall:	+ 7,0 kW
Loss due to new food storage:	+ 0,1 kW
Estimated rock heat load:	+ 39,1 kW
Excess heat load	+ 3,3 kW

Table 5 Heat budget for Kastbrekka cavern

Summation of the heat balance indicates that the energy delivered by the compressors is 3.3 kW less than the sum of all losses. If it is assumed that this discrepancy is due to an overestimate of the heat load from the rock and the rock mass thermal conductivity is reduced to eliminate all of the heat load, the computed thermal conductivity is 3.66 W/m-°C. Such a thermal conductivity is considered a reasonable approximation of the average £or the cavern which is influenced by the occurrence of greenstone (K=3.14 J/m-°C) in the cavern floor. It is also consistent with the back-analysis which was reported by Frivik (1978) during start-up.

5 DISCUSSION

The objective of this continuing study is to compare observed, full-scale, thermal behaviour of fractured rock masses with that which would be predicted from laboratory measurements and theoretical computations. Within the scope of this preliminary study, this has been demonstrated. More accurate comparison will require better definition of the elements of cavern heat budget and more precise modelling techniques.

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ARCHIVES IN ROCK CAVERNS

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ABSTRACT: The National Archive of Norway has since 1978 had its archives placed in underground storage facilities. 4-storied concrete buildings were erected in rock caverns, giving 100,000 m^2 storage area in which are installed 100 km of shelves.

PREFACE

The National Archive of Norway has its storage facilities in concrete buildings erected inside rock caverns. The caverns have a rock cover sufficient to resist impact stress from air raids with heavy bombing. All entrances and ventilation intakes are secured against chemical and biological weapons.

In the 100,000 m^2 storage area, a total length of 100 km of shelves are installed. Fixed racks as well as compact rack systems are used.

LAYOUT

The store complex consists of two parallel rock caverns, both with cross section 16.00 m width and 15.00 m height. One cavern is approx. 85 m long and the other 70 m long. The rock pillar between the two caverns is 16 m wide. The technical control centre, which furnish the store rooms with adequate climatic conditions, is located at the end of one of the caverns. The tunnel, which connects the store rooms to the administration centre, is placed perpendicular to the store caverns and leads to the elevator shaft.

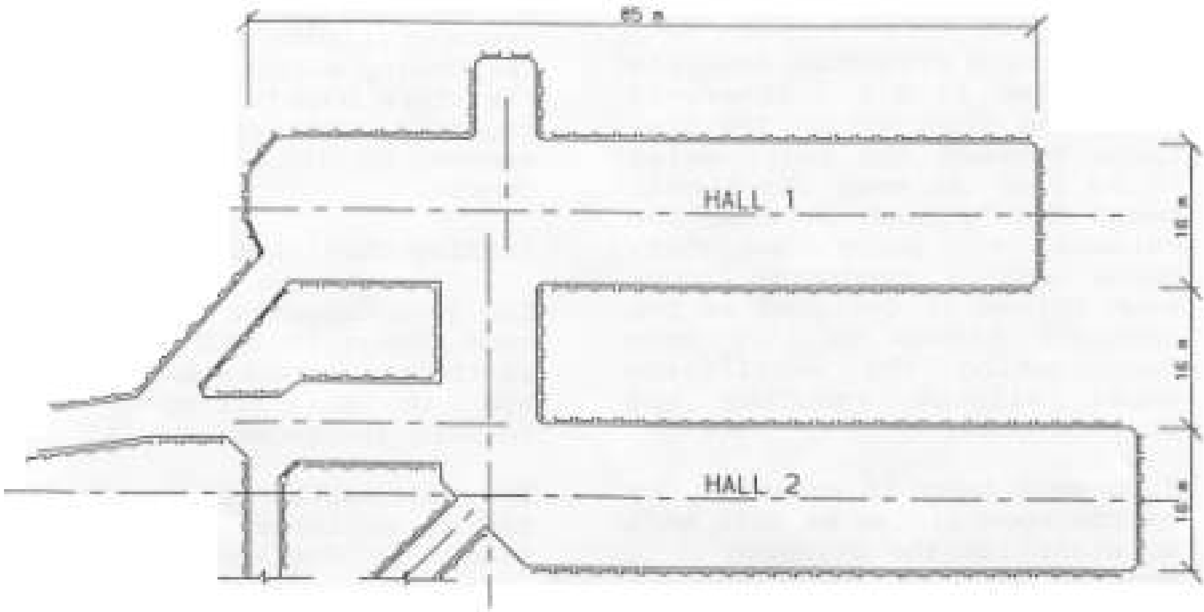

Fig. 1 Layout

Before deciding upon the necessary rock cover and orientation of the caverns a thorough investigation programme regarding the geological conditions in the area was carried out.

The ground water level had to be monitored to take proper precautions to prevent decrease of the gwl. and to design a proper drainage system from the store rooms.

ENGINEERING GEOLOGY

The top 7.5 m section of the caverns was blasted in one primary operation step. The rest of the 15 m high cavern): 7.5 m was then taken out in one final drilling and blasting operation.

The roofs of the caverns were secured against rockfall by systematic installation of rock anchor bolts.

The major part of the rock surface was also covered with reinforced shotcrete.

CONCRETE STRUCTURES (SECTION)

Inside each of the rock caverns a 4-storied reinforced concrete building was erected.

The concrete structure consists of columns at 5.6 m intervals along the outer walls. The distance between the outer walls is 14.5 m. At each level main beams are placed between the columns to carry secondary beams and a concrete floor. Each column is designed as two separate halves to give room for running the ventilation ducts without reducing the storage area.

Each main beam is extended out to the rock lining at both ends to stabilize the building against shaking in transverse direction due to explosion forces.

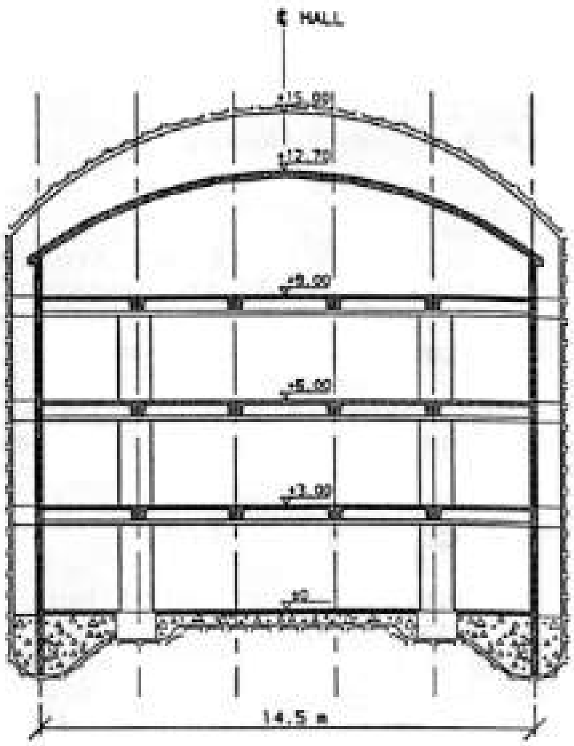

Fig. 2 Typical section

The concrete structures are divided by expansion joints at approx. 30 m distanse.

On top of the 4th floor concrete vaults are constructed and covered with a membrane lining to prevent leakage from the tunnel roof.

In the connecting tunnel concrete corridors are built corresponding with floor levels in the store room buildings, leading to the elevator shaft with access to the administration centre.

Loading Conditions

The load capacity of all store room floors is 1200 kpm pr sqm in the store racks area and 200 kpm in the transport area between the racks.

The roof vaults are designed to resist an impact from falling blocks of 3000 kgs.

HVAC

A very extensive air conditioning system is installed to ensure that all store rooms are given the ideal climatic conditions to take optimum care of the valuable archives.

EXECUTION OF WORKS

The excavating of the caverns was completed over a period of 12 months, and the construction of the store buildings was executed in 15 months.