NORWEGIAN ROCK EXCAVATION, PAST AND PRESENT

Norway is said to be the stone country. Of its surface area more than 50% are mountains and only 2.8% agricultural flatland where the bedrocks are hidden by soils and loose materials. In most areas elsewhere the loose cover is thin or lacking, thus leaving exposed rocks in the larger parts of the surface.

This leaves the rocks easy to explore and the early Norwegians had a good opportunity to examine the possibilities the rocks could offer for tools, weapons and gems. The problems for them were, however, that with their primitive tools almost all rocks were too hard for mining. The mining or other use of Norwegian rocks was therefore not practiced on a larger scale before the 16th century. But from then on the challenge of mining the hard bedrocks for underground works, has given rise to a continuous development in tools and equipment for rock excavation.

As in most other countries, the Norwegian underground works started with ore mining. Later, railway tunnels and tunnels for hydropower plants were excavated. Hydropower development became after the second world war the most important field of underground activity in Norway. Both design and construction had a rapid development in this period.

During the last three decades the construction of road tunnels has had a steady increase. Today three sub-sea road tunnels have been constructed and there are firm plans for several more, the deepest one is planned to go down to more than 500 meter below sea level.

The construction of underground rock storage caverns for hydrocarbon products started about 20 years ago. After the oil and gas exploitation in the North Sea, this activity has increased. Today, some 4 million m of hydrocarbon storage caverns are in operation or under construction.

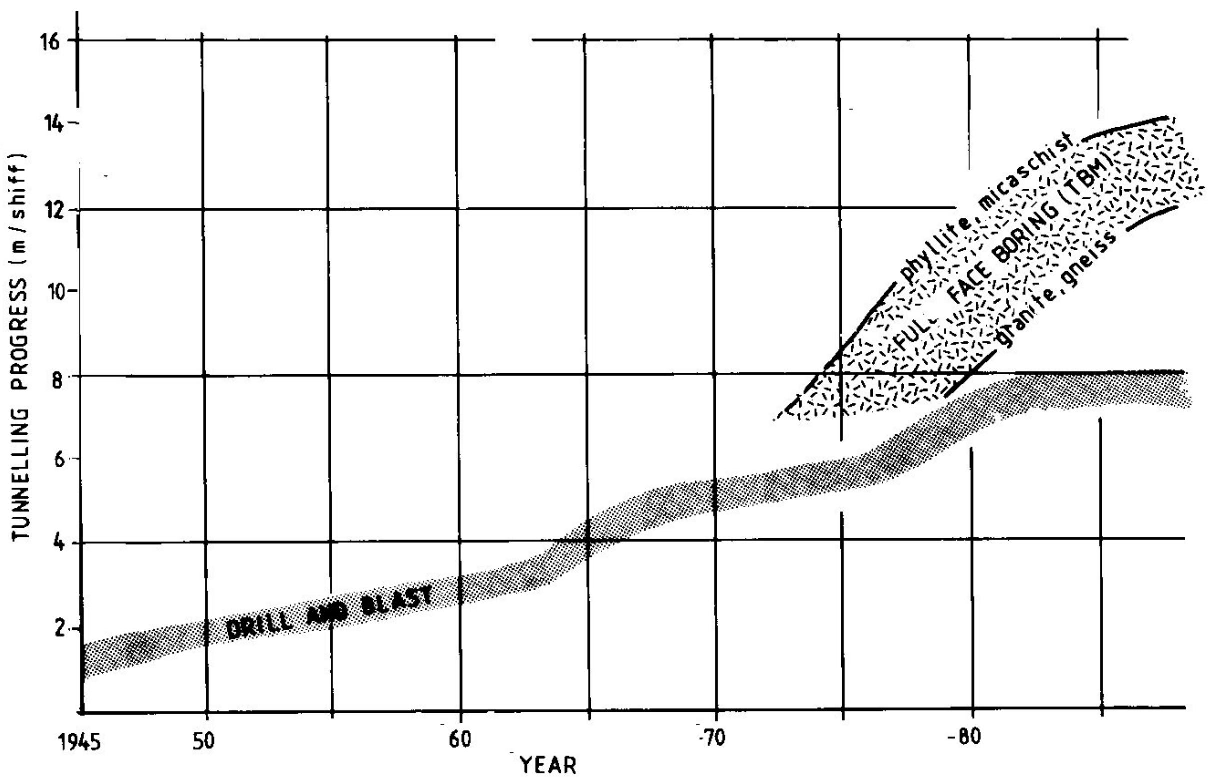


Fig. I. Development in Norwegian hard rock tunnelling progress (rock supporting works excluded}

The Norwegians have always been quick in adapting new technologies in underground construction and to find the possibilities the new improvements could offer in rock excavation. From the primitive use of hammer and chisel through hand drilling and gun powder till today's full face boring machines the tunnelling capacities have increased enormously. As an example a 5 km long transport tunnel in one of the larger Norwegian mines can be mentioned. It took 76 years to excavate this tunnel 200 years ago, today it would take only 1.5 years.

Vøringsfossen waterfall 182 meters high. Steep topography, abundant precipitation, natural lakes used as water storage reservoirs and predominantly hard rocks have given ride to an extensive exploitation of hydropower in Norway. (Statkraft)

HISTORY AND DEVELOPMENT

Morten G. Johnsen
Astrup & Aubert A/S

1. Early rock excavation

As in most other countries the rock excavation and tunnelling in Norway started with mining. The earliest signs of mining found in Norway are small quarries or tiny tunnels for tools in the Stone Age some 3-4000 years ago.

The more organized rock mining started in the 15th century. The rocks were mined by hammer and chisel, and the tunnelling work was done manually. In 1624 promising silver ores were discovered at Kongsberg, encouraging the development of a mining industry. For this purpose many German miners moved to Norway to teach the natives the modem mining technique of that time which had been developed in Germany's ore deposits in Harz.

From then on the Norwegian hard rock excavation technique has been keeping stride with the development internationally.

During the 16th to the 19th century most rock excavation was done by the so-called "stoking", meaning that the rock at the tunnel face was heated by building a fire. A lot of small cracks developed in the rocks which at the same time became brittle, enabling the miners to excavate some 10 cm of the rock, after it had been cooled down by spraying water on the hot surface.

This method was rather time-consuming since much work was involved not only in wood supplying and fire building, excavation and transport, but also in getting air into the tunnel and smoke out again. A usual progress rate was less than 1 meter per week. This method was in use up to about 1890.

2. The drill and blast method comes into use

The possibility of using gunpowder in mining had been known for a long time, but lacking was the know-how of making the holes in which to place the powder. In the beginning of the 18th century the first tool of iron for making blast holes was produced and up to 0.8 m long holes could be made.

The dynamite came about 1867 and at the same time the steel drill was presented. This was the beginning of a new era in tunnelling.

From the end of the 19th century the rock excavation technique developed in mines was transferred to other tunnelling fields such as rail- way- and hydropower tunnels. The drilling was all done by hand and the mucking was also done manually. Often the muck had to be wheeled out of the tunnel by hand carts, but horses were used wherever possible. Although drilling machines existed by 1900, they were apparently not used in Norway until some years later. The reason may be that the early drilling equipment was not powerful enough to penetrate the local hard rocks, or possibly that delivery and service was below an acceptable standard. The cost of labour was cheap in those days, and manual work common.

The 5,3 km long Gravenhalsen railway tunnel on the line between Oslo and Bergen was built between 1895 and 1905. The tunnel passes through a high mountain, and it was possible only to build the

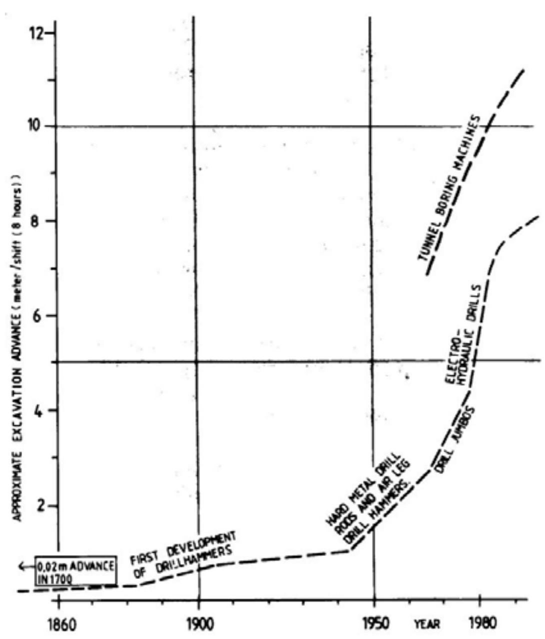


Fig. 1.1 Development of tunnel excavation

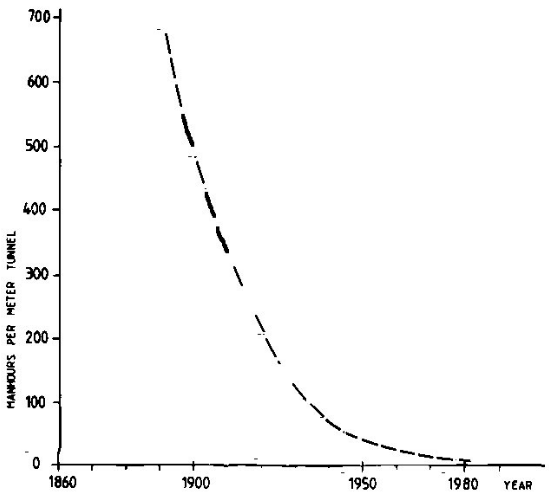


Fig. 1.2 Time consumption in tunnel excavation

tunnel from its two portals -East and West. Access from the West was easiest, and 4200 m were driven from that face. Even if the cheapest method of tunnelling at the time was to use as much manpower as possible, the contractor on the West portal acquired two hydraulic drill hammers which were powered by water from a nearby small waterfall. These achieved a progress rate of 52 m per month against an estimated 60 m, a reasonably satisfactory result.

The East side proved more difficult. Work started manually, but the rock was so hard that progress was only 9.5 m per month compared with the planned progress of 15 m per month. So the contractor brought in some electric drills of a type previously used in the Alps and with good results, but the hard Norwegian rocks proved to be too much for them. In the end compressed air (pneumatic) drills had to be used. These achieved a progress rate of 40 m per month in the 5 m^2 pilot tunnel, considered quite satisfactory at the time. The contractor of the Gravehalsen Tunnel records that it required 700 man-hours per metre of tunnel built. At that time tunnelling continued for 24 hours per day for 6 1/2 days a week. If tunnelling should have taken as many man-hours today, it is unlikely that many tunnels at all would be driven. The pilot tunnel probably had six men per shift per face, with nine men per shifts per face for the enlargement from 5 m^2 to the full face 24.5 m^2.

The technical development has speeded up progress continually. By 1940, the 700 man-hours per metre were reduced to about 70, and by 1950 to 60. Since then it has been nearly halved about every ten years, and today the time consumption is down to 7 man-hours per metre tunnel.

3. Mechanisation speeds up the tunnelling progress

As mechanisation was introduced in tunnelling, various machines were tried. Cyclope was one type of stope hammers, with the disadvantage that the driller had to turn the machine back and forth the whole time. Later the RWT 801 -Rotation Water Telescope -came into use. This was much easier to handle, but still too heavy and needed to be fastened to a pillar in the tunnel.

Another problem at that time was the drilling steel, which had to be forged and tempered by a blacksmith. Average life length of a drill rod at the time was 25 cm. A big stockpile of drills of all kinds was needed to drill just one round.

Shortly after the Second World War hard metal drills were introduced in conjunction with the air-leg support, making drilling much easier. Further development was to use air-leg machines on ladders so that the machine did not have to be held all the time. By 1949, advances of 120 m/month were reported from an 18 m^2 heading, and 95 m/month from a 30 m^2 drive. In both cases there were five men at a time on the face, with three shifts per day.

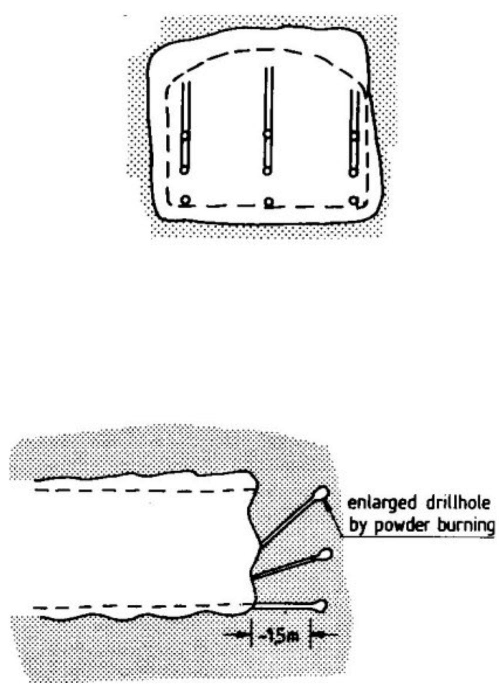


Fig. 1.3 Only 9 holes were used to blast the 12 m^2 tunnel at Follafoss power plant in 1919

To reach the entire cross-section of tunnels larger than 6 m^2, drilling jumbos were introduced. W hen the Nes power plant was bunt in 1963-67, two rigs were used in the 65 m^2 tunnel. 40 to 45 m/wk was achieved with five men on the face.

Fig. 1.4 Three drill hammers on airleg

By 1985, progress rates had reached 100 m/wk in a 16 m^2 tunnel, and without a full night shift. This is due not only to fast drilling, but also to fast and efficient loading and mucking out.

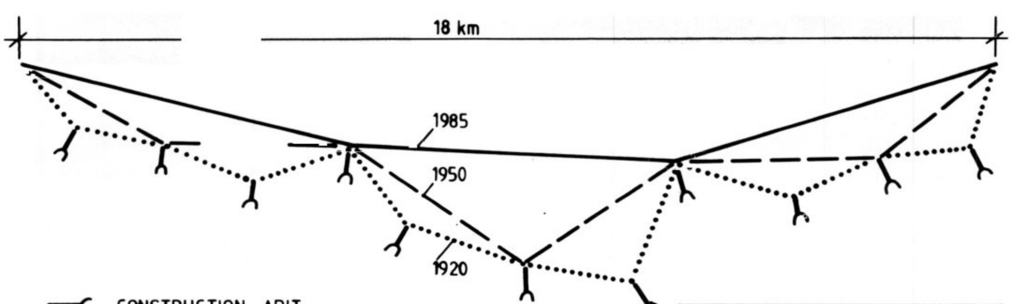
When hand drill was used it took one hour to drill a 20 cm deep hole of one-inch diameter. The heavy pneumatic machines could drill a two-inch hole, 60-80 cm deep, in one minute. The electro- hydraulic drills that followed increased penetration rates further to 200-300 cm a minute, depending on rock hardness. Today we can drill a two-inch hole, 300 cm deep, in one minute.

Norwegian rocks are mostly hard rock types and wear and tear on machinery is considerable. For this reason full face TBMs came late, contractors preferring the known and well proven drill + blast technology with which they were quite familiar. From the late 1970s TBMs came into use, first in greenstone and clay-schist, later in mica-schist, phyllite, gneiss and granite. The present-day advance rate with TBM in hard rocks is reportedly more than 150 m per week.

Many of the early Norwegian tunnels were long and had to be excavated manually. Access was often difficult because of the steep mountain sides. Where possible, access roads to several adits were built to give the maximum number of working faces, reducing the total construction time.

In 1913 the Aura power scheme was planned with 16 adits for a 17.9 km long tunnel. This tunnel was not, however, built until 1949 and then with only four adits, which also allowed the tunnel to be 1 km shorter. The improved excavation methods by then made this possible.

Fig. 1.5 Ladder drill rig at Nes power plant 1964



YEAR	TUNNEL LENGTH	LENGTH OF ADITS
1920	21,7 km	3,5 km
1950	20,4 km	2 km
1985	18,4 km	1 km

Fig. 1.6 Different tunnel alignments for a tunnel between the two reservoirs "A " and "B The construction time is 21/2 years in all cases

Another example is the 31.5 km long tunnel of 65 m^2 cross section at the Nes power station. It has four adits only and the distance between two of them is as much as 11 km. Not only has this saved more than 2 km length of the main tunnel, but there is also the saving in the total length of adits.

4. Shaft excavation development

A shaft sinking technique was developed at Killingdal mines in Norway. The method was first used for pressure shafts in 1947 at Abjøra power plant. The 660 m long shaft was driven from the surface at an angle of 41° .The same equipment was used later to sink a 960 m long shaft at 36° at Hjartdøla power plant. Both shafts were built with three shifts of five men each, including the lift operator at the top. The work included drilling + blasting, loading muck, pumping out water, and installing rails. These rails were later used for transporting the steel penstock liners and imbedding them in concrete.

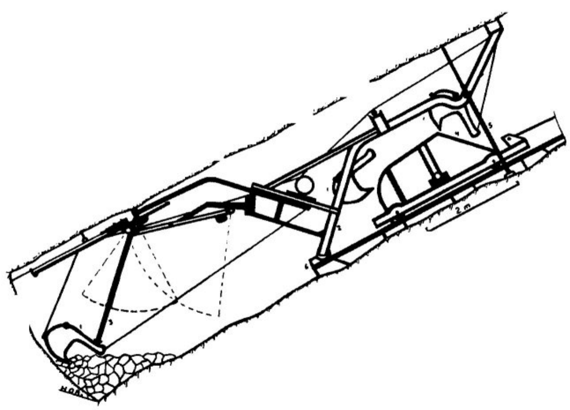


Fig. 1.7 The Larsen shaft sinking rig could be used at angles up to 42°

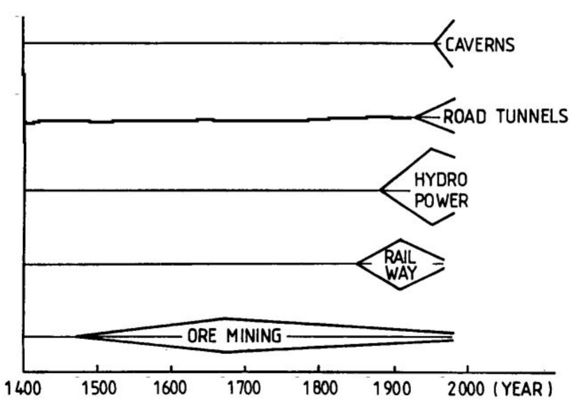


Fig. 1.8 History of Norwegian hard rock tunnelling

The building of shafts has become much simpler since 1950. The main development is the possibility of raising shafts from the tunnel below, rather than having the laborious job of sinking them from above. The advantage is that the muck will slide down and can be removed via the tunnel. Another advantage of raising shafts is that in mountainous regions it may be difficult or impossible to reach the top of the shaft except by helicopter, and the cost and trouble of such transport -building a road as an alternative -can be saved.

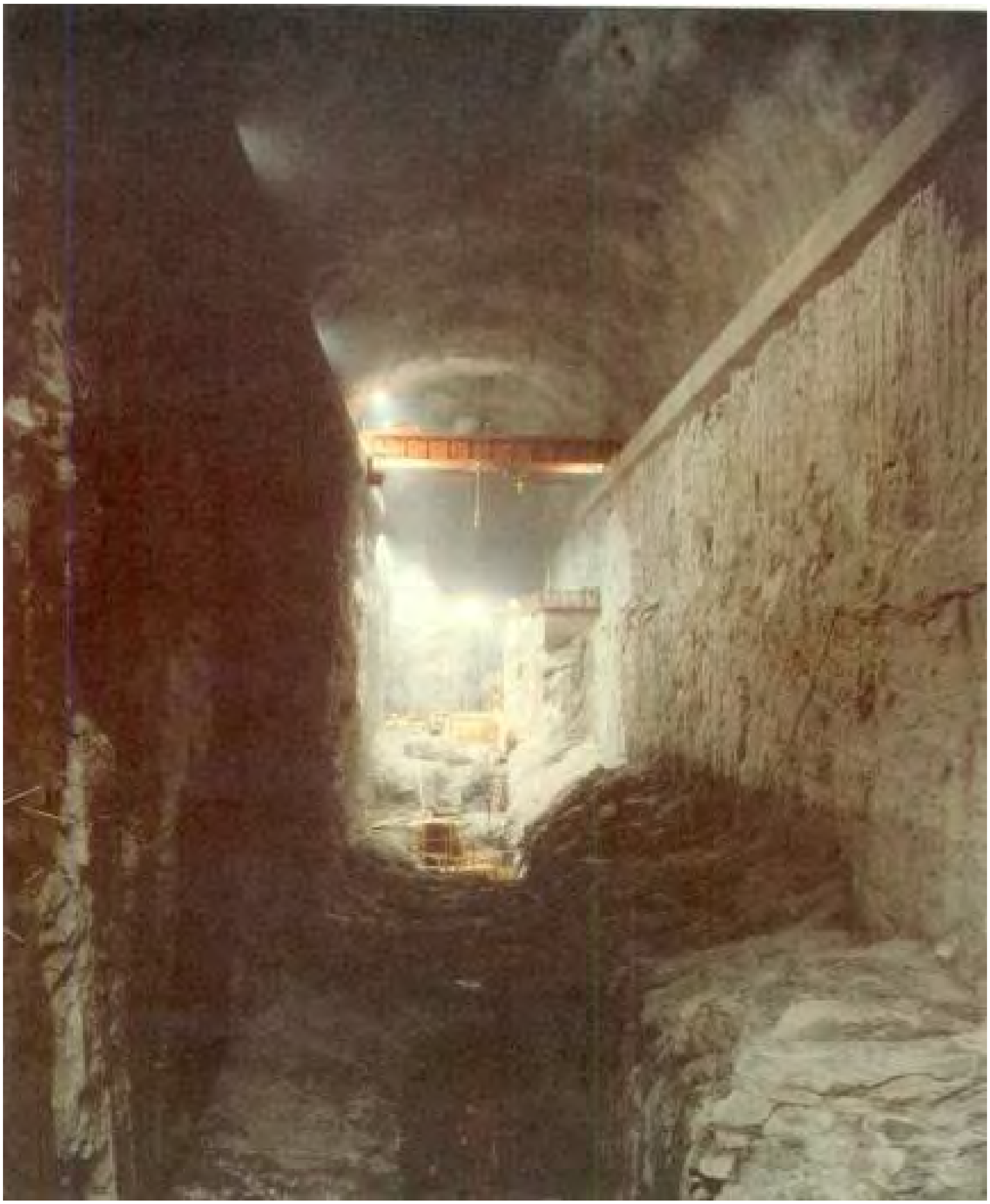
The first development was to drill a centre hole down as much as 120 m, and then blast the shaft to full final area from a platform manoeuvred by a winch at the top. The next development was the introduction about 1960 of the Alimak lift, which made shaft raising much easier, and also allowed angled shafts down to 40° to be run. At Borgund, a 1200 m long power shaft was driven in two parts -the longest 900 m section with two shifts of three men only, achieving a progress rate of 20-30 m/wk.

Later, also TBMs have be en employed to drive shafts. The 1250 m long shaft at Tjodan hydro-power plant was driven in 1984 by a Jarva TBM at an angle of 41°; and in 1985 the same TBM drilled a 1440 m long shaft of 3.2 m diameter at Nyset-Steggje hydropower plant at the rate of 80 m/wk.

5. Future trends

The cost of tunnelling has risen, partly because of better working conditions such as the elimination of night shifts, shorter working days and less shifts per week, and partly because tunnellers receive relatively high wages.

In spite of the lesser working hours per week the tunnelling progress has increased as mentioned earlier, and the relative tunnelling costs have decreased. This is explained by the steady improvement of the equipment, and the better planning and execution of the work. Norwegian contractors are in the forefront where it comes to hard rock tunnelling.



Saurdal underground hydropower station after excavation shown prior to installation work. Norway has nearly 200 underground hydropower stations in rock, a design with many advantages as described in chapter 12.

Eastern end of the Seljestad road tunnel. The development towards steadily cheaper and faster tunnelling makes it possible to shorten and improve old curved mountain roads.(Photo: Fjellanger Widerøe A/S)

TUNNELLING CAPACITIES AND TUNNELLING EQUIPMENT FACTORS

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1. INTRODUCTION

The current supply of in ex pensive hydroelectric power is the most important factor for the growth of the Norwegian industrial community. Hydropower development after World War II became in a short time the most important field of activity for Norwegian construction contractors. Later, road tunnels and storage caverns have attained increasing excavation volume.

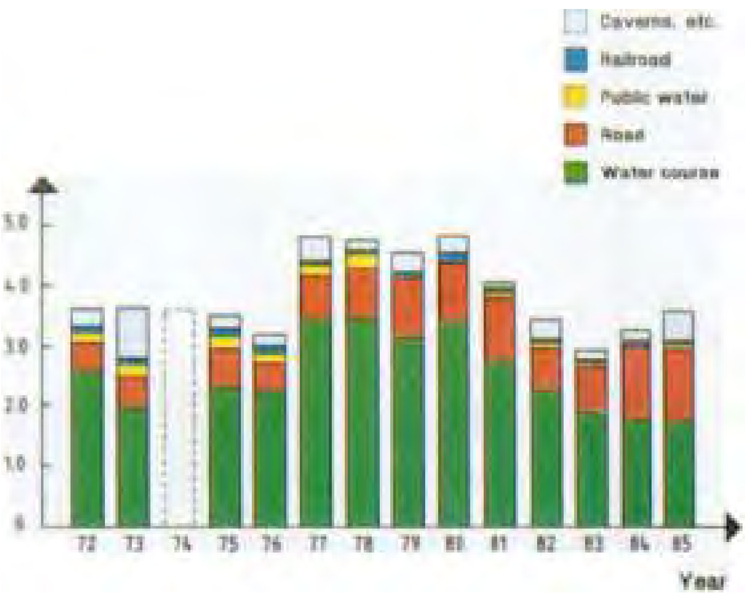


Fig. 2.1 Yearly volumes of underground rock excavations for construction purposes during the last 13 years (Mining not included}

2. EXCAVATED VOLUMES OVER THE YEARS

Most of the power stations are sited underground. One reason is the possibility of locating it in rocks well suited for excavation of tunnels and high-walled power houses without too ex pensive rock support. The excavated material is useable for building purposes in the area, especially for rock fill dams.

As shown in the block diagram Fig. 2.1 under- ground excavations for power houses and tunnels for hydropower plants normally amount to 2 to 3 million m^3 a year. In average, during the 1972 to 1985 period, rock excavation for these purposes are about 65% of the total volume excavated for construction projects in Norway (mining is not included).

In the first part of this century the building of railroad tunnels was the main underground excavation activity. In the last decades road tunnelling has became the major interest on the communication sector. Railroad tunnelling amounts to only 3% of the total yearly excavated volume, while road tunnelling in average has amounted to 21,5% during the last 13 years.

Several larger road tunnelling projects are presently under construction (1987) and many new projects are planned. The increasing activity on this sector is expressed in the diagram for the years 1984 and 1985 in Fig. 2.1.

Most Norwegian road tunnels have a cross section around 50 m^2. The majority of hydropower tunnels have 10-30 m^2 cross sections. Commonly, public sewers and water tunnels have cross sections between 2 and 10 m^2. The volumes excavated in 1985 are depicted in Fig. 2.2.

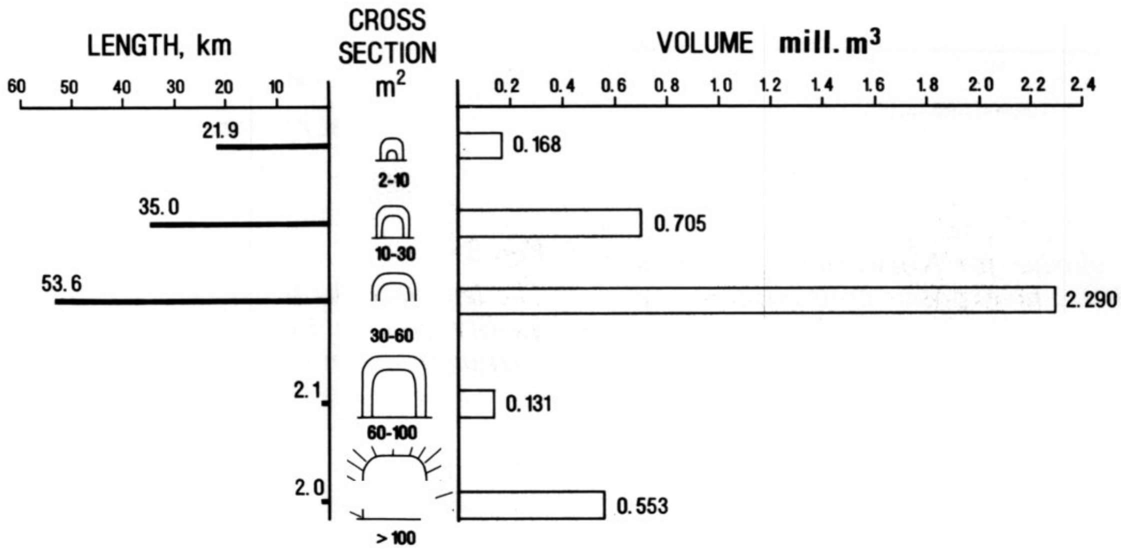


Fig. 2.2 Length and volume of various size tunnels excavated during the year 1985

3. DEVELOPMENT OF THE DRILL AND BLAST METHOD FOR TUNNEL EXCAVATION

Drill and blast methods in tunnelling have in principle not changed much during the last three decades. When looking at the separate factors on which this tunnelling method depends and their influence on the improvement of tunnelling progress, however, the development is consider- able.

In the 1960s

For tunnels with large cross section, the jacklegs were replaced by tunnel jumbos with two to three and even six booms. The first jumbos were "homemade" and specially designed by the con- tractors for each project. The bore hammers could now be made heavier and more powerful and that allowed drilling of 36/38 mm holes and later 43/45 mm holes with use of loose exchange- able drill bits. Wedge cuts were still mostly used. First at the end of the decade did the parallel burn cut come into use in larger cross-sections. Faster drilling rates, increased borehole diameters and subdivision of half-second-detonator series with millisecond series, made it possible to enlarge the blast rounds. The rate of advance in tunnel excavations was nearly doubled during this decade.

The blast rounds contained now more explosives than earlier and more rounds were detonated per week. The explosion fumes created unpleasant working condition, but not until the next decade did ventilation systems catch up with the actual need.

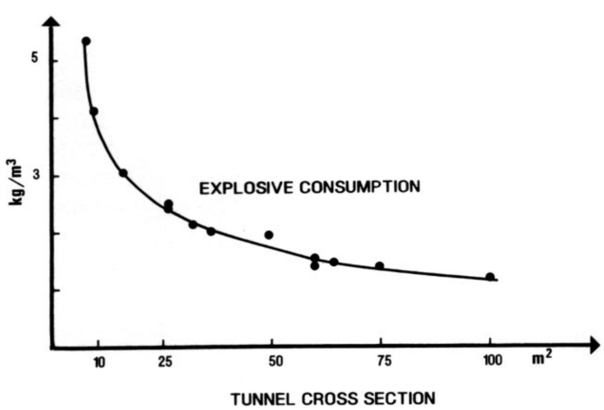


Fig. 2.3 The basic explosive for Norwegian tunnelling today is ANFO. The explosive consumption per m^3 of rock varies widely dependent on cross- sectional size and rock quality

In the 1970s

At the beginning of the 1970s the drilling equipment manufacturing, who had taken over the design and production of the drilling jumbos, developed effective tunnel rigs. The main interest for the contractors now was to increase the effectivity of the explosive charging process. Separate loading platform baskets, and later in the decade pneumatic ANFO-loading equipment, was mounted on the jumbo. The time needed for charging the holes of the tunnel round was reduced with about 20% and explosives to about half the earlier price, came into use.

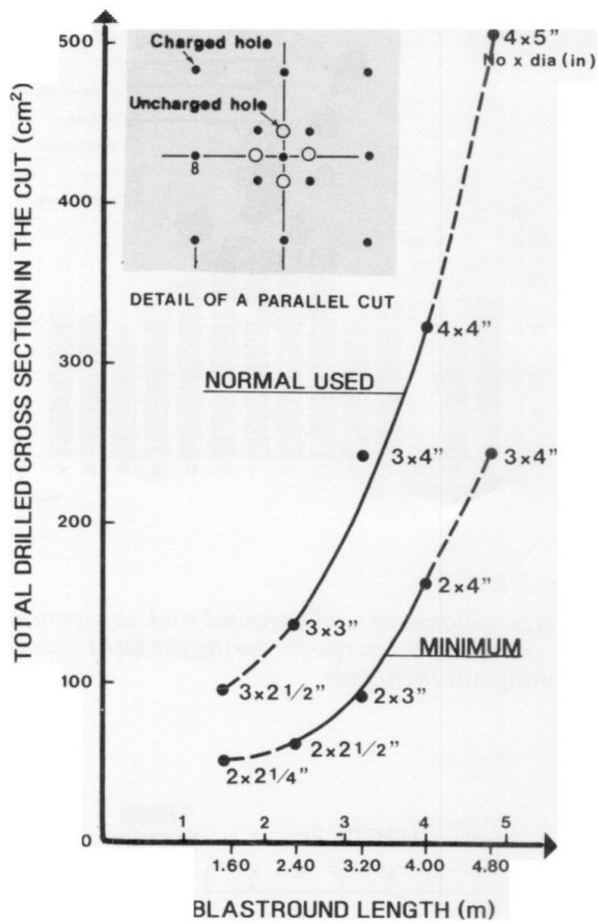


Fig. 2.4 The length of the blast round depends on rock quality and on the volume of the unloaded opening holes in the parallel cut

To reduce overbreak and to minimise rock support and lining costs, the owner called for smooth blasting for nearly every project. The drill booms used for the contour holes had to be equipped with instruments for direction and level control and the use of shorter spacing between the holes and the use of tube charges became a necessity.

By the middle of the decade the contractors became more interested in speeding up the drilling rate. The hit frequency of the conventional air-driven bore hammers had reached its speed limit. Higher feed pressure gave only destructed drill rods and bits and reduced drilling accuracy. Hydraulic bore hammers then came into use. With the new drifters the drilling rate was nearly doubled. The noise and mist level at the tunnel face dropped dramatically. The tunnel workers again got better working environment while the drilling, blasting and hauling cycles was shortened. The tunnel crew had to adapt to a new system in their work. The "floating cycles" -which means that operations are not fixed in time but rather continuously linked together -came into use. Thus tunnel crew members had to be skilled in almost any kind of tunnelling work in order to be able to take over and carry on at any stage of the works.

In the 1980s

This decade started with dim prospects for tunnelling projects, especially for hydropower development. The contractors interest was more focused on the efficiency of the house building process than on the tunnelling process. Still, some improvements in tunnelling methods have been made: laser-optic tunnel direction control, new electrical and non-electrical detonators specially adapted for tunnel blasting, better mixing- and loading equipment for ANFO- and ANFOpolystyrene-explosives and better noise isolated operator cabins on the drilling rigs have come into use.

Towards the end of this decade the drill and blast tunnelling method will be even more efficient. The drill hammer producers are launching a new generation of hydraulic hammers with extremely

high hit frequencies. With 45 mm hole diameter in rock of average drillability drilling rates up to 300 cm/min has been obtained in hard rocks. With this new generation of hammers tunnelling progress rates are expected to increase 10-20%.
Computerized steering systems for the entire drilling process have been developed and tested with good results.

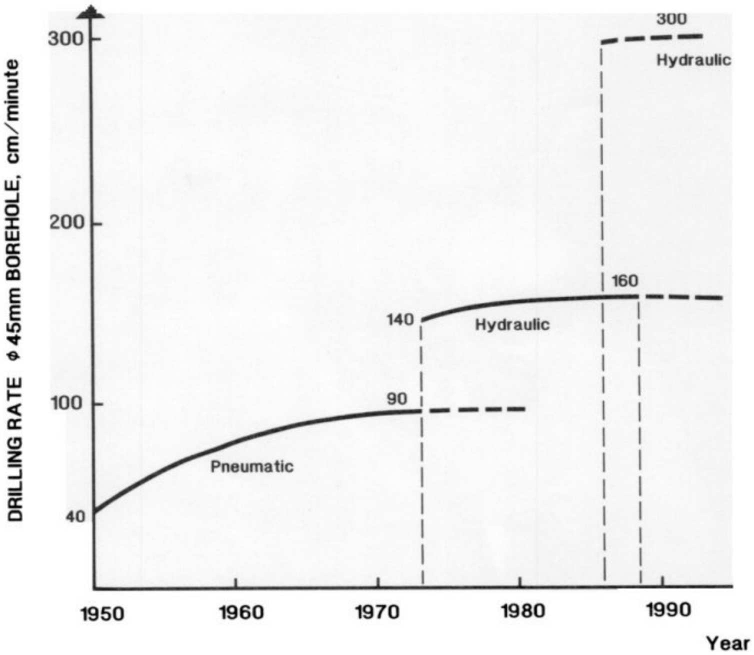


Fig. 2.5 Blast hole drilling rates have been greatly improved during the last decades

4. FULL FACE BORING OF TUNNELS

Up till today most Norwegian tunnels have been excavated by the drill and blast method. In 1972, the first tunnel-boring machine (TBM) was hired for full face boring in Trondheim green schist. Over the years, full face boring has become an accepted method for tunnel excavation especially for small tunnels even in hard rock. In the Oslo area alone about 38 km sewer-tunnels have been drilled through sedimentary rock. In Bergen City, road tunnels with diameter 7.8 m have been drilled through schist and granitic gneiss. So far the highest production reached in one year was 23.5 km in 1983 (Fig. 2.7). The average cross section of the full face drilled tunnels in that year was 12.5 m2.
With tunnel length above 4-6 km and cross sections below 12-15 m^2, full face drilling seems to be the most economical tunnelling method even in hard rock. For larger profiles the drill and blast method is still the cheapest method.
The TBMs have also been used for shaft drilling in hard rocks, where the method is profitable in shafts longer than 1000 meters. The high capacity this method offers, makes it interesting also for shorter shafts with diameter larger than 3.5 m.

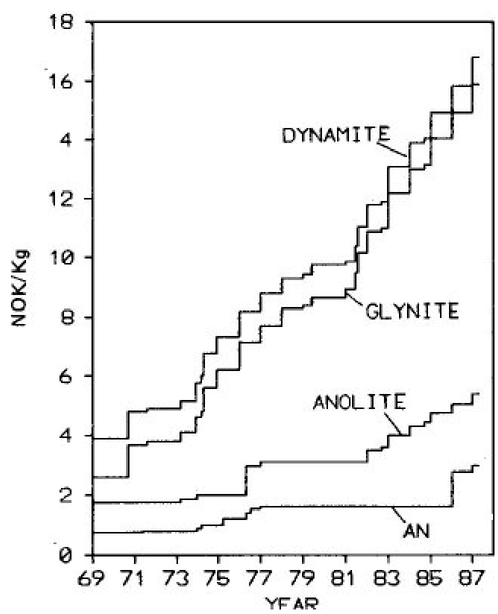


Fig. 2.6 The price gap between cartrigde-NG-explosives (dynamite, glynite) and bulk ANFO is still widening (Prices in NOK, 1987 exchange rate: $1 = 6,80 NOK)

5. FUTURE DEVELOPMENT

Today we have experienced tunnelling progress by drill and blast of more than 120 m/week working 100 hours.

The greater accuracy obtained with the computerized systems for drilling and the higher drilling rates will make it profitable to drill a greater number of holes in the parallel cut. In this respect it can be expected that the length of the blast rounds in tunnelling can be increased from today's 4,5-5,0 meters to 6-7 meters around 1990. This will increase the drill and blast tunnelling progress by 10-20%.

Today we have experienced weekly advances of TBM tunnels up to a little less than 200 m in hard rocks like gneiss and granite. The rapid improvement and development makes this method more and more attractive compared to drill and blast in hard rock tunnelling.

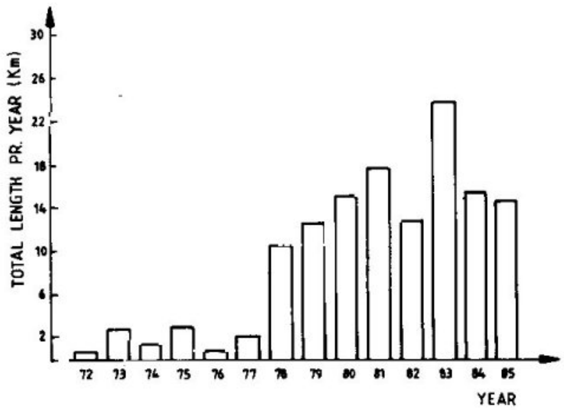
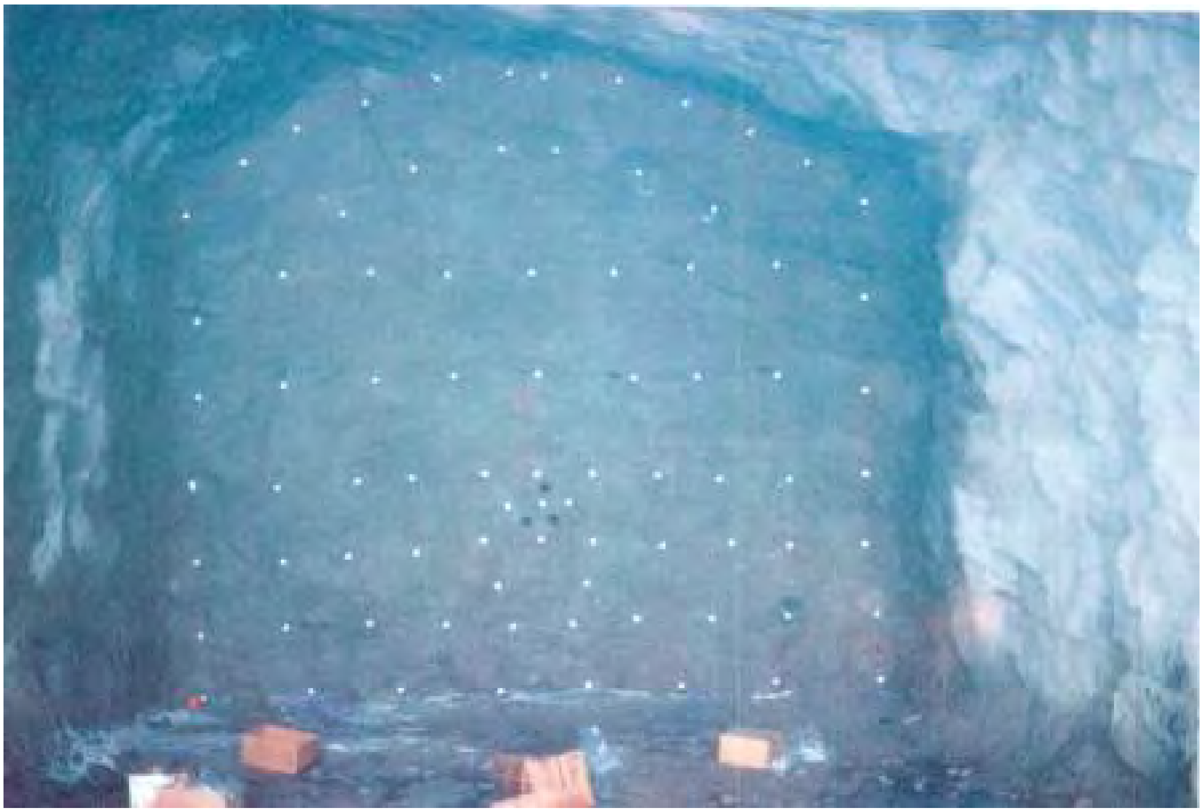


Fig. 2.7 Total lenght of full face drilled tunnels in km per year.



Drilling pattern in a 40m^2 tunnel drilled with a computer-controlled jumbo (Norwegian Geotechnical Institute)

The non-electric ignition system has many advantages and is coming more and more into use in tunnelling. (Norwegian Geotechnical Institute)

Old modern drilling jumbo with ANFO charging equipment built in 1974 (Norwegian Geotechnical Institute)

Blasting of a massive rock cofferdam at Raanaasfoss power plant. The intake gates are located from 15 to 35 meters distance from the cofferdam (Norwegian Geotechnical Institute)

II
NORWEGIAN BEDROCK CONDITIONS

Norwegian bedrocks are mostly made up of hard rocks. Another characteristic feature is that the rocks: are old, mostly older than 250 mill. years. During this vast period of time the rock masses have undergone many geological events, like a number of earth movements causing faults to develop. Such faults or weakness zones have been a main reason for many of the stability problems encountered in Norwegian tunnels.

Throughout the ages, numerous geological events have formed a variety of rock types. Apart from young, sedimentary rocks (found only in the North Sea and in the Norwegian Sea), most types of rocks are present in the Norwegian bedrocks. Among these is one of the oldest rocks found on earth, and also another type formed during the strongest metamorphism more than 50 km down below the surface.

A simplified engineering geological map of Norway would in fact be a map showing only one type of rock Rut the map would have a lot of lines which illustrate fracture zones, weakness zones or faults where tunnelling problems may occur. The rest are hard rocks, mostly with fair or good tunnelling properties.

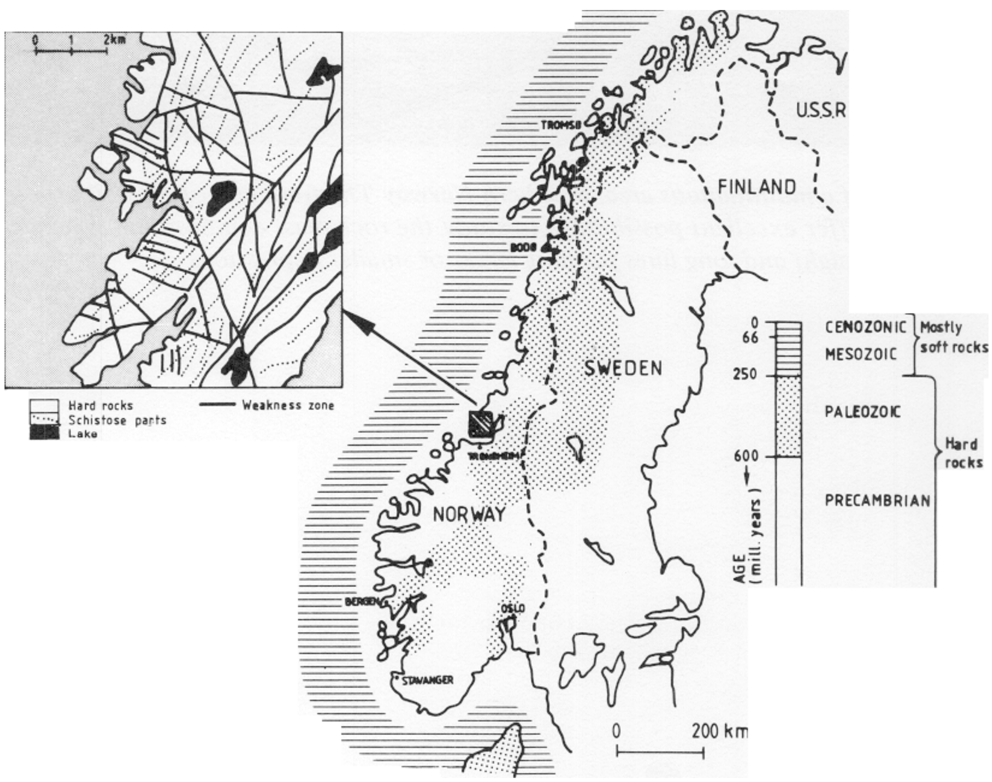


Fig. II.
A simplified engineering geological map of Norway

Deglaciation following several ice ages during the last 1 to 2 million years have eroded the surface of the rocks. All the earlier, weathered rocks that were outcropping, have been shuffled away along with the erosional products leaving an almost unweathered rock surface. As the ice erodes strongest where the mechanical properties of the rocks are weakest, faults and weakness zones form often depressions in the topographical picture and they are thus easy to point out from surface observations.

THE GEOLOGICAL HISTORY OF NORWAY

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1. INTRODUCTION

The Norwegian continent is part of the Baltic shield, one of the bigger and older continental shields in the world. It includes Fenoscandia (Norway, Sweden, Finland) and the western part of Russia. The dominating rocks originated in medium and late Precambrian. The Baltic shield is limited by the Caledonian mountain range on the western edge and by the much younger sedimentary rocks towards east.

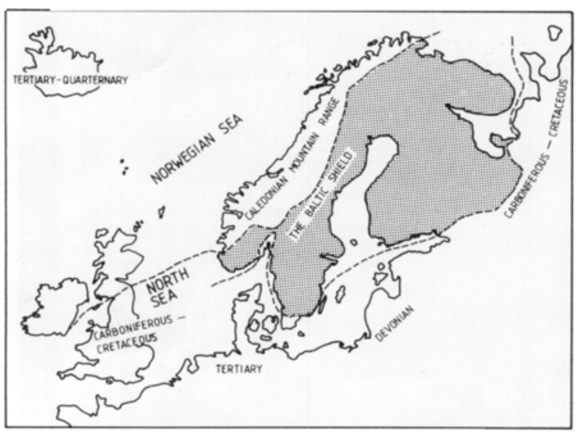


Fig. 3.1 The Baltic shield

Many of the geological events in Norway can be explained from the continental drift theory, i.e. the movement of two continents relative to each other during long periods, possibly 400-500 mill. years. Such a process starts with rifting of a larger continent into two continents which start drifting apart. As the distance between them increase, the ocean formed is acting as a sedimentation basin receiving erosion products from the surrounding landmasses.

An orogenic period begins when two continents start converging. All the sediments deposited in the basin are compressed and folded by this movement. When the two continents later collide, rocks are squeezed up to form a mountain range. Volcanism and hardening of magma in the crust are frequent processes during this orogenic period. During the following 50 -100 mill. years the mountains are eroded down to form a flat plain at sea level.

The geologists are able to trace four orogenic periods in Norway during the last 2000 mill. years. The last one, the Caledonian orogeny 450 mill. years ago, has been most important for the geology we find in Scandinavia today. Today Greenland is parting from Scandinavia and the

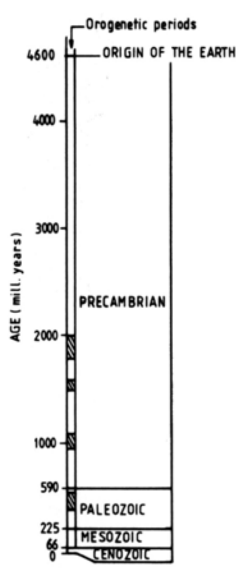


Fig. 3.2
General historical view

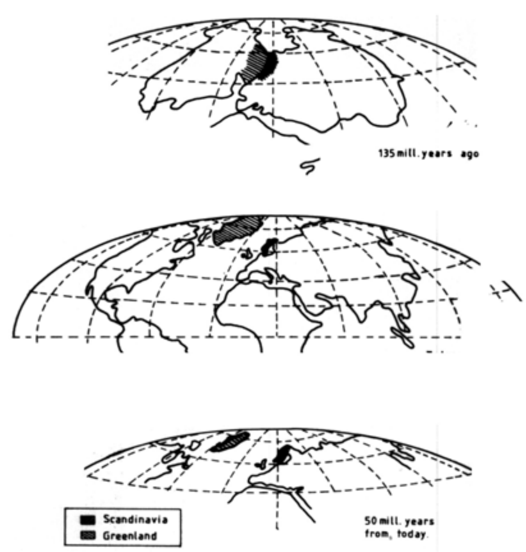


Fig. 3.3
Possible development of the continental drift

Norwegian Sea is again acting as a sedimentation basin. It is believed that we are now in an interim period, and that the next orogeny may possibly start in some 100-200 mill. years.

2. PRECAMBRIAN

2 500 mill. years back, the geologists believe, a number of small continents were present on the earth. Through a period of many collisions and orogenies these Precambrian continents were expanding. Because later events destroyed tracks from older ones, our past is still not fully under-stood. The closer we get to present time the more information we have.

Radioactive isotopes have given geologists a re liable method to determine the age of the rocks. As a result it is now possible to divide the following three orogenies :

2 000'- 1 800	mill.	years	ago
1 600 -1 500	-»-	-»-	-»-
1 100- 950	-»-	-»-	-»-

These events were the main reason for the tectonic activity in Precambrian. The first orogeny was important in northern Norway, the next two had greater importance in the southern parts of the country.

Precambrian rocks are occupying a great part of the continental crust in Norway. Precambrian windows are protruding where younger layers have been removed by erosion. Some of the oldest rocks on earth, up to 3 000 mill. years, are found in isolated places in Lofoten in northern Norway. These are mainly various types of gneisses, quartzites and more or less metamorphosed igneous rock types. Special attention should be paid to the rocks in the northwest- Norwegian gneiss-region where anorthosite, dunite and lots of the highly metamorphic rock called eclogite can be observed. It could be that these rocks have been formed under extreme pressure and temperature 40 -70 km down in the earth's crust.

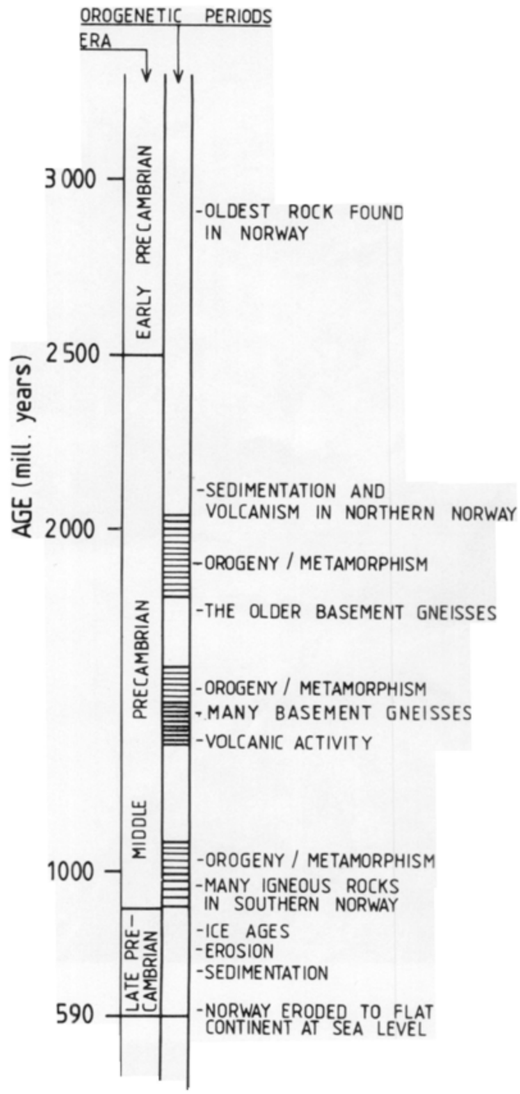


Fig. 3.4 Main geological events during the Precambrian era

Many ore and mineral deposits in Norway are found in connection with old Precambrian rock types. It is worth to mention:

- iron with quartzite
- molybdenium with granite
- titanium with anorthosite/norite
- copper with meta-greenstone
- quartz and feldspar in pegmatite of granitic origin

Towards the end of Precambrian it is assumed that the continent was eroded down to a flat area (peneplain) with small height differences. During the following geologic periods this peneplain was inundated by ocean and received coarse, later finer sediments.

Fig. 3.5 Pr
Fenoscand

In early Cambrian big fractures started to develop between the Greenlandic/Canadian and the Baltic shields. Norway and Greenland were drifted away from each other, and the ocean covered large parts of Norway. Huge amounts of erosion products like sand and gravel transported from the surrounding land areas were collected in this basin. The oldest sediments date 825 mill. years back, and the total thickness is 4 -5 km. Most of the rocks were formed from shallow water sediments. Petrified moraine (till) is a good proof of an ice age around 590 mill. years ago. A 400 m thick package of sedimentary layers between the tillite layers indicate a long period of glaciations. Tillite from this period has also been found other places in the world, and this type of rock has been used as a time limit between Precambrian and Paleozoic.

Fig. 3.6 Norway and Greenland start drifting apart at the end of the Precambrian era

3. PALEOZOIC

3.1 Cambrian -Silurian

As Greenland and Norway continued to part and the Precambrian crust was sinking, the size of the ocean increased. Layers of sandstones with traces from worms, fossilized shells and trilobites - the species that dominated the life in the ocean for millions of years -were deposited. The most characteristic rock type from Cambrian, is a shiny black shale called alunshale. This type of rock which often causes construction problems, contains 10 -40% carbon, traces of uranium, and sulphides. When exposed to air the sulphides are broken down to sulphates and sulphidic acid which "eat up" iron and concrete. Through the following periods, Ordovician and the oldest part of Silurian, the bottom of the sea continued to sink. New sediments brought in to sea from heights around were deposited in the basin. Powerful volcanism in the region was building up layers of basalt, often with sediments from the continent in between during more quiet periods. Sulphur from the volcanic eruptions

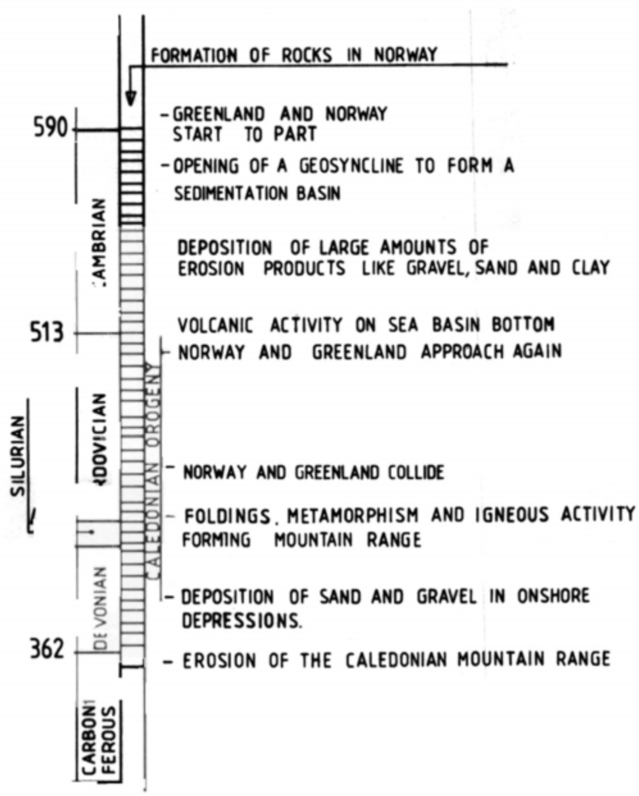


Fig. 3.7
Main geological events during the Paleozoic era

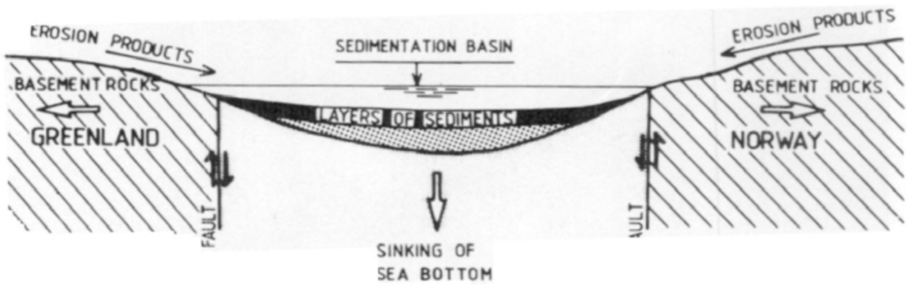


Fig. 3.8 Sedimentation of erosion products in the geosyncline basin

reacted with metals in mud and basalts to create bodies of pyrite. Today these are found in famous mines like Lokken, Røros and Sulitjelma.

As the environment on the surrounding continents changed, different types of sediments were deposited on the sea floor. Shells and organic remnants created clean limestones during quiet periods. An increase of erosional products like mud caused shale to form, of ten with layers of nodular limestone. The depositional process was slow. Geologists have calculated that it might have taken 1000 years for a 1 mm layer of limestone to be formed.

For unknown reasons, Greenland and Norway stopped drifting apart. In Silurian the two continents started to drift towards each other. The sedimentary layers on the sea floor were folded and compressed and continents were rising from the ocean. Basalts and lavas were erupted from volcanoes, and coarse materials were transported from the uplifted continents out into the ocean, to rest on top of the existing limestones with volcanic ash layers.

3.2 The Caledonian Mountain Range

Greenland and Norway collided in Devonian and created a new, enormous continent. The large layers of sediments and lava on the sea bottom were subject to compression and digenesis. Large bodies of rocks (napes) were pushed south-easterly on top of the continental crust. Along parts of the coastline greenstones, igneous rocks, and metamorphosed sedimentary rocks are representing the bedrocks today.

The movements continued, and towards the end of the Caledonian orogeny compression caused large parts of the old continental crust to be tom off and pushed southeast on top of younger strata. Napes found on the highest peaks in Jotunheimen and other mountainous areas show that they once covered large areas west of southern Norway.

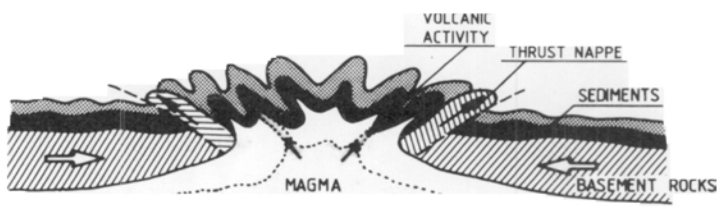


Fig. 3.9 The sea bottom with large amounts of sediments was squeezed, and rocks folded up to the Caledonian mountain range

3.3 Devonian and Carboniferous

The Devonian continent stretched from England/ Ireland to Spitsbergen, from Greenland to Russia. Large oceans of

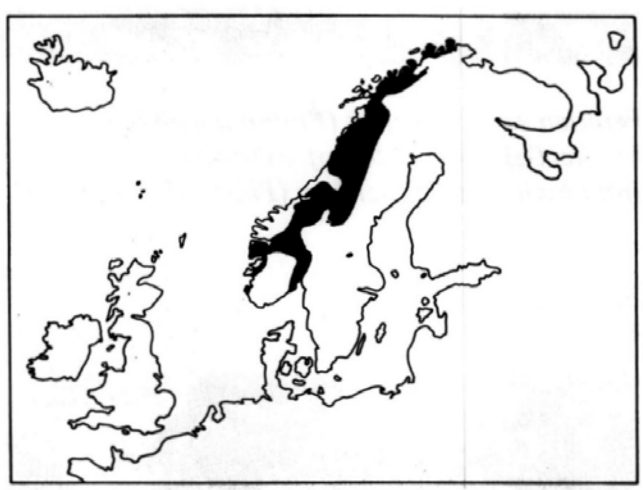


Fig. 3.10 Paleozoic rocks in Fenoscandia

fresh water formed on the continent. Eroded material from the young mountain ranges were transported in to these depressions. The weight of the sediments caused sinking of the bottom. Old Red is the most famous deposition of sands found in England. In Norway today only a few small Devonian fields are remnants of this large sedimentary sequence. The y are mostly found along the north-western coastline.
At the end of Devonian swamp forests grew in the southern North Sea region. In the next period, Carboniferous, forests spread all over the world and gave the origin to coal- and oil/gas deposits in many countries. Layers from Carboniferous are also found in the North Sea, although unessential for the formation of the fossil fuel deposits here.

3.4 Permian

After the Caledonian orogeny, the mountains were eroded down to a sub Permian peneplain. Greenland and Norway were drifting apart in Permian. Far east a new mountain range was rising in Ural, and rifting caused blocks in the south Norwegian crust around Oslo to sink as much as 1000 m. Rocks, eroded away elsewhere, were thus protected against later erosion in the young down- faulted basin. Lava erupted in the many regional fractures, many as long as 100 miles. It started with basalts, then rhomb-porphyry the interesting sequence which make the Oslo field famous among geologists. Plutonic magma bodies deep down in the crust caused the nearest surrounding rocks to be cooked to form migmatite. Limestone were re-crystallized to marble and shale to hornfeis. Parts of the magma containing gas and liquid, formed small ore bodies along numerous contacts. Erosion has later exposed the deep plutonic rocks like Larvikite and Nordmarkite after the Permian period.

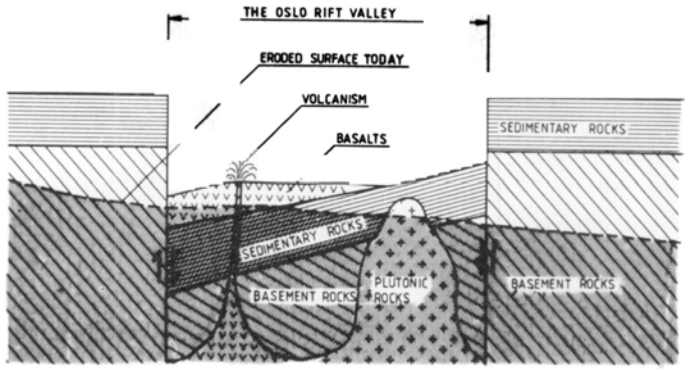


Fig. 3.11
Rifting with graben formation in the Oslo region

4. MESOZOIC

After a desert like climate in Triassic the weather changed to be more humid and the rivers again transported sands and gravel in to the North Sea and the Norwegian Sea which was beginning to open up.
In Cretaceous erosion made Norway once more to a flatland with rounded hills and shallow depressions. The less mass transport into the sea caused mainly organic sedimentation in the Jurassic ocean which is believed to be the main sources for the oil and gas reserves in the North Sea. The total thickness of sedimentary layers from Mesozoic age in the North Sea and the Norwegian Sea corresponds to 5-6 km.

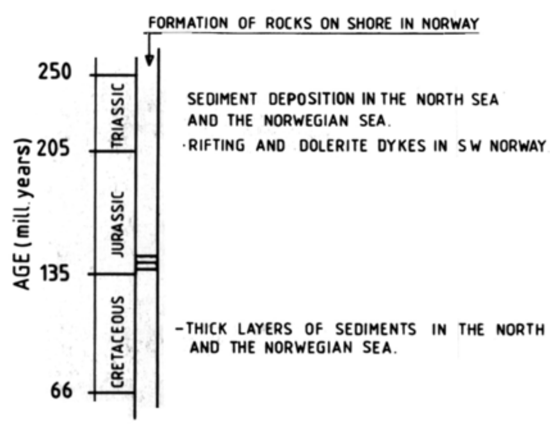


Fig. 3.12
Main geological events during the Mesozoic era

5. CENOZOIC

5.1 Tertiary

60 mill. years ago, in the beginning of the Tertiary age, Greenland and Norway were separated by a shallow ocean on the continental shelf.
58 mill. years ago Greenland drifted further away from Scandinavia and the Norwegian Sea opened up again to a deep ocean parallel to the mid- Atlantic ridge.
In early Tertiary the distance between Greenland and Norway increased with 2 cm each year. In the fracture zone along the mid-ocean ridge between the two continental plates, basaltic lava poured out and created a oceanic crust. Iceland is located on the mid- Atlantic ridge. On cross faults, some distance away from the ridge, we find Jan Mayen and the Faroe Islands.
As the continental plates were sliding away from each other, the Scandinavian plate was uplifted some 2000 m along the western coast. At the same time the stiff continental crust was tilted with a ridge formed along the western coast and a moderately dipping topography in south- easterly direction. This is still the main topography in Scandinavia today 50 mill. years later.
The uplifted landmasses were vulnerable to erosion. Powerful rivers eroded their way from west and transported huge amounts of sands and gravel to the sea. These Tertiary layers are up to 5000 m thick.

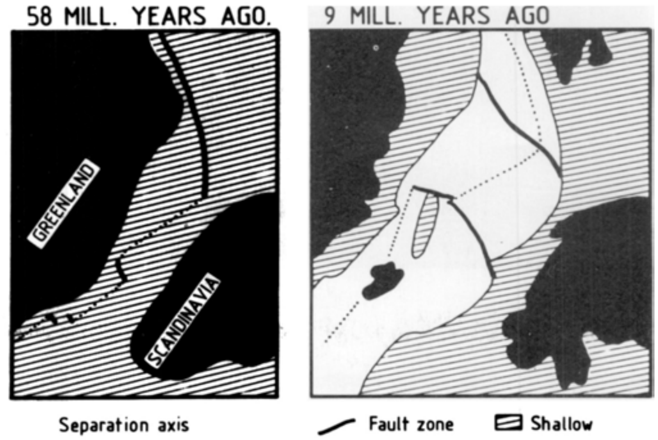


Fig. 3.13 Norway and Greenland once more start drifting apart forming the Norwegian Sea and the North Sea as sedimentation basins

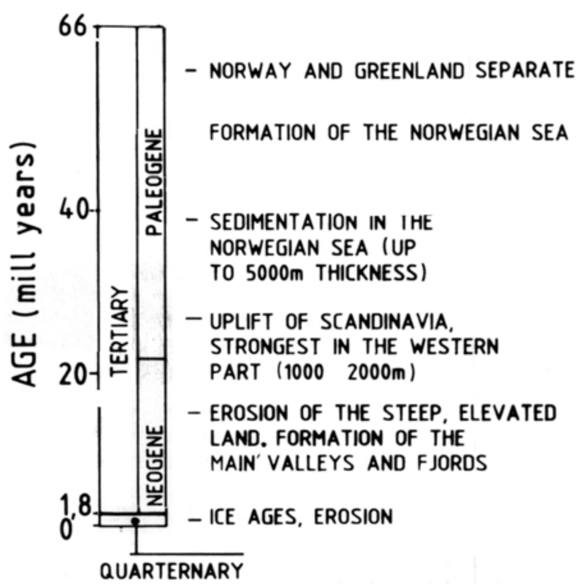


Fig. 3.14
Main geological events during the Cenozoic era

In early Tertiary a warm climate with large forests was characteristic for Europe all the way north to Spitsbergen. Towards the end of Tertiary the climate gradually became colder and more humid. The geologists still disagree on where to draw the line between Tertiary and Quaternary , the start of the first major ice age. It is generally believed to have taken place some 2.5 mill. years ago.
It is an open question how many glacial ages there have been in Quaternary, because the second last one so effectively eroded down the evidences from the earlier ones. In the Alps it is still possible to trace 4 or 5, but many professionals believe it could have been as many as 1 Q. In between the ice ages there were longer inter-glaciation shorter interstadiale periods, with the whole of Norway free of ice.

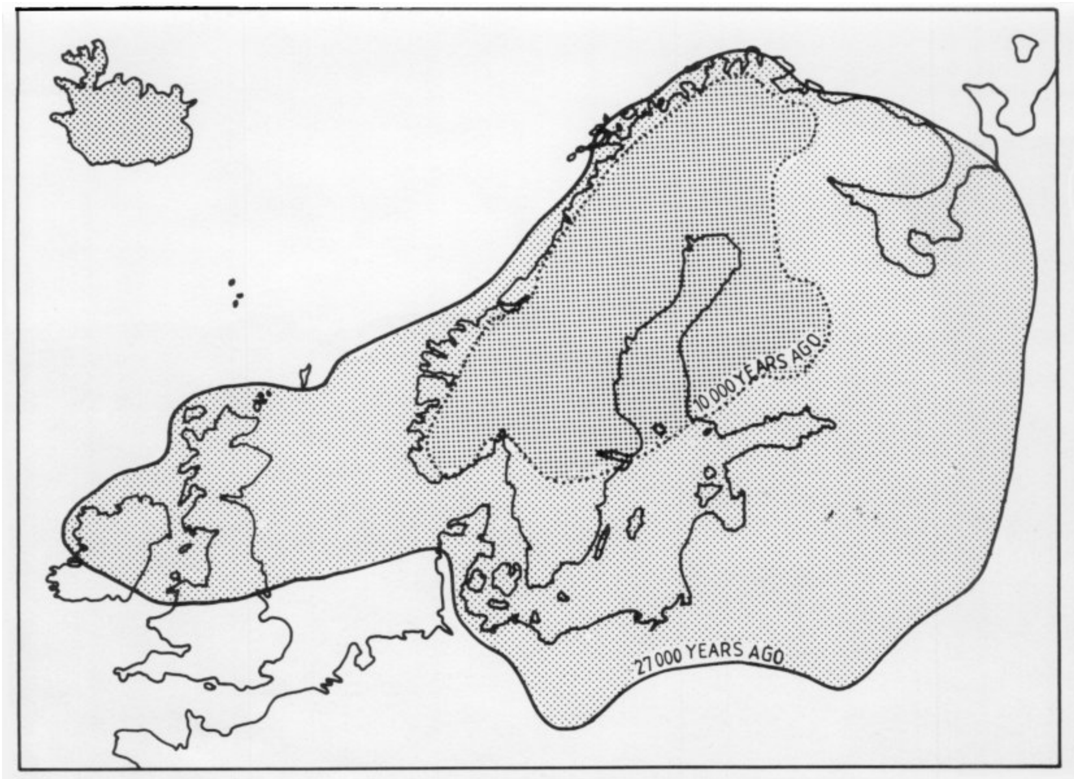


Fig. 3.15 De-glaciation of northern Europe during the last ice age

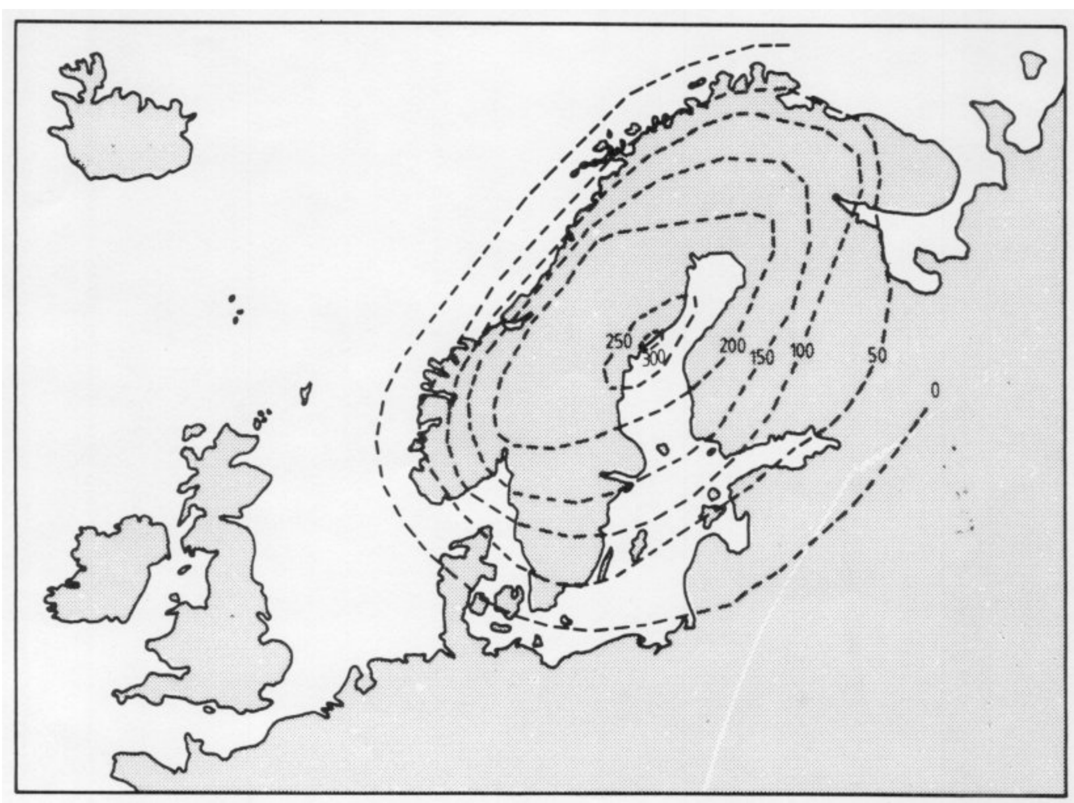


Fig. 3.16 Uplift of Fenoscandia caused by the removal of the ice. (contours in meters)

At maximum of the last glacial age, some 20 - 22 000 years before our time, the icecap covering Norway was about 3 km thick, floating towards the melting front south and west, independent of the topography underneath.
Under maximum glaciation, with the ocean bereaved as ice, the sea level dropped 100 m. Large areas of the North Sea and the continental shelf dried up or became shallow water regions. The continent on the other hand was pressed down as much as 300- 400 m by the enormous weight of the ice masses.
During erosion by the glacier the earlier environment shaped in the Tertiary has been further developed and changed. Weaker layers or zones were strongly eroded to become depressions or U-shaped valleys. Differences in the crust's mechanical properties can be seen from the topographical features today.
The glacier's ability to erode was proportional to weight and speed; it would erode deepest where the speed was high and the ice was thick. This explains the deep profile of Norwegian fjords. The ice was thick close to its maximum where the inner part of the fjords are. Because of this, the fjords are very deep farthest in, with depth decreasing in the outer part where the ice was thinner towards the melting front. At the continental shelf the fjords have a shallow sill. The glaciers have effectively cleared the region, and the sediments from the earlier period are nearly all removed. The ice erosion has also re- moved the old weathered rock surface. The sediments covering Norway today are mainly materials from the last melting period of the glacier settled around 10 000 years ago. The Quaternary geologists have developed a complicated history, showing the pattern for retreat and still stand of the glaciers from records of continental and oceanic lifting. This present history for the geologist is the most important knowledge we can get, because: the present time and the processes we can study today will always be the key to find out the past.

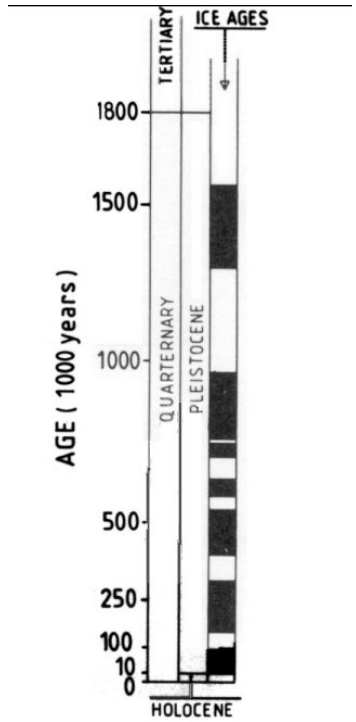


Fig. 3.17 Probable ice ages during the Quaternary period

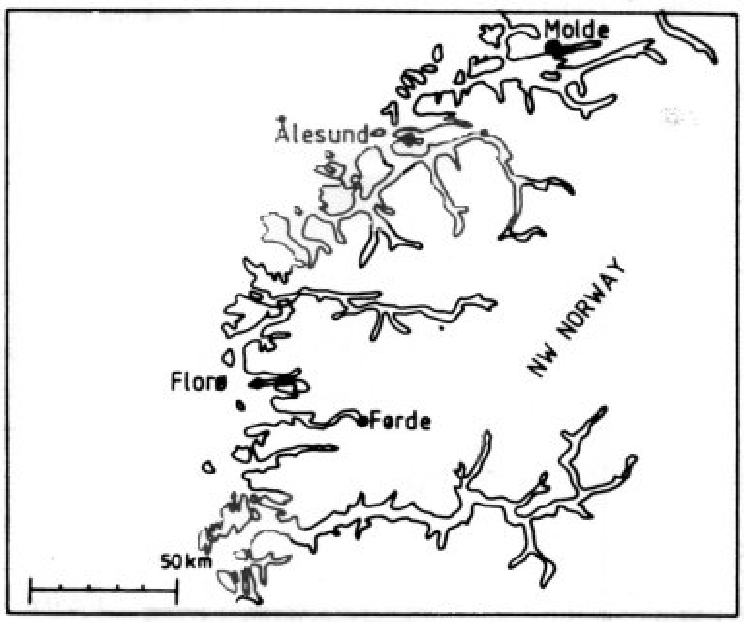


Fig. 3.18 Part of the Northwest coast of Norway. Erosion has formed a lot of long and up to more than 1000 meters deep fjords along the Norwegian coast

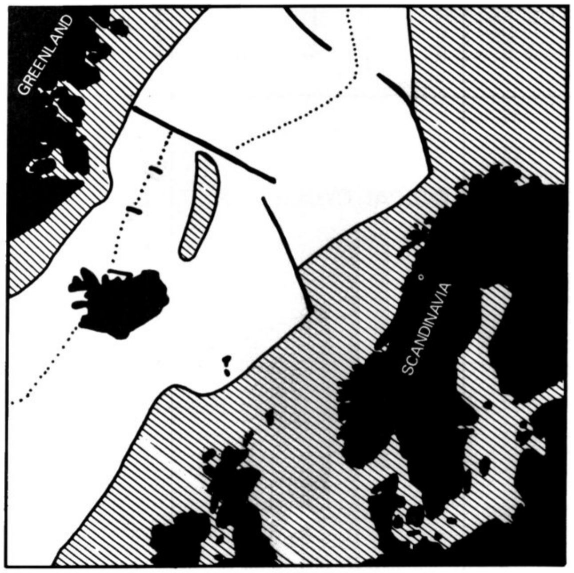


Fig. 3.19 Greenland is still moving away from Norway by 2 cm per year. The Norwegian Sea has become a large sedimentation basin

...... –separation axis
—— –fault zone

–shallow

–deep

Aerial view of a mountainous area. Southern Norway. The fresh and often well exposed rocks at the surface offer excellent possibilities to study the rock mass composition. Weakness zones can be seen as straight and long lines forming larger or smaller depressions.

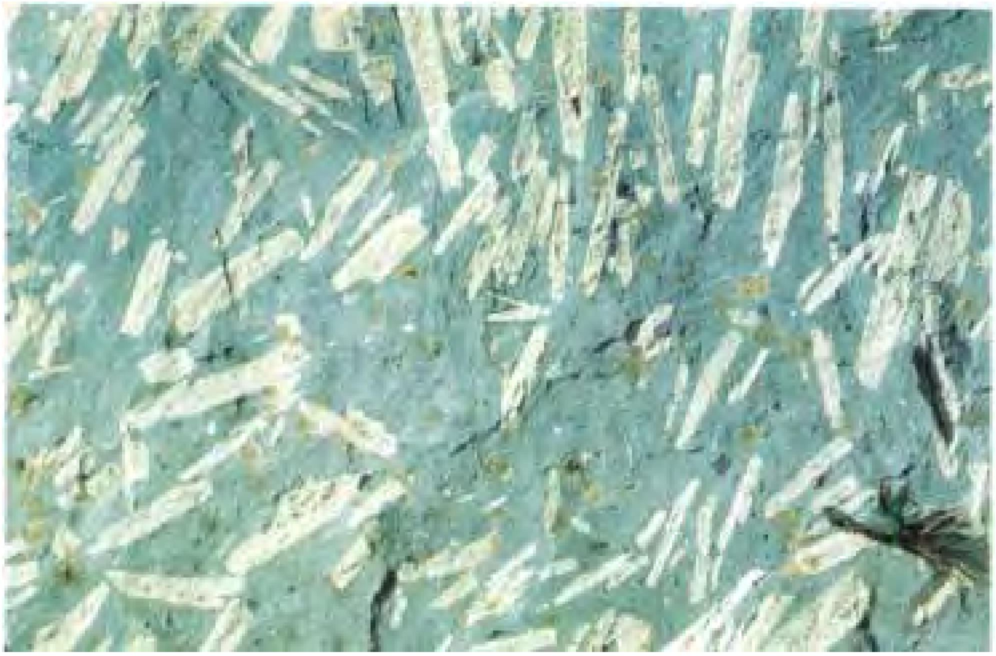
Some of the older rocks on earth are found here in the Lofoten area, in northern Norway

Various Precambrian rocks. The bedrock sometimes show that they have undergone an eventful and long geological development.

Banded gneiss. This type of rock contains grey micagneiss alternating with dark amphibolite. In the foreground is a small, light lens of granite.

Precambrian migmatite. The white veins have intruded the older darker gneiss; at the same time the new rock have been strongly folded. (Photo: J. Naterstad)

Lime-containing, metamorphosed schists and sandstone of late precambrian origin in Varanger, northern Norway.



Permian extrusive rock (rhomb porphyry) In the Oslo area a lot of extrusive rocks were formed from high volcanic activity. (Photo: J. Nakestad)

Folded layers of shale and limestone near Oslo.
The originally flat layers of the sedimentary rocks have been folded during the Caledonian orogeny. (Photo: J. Nakestad)

A small stream is eroding its way down in a weakness zone in Precambrian gneiss. (Photo: Torgeir Garmo)

The Nigardsbre glacier, a branch of the country's largest glacier; Jostedalsbreen. This famous curved glacier clearly shows its eroding capacity. Large amounts of sand and gravel are brought into the lake. (Photo: Torgeir Garmo)

The pulpit rock in Lysefjord, south-eastern Norway. Glacial erosion has produced this spectacular scenery. (Public Roads Adminstration Rogaland)

Devonian metamorphosed sandstone. The bedding of the rocks can be clearly seen in this glacier-eroded landscape.

ROCK PROPERTIES

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1. INTRODUCTION

The Norwegian Institute of Technology have been testing the mechanical properties of some 2000 rock specimens over the last 25 years. The results given are from tests of cylindrical specimens with diameter d = 62 mm and length to width ratio of 2,5. The uniaxial compressive strength is the maximum recorded stress, and the modulus of elasticity is defined as secant modulus at a stress of 20 MPa. The tensile strength is based on the point load index without any size correction for cylindrical specimens with a diameter of l8 mm.

2. RESULTS

The modulus of elasticity plotted versus the uniaxial compressive strength behave rather erratically, Fig. 4.1. No systematic trends are apparent. Some rock types exhibit increasing modulus of elasticity as the uniaxial compressive strength increases. Others do not follow such a rule. For gneisses the relationship between uniaxial compressive strength (σc) and modulus of elasticity (E) given in GPa can be described fairly well by the equation :

$$E = 20 + 1/7\ \sigma c$$

Some other rocks also indicate similar trends but no general relationship between uniaxial compressive strength and modulus of elasticity can be given.
The point load index (Is) plotted versus the uniaxial compressive strength (σc) do not show good consistency, Fig. 4:2, although the bulk of the data presented here falls between the lines:

$$Is = 1/20\ \sigma c$$

and

$$Is = 1/3\ \sigma c$$

These results emphasize that no general relation between the point load index and the uniaxial compressive strength exists. Consequently such a relationship must be established for the actual rock(s) subjected to testing, as stated by the International Society for Rock Mechanics Suggested Method for Point Load Testing (1985). The tendency is that the stronger and more homogeneous the rocks are, the higher is the ratio between the uniaxial compressive strength and the point load index.
The various rock types in Table 4.1 have be en ranked such that the uniaxial compressive strength increases from left to right in Fig. 4.3 which shows that the rocks are grouped according to geologic origin as indicated below the diagram.
The same ranking is used for the modulus of elasticity, Fig. 4.4, but this does not yield any meaningful information. A general trend is, however, that the mean modulus of elasticity for gneisses of all kinds are generally low, and for magnetite- and sulphide ore high.

Table 4.1
Major rock types that have been tested. Numbers refer to Figs. 4.3 and 4.4.

1.	Amphibolite	16.	Granite	31.	Mylonite
2.	Amphibolitic Gneiss	17.	Granitic Gneiss	32.	Nepheline-syenite
3.	Anorothosite	18.	Granodiorite	33.	Pegmatite
4.	Arkose	19.	Greenschist	34.	Sandstone.
5.	Basalt	20.	Greenstone	35.	Serpentinite
6.	Diabase	21.	Greywacke	36.	Sparagmite
7.	Diorite	22.	Hyperite	37.	Black Shale
8.	Phyllite	23.	Limestone	38.	Syenite
9.	Gabbro	24.	Conglomerate	39.	Talc-schist
10.	Mica Gneiss	25.	Quartzite	40.	Augen-gneiss
11.	Mica Quatzite	26.	Quartzitic Phyllite	41.	Coral Chalk
12.	Mica Schist	27.	Quartz Sandstone	42.	Magnetite Ore
13.	Gneiss	28.	Mudstone	43.	Sulphide Ore
14.	Gneissic Granite	29.	Marble	44.	Zinc Ore
15.	Garnet-mica Schist	30.	Monzonite	45.	Lead Ore

3. EXPERIENCE
Nearly all Norwegian rocks are typical hard rocks. The compressive strength is mostly higher than 50 MPa, and values up to 470 MPa has been recorded.
Preliminary conclusions from continuing study of mechanical properties of Norwegian rocks indicate that for certain rock types it is possible to establish relationships between the different material properties. It is not yet verified that these relationships are generally valid.

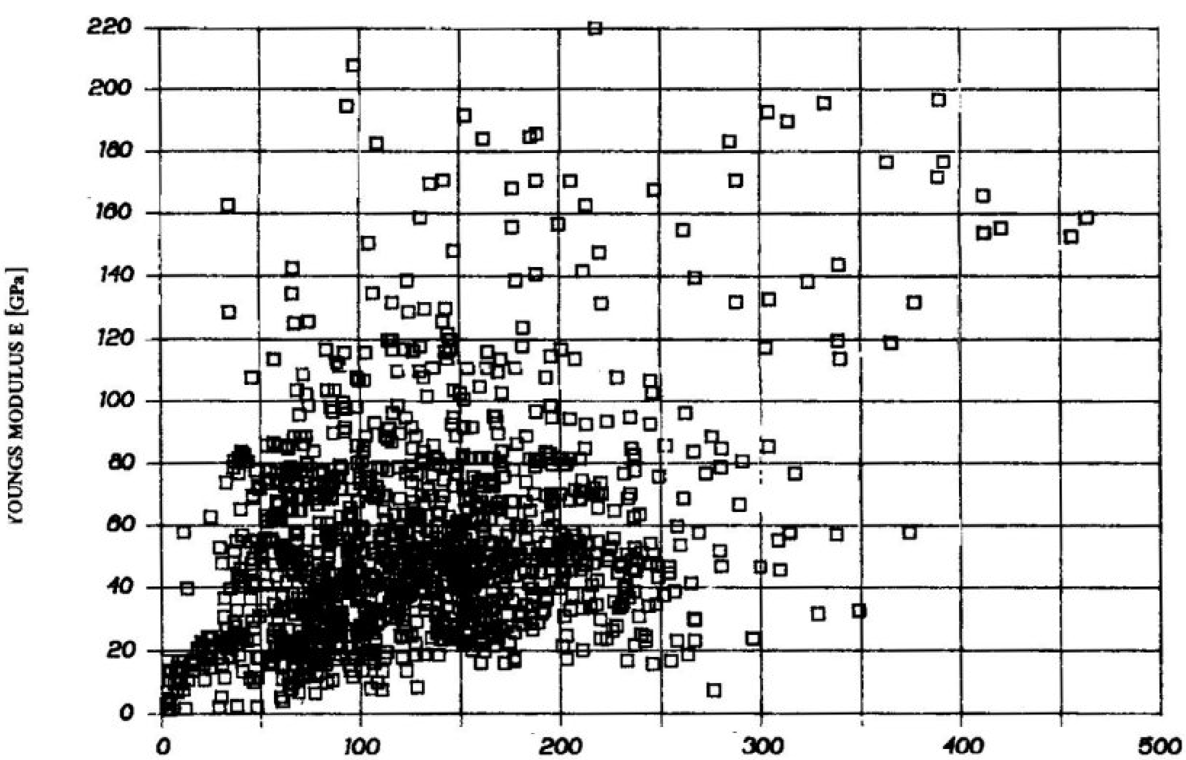

Fig. 4.1 Modulus of elasticity vs. compressive strength.

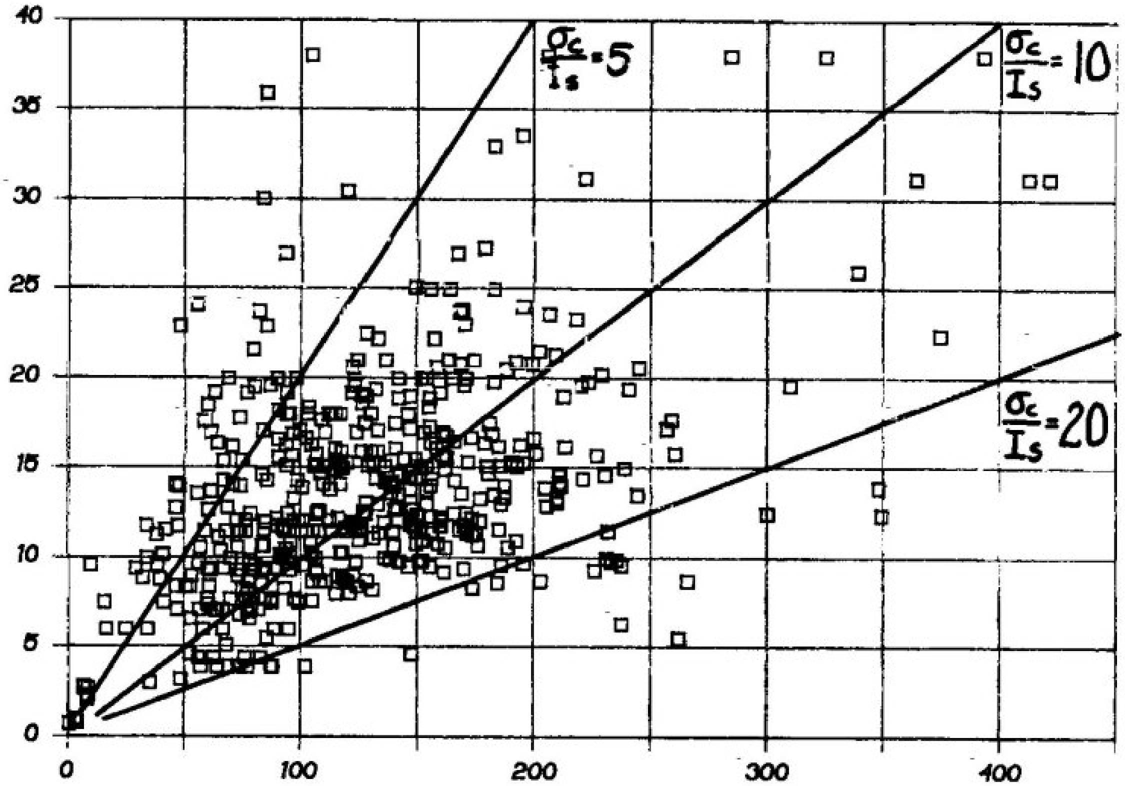

Fig. 4.2 Tensile strength vs. compressive strength.

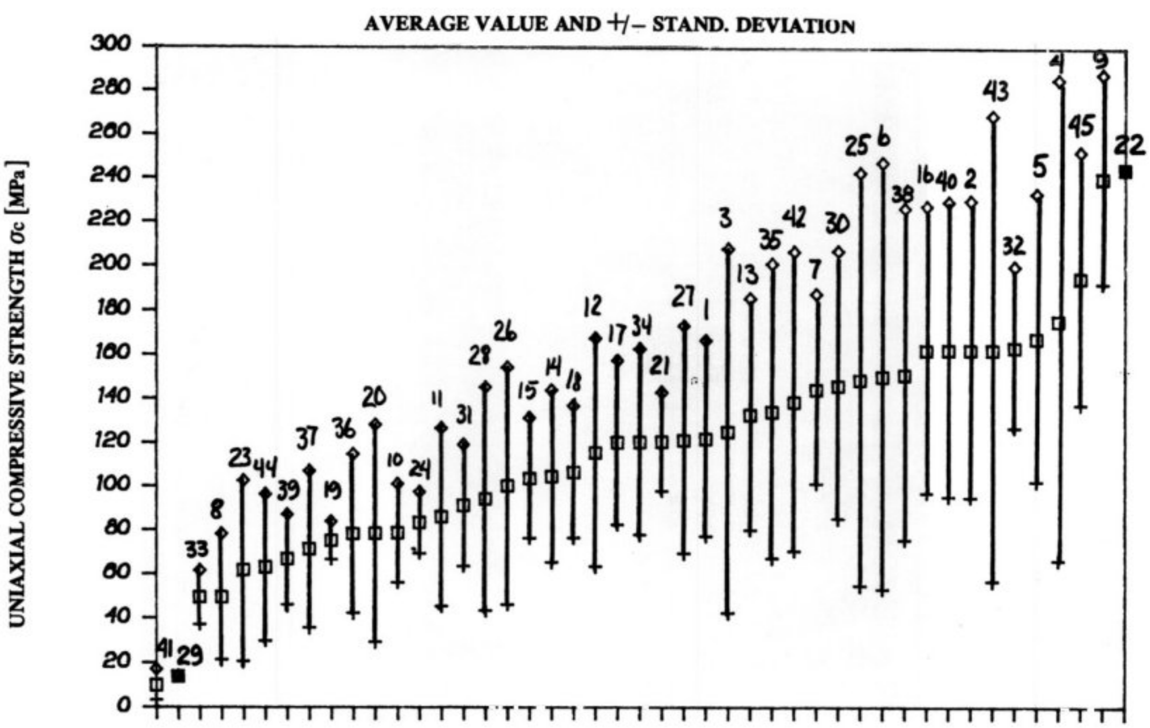

Fig. 4.3 Compressive strength for various rock types.

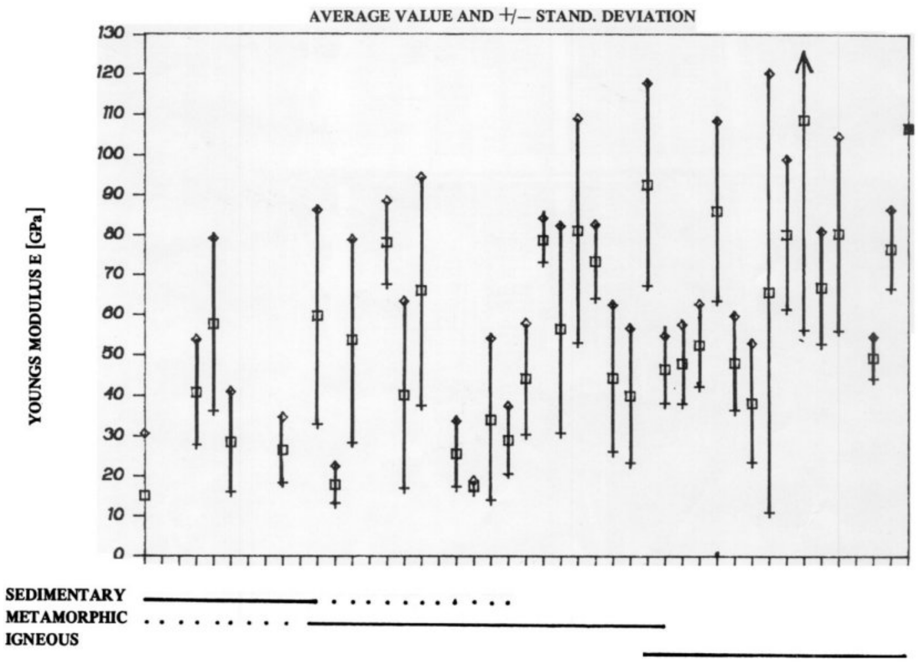

Fig. 4.4 Modulus of elasticity for various rock types

Installation work in the Alta underground powerhouse. The span is 16 meters the height is 35 meters. (Photo: Ole M. Rapp.)

ROCK STRESSES

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1. INTRODUCTION

The Norwegian Institute of Technology have undertaken measuring of in situ rock stresses over the last 20 years. At some 80 sites, three-dimensional stress measurements have been conducted by a modified Leeman over coring method, Myrvang (1983).
Experience indicates that rational analysis is only possible on a regional scale. In Norway there seems to be from 10 to 15 regions in which stresses are reasonable consistent. In such regions the behaviour of engineering structures can be predicted on the basis of statistical trends of stress patterns. The reliability of such predictions, however, can be verified and improved by local measurements.

2. RESULTS

The results from rock stress measurements in Norway and other parts of the Fenoscandian Shield (Norway, Sweden and Finland), and their interpretation indicate considerable variation depending on both geological and topographical settings.
In Figure 5.1 the measured rock stresses are plotted, indicating local anomalies where stresses are higher or lower than w hat should be expected from the overburden (gravity). In some places the minor principal stress is negative.
After decomposition of the measured stresses in vertical and horizontal components, the major horizontal component is often higher than the. vertical, Figure 5.2. The minor measured principal stress is normally higher than the in situ ground water pressure, Figure 5.3.
The rock stress pattern in the Precambrian culminations in Northern Norway is dominated by high horizontal tectonic stresses. The major principal stress is oriented approximately E -W, while the intermediate is trending N -S. The horizontal stresses are higher than the gravitational vertical stress, at least down to a depth of 800- 1000 m.

3. EXPERIENCES

Preliminary conclusions from continuing study of the in situ rock stresses and their distribution in Norway are:

- The vertical measured stress equals the overburden stress.
- The minor measured principal stress is normally higher than the in-situ ground water pressure.
- The sub-horizontal principal stresses are often higher than the vertical stress.
- In the local regions, there often exist global stress patterns governing the orientation of the stresses.

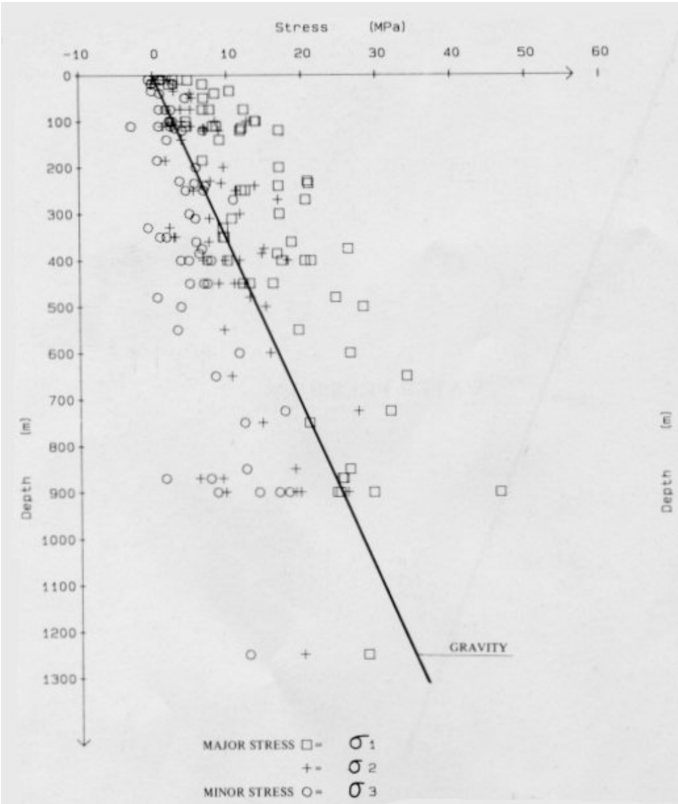


Fig. 5.1 Measured in situ principal stresses

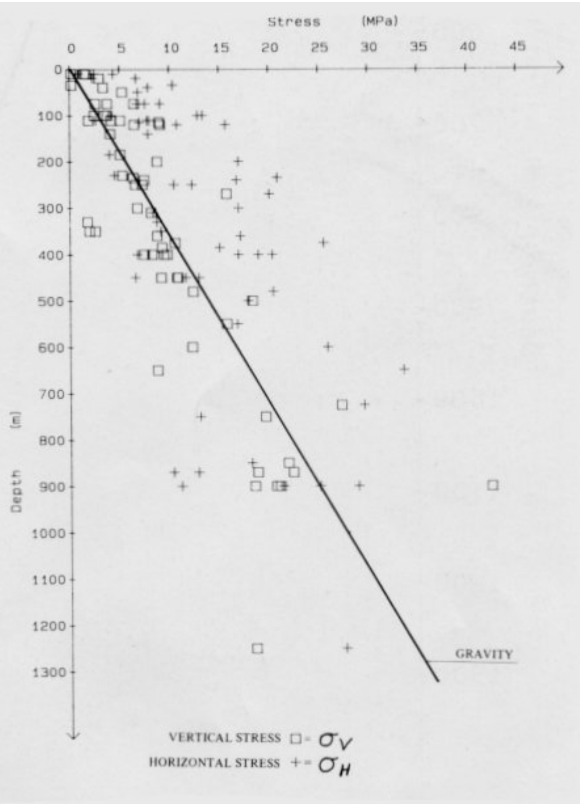


Fig. 5.2 Maximum in situ horizontal and vertical stresses

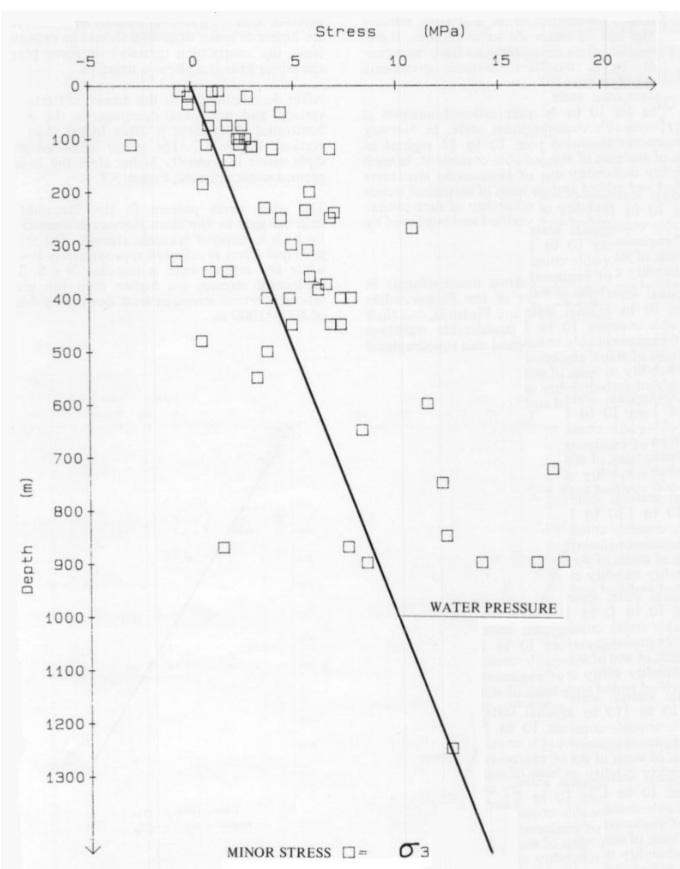


Fig. 5.3.
Minor principal stress

III

NORWEGIAN TUNNELLING DESIGN

In Norway, the erosion during several ice ages in the past 1 to 2 million years has left a more or less fresh bedrock surface where the geological features of importance for underground works can be studied. This situation has at least two great advantages:

- The costs of field investigations is relatively low. The Norwegian experience is that the costs of geological investigations are about 1% of the construction costs for simple tunnels and about 3-4% for storage caverns. For special projects the costs have been up to 5-6%.
- The surface exposure of the rock mass conditions gives the opportunities to discover the construction possibilities the underground may offer. This has given confidence w hen utilizing the rock masses as construction material.

A close cooperation between tunnellers, contractors, design engineers and engineering geologists are important for developments in design, excavation methods and rock supporting measures. The engineering geologist plays an important role during all stages of a project. Not only does he determine the rock distribution, but he also interprets the geological data recorded for the actual project. Then he finds the most suitable location and orientation of the structure from a geological point of view. Be also follows up during construction and gives advice on the measures to be taken for a safe project execution.
Another important feature in Norwegian tunnelling practice is that of carrying out the rock supporting work according to the actual rock masses encountered in the tunnel. First the rock support to ensure safe working conditions for the tunnellers is installed, and later the permanent rock support for a safe operation of the project is performed. The tender documents are worked out such that the various and changeable conditions during tunnelling can be systematically met.

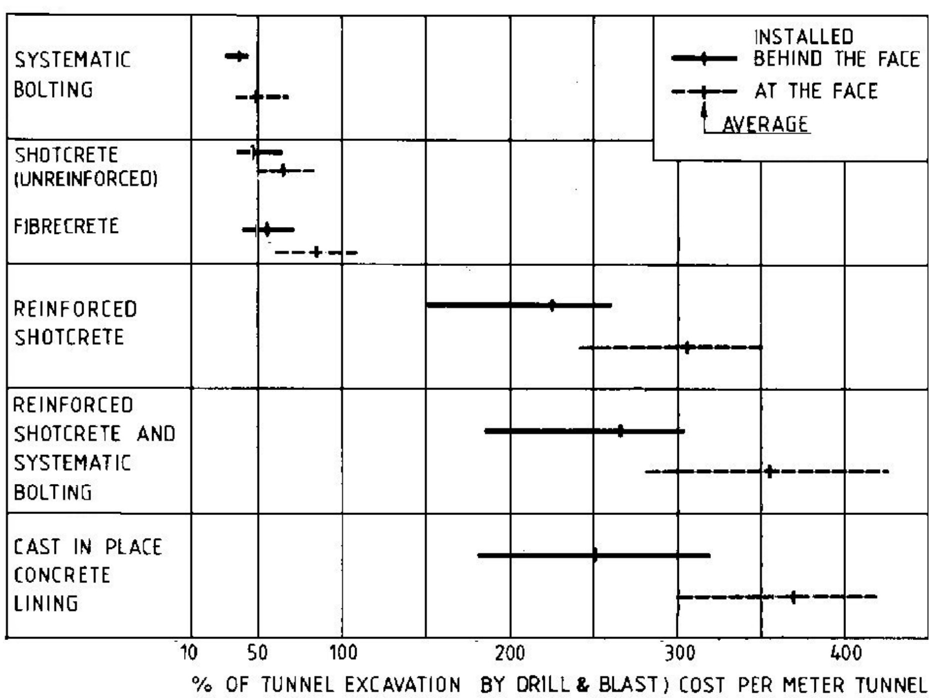


Fig. III Important tunnelling items: Relative costs of rock support and tunnel excavation (drill & blast and mucking out)

SITE INVESTIGATIONS

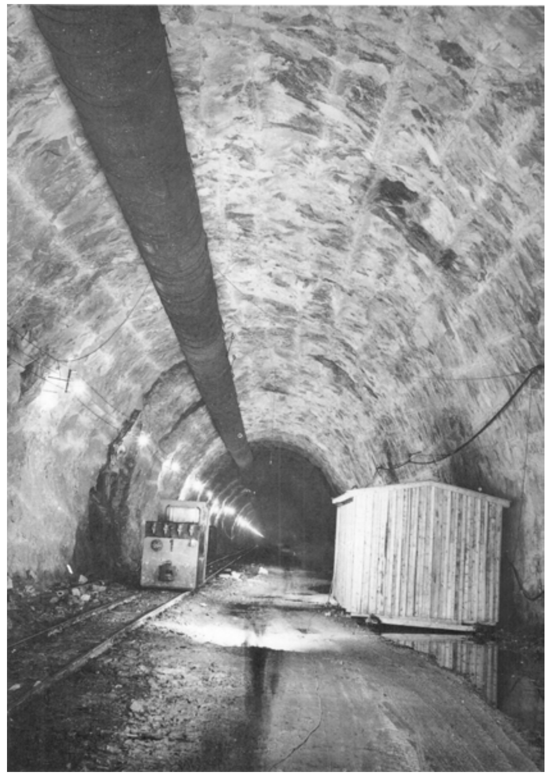
Einar Broch
The Norwegian Institute of Technology

l. INVESTIGATION STAGES

Norwegian engineering geological investigations related to tunnels and underground openings can be divided into two main stages :

- Pre-investigations. Tunnelling has not yet started and all information has to be collected on or from the surface.
- Post-investigations. Through tunnels being excavated the rock masses are accessible for inspections.

As shown in Table 6.1 each of these main stages can be divided into two sub-stages. The characteristic investigations for each of the four stages are briefly listed in the table. Types of reports are also indicated. Not all kind of investigations will be carried out for all tunnels. A short road tunnel through rocks which can easily be mapped on the surface does not necessarily need a two-stage pre-investigation phase. On the other hand, for a hydro power scheme with several alternative tunnel alignments, or for a complicated sub sea tunnel, a subdivision of the pre-investigations into more than two stages may be considered.
In the following chapters the four stages defined in Table 6.1 will be further described.



View from the 65m^2 headrace tunnel at Tokke hydropower plant (Statkraft)

Table 6.1 Various site investigation stages.

PRE-INVESTIGATIONS Information collected on or form the surface	
PRELIMINARY SITE EXPLORATION	**DETAILED SURFACE INVESTIGATIONS**
Desk studies of: - geotechnical literature - topograpichal and geographical maps - air photoes **Walk-over survey for preliminary mapping of soil cover, rocks, jointing, and weakness zones.** G**eophysical investigations at key points for tunnels:**	**Engineering geological mapping along tunnel alignment.** - types and quality of rocks - orientation, spacing and quality of joints - orientation, thickness and quality of weakness zones - ground water condition **Special investigations:**

- entrances - intakes and outlets in lakes, fjords and rives - areas of low rock cover - check of soil thickness in critical points	- refraction seismic survey - core drilling Sampling and laboratory testing of rocks: - strenght - drillability - blastability
Preliminary report: - review of geological and geotechnical conditions. -evaluation of feasability for different alternatives - plan and cost estimate for detailed investigations -need for more maps and air photos.	**Pre-investigation report:** - description, with maps an cross sections, of all topographical and geological factors that may influence construction and use of tunnels and openings - estimates and preliminary plans for excavation requirements, rock support and lining - plans for use of rock material

POST-INVESTIGATIONS

Rock masses can be inspected in the subsurface

DETAILED SUBSURFACE INVESTIGATIONS	TUNNEL MAPPING
Sampling and testing of rocks and infilling materials from joints and faults. **Supplementary investigations:** - rock stress measurements - permebaility test s of rock masses - covergence measurements of openings **Control and revision of reports from pre-investigations.** **Recommendations of permanent rock support and lining.** **Recommendations for grouting.** **Recommendations for excavation throught highly unstable rock masses.**	**Mapping in tunnel of:** - types and quality of rocks - orientation, spacing and quality of joints - orientation, thickness and type of weakness zones - seepage of water - stress induced problems **Registration of all rock support, lining and rock improvements** **Evaluation of excavation progress.**
Supplementary reports from post-investigations. **support.** **Report on permanent rock support and lining.**	**Final report with tunnel map and review of rock** **Evaluation of pre-investigations**

2. PRE-INVESTIGATIONS

The Preliminary Site Exploration for Feasibility Evaluations

The preliminary site exploration is carried out in the early phase of the planning of a project. The aim is either to study the feasibility of a planned tunnel, or, more often, to evaluate and reduce the number of possible alternatives in a scheme based on geotechnical information. Few decisions are yet made, and sketches are more typical than drawings. This is a highly challenging phase.

Important decisions are taken, often based on limit ed information. Experience from similar projects and similar sites are therefore of particular value.

At this early state it is important to collect all existing relevant information from literature and technical reports. Also desk studies of topographical and geological maps as well as air photos are done. Such studies will give the first answers to questions about where the bedrock is covered with soils, what the locations and directions of the more important weakness zones are, and what the stress situation in the area may be. The studies will normally be followed by a walk- over survey to investigate certain key points in the actual area. Rock sampling for simple classification tests are done, and the most important joint information is collected. Depth of weathering and the ground water conditions are also studied during this walk-over survey.

In the report that concludes the preliminary site exploration all collected information is presented and the different alternatives are discussed. Plans and cost estimates for further investigations are presented, and the need for maps are given.

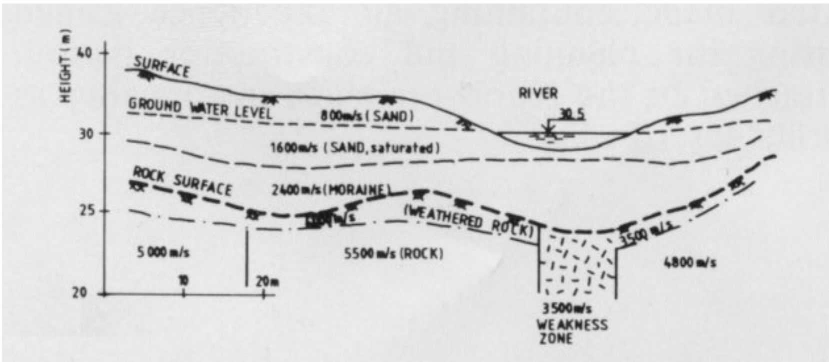


Fig. 6.1 Refraction seismic measurements are often used for rock mass quality evaluation and to find depth of loose materials.

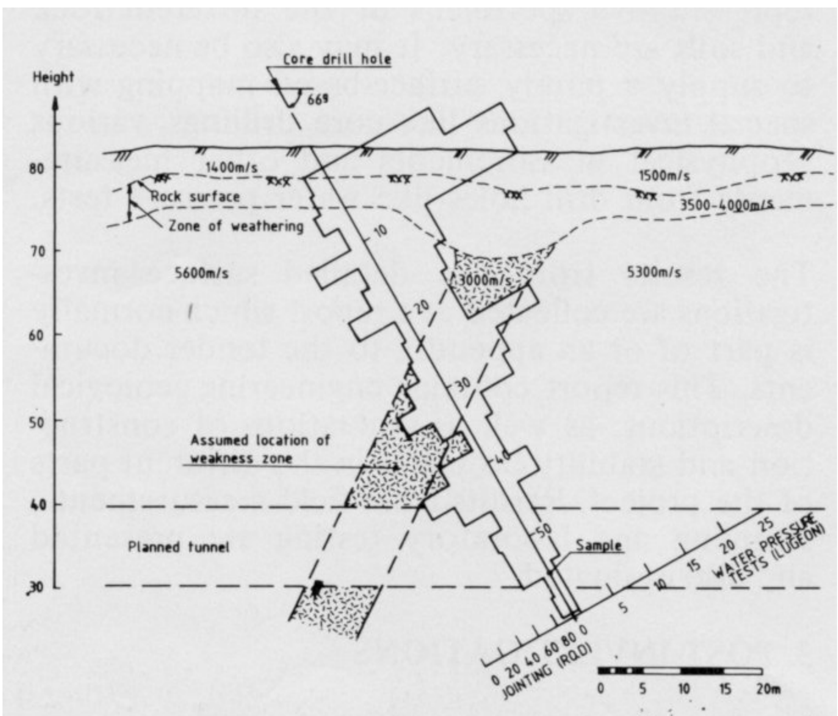


Fig. 6.2 Presentation of results from core drilling and seismic refraction measurements.

The Detailed Surface Investigations

Based on the preliminary report the client, in co- operation with his consultants, will decide if further planning should be carried out, and if so, w hat alternatives should be investigated. Additional air photos are taken if required and better maps drawn. The engineering geologist will

normally need air photos and maps that are covering a larger area than is strictly necessary for the other planning operations.
At this stage the most important task for the engineering geologist is to produce engineering geological maps and cross sections that are covering the different parts of the project. For this purpose mapping, sampling and analysing of representative specimens of the different rock and soils are necessary. It may also be necessary to supply a purely surface-based mapping with special investigations like core drillings, various geophysical measurements and other measurements from drill holes like water pressure tests.
The results from the detailed surface investigations are collected in a report which normally is part of or an appendix to the tender documents. This report contains engineering geological descriptions, as well as evaluations of construction and stability problems in the different parts of the project. Results from field measurements, sampling and laboratory testing are presented and also evaluated.

3. POST-INVESTIGATIONS

The Detailed Sub-surface Investigations

When the construction work has started and the tunnel can be entered, the possibilities for the engineering geologist to get better information increases considerably. The post investigations are therefore started as early as possible. During the planning of an underground project, important decisions have to be taken about w hat investigations should be carried out before the start of the operations, and w hat investigations could be postponed.
A high degree of flexibility and relatively simple pre-investigations have of ten characterised tunnelling operations in Norway. Under such conditions it is particularly important that the post-investigations are started early in the construction period. In many cases ex pensive pre-investigations like for instance deep core drillings can be replaced by the much cheaper pilot borings from the face of the tunnel during construction.
Stress measurements in rock masses will normally have to be done in underground openings and tunnels. Such measurements are therefore good examples of the type of detailed investigations which have to be delayed until tunnelling has started. For unlined high pressure hydro tunnels hydraulic jacking tests are used to decide length of lining.
Detailed sub-surface investigations should not be delayed pre-investigations, but should be planned as a control of and a supplement to pre-investigations. For sub-surface works a pre-investigation report will always have to be based on a number of assumptions. The sooner these are verified, the better for the remaining part of the underground works. The reports should thus be under continuous revision during the entire construction period.

Tunnel Mapping

A number of underground openings and tunnels will be difficult to inspect after they have been put in operation (for instance hydropower tunnels). For the owner it is therefore useful to have maps and drawings describing the inaccessible parts of the

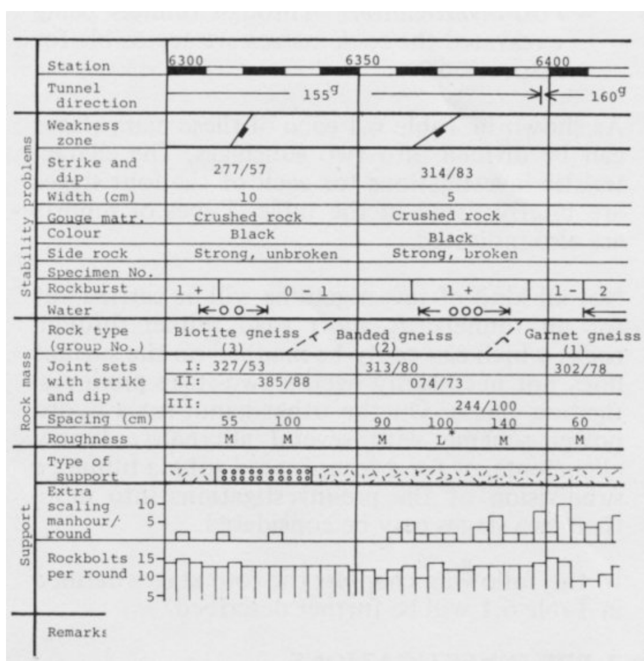


Fig. 6.3 Examples from tunnel mapping.

project. Such maps should contain all geological elements that may influence the stability of the tunnel such as major joints, faults, crushing zones, water leakages and areas with rock burst problems, in addition to rock types and information about support work.

The post-investigations and especially the tunnel mapping are important parts in the process of building up engineering geological experience. The method of pre-investigations can only be improved if the prognosis are carefully controlled through the post-investigations. Especially where the prognosis have been wrong, it is important to find the reasons for this so that similar mistakes can be avoided in the future. W hen an underground project is completed and the post-investigations carried out, a final report is often made containing all experience gained during the planning and construction period. Attached to the report are maps and drawings as earlier described.

Near Beiarn, Northern Norway. From the studies of aerial photographs it is often possible to discern many geological features thanks to the well exposed, fresh rock surface.
(Photo: Fjellanger Widerøe A/S)

GENERAL ENGINEERING GEOLOGICAL DESIGN PROCEDURES

Rolf Selmer-Olsen
Geoteam A/S

1. INTRODUCTION

During construction of a rock facility the properties of the involved materials can never be chosen so as to comply with defined criteria. The goal is to try to make use of the fractured rock mass such as it is, with its known orientation, to provide the best possibilities for satisfying the requirements towards stability and operation of the facility.
Even though stability-reducing structures always will function together with the stress and strain abilities of the given rock mass, it has been beneficial to evaluate the stability-reducing factors individually in order to find the best location, orientation and shaping of the rock cavern. Such a division require a well organized set of individual conclusions from the study to satisfy the functional demand. The cost impact of each parameter should be evaluated.
A close cooperation between the designer and the engineering geologist is important from the early stage in a project, w hen the conceptual ideas are developed. During the later planning period and even during the construction process, the geological, functional or economical criteria and conditions may have to be adjusted, causing revisions in the design. Continued cooperation of the planners will ensure against delays or cost overruns attributable to such revisions.

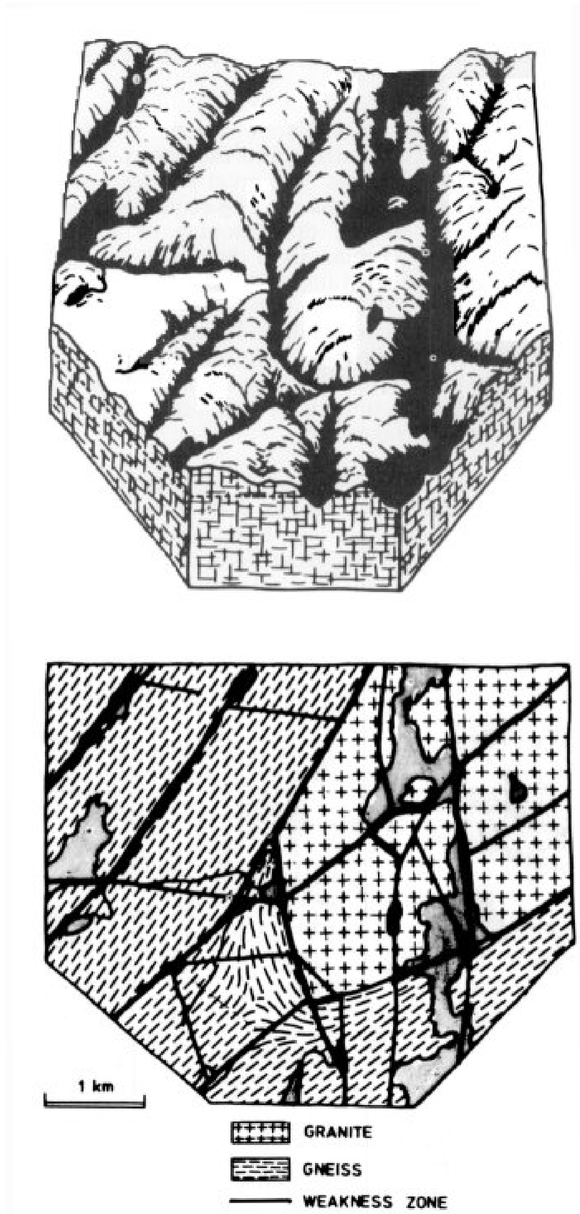


Fig. 7.1
The topographical features in Norway give often valuable information about the engineering geological conditions. Surface observations combined with existing geological data play an important part in the evaluations made for facilities to be constructed in rock

2. ENGINEERING GEOLOGY OBSERVATIONS

The Quaternary icecap which covered Norway left behind a hilly terrain with huge exposed rock areas. The glacial deposits are intermittent and thin. The less resistant rock types and the weak gouge materials in fault zones and crushed zones often appear as well defined depressions in the topography.
The fresh rocks, often well exposed at the surface intakes it possible to study the rock mass conditions

through simple geological mapping. It is therefore easy to judge the possibility of utilizing a given rock mass for subsurface use.
These surface conditions also give good possibilities for three dimensional mapping of rock types and weakness zones without extensive use of coring. They are helpful for finding a location for a rock facility where major problems can be reduced or avoided.

3. GEOLOGICAL CONDITIONS THAT AFFECT THE STABILITY

Crushed zones

Crushed zones are tectonic zones with high or very high degree of jointing. They can contain clay, swelling clay and may have low frictional angles because of talc and chlorite. Greater swelling clay zones are expensive to reinforce by rock support, especially in larger rock chambers. They should, if possible be avoided, alternatively be intersected at a favourable angle where unstable length is short to reduce the rock support expenses.
The swell pressure of a zone material is dependent on the composition and the consolidation from the rock stresses. The great potential pressure of a well developed swelling clay, can have a major effect on the dimensioning of a rock support. The clay minerals in the weakness zones are therefore of great importance in the planning of the project. It is often necessary to do analyses of the clay materials and evaluate the consolidation in addition to a thorough mapping of the zone during construction so a decision on the extent and strength of the permanent rock support can be taken.
Zones filled with talc can be observed with an angle of friction as low as 2-4°. The result is often near zero stand-up time. This fact puts fractured serpentine, soapstone and olivinite which often are observed with such fractures, in a less favourable position and should be avoided if possible.

Jointing

Norwegian rock masses have through long geologic periods repeatedly been subjected to faulting. In addition to faults, we can observe the result today as joints, cracks and fractures. In hard rocks the joints play a very important role regarding the rock mass quality and hence the amount and methods of rock support. The Norwegian experience is that the costs for rock support can vary with the orientation and the degree of jointing, and special care should be taken to record this property during the geological mapping.
During the blasting of a rock chamber, smooth, planar and clay-filled fractures will give an over- break in case they are at small angles with the longitudinal axis of the rock chamber. An average overbreak from a single fracture is in the order of 3% pr. meter tunnel, valid for angles less than 15°. This is over 5 times as much as for angles larger than 50°. If there are more sets of unfavourably oriented fractures, the percentage of overbreak can locally be over 15% or there is a potential danger for block fall of a corresponding extent.
In circular TBM-drilled tunnels fracturing has less influence on the stability; only the most un-favourable cases of fractures and stress have influence on overbreak and stability.

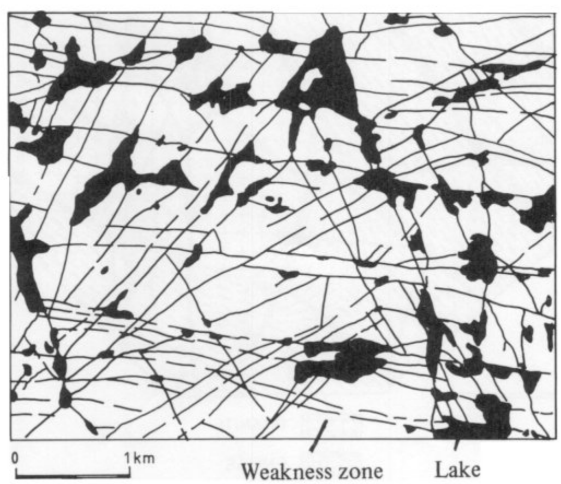


Fig. 7.2
The study of aerial photos is used in an early stage of the investigations to find larger faults and weakness zones. Most important in planning a rock facility is -it possible -to avoid such features or to intersect them at a favourable (great) angle

Conditions with very low or very high rock stress in jointed rock masses, can increase the overbreak substantially and the block stability will be considerably worse.
The degree of rock support increases with the size of the rock chamber. A favourable orientation of the long axis of the rock chamber in relation to the dominating joint sets, can to a large degree help to reduce the cost of rock support. This is especially valid for large blasted rock chambers.

Thrust faults
Thrust faults are often composed of different types of schists intensely folded and brecciated by shear stress and tectonic movements. The same stability principles as for crushed zones or jointing are used for thrust faults. Usually we find thin layers of schist which are crushed to channels of clay in between more resistant rock types. If the clay zone is relatively horizontal, it can be difficult to avoid which may cause great stability problems along considerable lengths of tunnels and rock chambers.

High rock stress
High and anisotropic stress can cause spalling in the resistant and brittle rock types with poor creep characteristics. In weak mica- and chlorite- rich rock types this may cause squeezing.
The stresses in the crust can be a combination of one or more of the following:

- Gravitational stresses caused by difference in topography
- Tectonic stresses
- Residual stresses

Experience from rock stresses through rock burst and spalling in a number of tunnels and rock chambers in Norway, has given us a good knowledge about the interaction of stress and strength conditions. For evaluations of the possible stress problems for a given project, also experience from other nearby facilities will give good information about the general rock stress conditions in the area.
Calculations on ideal FEM-models give us the size of the expected tangential stress in circular tunnels. The size of the stresses for spalling to occur, has been found empirically for blasted and TBM-drilled tunnels. This method can also be used to predict the potential for spalling in specific rock chambers.
The spalling in rock chambers in hard rocks such as granite and gneiss, located close to the valley floor, might be a problem in areas where the height of the valley sides is more than 400 m and dipping more than 20°. The tangential stresses and the chances for spalling are the least when the angle between main stress direction and the long axis of the chamber is relatively small. Large problems can be avoided by giving the axis of the rock chamber such a favourable orientation.
Another way to reduce the ex tent of the rock support from high, anisotropic stresses, lies in a rational design of the tunnel profile. By reducing the curve radius in the roof where the largest in situ stress is expected to be tangent, it is possible to reduce the area in the profile which is expected to have high, tangential stresses. This modified shape of the regular horseshoe-shaped tunnel profile reduces both the particular spot that has to be supported and removes the part of the rock volume which is most likely to be come unstable

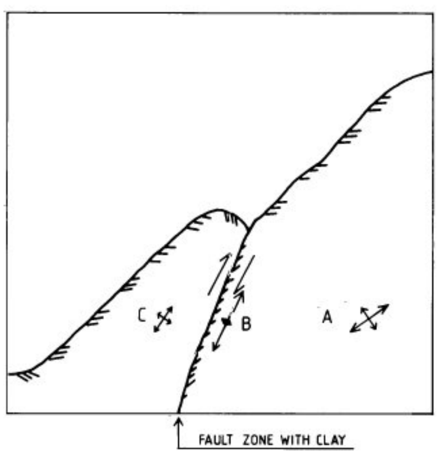


Fig. 7.3
A fault or weakness zone in a valley side can cause large local variations in the rock stresses, especially if it is orientated parallel to or at a small angle to the overall main principal stress. At "A" there is a general stress situation, at "B" highly anisotropic and at "G" a de-stressed situation

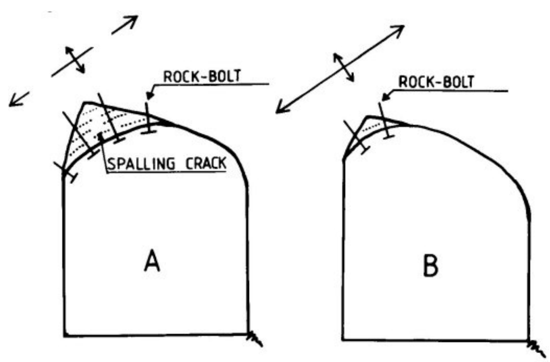


Fig. 7.4
If high anisotropic stresses occur the extent of rock burst or spalling, and hence the rock supporting works, can be reduced by a favourable shaping of the opening. "A" shows the situation for a normal horse-shoe tunnel profile, while "B" shows how the high stress area is reduced to a limited area of the roof with an asymmetric tunnel shape

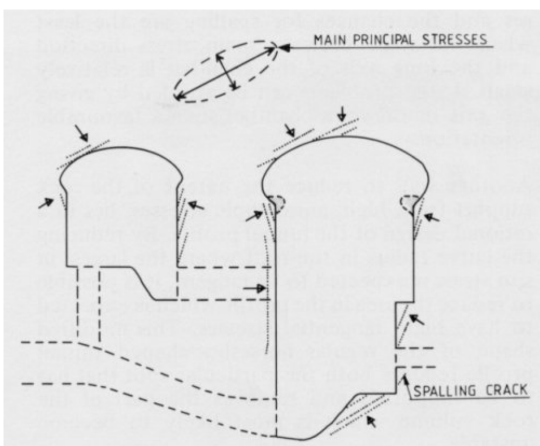


Fig. 7.5
Arrows indicate unstable parts of an underground powerhouse subjected to high anisotropic stresses. Ledges, intersectional areas with tunnels or shafts, overhang etc. are parts particularly exposed to rock stress problems.

The shaping of rock chambers exposed to high rock stresses, has to be made simple without ledges and overhang to avoid spalling.
Clay zones will often change the usual stress situation. Steep dipping zones of clay striking parallel to a valley side will cause the stresses in the outside region not to be more independent upon the valley side height. This is because clay zones are not able to transfer shear forces. These circumstances are valuable in making a decision on location for rock chambers in valley sides where the stress will be less on the outside of the zone than on the inside.
The rock stresses are also valuable for localization of unlined pressure shafts and air cushion surge chambers to avoid hydraulic ground failure. The determining factor is the minor principal stress.
In relatively flat terrain the required rock overburden is estimated as a function of gravitational stresses. In valley sides the smallest expected gravitational stress is estimated from equilibrium analysis and FEM-models as dealt with in Chapter 10.
In rocks with poor creep properties a local, high tectonic stress anisotropy may determine the size of the minor principal stress. (Possible water leakages may, however, occur independently of the orientation of the rock chamber with regard to geological structures)
When the orientation of the rock chamber is done as recommended earlier in this paper, it is possible to check the assumptions for rock pressure under construction by rock stress measurements and hydraulic splitting tests on critical places.

Water leakage

In some parts of Norway problems with water can be substantial in tunnels going down dip in hard rocks. The leakage is most often connected to tectonic stress anisotropy and occur in close to vertical fractures.
The leakage during tunnelling can in certain cases be up to 300 l/sec. at the face at depths a couple of 100 m down.
The chances for large leakages can be estimated by studying the regional faulting, the local structures, fracturing pattern and rock types. Also experience from nearby tunnels can give information of the possibilities for water-bearing zones.

4. EXPERIENCE

Information gathered from different facilities in the nearby region of a planned project, is a good indicator of how to prepare and solve upcoming problems.
Rock stresses, crushed zones, jointing, foliation and bedding are all typically oriented phenomena in the rock mass. The effect they have on stability in a chamber is largely governed by the location and orientation of the facility.
If joints, fractures and orientation of stress are known in a rock mass, it is possible to minimize their effect by applying a favourable shape and orientation of the chamber.
The best orientation of the long axis in a rock chamber is found by using a fracturing diagram in the area where foliation and main stress are given. It can be a single joint rosette or a stereographic projection of the intersecting structure poles. This will give the necessary overview of the possible angles of the long axis in relation to the characteristic jointing, weakness zones, and main stress.
The larger weakness zones mostly follow planes and have limited thickness. The position of these zones down in the rock mass can be predicted and avoided if the tunnel facility has some degree of flexibility and the final location can be determined during construction.

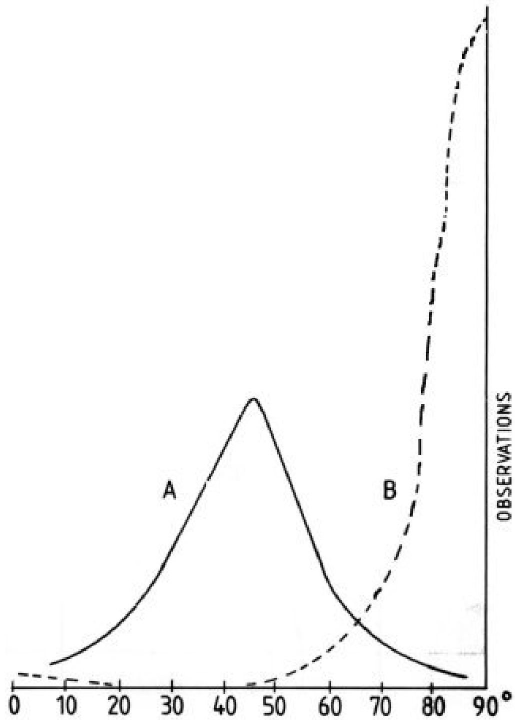


Fig. 7.6
Experience from 600 observations of water leakages connected to fractures. "A" shows the angel between the strike of the fracture and the adjacent regional fault. "B" the dip of the fracture.

The total cost of rock support in the many km's of water tunnels in Norway has varied from less than 10% to more than 120% of the cost for drilling, blasting and mucking out. In tunnels and chambers for vehicular traffic the supporting cost is often higher due to stricter safety requirements to avoid even small rock fans. Additional cost may be added from possible water and frost problems in such tunnels.
The active use of engineering geologists in tunnelling is important not only to solve rock stability problems and to recommend rock support, but also to find the best location and rock support design to meet the requirements for the actual rock mass conditions.

Rock slide in a water tunnel. An engineering geologist inspecting the failure caused by insufficient rock support of a weakness zone containing swelling clay.
(Norwegian Geotechnical Institute.)

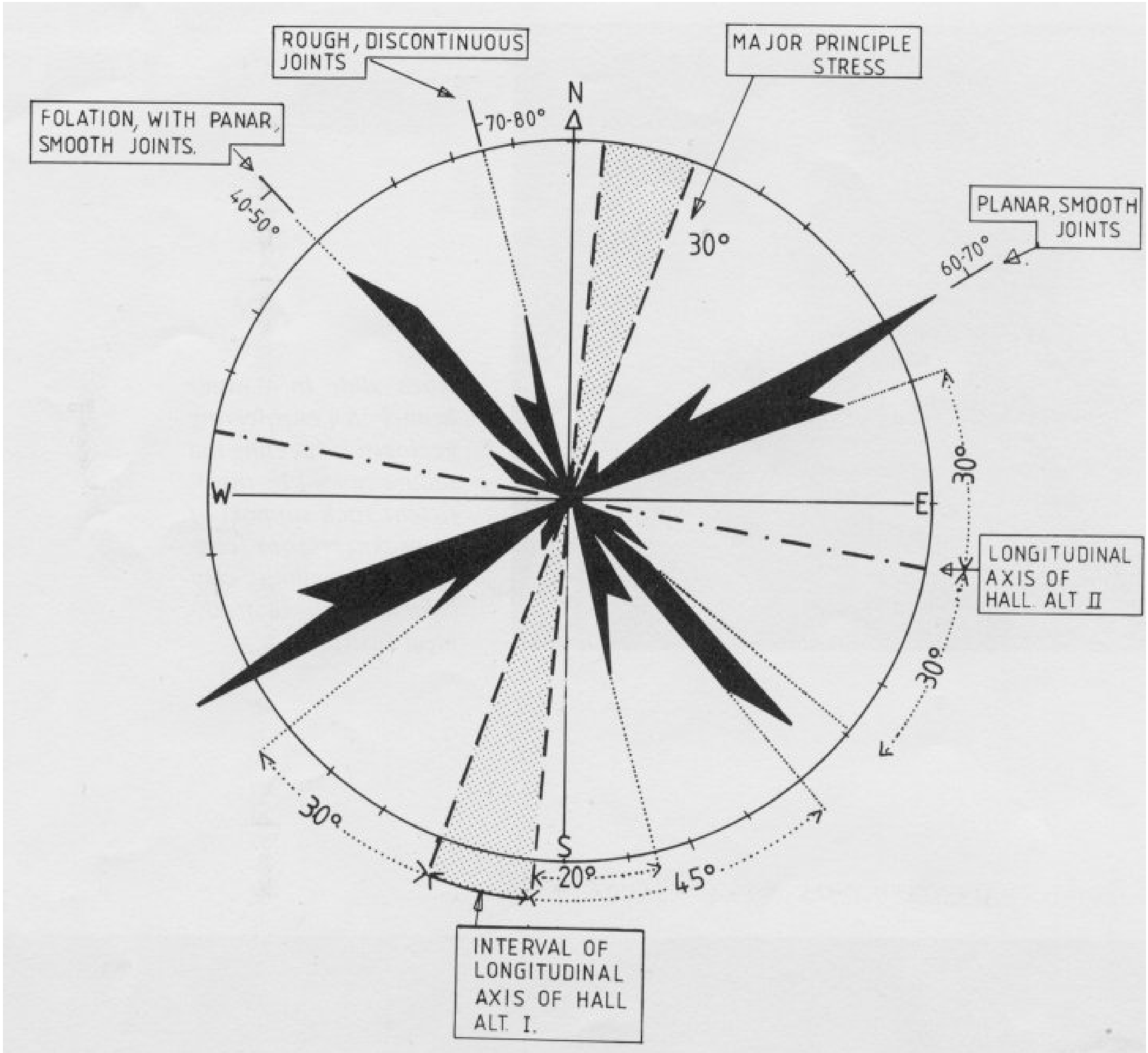


Fig. 7.7
A joint rosette is a simple tool for carrying out evaluations of the most favourable orientation of an underground opening with regard to rock stresses and jointing. (The longitudinal axis of hall alt. II is, however, unfavourable if higher major principal stresses occur)

The entrance of Fjone underground power station. This plant, subjected to high in situ rock stresses, had rock burst problems during construction. (Berdal)

DESIGN AND METHODS OF ROCK SUPPORT

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1. INTRODUCTION

Norway's bedrock consists predominantly of hard rock and therefore a greater part of the tunnel lengths are unlined. But in a majority of the tunnels there are some zones of weakness like faults, joint swarms and alteration zones with swelling clay, or problems with high stress causing slabbing and popping. These occurrences require different kinds of rock support.

The rock support methods used in tunnels and large rock caverns vary to a large extent with the purpose of the excavation and the intended working life of these constructions. Power stations and major highway- and railroad tunnels require for instance far more safety than water tunnels and temporary tunnels and openings.

Only minor parts of the Norwegian tunnels are usually line d. In search of optimum economy combined with safety, Norwegian tunnellers try to find the support measures most appropriate for the given excavation and rock mass quality. Hence, there is great flexibility with respect to the support methods used.

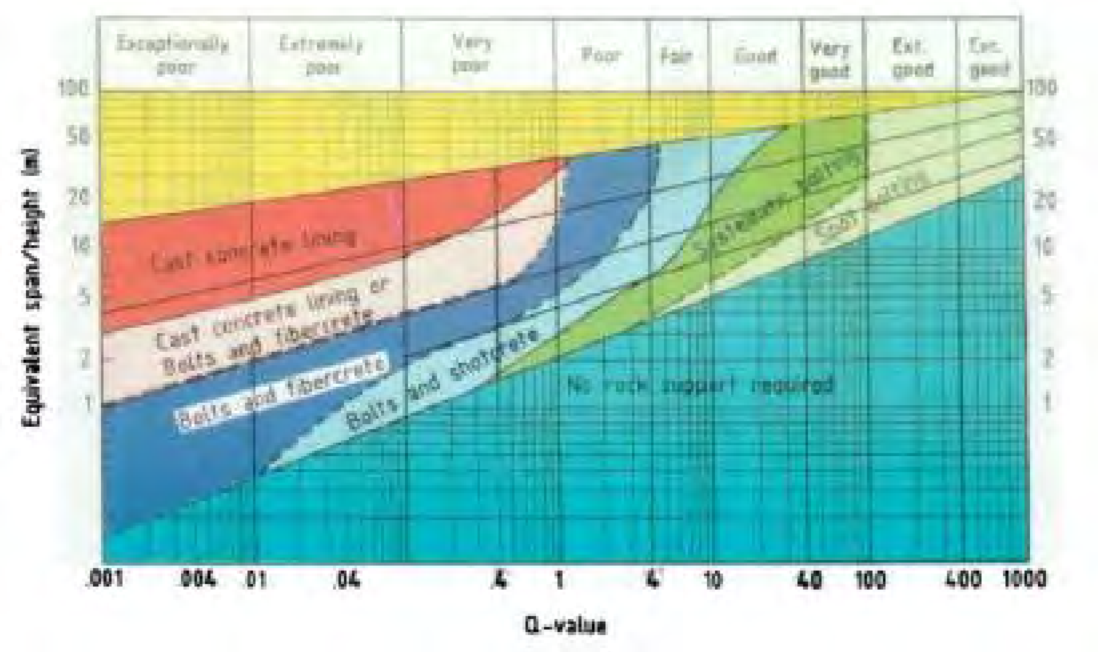


Fig. 8.1 Simplified diagram for design of rock support based on the Q-system

2. THE Q-SYSTEM FOR ROCK SUPPORT DESIGN

The support recommendations are in increasing degree based on the Q-system which is a rock mass classification system developed in Norway in 1974, and since then used widely internationally. The Q-system is an empirical method based on the RQD method and five additional parameters, which modify the RQD-value for the number of joint sets, joint roughness and alteration (filling), the amount of water, and various adverse features associated with loosening, high stress, squeezing and swelling. The rock mass classification is associated with support recommendations based on 212 case records. The Q-value is expressed by

$$Q = RQD/J_n \cdot J_r/J_a \cdot J_w/SRF$$

The numerical value of Q ranges from 0.001 for exceptionally poor quality squeezing-ground up to 1000 for exceptionally good quality rock which is practically without joints. The six parameters, each of which has a rating of importance, can be estimated from surface mapping and verified during excavation. In combination they represent:

1. The block-size, given as the quotient
   RQD/J_n = degree of jointing / number of joint sets

2. The inter-block shear strength
   J_r / J_a = joint roughness/joint alteration of filling

3. The active stress
   J_w/SRF = water pressure or leakage/rock stress conditions

The detailed recommendations for support measures include various combinations of shotcrete, bolting, and cast concrete arches together with the appropriate bolt spacing and lengths, and the requisite thickness of shotcrete or concrete.
The use of the Q-system require an engineering geological understanding and the evaluations must be carried out with analysis of all the important geological features involved. The rock support evaluated from the Q-value and the corresponding tables gives the appropriate amounts and support types to be used. The preliminary Q-value and the corresponding rock support obtained from surface mapping or drill cores should be adjusted to the conditions found during excavation w hen the actual rock masses can be studied.
The equivalent span/height in Fig. 8.1 is found by dividing the actual dimension by a factor representing the safety requirement for the use of the excavation. For underground power stations the factor is 1.0, while it is 1.0- 1.3 for road tunnels and 1.6 for water tunnels.

3. NUMERICAL METHODS OF ROCK SUPPORT DESIGN

An advanced method of adapted numerical design of rock support has recently be en developed by the Norwegian Geotechnical Institute by com- bining Cundals finite element program UDEC (universal distinct element code) with NGI's joint models (joint roughness coefficient, JRC, and joint compressive strength, JCS). In this way the joint strength, deformation and conductivity changes as a result of excavation can be tracked. An example of a simple investigation of a potential failure mechanism in an unlined tunnel using this method is illustrated in Fig. 8.2.
Fig 8.3 illustrates a complex model. Joint conductivity changes with and without rock bolting are being investigated, followed by

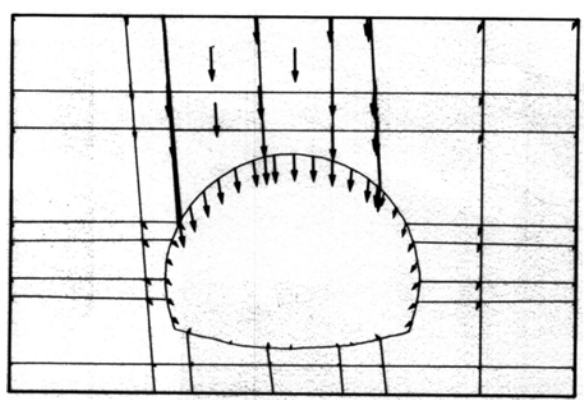


Fig. 8.2 Discontinuum modelling of tunnel stability problems in jointed rock. Arrows show deformation vectors (max. 9. 7 mm) and line thickness shows shear magnitudes (max. 6. 7 mm)

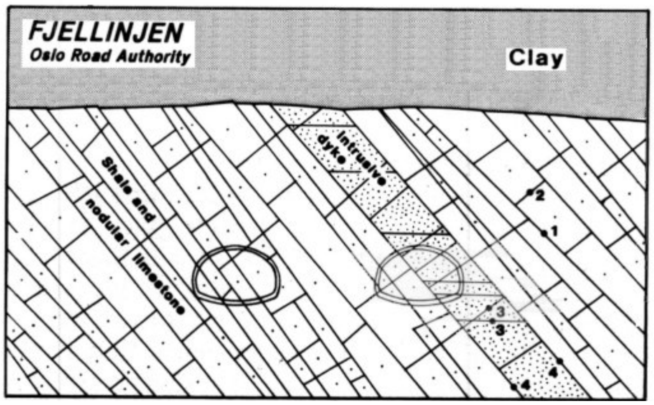


Fig. 8.3 Assumed rock geometry for discontinue modelling of the Fjellinjen road tunnel.

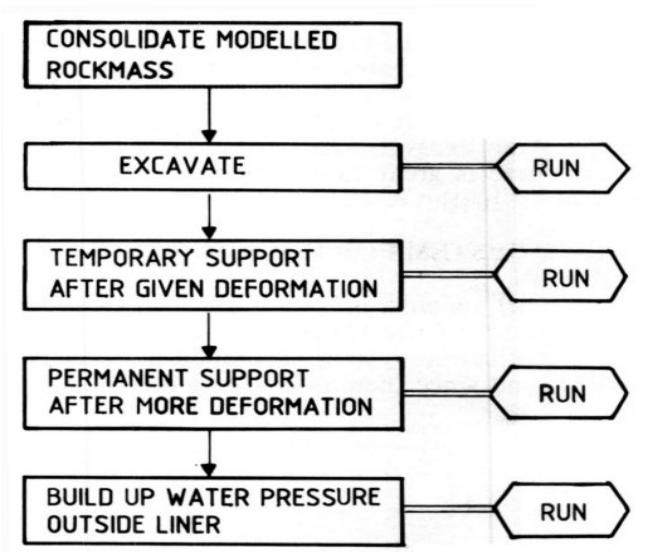


Table 8.1 Numerical modelling, steps.

4. TUNNEL SUPPORT METHODS

In spite of predominantly hard rock tunnelling in Norway, many fault zones, alteration zones, rock burst etc. require rock support ranging from spot bolting to very heavy rib- and rock- bolt reinforced cast concrete. W hen selecting support methods during the excavation of the tunnel it is of great advantage to choose a temporary support system that can act as a permanent support later, or act together with other permanent support methods.

Rock Bolts

Rock bolting is the most common rock support method. We have to distinguish between permanent and temporary rock bolts. For temporary support there is a need for immediate effect on the rock stability, and for a quick and simple installation.

concrete lining design for resisting full ground water pressure.

The numerical modelling philosophy adapted to simulate real conditions as closely as possible, the steps are summarized in Table 1.

In contrast to empirically assisted design, the above numerical procedures provide a wealth of information concerning stresses, deformations and joint displacements, which assist the designer in assessing the effect of the chosen rock support. The above calculation stages can be repeated a limited number of times to investigate the effects of changed rock mass conditions, stress variations etc. However, the day to day changes encountered w hen the tunnel is driven are best accommodated by empirically assisted design (i.e. rock mass classification). The numerical calibration is as much an education for the designers, as a specific solution for an idealized set of assumed or partially measured parameters.

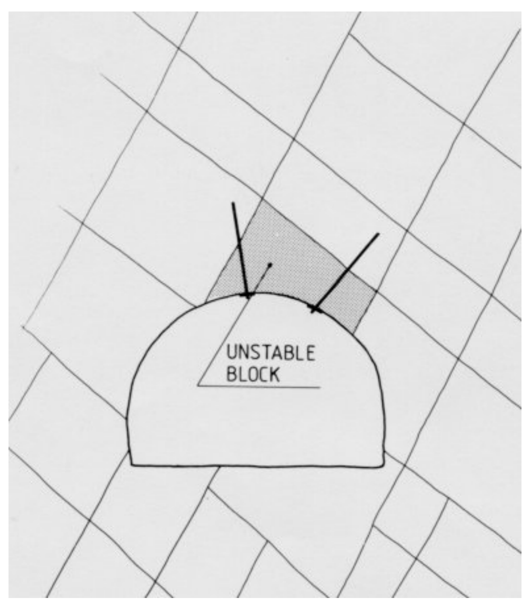


Fig. 8.4
Examples of local rock bolt support.

For permanent support the most important requirement is long time reliability. In many cases much attention is paid to combining all these demands in one single type of rock bolt. A lot of money is saved by such a solution. Because the Norwegian tunnels are mainly driven in hard rock, point anchored rock bolts are widespread as temporary rock support. In hydropower constructions rebar bolts with expansion shells are totally dominating as temporary support. As permanent support un-tensioned grouted rebar bolts are equally dominating. In road tunnel constructions hot galvanized re bar bolts with polyester resin cartridge anchoring are mainly used. In most cases such bolts act both as temporary and as permanent support. This bolt type, when equipped with large triangular plates, is in widespread use against rock burst and slabbing problems.
During the last few years there has been a renaissance for tube bolts provided with expansion shells, installed as temporary support and grouted afterwards so that they can act as permanent support, both in hydropower- and road tunnels. The split and wedge bolts, and the perfo-bolts are now more or less obsolete. The bolt types mentioned above are listed in Table 8.1

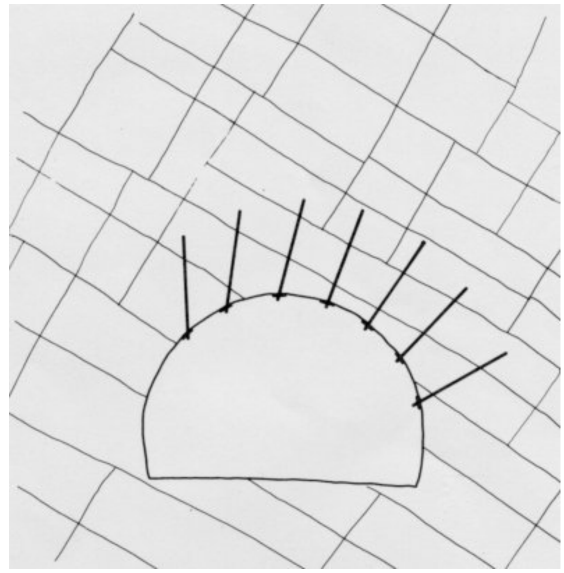


Fig. 8.5
Examples of systematic rock bolt support.

Wire Mesh and Steel Straps
When the rock mass quality is poor, there is a need for surface strengthening between rock bolts. The traditional method has been the installation of wire mesh and steel straps. Usually the wire mesh cannot prevent the rock from loosening, but it can prevent it from falling down. These methods are normally used in tunnels with rock burst problems and in moderately jointed rock in combination with bolts.

Type of Bolt	*Pretensioned*	*Simple and rapid insertion*	*Permit quick tensioning*	*Permanent support*	*Widely used in Norway*
Rebar bolt with expansion shell	+	+	+	--	+
Split and wedge bolt	+	+	+	--	--
Polyester resin cartridge anchored rebar bolt	+	+	+	(+)[1)]	+
Perfo-bolt	--	--	--	+	--
Untensioned grouted rebar bolt	--	+	--	+	+
Grouted tube bolt with expansion shell	+	(+)[2)]	+	+	+
Mechanical or chemical anchored and grouted bolt [2)]	+	--	+	+	--

1) In most cases when hot galvanized
2) Installed in two working cycles

Table 8.2 Application of rock bolts

Shotcrete
Under poor rock quality conditions shotcrete has been used to stabilize the rock surface in tunnels. During the last years steel fibre reinforced shotcrete, combined with rock bolts, has become the dominating way of stabilizing underground structures. The wet process shotcrete is superior to the dry process which is on its way out. The use of concrete additives such as micro silica and fibre reinforcement together with advanced equipment has made it possible to produce a high strength shotcrete of homogeneous quality. When rapid support is required, a mix with accelerator is used, resulting in high early strength.

New experiences have shown that the concept of combining information on rock mass conditions in the Q-system with use of steel fibre reinforced shotcrete and rock bolts, gives a rapid estimate of cheap and reliable tunnel support.

There are many reports of conditions where a conservative estimation of necessary rock sup- port would have resulted in cast concrete lining, but where the new concept of fibre reinforced shotcrete and rock bolts was used successfully. The same concept has been used for bursting rock conditions.

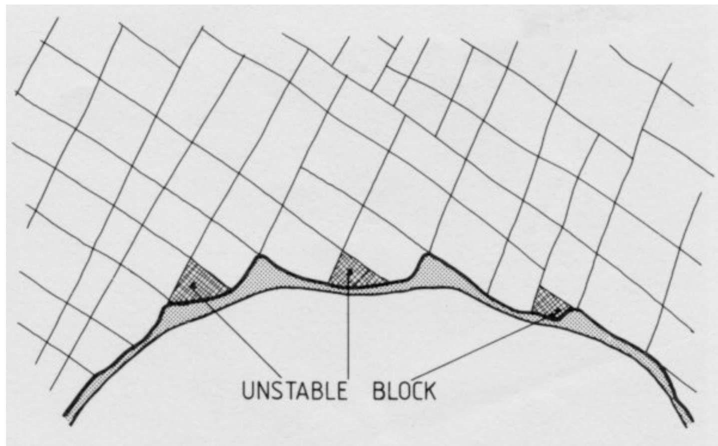


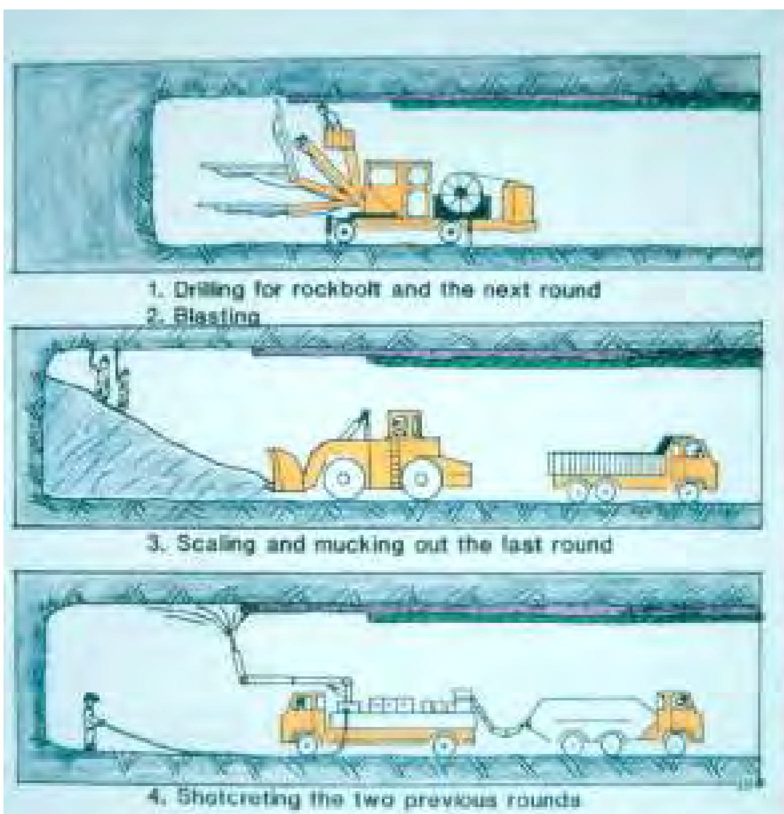
Fig. 8.6
Example of shotcrete support

The older, labour intensive method of shotcrete reinforced by welded wire mesh is now largely replaced by steel fibre reinforced shotcrete.

Cast Concrete
In fault zones with swelling clay or crushed rock with very poor rock mass conditions the traditional support method has been in situ cast concrete in steel moulding. This is ex pensive and time consuming work, which in many cases is executed close to the tunnel face.

Rock bolts equipped with triangular plates and net, in a tunnel subjected to rock burst.
(Norwegian Geotechnical Institute)

Rock bolts, steel straps and net, in a tunnel subjected to heavy rock burst. (Norwegian Geotechnical Institute)



Combined use of rock bolts and shotcrete. Shotcrete, and fibercrete, are used more and more in Norwegian underground works. (Norwegian Geotechnical Institute)

The top heading of the Saurdal underground powerhouse. Roof support is carried out with fibercrete and systematic rock bolting. (Statskraft.)

NoTCoS

NORWEGIAN TUNNELLING CONTRACT SYSTEM

Erland Kleivan
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1. INTRODUCTION

This paper deals with such contract provisions only that are regarded as being particular for the Norwegian tunnelling industry .Although hard to prove, it is believed that these provisions which are related to risk-sharing, have served to minimize both cost and time for implementation of tunnel projects.

Risk in this context is to be understood not as the exposure to bodily injury in tunnelling but solely as the risk of financial losses for the case that the ground conditions along the tunnel should turn out to be more difficult than anticipated.

Over the last decades, tunnelling in Norway has been progressing at a rate of between 100 and 200 km per year, most of it as contract work and nearly all of it hard rock tunnels that are basically unlined. Hard rock conditions, how- ever, do not always mean stable ground. There are examples to the opposite, one in particular with conditions so adverse that 80% of a 30 km tunnel waterway had to be shotcreted or concrete lined, against the normal 3-10%.

The contracts practise used in the past, until the late '60s, did not provide for enough flexibility in regard to the sharing of risk and responsibility between owner and contractor w hen ground conditions became adverse and penalty dates were being overrun. Still more time could be lost during the discussions that followed, and in some instances the parties had to go to court.

The principle of risk-sharing that was first introduced 15 to 20 years ago has since then been accepted and appreciated by both owner and contractor and is, in its present-day refined form, a part of nearly all Norwegian tunnel contracts.

It should be' mentioned here that the unit price system with flexible quantities is used as the guiding principle in all tunnel contracts in Norway.

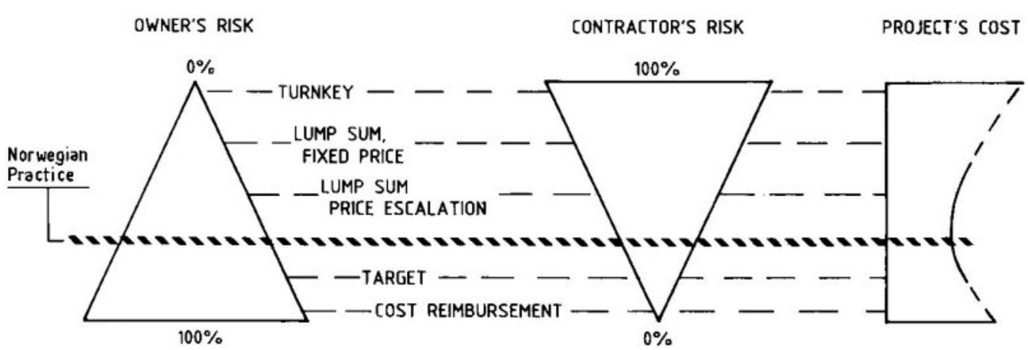


Fig. 9.1 Risk sharing according to type of contract and assumed influence on project cost

2. WHY THE RISK SHOULD BE SHARED

ome significant arguments for a sharing of the risk in tunnel contracts are the following:

The ever present uncertainty of what the actual ground conditions are: Field investigations normally include few, if any, core drillings, and from surface observations alone it is limited how accurately the conditions at tunnel level can be predicted at the time when the contract is signed.

The cost: If the contractor should assume all risk he would necessarily ask a very high price in order to be safe, a price that most likely exceeds the actual cost, thus in the end giving the owner less value for his money. Should on the other hand the actual ground conditions be such that the contractor goes broke during the contract period, also that situation may have an undesirable effect on the owners economy.
Court proceedings in the wake of a contract are costly and time-consuming, and may do more good to the lawyers than to the two parties involved.

3. HOW THE RISK CAN BE SHARED
The main principle of risk-sharing in tunnel contracts is to give to the parties a tool for converting work into time-equivalents, a tool with which they can later meet any contingency arising from changed ground conditions, eliminating any future discussions over regulation of the construction time where needed, or over costs incurred. All such problems are foreseen and the answers are to be found in the contract.
The formula developed to meet this end contains the following ingredients:

- Contract documents where all types of rock support and other work that may possibly be needed are described. Bach one of these activities shall be specified in the Bill of Quantities, as a separate pay-item. The unit price for that item shall maintain its valid it y even if the actual quantity should deviate widely from the one given in the contract documents.
- Time parameters, quoted in the contract documents, by which any one of those work activities expected to influence progress can be converted into time intervals that reflect the time this activity consumes, such as the placing of one rock bolt of specific length, or the lining with concrete of 1 linear m of the tunnel. Such work will have different impacts on the construction time w hen it is performed at the tunnel face where it interferes directly with the excavation cycle, and w hen it can be postponed to be installed behind the face where it interferes only marginally with the excavation cycle. Therefore, two sets of time parameters must be quoted to cover both situations.
- Provisions in the contract for adjusting the contractual construction time, calculated by means of the work volume presented in the Bill of Quantities (the reference ground conditions) and the time-equivalents system described above, to the actual ground conditions encountered during construction.

An example of how the time equivalents system work is shown in Fig. 9.2, where the object is a tunnel of approx. 5 km length and 32 m^2 cross section, excavated from one heading. The reference ground conditions can be read from the "Contractual quantities" column which is in fact a summary of those pay-items from the Bill of Quantity that may influence progress.
The actual quantities as recorded during construction exceed the anticipated quantities by more than 13% in this case, indicating an extension of the construction time by a little less than 15 weeks.
A great advantage of this "time-equivalents system " is that the contractors incentive to meet the penalty deadline will be maintained; changed ground conditions and changed volumes of rock support may shift the completion date one way or the other, but then strictly in accordance with the rules and regulations laid down in the contract.

4. THE ROLE OF THE ENGINEERING CONSULTANT
The most important contributions from the consultant come in the prediction of the geology and in the assessment of parameters for converting work into time-equivalents.
The so-called reference ground conditions, reflected in the Bill of Quantities for which the consultant is responsible, are based on the best judgement of the engineering geologist after an evaluation of all reports from the field. Normally it is also the consultant who is responsible for

Activities/work that influence progress	Time equi-valents	Contractual conditions		Actual conditions	
		Quantities	Time	Quantities	Time
	Shifts per unit		Shifts		Shifts
1 *Excavation of tunnel, excluding downtime caused by rock support or pregrouting*	0.156	5 000 m	778.0	5 040 m	786.2
2 *Rock support at the face*					
Rock bolting	0.017	4 000 no	68.0	8 332	141.6
Wire mesh	0.017	500 m²	8.5	630 m²	10.7
Rock straps	0.004	400 m	1.6	910 m	3.6
Shotcreting	0.026	340 m³	8.8	1 012 m³	26.3
Concrete lining (roof and walls)	0.313	500 m	156.5	389 m	121.8
Extra scaling (by 3 men)	0.044	1 550 mhrs	68.2	2 690 mhrs	118.4
3 *Rock support behind the face*					
Shotcreting	0.022	435 m³	9.6	775 m³	17.0
Concrete lining (roof and walls)	0.278	50 m	13.9	123 m	34.2
Total no of shifts			1 115.1		1 259.8
Total no of weeks			111.51		125.98
		Extension in contractual time		14.47 week	

Shift Regulation: 7.5 hrs shifts, 10 shifts per week, 45 weeks per year

Fig. 9.2 Sample tabulation for converting work into time

establishing the time parameters that are laid down in the tender documents. These parameters are highly sensitive figures; if it should seem to the bidders that the parameters are on the tight side and may become hard to live with, this could be reflected in higher bid prices. Proof of well estimated parameters can be obtained during the work, by recording progress including support and other impediments over a given time period, for instance one week which in Norway normally equals 10 shifts. If that progress is converted to "contract time" by using the time parameters, the average outcome should be near the actual 10 shifts, say between 9.5 and II shifts.

Fig. 9.3 demonstrates, in part, well estimated parameters and well predicted geology on the right hand side. In the left portal area, however , the difficulties were grossly underestimated. In addition the rock mass conditions along the deep part of the tunnel were worse than anticipated. The extension of the construction time came to 23 weeks. This case history from the offshore industry is one of three consecutive fjord crossings in tunnel of a gas pipeline from the North Sea, cross section 27 m^2.

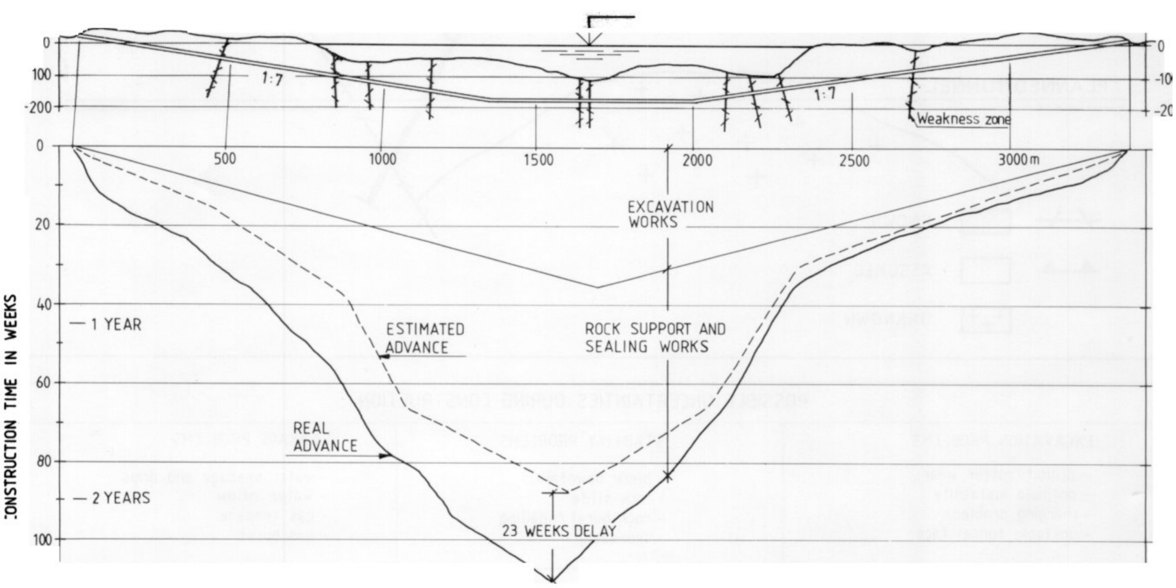


Fig. 9.3 Advance diagrams. Sub sea tunnel driven from both sides

5. THE ROLES OF OWNER AND CONTRACTOR

It is essential that the two parties in a tunnel contract maintain a close contact in the field during the construction period, and that their field representatives are fully experienced in the work at hand and are furnished with the necessary authority to make binding decisions at the site.

Rock support is normally installed in two steps: An initial support to provide safe work conditions for the crew, and a final support for the purpose of preparing the tunnel for its operation condition. The initial support, normally installed at the tunnel face, is the responsibility of the contractor, even if these installations later become an integral part of the final support system.

The final support measures, in most cases installed behind the face, are on the other hand directed by the owner.

The fact that the ground conditions may change continually necessitates frequent counsel by owner and contractor to decide on what support measures to apply for an optimum solution, technical and economical, and to prevent loss of time.

6. EXPERIENCE FROM PRACTICING RISK- SHARING

There is no evidence of any risk-sharing system similar to the one described here having ever be en practiced outside Norway, and the practice may therefore in fact be unique.

In the period after the risk-sharing provisions became common in Norwegian tunnelling con- tracts, a total of 2600 km tunnel has been bunt here. As some owners prefer directly employed labour to contracting, only an estimated 80% of this total may have be en contracted, meaning that approx. 2000 km are implemented with provisions of equivalent time risk-sharing in one form or another bunt into the contract. No lawsuits or arbitrations with relevance to changed ground conditions are reported from this period. One cannot guess how many cases would have gone to court without such provisions in the contracts, but there would undoubtedly have been some: In the period prior to the time when risk-sharing came into common use there were several instances of legal proceedings in the wake of tunnel contracts, although the number is uncertain.

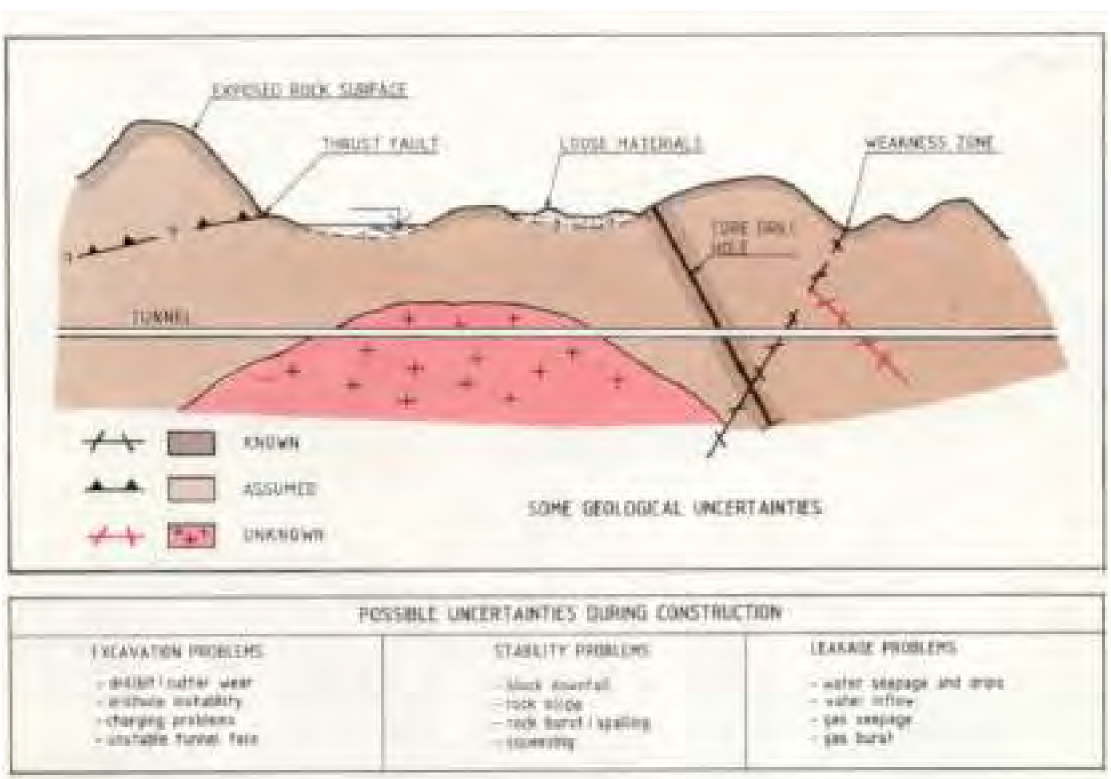
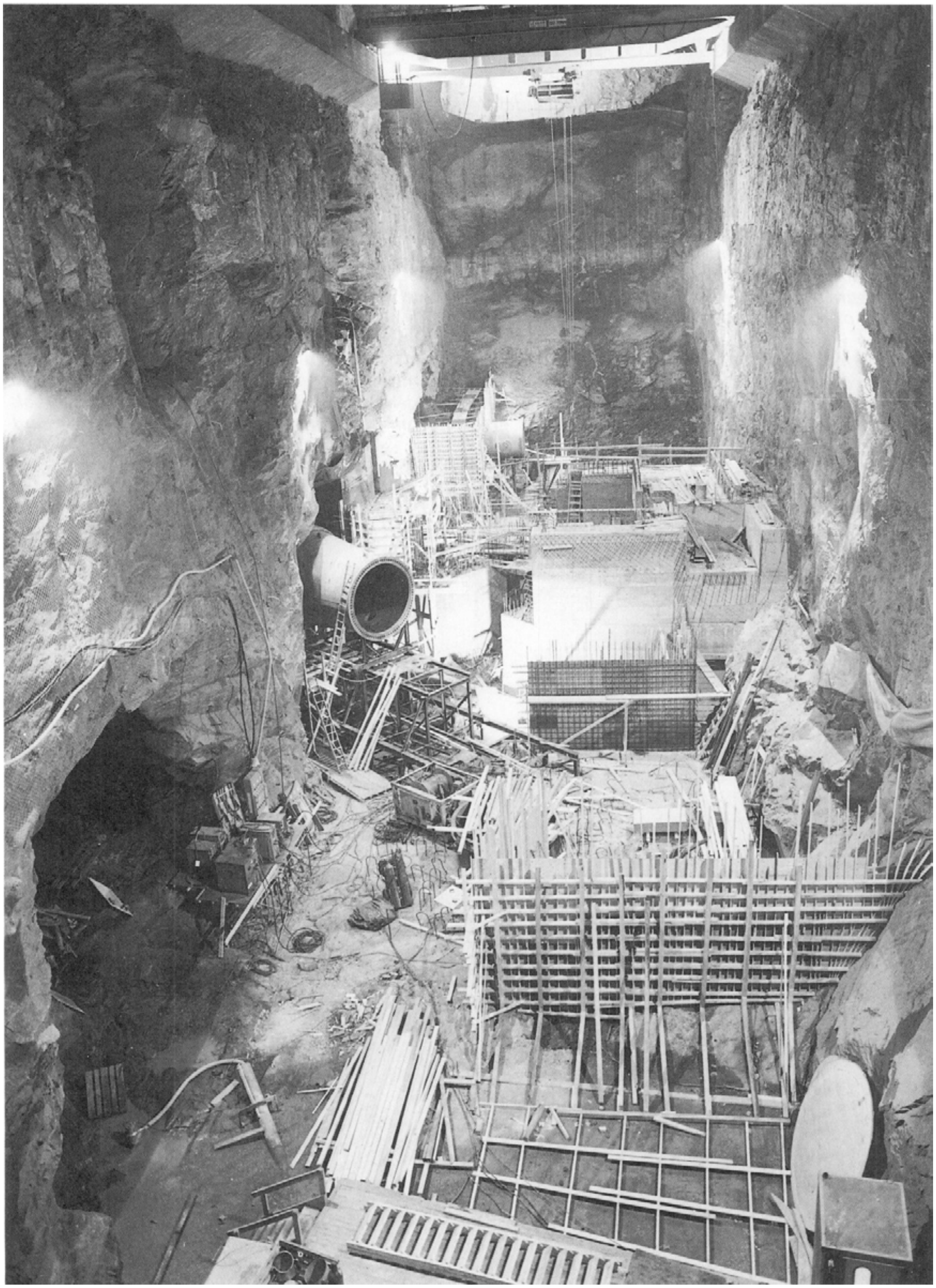
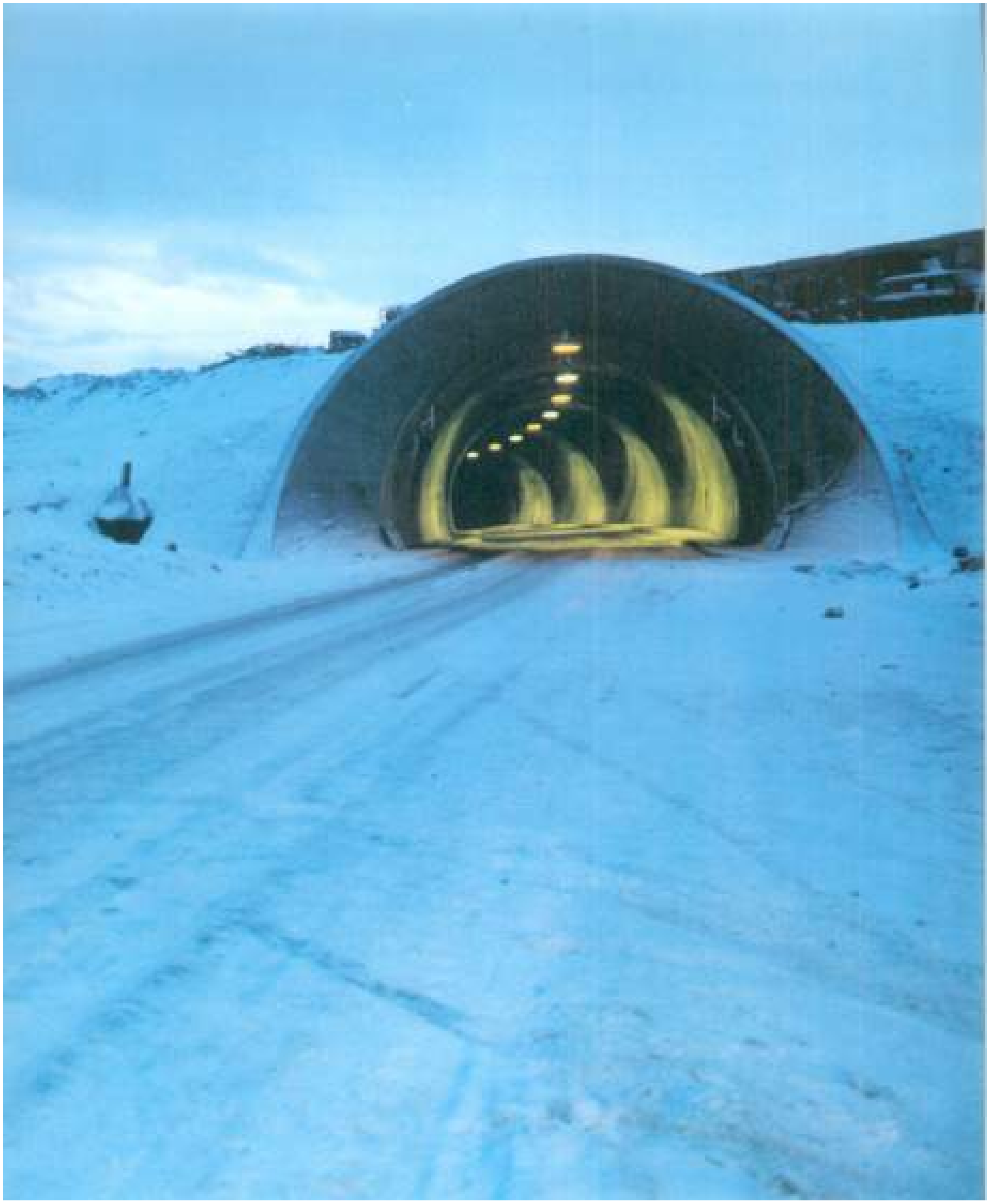


Fig. 9.4 There are many geological uncertainties to face in rock tunnelling contracts



The Alta underground powerhouse during construction (Photo: Ole M. Rapp)

Intake structure at Hylen power plant (Statkraft)



The Vardø sub sea road tunnel. The eastern portal of the 2.9 km long $50m^2$ tunnel in northern Norway. (Selmer-Furuholmen Anlegg)

Lake tap blast in the Rembesdalsvatn reservoir (Statkraft)

UNLINED HIGH PRESSURE TUNNELS AND SHAFTS

Arild Palmstrøm
Ing. A. B. Berdal A/S

1. INTRODUCTION

Unlined means that no steel or continuous concrete lining is installed in the shaft or tunnel, with the result that the rock itself is under direct pressure from the water .

The application of unlined pressure tunnels and shafts in Norwegian hydropower construction started as early as 1919. The main reason was shortage of steel for penstocks during and after the First World War.

The benefits of the unlined design became more evident when Norwegian power houses were put underground in the 1950s, and from the mid 1960s the unlined pressure shaft solution became traditional. From the late 1960s the design with unlined pressure tunnels and surge chambers with air cushion was introduced. Fig. 10.1 shows the development of steadily increasing heads in Norwegian unlined pressure conduits till today when more than 80 unlined pressure conduits with water head in excess of 150 m are in use.

Fig. 10.2 shows the three main types of design solutions in current use. When the power house is located underground, the distance with steel pipe from the turbine to the unlined tunnel shaft portion can be made very short. This is highly beneficial since the cost of such high pressure steel conduits and their installations is often very high.

The total length of unlined high pressure shafts and tunnels in operation in Norway today is not known exactly, but is estimated to exceed 100 km.

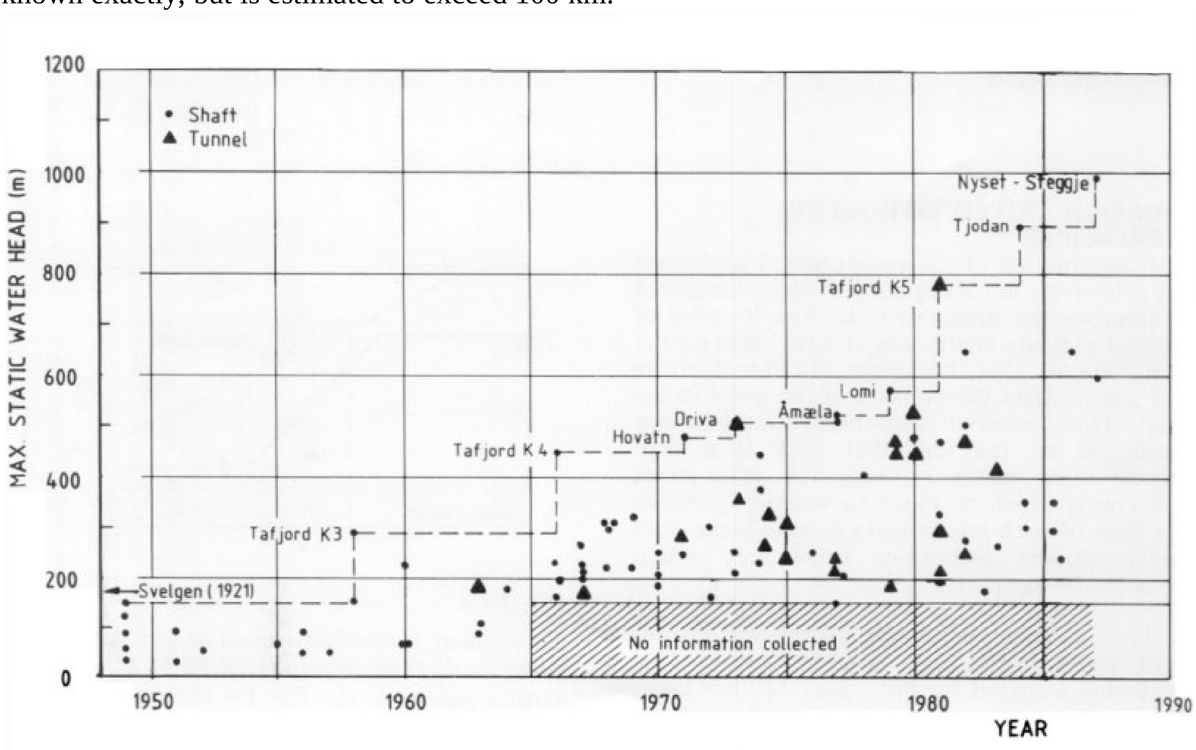


Fig. 10.1 Development of Norwegian unlined high pressure tunnel and shafts

2. ROCK CONDITIONS REQUIRED

An unlined pressure conduit requires rock conditions able to withstand the internal water pressure both with regard to leakages and to deformations which can lead to failures. The rock must therefore have low permeability to ensure small leakages only. Even where the rock mass permeability is low, water will migrate into or out of a tunnel depending on the relation between natural ground water pressure and the pressure in the tunnel, i.e. the gradient.

As for all rock tunnel waterways the rock mass conditions must be suitable for tunnelling. In most Norwegian hydropower projects there are portions of poor rock mass conditions where comprehensive rock supporting has to be installed. In such rock masses also seating works must of ten be: carried out in the unlined conduits to reduce possible water leakages and prevent washing out of soft gouge materials.

3. DESIGN AND CONSTRUCTION PRINCIPLE

The construction of the many unlined waterways has provided a lot of experience which has served to improve the design criteria. The location of unlined pressure shafts was at first based on the simple theory that the weight of the rock above was greater than the pressure of the water in the shaft. This somewhat conservative method was ascribable to the fact that rock is a non- homogeneous material intersected with joints and cracks which do much to weaken it. Along the lines of such cracks leaks tend to occur, and under adverse conditions these may attain considerable proportions.

In 1972 a better simulation model was introduced, based on the finite element method. This work was initiated by Prof. Rolf Selmer-Olsen. The model makes use of the principle that the minimum main stress in the rock should not be

is arrived at by transferring the scheme to topographical models adapted to local conditions. In determining the final placing of the scheme, however, special attention has to be paid to any significant geological factors that may be present.

A set of standard two-dimensional FEM diagrams that have been worked out represent a useful tool in the feasibility stage of the project. They make it possible to find a preliminary location of the

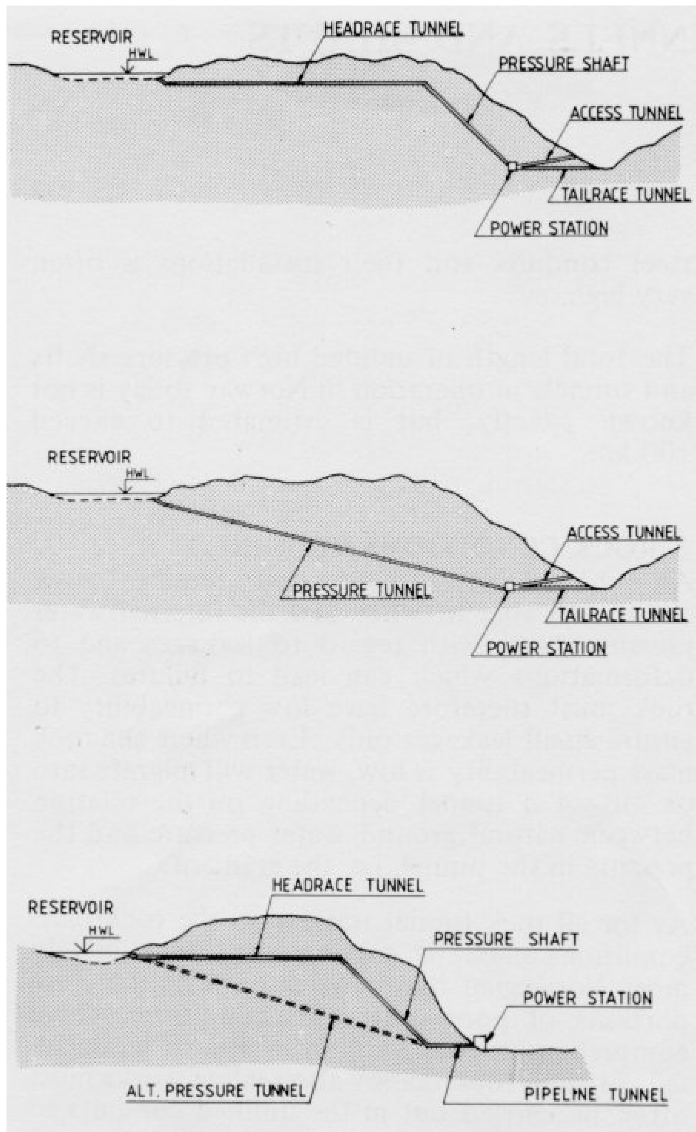


Fig. 10.2 Different design solutions with unlined pressure tunnels and shafts

pressure tunnel/shaft, a location which in many cases turns out to be the final one. As most power houses are located inside valley sides, these diagrams represent valley slopes varying from 14- 75.º
A controlled and slow filling up of the waterway is an important part of the safe construction of an unlined pressure system. Normally a shaft or tunnel is filled in steps with intervals of 10-30 hours. - During the pauses the water level is continuously and accurately monitored by an extra sensitive manometer. This makes it possible to calculate the net leakage out of the unlined pressure tunnel/shaft into the surrounding rock masses.

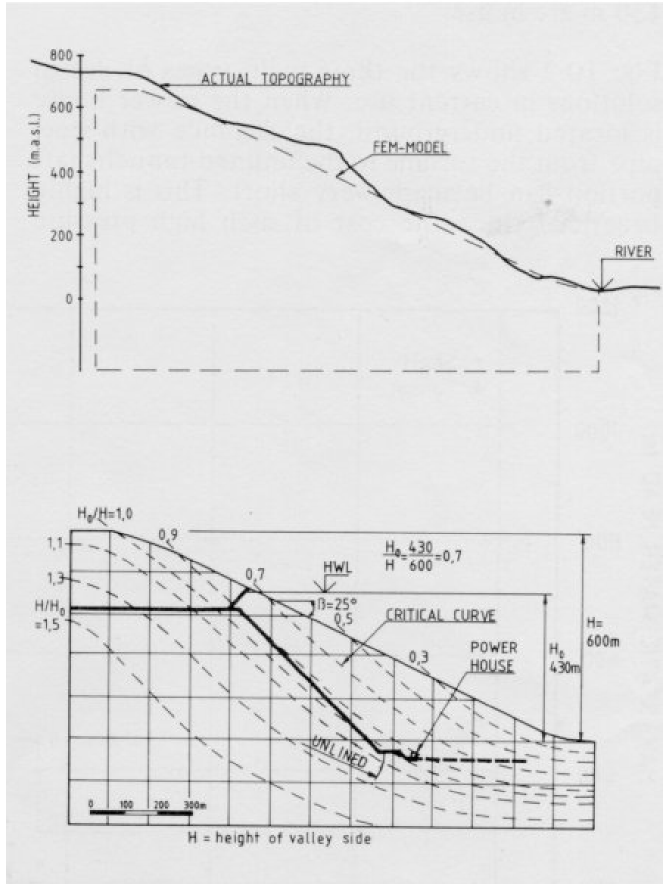


Fig. 10.3 Example of preliminary location of an unlined pressure shaft based on a standard FEM model. With a safety factor F = 1.4 the stress curve $H_0/H = 0.9$ is applied

Table 10.1 Some of the Norwegian unlined shafts / tunnels ⇩

Name of hydropower project	**Date of commiss-ioning**	**Max static head (m)**	**Unlined section**	
			Type (inclin. –cross section)	**Rock type**
Svelgen I	1921	152	Shaft (45º - 4,5 m^2)	Quartzite
Balmi	1958	150	Shaft (45º - 16 m^2)	Phyllite
Tafjord III	1958	286	Shaft (32º - 6,2 m^2)	Gneiss
Byrte	1968	303	Shaft (60º - 62 m^2) (failure occured)	Granite Gneiss
Hovatn	1971	475	Shaft (45º - 7 m^2) Tunnel (1:14 – 12 m^2)	Granite Gneiss
			Tunnel (1:12 – 22 m^2)	
Driva	1973	510	Shaft (45º - 4,5 m^2) Tunnel (1% - 8 m^2)	Gneiss
Tafjord V	1981	780	Shaft (45º - 8 m^2) Tunnel (1:10 – 15 m^2)	Gneiss Dunite
Tjodan	1984	875	Shaft (41º - 7,5 m^2) (TBM-driven)	Gneiss
Nyset-Steggje	1986	964	Shaft (45º - 8 m^2) (TBM-driven)	Gneiss Granite

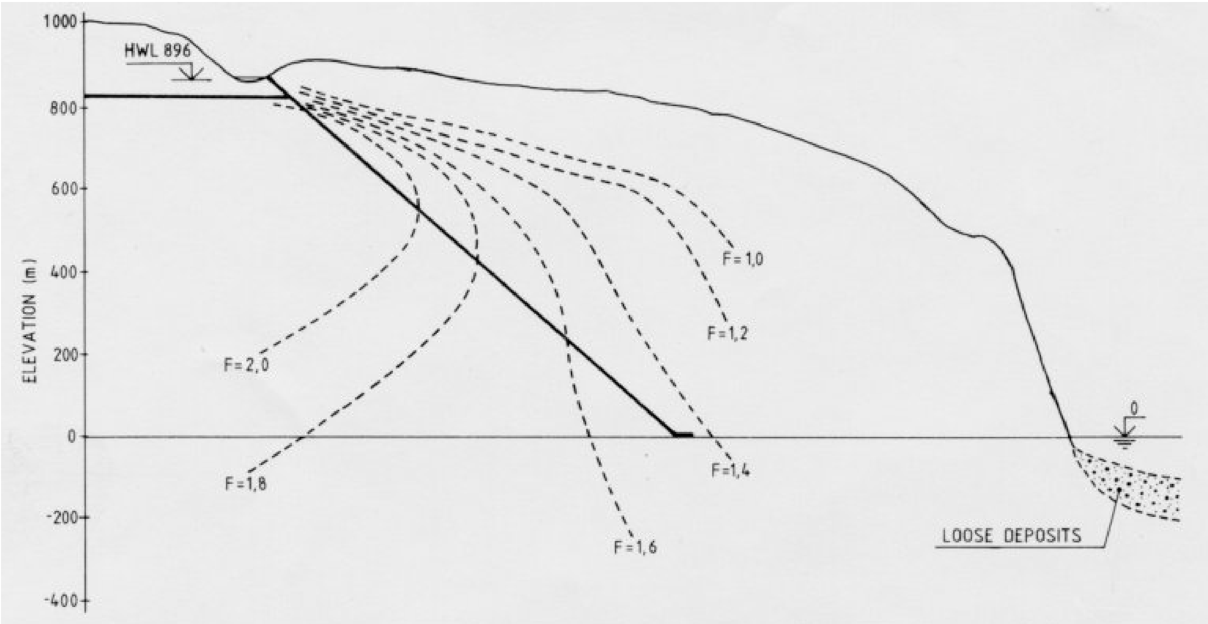


Fig. 10.4 From the FEM analysis and the measured in situ rock stresses it is possible to calculate the factor of safety along the unlined pressure shaft with regard to hydraulic splitting and failure. The example is from the Tjodan power plant with 880 m head on unlined rocks

4. EXPERIENCE

From the six pressure tunnels/shafts where leakage measurements have be en carried out, a leakage of 0.5-5 l/s per km has be en measured.

The benefits from the concept pressure shaft/tunnel are these :

of unlined

- Cost savings in construction caused by the fact that the lining with concrete embedded steel penstock is omitted.
- Shorter construction time meaning an earlier start-up of the power plant, and reduced capita] costs.
- Simpler design of the water ways. In many cases it is possible to omit constructions adits which in areas with steep topography can be of substantial costs.

Norwegian experience shows that up to 5% of the construction cost can be saved applying the unlined design. A high portion of this is gained from the possible earlier start-up of the power plant.

EXPERIENCE FROM AIR CUSHION SURGE CHAMBERS

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SINTEF,
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1. INTRODUCTION
The application of unlined air cushions as surge chambers is one of the latest developments in Norwegian hydropower design. Under certain topographical conditions the possibility of using the air cushion surge chamber in the unlined headrace tunnel offers interesting constructional cost savings. This is so because the usual surge chamber vented to the atmosphere at the top of a pressure shaft can be omitted, and construction costs related to access to this area are saved. At the Kvilldal power plant a cost reduction of some 5 mill US$ (1987) was achieved, which was the main reason for choosing the air cushion concept in combination with an unlined pressure tunnel. Compressed air surge chambers have been used since 1973. As of 1986, this concept has been chosen for a total of nine power plants.

2. REVIEW OF MAIN FEATURES
Air surge chambers have been constructed for internal pressures ranging from 19 to 78 bar, storing air volumes up to 80 000 m^3. These unique gas storage facilities are all located in Southern Norway. The host rock is mainly little to moderately jointed gneiss, except for Brattset where moderately to highly jointed phyllite dominates. Within the bounds given by operational requirements, the optimal location is based on an evaluation of the rock mass and the rock stress conditions. The surge chambers are carefully sited away from weakness zones and extensive discontinuities.

There are no special geological requirements connected to surge chamber compared to other underground openings. Locating the chamber is mostly done using the same evaluations as for unlined pressure shafts and tunnels mentioned in Chapter 10. To avoid air leakage through the host rock, the air pressure must not exceed the ground water pressure at any point in the chamber. The final location is often based on supplementary geological data and ground water recordings obtained by exploratory drilling and rock stress measurements performed from the pressure tunnel while that is being excavated.

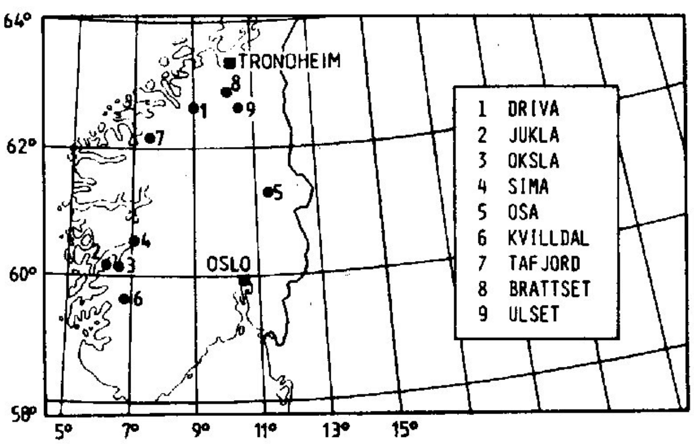


Fig. 11.1
Location of hydropower plants with air surge chambers {Southern Norway)

Main features for the nine Norwegian air cushions are given in Table 11.1. The ground water pressures used for the pressure ratio column are in some cases estimated on the basis of topography and probable flow pattern. Air loss given is defined as the total loss of air mass as a proportion of the average mass stored. This includes leakage through the rock and air dissolution in water bed.

At Osa, the air leakage through the rock mass was reduced by an order of magnitude by grouting. The air loss after grouting is given in Table 11.1.

Table 11.1
Main data for air surge chambers constructed by 1985. Cover is vertical thickness of rock above the cavern. Air pressure and volume are maximum values. Water pressure is that estimated for the host rock during operation.

Site	**Rock cover m**	**Air pressure bar**	**Air volume 1000m³**	**Pressure ratio Water/Air**	**Air loss %/Year**	**Rock type**
Driva	1110	42	3	>1,4	11	gneiss
Juksla	350	24	5,3	0,6	<100	gneiss
Oksla	655	44	12	0,9	9	gneiss
Sima	425	48	7	0,7	8	gneiss
Osa	145	18	10	0,5	320	gneiss
Kvilldal	522	41	80	1,2	2	gneiss
Tafjord*	658	78	1,2	0,4	1600*	gneiss
Brattsett	150	24	6	0,6	60	phyllite
Ulset	264	27	3,6	0,8	<10	gneiss

*Not in operation due to excessive air leakage

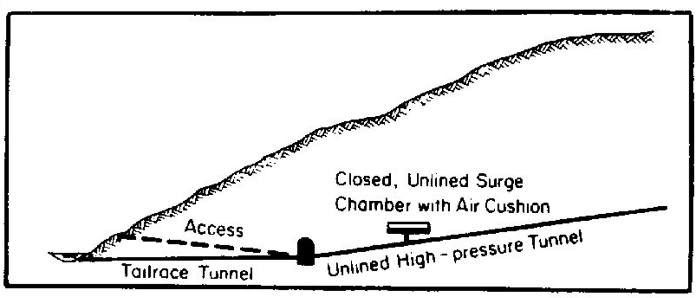


Fig. 11.2 Air surge chamber located at the end of the headrace pressure tunnel

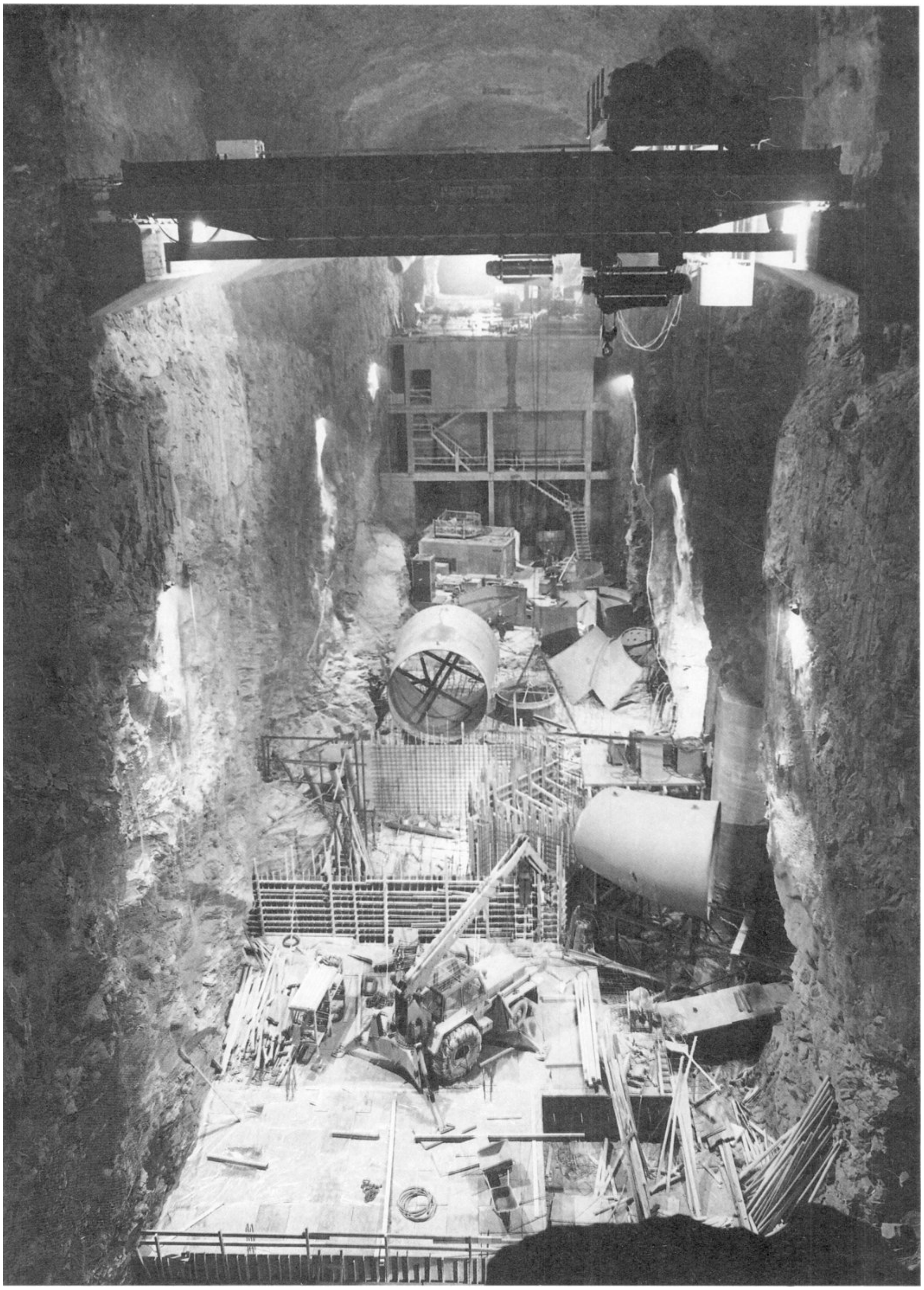
At Kvilldal it seems that the initial air leakage of about 60% per year through the rock mass has been eliminated by installing a water curtain. Kvilldal is the only site where artificial ground water control by water curtain is used.
The air loss rate at Tafjord is based on rather few observations as the air cushion was never filled with air. An attempt to reduce leakage through the rock mass by grouting has not been successful. The air cushion is not currently in use due to excessive air loss and because it is not essential to power plant operations. Deformation caused by pneumatic jacking has been suggested as the reason for the high air loss rate.

3. EXPERIENCE

Of the nine air cushions, six have operated without maintenance since initial construction. One has been improved by grouting and one by a water curtain installation. One of the air surge chambers is not functional due to excessive air loss.
The conclusions from the experience so far of the air surge chambers in Norway are as follows:

- The annual air loss rate between 2 and 320 percent of the average amount of gas stored per year is dependent mostly on the hydro- geological conditions.
- A minimum air loss rate of about 2% resulting from dissolution of air in the water bed appears to exist. At least two, if not as many as five, of the surge chambers seem to be operating in this fashion. One of these facilities with low air loss rates is equipped with the artificial water curtain mentioned above.

- Water curtains can be used to reduce air loss to the minimum dissolution loss rate. The effect of seating by grouting is more difficult to predict; in one case this method has been used with success.



The Alta underground powerhouse during installation of the draft tubes.
(Photo: Ole M Rapp)

IV

NORWEGIAN TUNNELLING SPECIALITIES

In Norway, tunnelling specialities have been developed during many years of active underground construction. The close cooperation between the different professions involved in planning and execution of tunnel projects have been most important for the techniques and practices developed. In spite of a thorough geological mapping of well exposed rocks at the surface, and comprehensive field investigations, there will always be uncertainties attached to the estimated rock mass qualities for constructions deep below the surface. In Norway the problems such uncertainties may cause are solved by applying contractual risk sharing drafted such that they meet the variations in estimated and actual quantities. The contractual risks are shared between the owner and the contractor both with regard to costs and construction time. This practice has not only eliminated lawsuits, but also lowered overall construction costs.

Some Norwegian tunnelling specialities are mostly a result of design improvements, while others are merely constructural refinements. Many of the developments have been initiated from hydropower construction after the World War II. Also the other underground works like road tunnels and rock cavern storage installations have benefited greatly from the experience earned in underground hydropower construction.

Today's high activity in underground construction and the many challenges to be met in Norwegian tunnelling, surely will move the tunnelling development further onwards. Of outmost importance for this exiting Norwegian development is the interdisciplinary cooperation between the professionals engaged in tunnel planning, design and construction.

Fig. IV The planned Eiksund fjord tunnel will improve communication to a large island. Sub sea tunnelling is a Norwegian speciality (Eiksundsambandet)

UNDERGROUND HYDROPOWER STATIONS

Erland Kleivan
Ing. A. B. Berdal A/S

1. INTRODUCTION

The partiality for sub-surface powerhouses in Norway has brought nearly 90% of the country's generating capacity underground (1987). Both safety and cost aspects are strong arguments in favour of going underground.

Contributing to this accomplishment is, aside from the generally suitable rock conditions, a fore fronting geo-engineering discipline. Compact design of powerhouses is achieved through reducing the cavern span by various means, by omitting any facilities that may be superfluous, and by converting excavated space needed for construction such as auxiliary adits to permanent use.

As early as 1916 the first hydropower unit was installed in a rock cavern. It was a small size auxiliary unit, part of the larger above ground located Såheim Power Station. The first full-fledged underground hydropower installation, Bjørkåsen Power Plant, was commissioned in 1923. But only from about 1950 it became normal procedure to place powerhouses below the surface. By 1987 there are an estimated 180 such installations in Norway, and of the new powerhouses presently being designed or built, nearly 100% are to be put underground.

2. WHY UNDERGROUND?

Among the first hydropower schemes developed were the low-head river plants, the majority of which were built before 1950, and with their powerhouses placed above ground as the most suitable solution. W hen time then came for the medium and high-head projects, the Second World War had just made it quite clear, both how vital the energy supply was to a nation and how vulnerable it was to acts of war. And since these new power projects were equally well suited for the underground, the logical solution was to place the powerhouse, with the components that are most indispensable and least replaceable in the supply system, safely below the surface.

It quickly became evident that the underground location had more advantages than sheltering only. The rock excavation technology was advancing rapidly, cutting costs, relatively more for underground locations than for above ground ones. Also, without the adverse impact from the weather, the reduced wear and tear to the structural components of the underground powerhouse served to reduce maintenance costs. The underground therefore has become the preferred solution also from an economical view point.

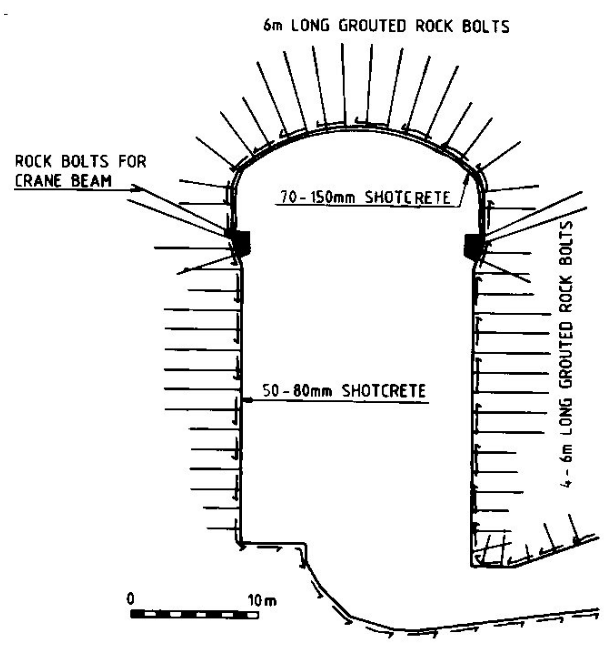


Fig. 12.1 Modem rock support of cavern roof and walls, and rock bolted crane girders

With the great emphasis on environmental issues seen in recent years, an above ground power-house in a wilderness setting might be considered an eyesore today. The underground location has solved that problem once and for all.

Obstacles, related to rock stresses and support measures in large underground openings, had to be overcome as powerhouses were placed steadily deeper into mountains. Engineering geologists met this challenge and developed the new technology in cavern orientation, bolting, shotcrete, etc. which is today a precondition for the constructability and economy of caverns.

3. DESIGN OF CAVERN POWERHOUSES. DEVELOPMENT AND TRENDS

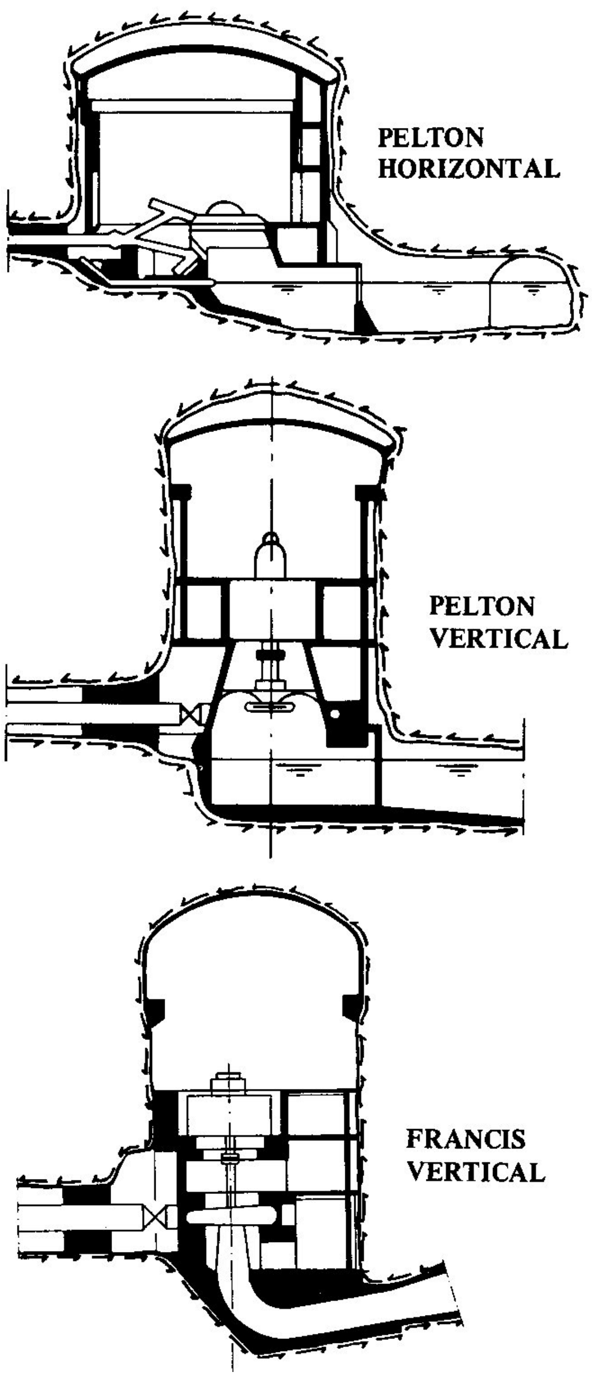


Fig. 12.2 Development of cavern shape

Shape of caverns
Nearly all hydropower units installed since the 1950s have vertical shafts, white also horizontal shafts were commonplace before that. This change in turbine setting has served to reduce the width and increase the height of caverns, as illustrated in Fig. 12.2. The reduced width often means a substantial saving in the cost of support measures for the ceiling, depending on the quality of the rock mass.

Transformer location
Several options are available if the transformers are to be located underground, in or in close connection to the powerhouse:

1. in a gallery along the access tunnel approach to the powerhouse, requiring a substantially wider access tunnel over the length of the gallery.
2. in the powerhouse on the machine hall floor level, opposite the power units, requiring increased width of the cavern.
3. in the powerhouse below machine hall floor level, between the units, requiring increased spacing of the units.
4. in a separate cavern, preferably down-stream from the powerhouse and combined with the draft tube gate chamber as shown in Fig. 12.3.

From a safety point of view this last arrangement is preferred.

Compact design features
Since the cost of the civil works in an under-ground powerhouse is more or less directly proportional with the excavated volume, great emphasis has been placed on finding solutions that serve to reduce the space requirements. Among these solutions are the following:

- For Francis-type turbines with closure valves; to position the pressure conduits' centrelines non-perpendicularly to the powerhouse axis, as indicated in Fig. 12.4, in order to improve accessibility to the valves and reduce the cavern width.

- To reduce to a bare minimum the workshop- and repair facilities in the caverns. Such space is now largely wasted, since modem power units, although extremely reliable, are also very complex and are better shipped back to the manufacturer for any revision work to be done.

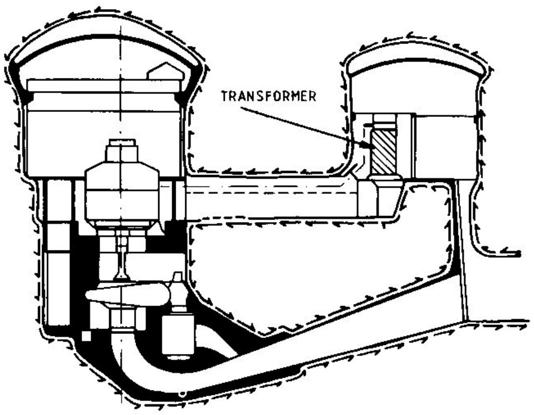


Fig. 12.3 Transformers in cavern with draft tube gate hoists

- To keep an eye on the possibility of a permanent use of any of the auxiliary adits that are frequently excavated to gain temporary access to various points of attack. An adit from the access tunnel down to the tailrace may for instance be converted later into a tail water surge chamber. Likewise may an adit from the access tunnel up to the top heading for the power-house cavern be used later as a permanent reservoir for cooling water, that is either pumped up from the tail water (Pelton units) or conducted as leakage water from the labyrinth seals of the upper turbine covers (Francis units). Fig. 12.4 demonstrate these situations.

As a basis for comparison, the excavated volume of rock per MW installed power has been calculated for a number of hydropower caverns, indicating that this ratio (m^3/MW) varies between 70 and 100 for Pelton installations and between 80 and 200 for Francis installations. Caverns with large-size installations have generally the lowest ratios, just as could be expected. For large Pelton installations, a volume/effect ratio of 50 m^3/MW should be attainable in the future.

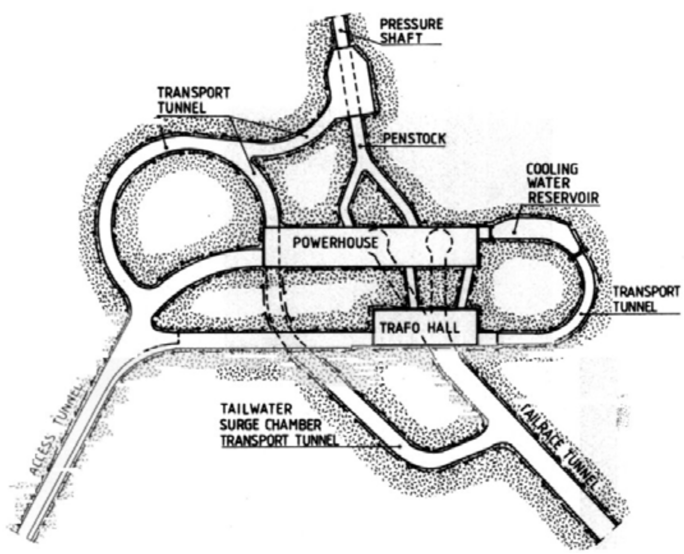


Fig. 12.4
Example of layout for an underground power station

Other cost-saving design features

The following concepts have served to cut costs either directly through reduced construction time or by saving downtime during maintenance operations:

- The roof arch support system shown in Fig. 12.1, with systematic bolting of the ceiling immediately after excavation of the top heading, followed by placing of fibre- rein force d shotcrete, is less ex pensive than the in situ concreted arches of the past, and reduces the construction time by 2 to 3 months.

- The rock bolted crane girders shown in Fig. 12.1 ensures early erection of the permanent overhead crane, which will then be available also for the initial concrete works, the erection of the spiral casings etc., serving to reduce the construction time.
- A design feature particularly related to high-head Francis-type turbines is the dismantling method which permits the lower cover, runner and guide vanes to be removed without dismantling the upper cover or the generator rotor. The work can then be done very quickly, and the worn parts maintained or replaced as required with the shortest possible downtime.

4. IMPLEMENTATION OF CONCRETE WORKS

It is always preferable to have the major part of the concrete work in the powerhouse cavern completed before the start of erection of the permanent equipment. The dust-filled environment from such work will often create adverse conditions for assembly of electromechanical components and may impair the quality of the product.

The concrete embedment for the spiral cases and the concrete support for the generators will necessarily have to be placed after the spiral case installation. Even so, it is normally no problem to plan the main concrete structures in such a way that they provide the needed support for the floors, including the assembly area on the powerhouse floor, before the spiral case is erected and embedded.

Part of an access tunnel for an underground power station (Berdal)

Drilling of blast holes for benching of an underground power station (Berdal)

HYDROCARBON STORAGE ON UNLINED ROCK CAVERNS

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1. INTRODUCTION

In Norway, six large-sized unlined cavern storage plants for crude and refined oil products are presently in operation, and two more are under construction.
The caverns, constructed mostly in good quality hard rocks have spans up to 22 m and heights up to 40 m. Bolting and shotcrete are the main supporting methods used to increase rock mass stability. For installations larger than roughly 50 000 m^3, Norwegian experience is that unlined rock caverns would be the most cost advantageous alternative. Other advantages are better environ- mental protection, lower fire, explosion and sabotage hazards, and lesser land requirements.

2. STORAGE PRINCIPLE AND PREREQUISITES FOR USING THE TECHNIQUE

In essence, the technique of storing hydrocarbon compounds on unlined rock caverns applies the pore water pressure of the surrounding rock to contain the oil product inside the cavern. The cavern design must ensure such rock conditions that its cracks, joints and fissures always convey water leaking into the cavern rather than product leaking out of it.
For successful application, four prerequisites must be met :

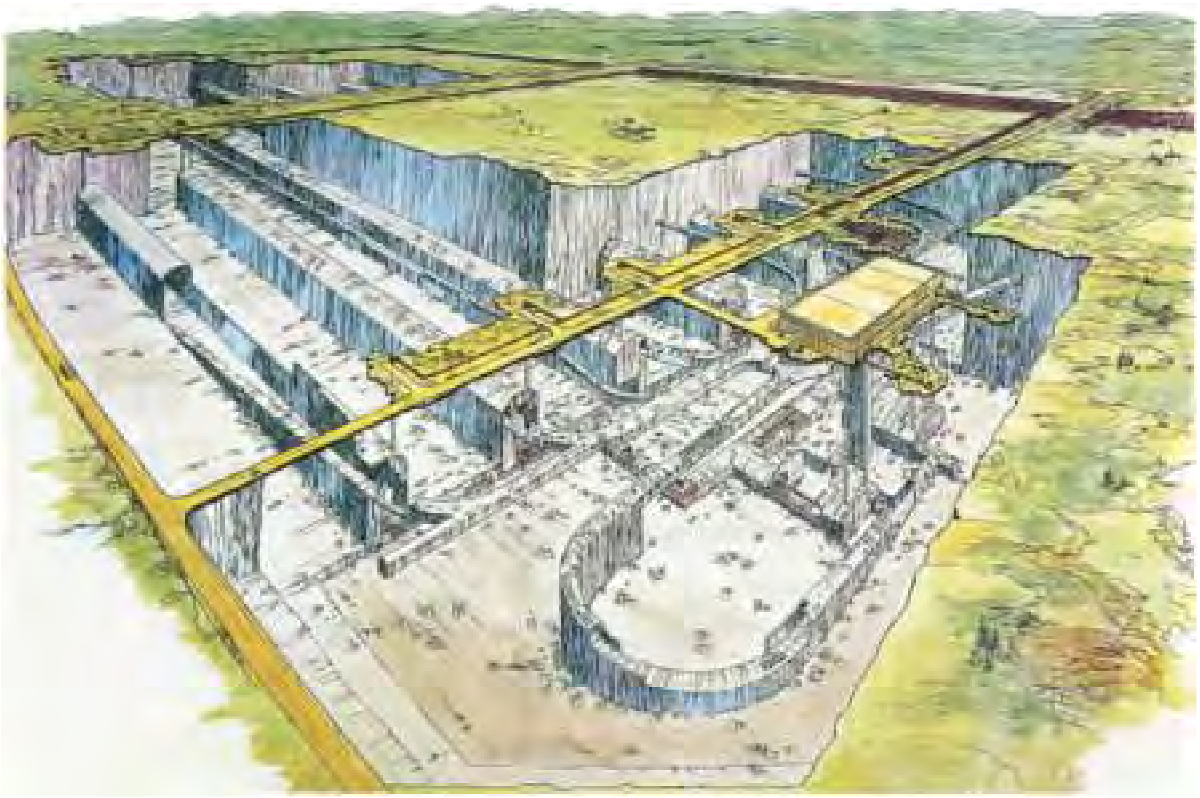
1. Competent and stable rock conditions which are suitable for construction of large openings.
2. The stored product must be higher than water
3. The stored product must be insoluble in water
4. The rock surface in contact with the stored product must hold a pore water pressure higher than the static pressure exerted by the stored product.

Table 13.1 Some Norwegian unlined rock caverns for hydrocarbon storage

Type of Product Stored	Name of Installation	Year Comm.	Dimentions Height x Width (m)	Temp. (ºC)	Abs. Pressure (bar)	Rock Type
Crude oil	Mongstad (Bergen)	1975	22 x 30	ca. 7	1	Meta-anorthorsite and gneiss
Diesel and gasoline	Ekeberg I (Oslo)	1969	12 x10		0,1	Granittic gneiss with diabase dykes
Diesel and gasoline	Ekeberg II (Oslo)	1978	15 x 10	60	1	Granittic gneiss with diabase dykes
Diesel and gasoline	Ilsvika (Trondheim)	1976	12 x 15		0,1	Quartzdiorite
Diesel	Namsen	1983	17 x 30		1	Gneiss
Diesel	Olavsvern	1984	8 x 8		1	Gneiss
Propane	Rafnes (Porsgrunn)	1977	19 x 22	9	6,5	Granite

3. GENERAL ARRANGEMENTS

Conventional cavern storage plants are completely closed and sealed by an overburden of water-saturated rock. It is considered a safe method, provided that adequate safety measures are implemented. The main safety provisions are pre- cautions to prevent gas and/or product leakage, i.e. to prevent fires and explosions and to protect the environment in general.



Layout example of a crude oil storage cavern (Berdal)

Although all unlined caverns for storage of hydrocarbons work on the same basic principle, their configuration often vary, depending on the product stored.
Caverns for crude oil storage usually have an overpressure varying between 0.5 -2.5 bar abs, and utilize fixed water beds. Special heating devices are often installed to provide possibilities for melting and removing wax and paraffin accumulations.
Caverns for storing of distillates have different configuration depending on product stored, e.g. auto and marine diesel, kerosene, and motor gasoline etc. Their utilization range from commercial storage installations with an extensive circulation of the stock to emergency storage depots with turn-around-times of several years.
Depending on the product type, the caverns might have variable water beds when gasoline or other volatile products are stored, or more commonly, fixed water beds for heavy fuel oil and diesel fuel etc.
Storage of (liquefied) gases. At present, only one gas storage plant is in operation, a 100 000 m^3 volume cavern storing propane at 1.9 bars abs. pressure. The plant has been in operation since 1977 with no operational problems reported.
A 60000 m^3 storage plant for butane is presently under construction.
Ballast water caverns are used at export terminals receiving large amounts of ballast water. These caverns have rectangular shape and are used both for equalization of large oil contaminated ballast water streams prior to treatment and for gravitational separation of oil and water.
Pumping arrangements. Both dry mounted and submerged pumps are used. Submerged pumps, which are suspended from the discharge pipes located in vertical shafts leading into to the caverns, have had a rapid development and are now extensively used.
Ground Water Control Systems. Nearly all unlined Norwegian hydrocarbon storage caverns have installed artificial ground water control systems. Such systems are usually implemented before

excavation starts, because it may be difficult and time consuming to establish a drawn- down ground water level.

4. FIELD INVESTIGATIONS AND DESIGN

In Norway, geological mapping and sampling is fairly easy. Due to glacial activities, outcrops are often exposed, and weaknesses, faults and gouges can be easily detected.
For a cavern project, an usual site investigation follows the steps listed below:

1. Geomapping
2. Refraction seismic survey
3. Core-drilling with water loss measurements, and water table observations
4. Rock mass quality assessment
5. Laboratory tests
6. Inspection and control of ground water level during excavation.

The actual design consists of four distinct steps:

1. *Selection of Cavern Location.* This is the most important single step in the design chain, and all efforts should be made to make the right selection with regard to rock mass quality and operational requirements.
2. *Orientating the Length Axis of the Cavern.* This is to be done such as to give minimum stability problems and overbreak.
3. *Shaping the cavern.* Properly shaped openings will favourably distribute stresses along the periphery.
4. *Dimensioning the Cavern.* In Norway, this is usually based on empherical techniques. Also, economical and constructional considerations have to be taken into account here.

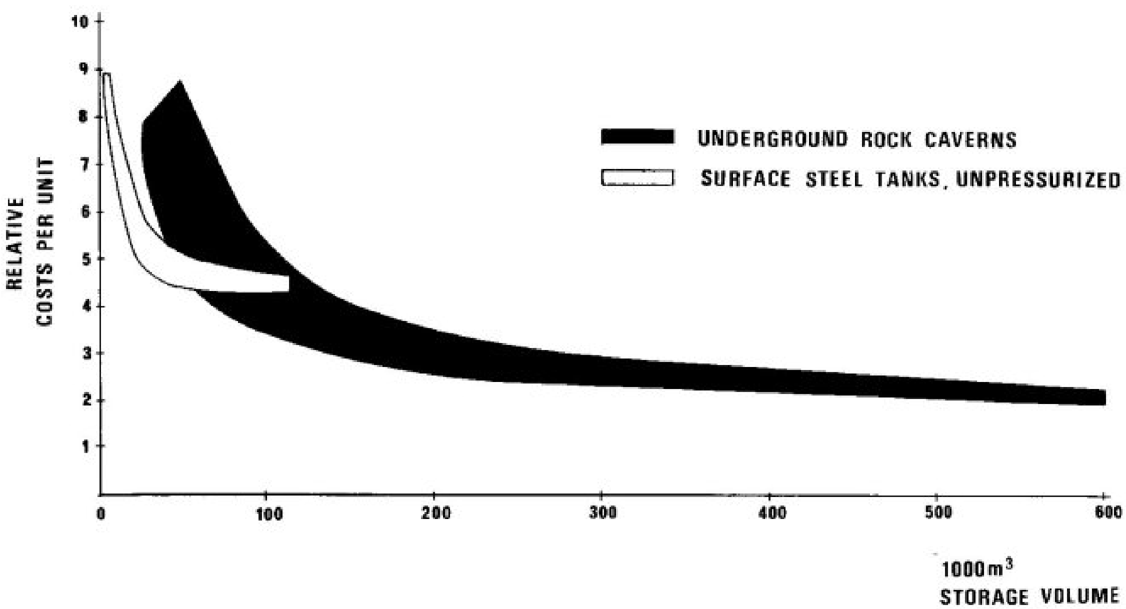


Fig. 13.1 Relative rock cavern/steel tank costs

5. COST ASPECTS

Numerous cost compilations and comparisons of rock storage caverns and alternative solutions, i.e. steel tanks, have been made.

Unlined rock caverns in Norway are considered economically favourable for storage volumes 50 000 m^3 (300 000 bbl.) and larger. Relative cost curves for caverns and steel tanks are given in Figure 13.1.

Over the past two to three decades, innovations in rock excavating techniques have increased the economic advantages of rock-caverns when compared to steel tanks. This trend is expected to continue.

Cavern cost is also a function of span width. For a Norwegian cavern installation, the relationship shown in Figure 13.2 was found. The discontinuity shown on the figure is due to limits for use of heavy excavation equipment.

For a given span width, the cost variation width depth can be determined and the solution optimised with respect to cost. In Figure 13.3, each bar is the sum of:

A. Excavation cost, incl. mucking and dumping.

B. Rock support cost.

C. Pumping cost.

The graph shows that the top heading is expensive, while average unit excavation cost is reduced once one or two benches are added. Going further down, the gain in reduced excavation cost is lost by rising rock support cost caused by increased wall height in addition to rising pumping costs, and rising costs of shafts and construction tunnels.

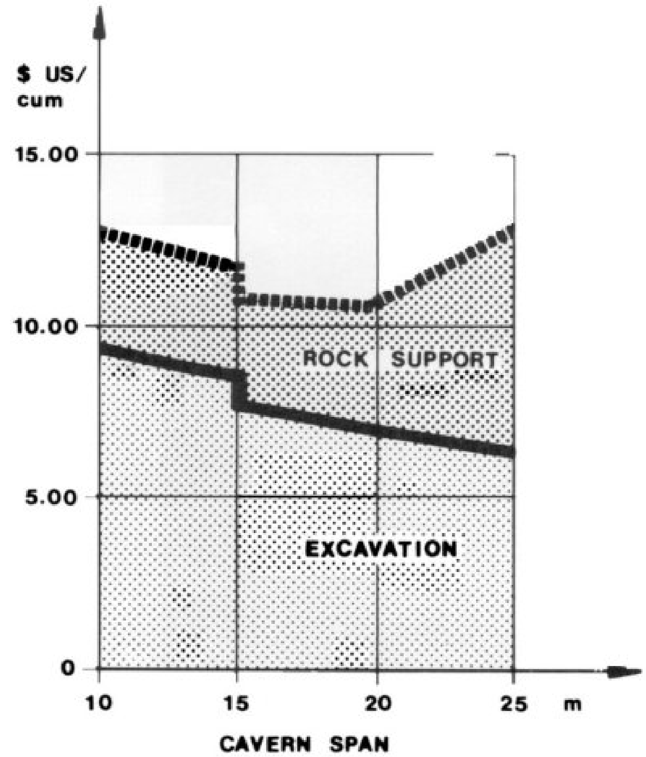


Fig. 13.2 Cavern cost as function of span width

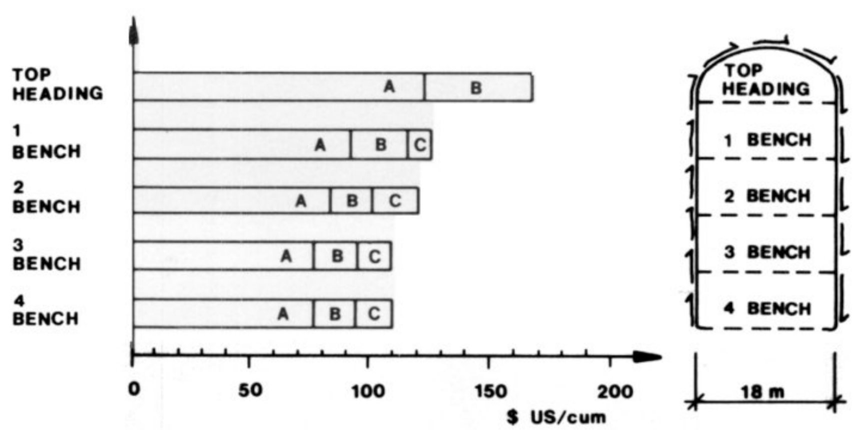


Fig. 13.3 Cavern depth/cost relationship

An underground oil storage cavern. These caverns with cross section 530 m^2 was excavated in four stages: Top heading and three benches (Berdal)

Submerged pumps in an underground oil storage cavern (Berdal)

Gallery and service tunnel for underground oil storage caverns (Berdal)

Before and during blasting of the cofferdam, at Solbergfoss hydropower plant.
The cofferdam is mostly formed by rock masses, with a distance from the intake structure of 5-10 meters (Oslo Lysverker)

LAKE TAPS

Bjørn Buen
Ing. A. B. Berdal A/S

1. INTRODUCTION

Lake tap, a submerged tunnel piercing, is the process of connecting a tunnel system to a reservoir or a recipient. The major use is draw down reservoirs for hydropower development.

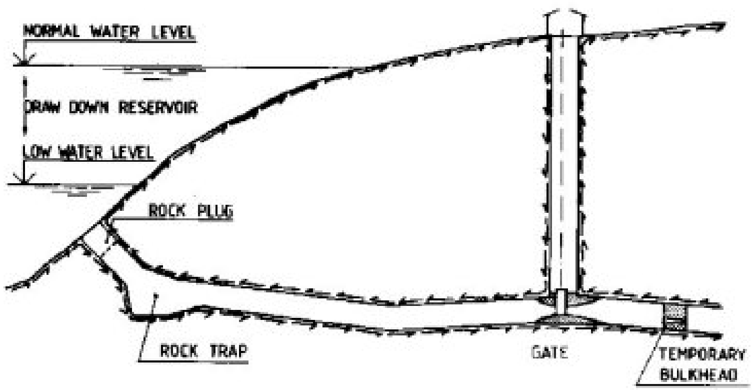


Fig. 14.1
Typical layout for a lake tap

The method is attractive basically because the cost is low. This low cost method has been one important factor in the development of Norwegian hydropower. The estimated number of taps performed in Norway, between 500 and 600, over a period of approximately 90 years under- lines this. A selection of lake taps performed is presented in Table 14.1. Recently the method has been applied at Hjartøy for a crude oil terminal on the Norwegian west coast where oil from the offshore Oseberg field will be piped through a 2.3 km long sub sea tunnel opened to the North Sea at a depth of 80 m bsl.

Table 14.1. Selection of submerged piercings

Location	Year	Depth m	Cross Section m^2	Rock Type
Demmevatn	1895	20	---	gneiss
Storglomvatn	1920	---	16	schist
Krokvatn	1922	---	4	gneiss
Jukla West	1973	105	10	gneiss
Jukladalsvatn	1977	93	60	gneiss
Ringedalsvatn	1980	86	40	granite
Tyee Lake, Alaska	1983	50	8	granite
Storevatn	1986	116	40	gneiss
Hjartnøy	1987	80	150	gneiss

2. GEOLOGICAL CONDITIONS

The geological conditions influence the selection of a site for a lake tap. Both rock mass quality and soil. deposits at the location of the piercing must be considered. Rock mass quality includes both the rock itself and the nature and frequency of its discontinuities. As a rule the rock itself will have a strength and a permeability which are more than sufficient for stability and leakage control. Rock types are therefore of interest mainly because of the effect their mechanical and physical properties

will have on the nature of the discontinuities. It is a fact that stiff rock types will have joints which at least near the surface will be more open than is the case with more deformable rock. In practice this means that for example granite and quartzite may be more problematic leakage wise than phyllite.
Faults and heavily fractured areas should be avoided in lake taps if possible. The combined problems of stability control and water control in low quality rock may be solved construction-wise, but may be prohibitive cost wise.
The existence of sediments in a tap site is not a problem per se. A thin layer of silt or moraine may reduce water inflow on open joints and simplify the drilling of exploratory holes through to the reservoir. There is, however, an upper limit to the acceptable overburden thick- ness. An overburden of the same order as the diameter of the opening is normally considered as the maximum acceptable for a successful piercing.
Erosion around the tunnel opening will because of the low velocities, normally not be significant. Care should, however, be taken not to undercut large and potentially unstable sedimentary deposits where possible slides may fill the opening. Likewise should sites close to active talus fans be avoided.

3. INVESTIGATION
Investigations for lake taps are based on available reservoir maps, reflection- and refraction seismic methods, soundings, drillings and inspection by divers or by a remotely operated vessel (ROV) and geological mapping of surrounding area. The combination of methods used depends on site conditions and depth of piercing. Apart from a geological evaluation refraction seismic profiles and sounding are used on almost all piercings.

4. LAKE TAP METHODS
There are two main types of methods; the closed and the open type. The two methods are shown in principle in Fig. 14.2.

In the closed type piercing the tunnel system is closed to the atmosphere. This is also called the "dry method" because a large air volume is necessary in order to control the pressure rise in the system w hen water jets into the opening after the final blast. Characteristic for the closed system piercing is a large and high velocity inflow of water. This may require large and complicated rock traps to catch the debris from the final plug.

In an open system piercing the tunnel filled with water save a small compressed air cushion directly beneath the plug. Therefore this is also called the "wet method". Typical for an open system piercing is a small low velocity water inflow and simple trapping of spoil in a rock trap directly beneath the plug.
Both methods have been used frequently and successfully.

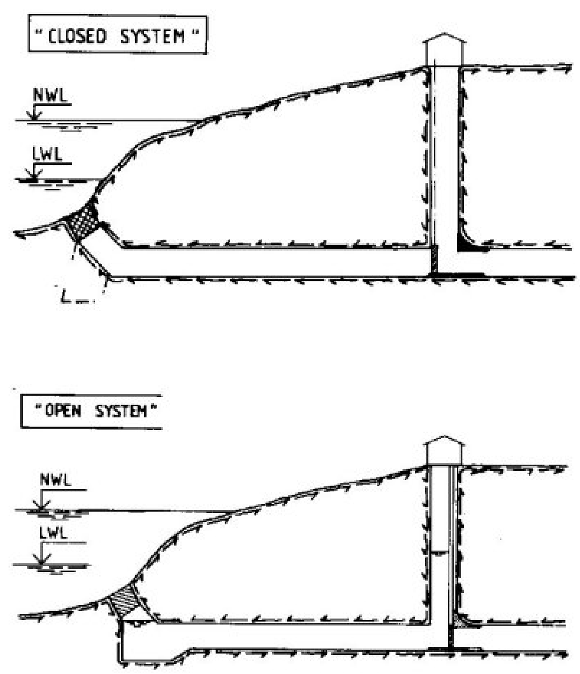


Fig. 14.2
Closed system and open system piercing

In general the closed system method is best suited where the distance between the closing structure and the opening is long, while the open system method is best suited for short distances.

5. CONSTRUCTION

The excavation for lake taps is done by drill and blast. Because of the need for exploratory drilling and grouting and because last minute changes in alignment and geometry may be required, only the drill and blast method gives the necessary flexibility.

When the tunnel is approaching the location of the piercing a systematic drilling of exploratory holes is begun. The main objective is to detect water bearing faults and joints ahead of the face. Major leakages are grouted ahead of the face, minor leaks not seriously affecting working conditions being left untreated.

W hen close to the surface a number of holes that depends on the cross-section is drilled through to the surface to locate it exactly. On this basis the final trimming, design and drilling out of the final round is done.

6. DESIGN OF FINAL BLAST

A typical plug is 3-6 m thick depending on cross section. For the final blast special water-resistant explosives and detonators are used.

The drilling pattern is designed on the basis of the tunnel cross-section, the thickness of the plug, the rock type, the thickness of the over- burden, the water pressure and the water leakage. Parallel-hole cuts are used and the minimum number of cuts is two. The holes are drilled 0.3-0.5 m short of breakthrough. Depending on the factors mentioned above, the specific charge will typically vary between 3 and 8 kg/m^3 of rock mass. Double and independent ignition systems are used.

7. EXPERIENCE

Underwater piercing is a proven and low cost operation, but a successful piercing is dependent on a well-selected site and carefully planned and executed construction and final blast. The method is suitable for hard rock with limited overburden or where overburden may be removed, and the opening above the rock surface protected for example by caissons. A close follow-up of the works is an important measure for a safe excavation .

SUB SEA TUNNELS

Arild Palmstrøm
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1. INTRODUCTION

Special for sub sea tunnels is that they pass under bodies of water that are inexhaustible and where drainage into the tunnel has no lowering effect whatever on the groundwater line. The down-grade excavation of such tunnels therefore include the possibility of having large inflows of water to cause great problems for the excavation works and even to drown the tunnel.
The special problems in sub sea tunnelling are therefore:

- Short distance to the inexhaustible body of water
- Limited knowledge of the rock mass conditions
- All leakage water has to be pumped out both during construction and during the operation condition.

A subsea tunnel project therefore require thorough planning of the works and include special safety measures.

2. EARLIER SUBAQUEOUS TUNNEL EXPERIENCE

Looking back at previously built "over-land" tunnels, there are many projects that today could be classified as sub sea tunnels, for example the tunnelling of water conduits for hydropower developments located below rivers or lakes, Fig. 15.1. The most useful early experience from such sub sea tunnelling has been the construction of submerged tunnel piercings, or lake taps which is a Norwegian speciality in hydropower works. The piercing is effected by excavating a tunnel in rock under the lake bottom, up to a pre-selected point, from where a controlled hole-through is made by a final round of blasting. In this way a reservoir is made accessible for hydropower exploitation using the storage volume available below the original water level
Lake taps are further described in <14>.

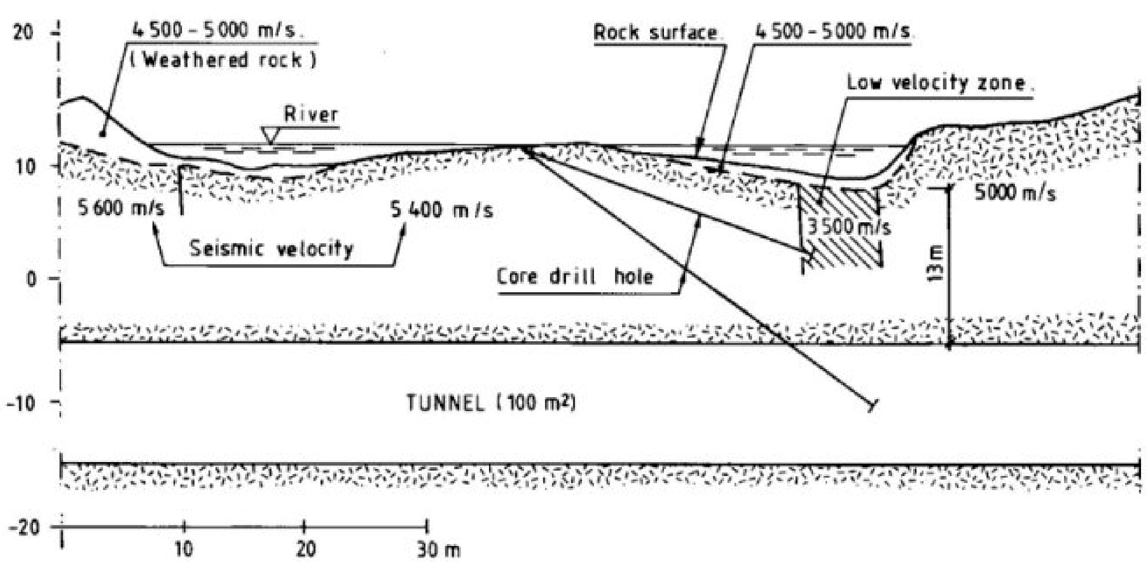

Fig. 15.1
Rygene power plant (1974- 76): The 2 km long, 100 m^2 tailrace tunnel crosses under the 100-200 m wide Nidelven river in two locations with a minimum rock cover of 13 m

3. SOME FACTS FROM RECENT SUB SEA TUNNELS

The most serious uncertainty associated with sub sea tunnelling, is related to sudden water infiltrations. Neither the number of water-bearing zones, nor the maximum inflow rates can be predicted beforehand. Norwegian experience shows that the risk of large water inflows is greatly reduced when sealing of water-bearing zones is performed as pregrouting ahead of the tunnel face. This pregrouting is included in the exploratory drilling programme. When a water-bearing zone has been detected by the exploratory drilling, additional holes are drilled and grouting is done through all the boreholes showing water leakage. After the grouting is completed, the tunnel can be further excavated through the sealed zone. The grouting process is also described in Chapter 17.
One major uncertainty for the tunnel cost are the supporting and grouting works, governed by the actual rock mass conditions. This is demonstrated in Fig. 15.3 where the recorded variations in construction costs with different rock mass qualities are shown. For sub sea projects the amount of the rock supporting works can never be known until the tunnel breakthrough. It is therefore of the greatest importance to work out tender documents with risk sharing provisions as described in<9>.

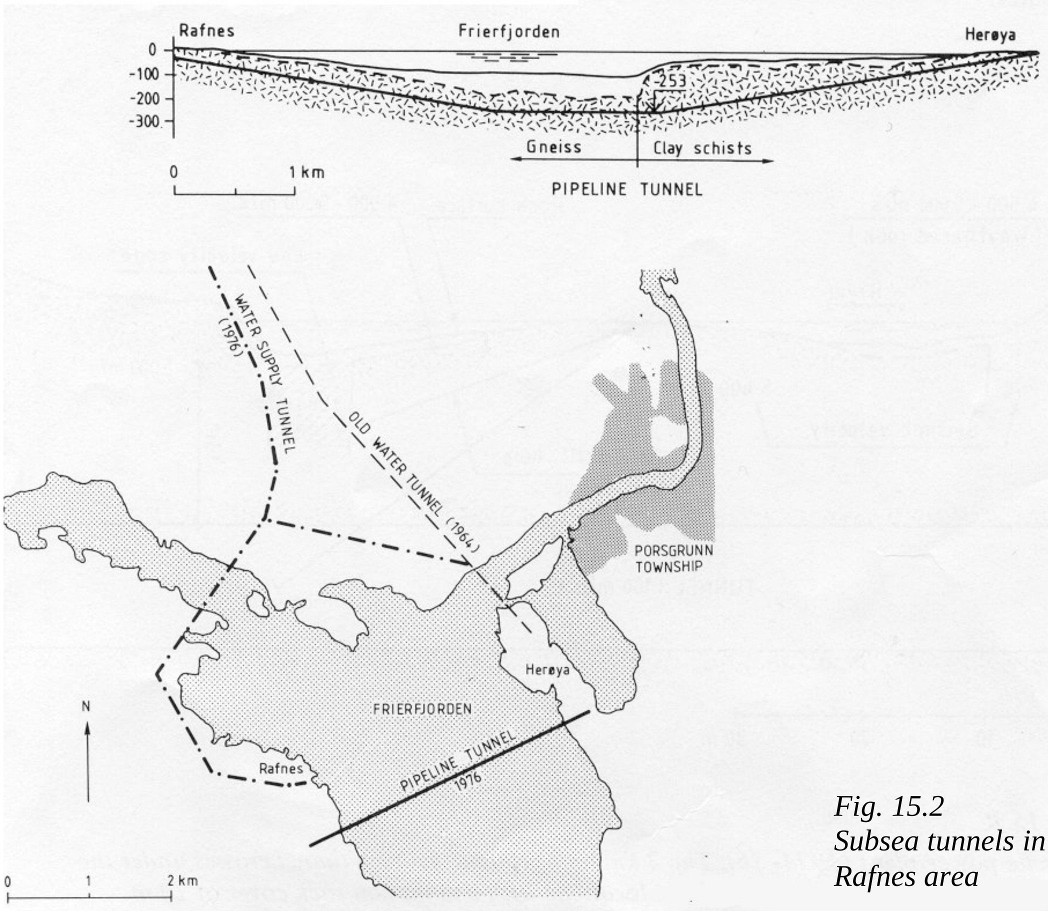

Fig. 15.2
Subsea tunnels in the Rafnes area

4. EXPERIENCE

The experience from nine Norwegian sub sea rock tunnels is that the overall rock mass quality has been fair to good for tunnelling. In weakness zones varying from 5 to 400 m in width, the quality has been poor to very poor. Special rapid rock supporting concreting methods were used successfully in some instances, making a safe advance possible even where the stand-up time of the rock masses was very short.

Experience from the sealing of water leakages shows that normally about 8-10% of the tunnel length has had to be pregrouted. The average water leakages into the permanent tunnels vary between 75-400 l/min per km tunnel.
There is at present experience from sub sea tunnels down to 250 m below sea level, and firm plans for 7-8 long tunnels down to more than 300 m. With the experience from several long "over-land" tunnels with ground water heads up to 1000 m it is expected that within 10 years sub sea tunnels may be constructed down to as much as 500-600 meters below sea level, with reasonable costs and construction time.

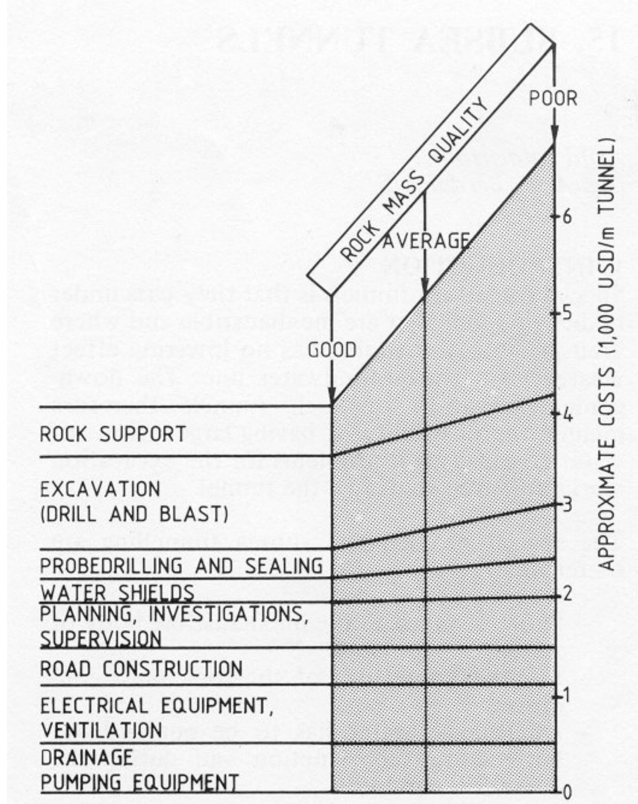

Fig. 15.3 Breakdown of costs for a two-lane sub sea road tunnel

Table 15.1 Data from eight Norwegian sub sea tunnels

Project	**Year**	**Purpose**	**Lenght (km)**	**Cross section (m^2)**	**Deepest point (m)**	**Pre investi-gation (*)**	**Weekly progress-rates *average* (m)**	***max* (m)**	**Working hours week**	**Rock type**
1. Rafnes-Herøya	1974	gas pipe-line	3,6	16	-253	1,7	37	97	130	gneiss
2. Vardø	1982	traffic	2,6	50	-88	5,0	17	60	75	schist
3. Kamsundet	1984	gas pipe-line	4,7	26	-180	1,9	33	92	105	greenstone, greenschist, gneiss
4. Førdesfjord	1984	gas pipe-line	3,4	26	-160	1,9	26	63	105	gneiss
5. Førlandsfjord	1984	gas pipe-line	3,9	26	-170	1,5	36	85	105	gneiss phyllite
6. Shore approach. Hjartøy	1987	oil pipe-line	2,3	26	-110	4,0	39	92	105	gneiss
7.Ålesund-Ellingsøy	1987	traffic	3,5	70	-140	1,0	35	55	105	gneiss
8. Ellingsøy-Valderøy	1987	traffic	4,3	70	-140	1,0	38	65	105	gneiss

(*): percent of total construction cost.

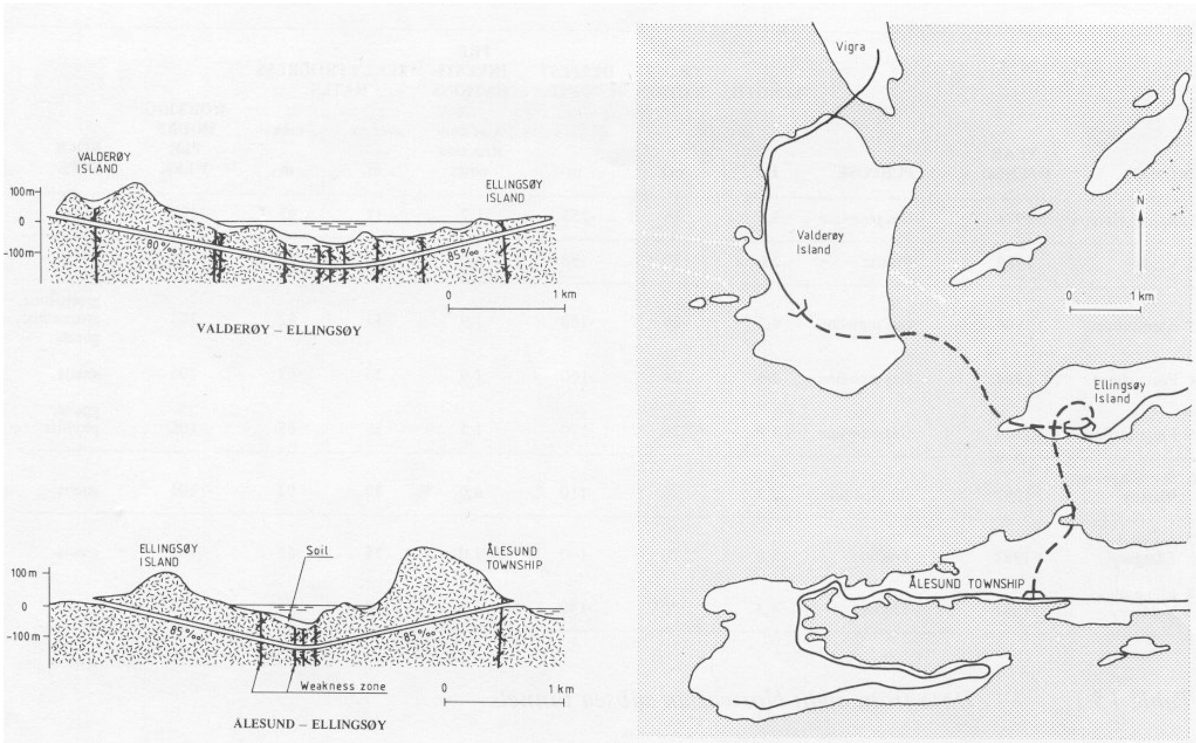

Fig. 15.4
The sub sea road tunnels Ålesund- Valderøy (project no 7-8 in Table 15.1) Tunnel cross section 70 m^2

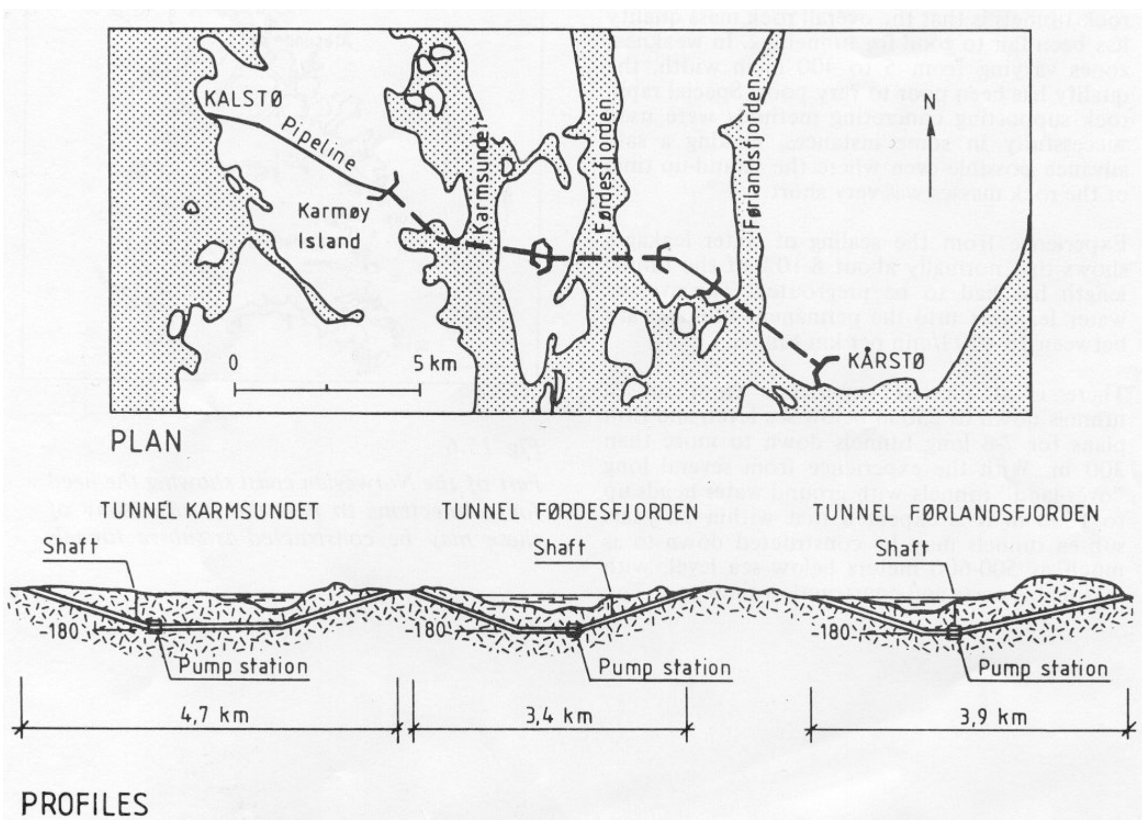

Fig. 15.5
The Kårstø sub sea gas pipeline tunnels (project no 3-4 in Table 15.1) Tunnel cross section 26 m^2

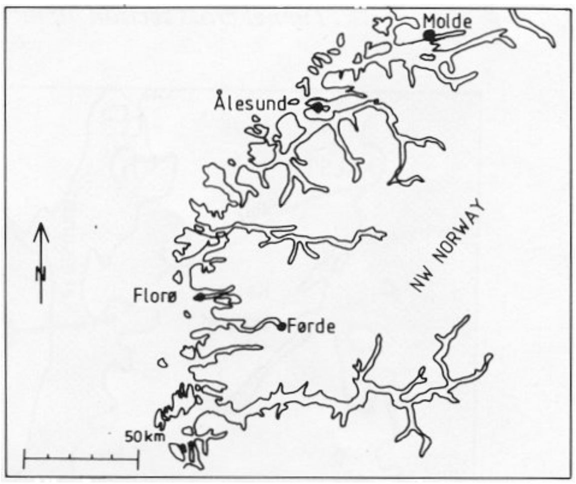

Fig. 15.6
Part of the Norwegian coast showing the need for connections to pass the fjords. Many of these may be constructed as sub sea tunnels

REAMING OF SHAFTS AND TUNNELS

Steinar Roald .
Ing. A. B. Berdal A/S
Trond Oiseth
Entreprenørservice A/S

1. INTRODUCTION

The pressure shaft at Lomen Hydro Power plant is one among numerous successful applications of raise drilling in Norway. The shaft (ref. fig. 16.1) is 410 m long with an inclination of 41° . The diameter of the shaft is 3.5 m and the rock is phyllite with lenses of quartz. Compared to conventional shaft excavation (drill & blast) the reaming or raise drilling method offered considerable advantages for this shaft, such as:

- Improved hydraulic conditions
- Simpler construction
- Reduced rock support work
- Lower construction cost
- Reduced construction time

Due to one or more of these advantages being obtained, the raise drill method has been selected for a number of shafts in Norway.

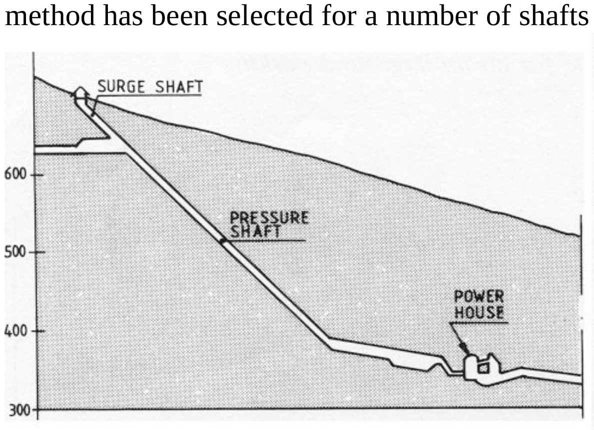

Fig. 16.1
The 410 m long pressure shaft at Lomen hydropower plant was excavate by reaming.

2. RAISE BORING IN NORWAY

Since 1970, when the first raise boring machine was introduced in Norwegian underground construction, considerable experience has been gained with this technique. Altogether there are now more than 20 km of raise bored shafts in Norway.

Raise boring is achieved by drilling a pilot hole with a diameter at approximately 11 inches. When the pilot hole is completed the reamer is mounted and the machine pulls and rotates the reamer back (ref. fig. 16.2).

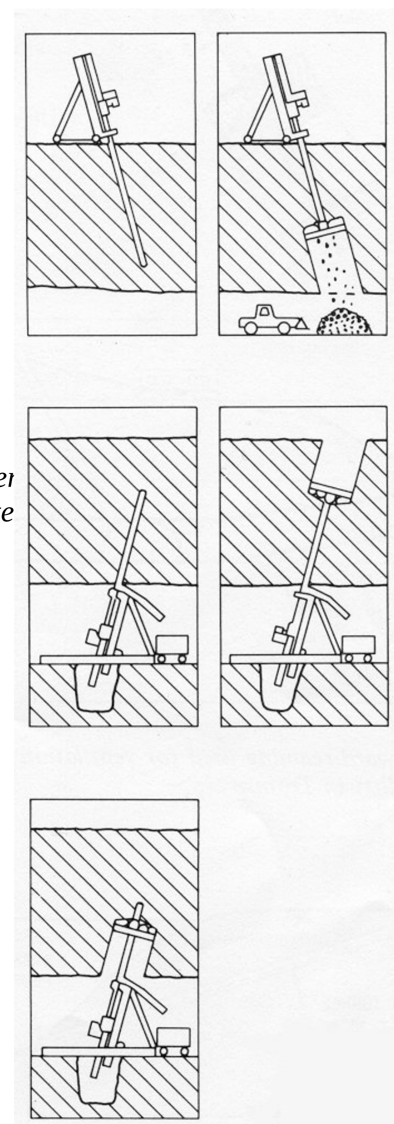

Fig. 16.2
Various applications of raise boring equipment:

Top: Pilot boring downwards with upward reaming
Middle: Pilot boring upwards with downward reaming
Bottom: Full face reaming

In Norway the method has been used in subsurface projects of many types, such as :

- Pressure and surge shafts in hydro power (like the shaft at Lomen, Fig. 16.1)
- Ventilation shafts for underground car parking and highway tunnels (Fig. 16.3)
- Cable shafts
- Various types of shafts in mining
- Shafts in connection with underground oil storage
- Dewatering shafts for sub sea highway tunnels

The raise boring technique can also be used for other applications, such as the shore approach tunnels for offshore pipe lines (The Berdal Offshore method) shown in Fig. 16.4.

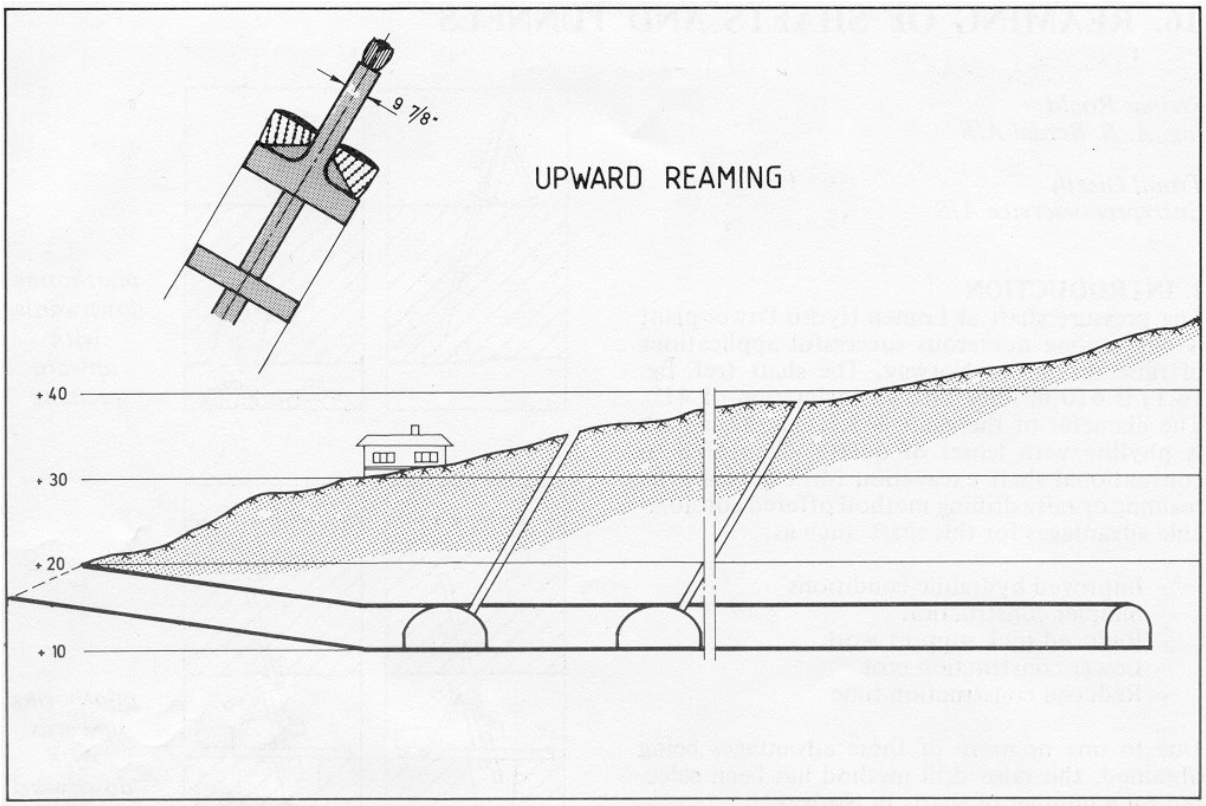

Fig. 16.3
Upward reaming used for ventilation shafts for an underground parking facility in Tromsø

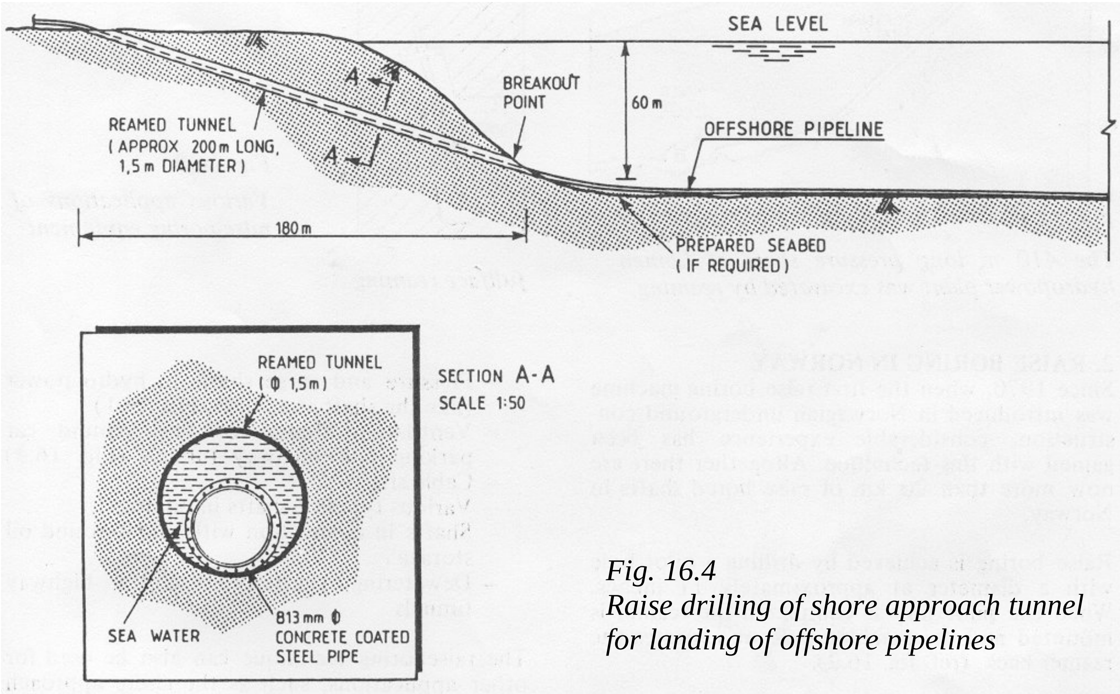

Fig. 16.4
Raise drilling of shore approach tunnel for landing of offshore pipelines

3. EQUIPMENT CAPACITY. EXECUTION

Compared to conventional shaft excavation methods, the construction time can be substantially reduced by application of raise drill equipment. Normal capacity for the pilot hole is between 1.0 m and 3.0 m per hour and the reaming capacity is normally between 0.5 m and 2.0 m per hour in hard rock. This means that a 350 m long shaft can be made in three months instead of seven months for excavation with the drill and blast method, mobilisation and demobilisation included.

Maximum drilling length depends on choice of method for drilling/reaming, shaft diameter, inclination and rock type. Pilot holes of 400 m and reaming length of 350 m with diameter smaller than 2.0 m are normally executed without any problems. Shaft lengths of more than 600 m are feasible under favourable drilling conditions.

The raise drill equipment is operated by two men. In addition to these at least one man is required for mucking and transportation of debris from the shaft.The equipment can be erected ready for operation within one week of arrival at the construction site. If needed, the equipment can be lifted by helicopter which allows operation on hillsides and in mountainous terrain with no road access (see photo 16.1). The operation of the equipment requires supply of water and electricity. Electricity is often supplied by diesel/electric generators.

4. HOLE DEVIATION

Deviation of the pilot hole is most often less than 2% of the drilling length. The deviation as a percentage of the drilling length normally increases with the length of the pilot hole. In close to 80% of the cases deviation has been less than 1.0%.

In cases where only small deviations can be accepted, the drilling accuracy can be improved by increasing the rotation speed and decreasing the feed pressure of the drilling bit.

The length of a shaft may be limited by the expected hole deviation in cases where it is strictly required to hit a pre-selected location, e.g. in an existing tunnel. It is, however, possible to measure the deviation of a pilot hole by special methods, and if then a branch tunnel is to be excavated later, it may be directed so as to intersect the known position of the pilot hole. In such cases the length of the shaft, between two tunnels or between the surface and a tunnel, can be extended to more than 600 m.

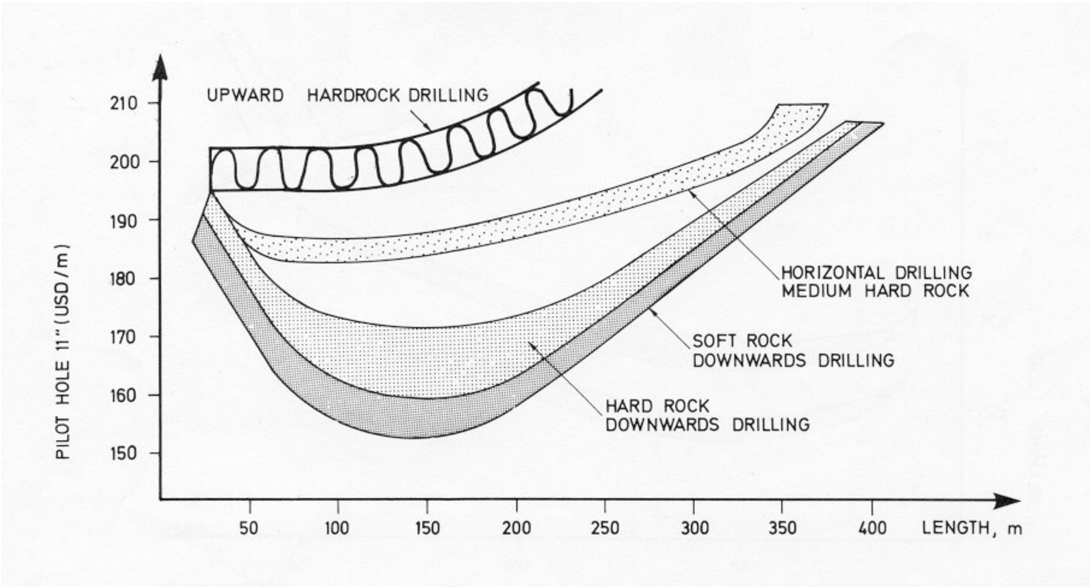

Fig. 16.5
Cost of pilot hole drilling (mobilisation and equipment erection excluded}

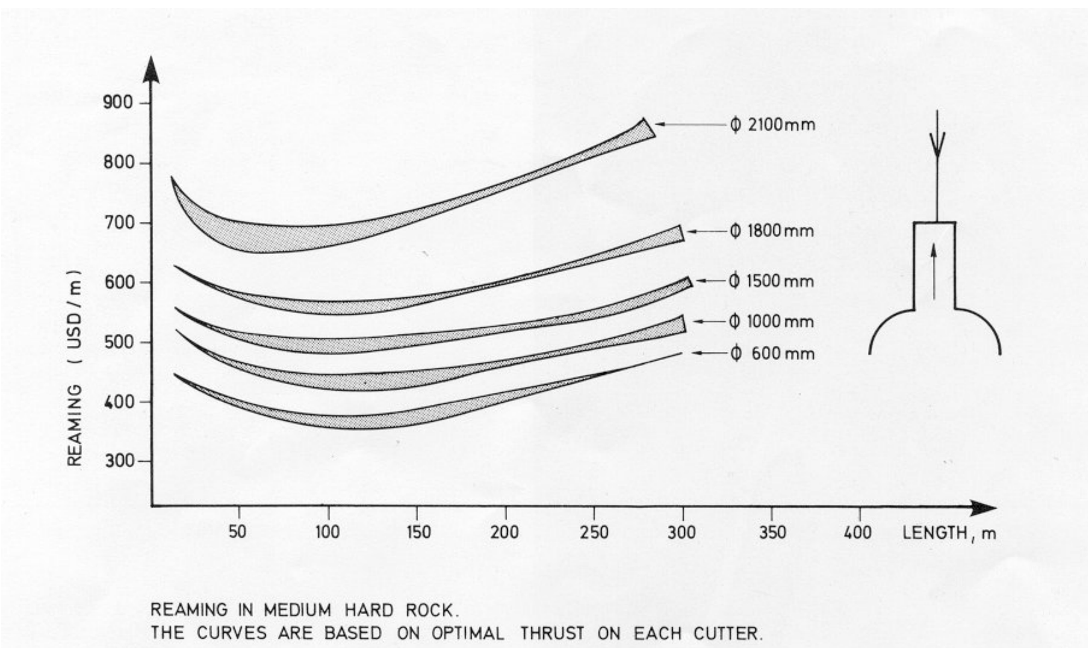

Fig. 16.6
Cost of drilling of vertical pilot hole with reaming (exclusive of mobilisation, equipment erection and muck transportation)

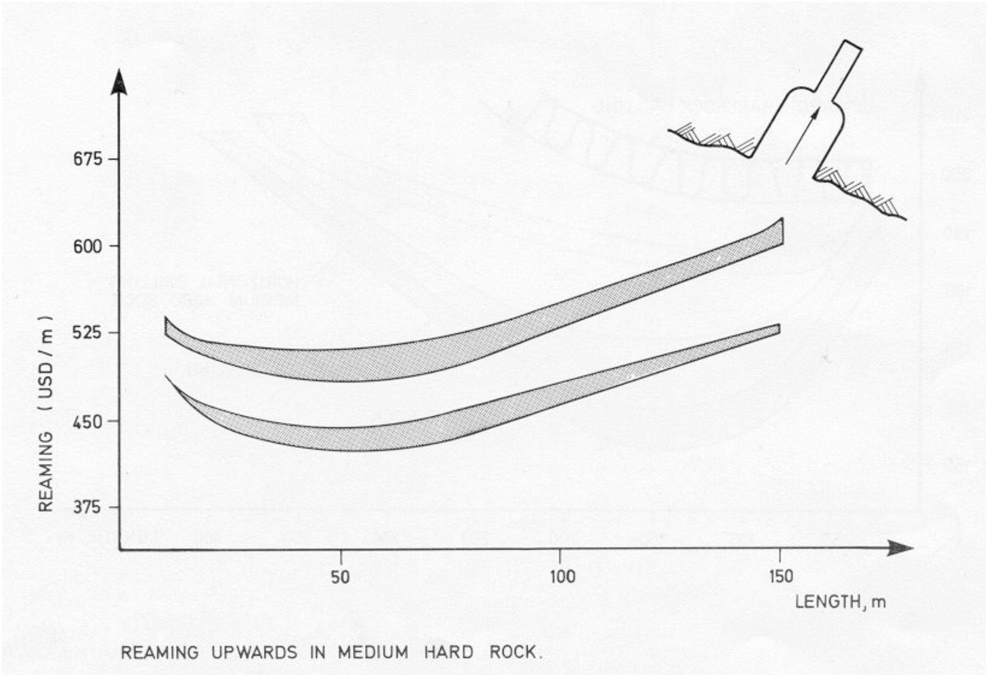

Fig. 16.7
Cost of upward reaming. (Pilot hole and reaming in the same operation) Mobilisation, equipment erection and muck transportation have to be added

5. COSTS
The cost of raise drilled shafts is dependent of:

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

Typical drilling and reaming costs are shown on next page. The cost of the pilot hole (Fig. 16.5) is a function of drilling length, drilling direction and rock conditions. Fig. 16.6 shows the cost of reaming incl. pilot hole in medium hard rock for various diameters, while Fig. 16.7 shows cost for upward pilot boring and reaming in the same operation.

Table 16.1 Some examples of reamed shafts completed in Norway

LOCATION	INCLINATION	DIAMETER (mm)	LENGHT(m)	ROCK TYPE
Fet	1º	250	321	Granite
Ulla-Førde, Kvelven	36º	1400	265	Granite/Phyllite
Ulla-Førde, Pjåkvatn	41º	1400	304	Granite/Phyllite
Holsfjorden	63º	130	187	Porphyrite/Basalt
Kårstø	90º - 85º	250	1300*	Gneiss
Lomen	41º	3500	410	Phyllite

*) several shafts, each approx. 130 m long

Photo 16.1 Transporting drilling equipment by helicopter (Entreprenørservice A/S).

Photo 16.2 Drilling of the pilot hole (Entreprenørservice A/S)

Photo 16.3 A reamed shaft, diameter 1,5 meter (Entreprenørsevice A/S)

ROCK GROUTING

Oddbjørn Aasen
Astrup-Høyer A/S

1. INTRODUCTION
Rock grouting in connection with tunnelling has the main objective of seating against water leakages into or out from the tunnel. The reasons for that effort can be :

1. To reduce the amount of permanent water to be pumped out of the "dry" tunnel in its operation condition
2. To reduce the drainage of the ground water reservoir above, avoiding settlements of buildings founded on clay
3. To reduce water loss out of water filled (pressure) tunnels
4. To avoid undesirable water leakages into a powerhouse, cavern or a tunnel
5. To avoid possible larger water inflows which can "drown " the tunnel during excavation.

The various grouting operations or seating methods to achieve these ends are usually carried out in one of the four ways described in the following.

Photo 17.1
Potential water leakage detected with a probe hole. (Astrup-Høyer A/S)

2. PREGROUTING DURING SUB SEA TUNNEL EXCAVATION
Systematically probing ahead of the tunnel face is an important measure to detect water leakages. Probing is intensified w hen necessary and when larger rock discontinuities are expected.
The probing fan is supplemented with grouting holes w hen water is observed in probe holes. Multi-hole grouting is performed with the use of ordinary and fast-setting cements, coarse filler additives, and in some situations also with polyurethanes.
The grouting pressure used is normally high, 20-60 bars, but is depending on geological conditions, overburden, etc. The length used for grout holes are mostly approx. 15 m. All grouting ahead of the tunnel face is finished with fast- setting cements, so that the drill- and blast sequence can restart immediately.
A length of about 1.5 km out of more than 15 km of sub sea tunnels in Norway has been pregrouted (1987). Sub sea tunnels are further dealt with in <15>.

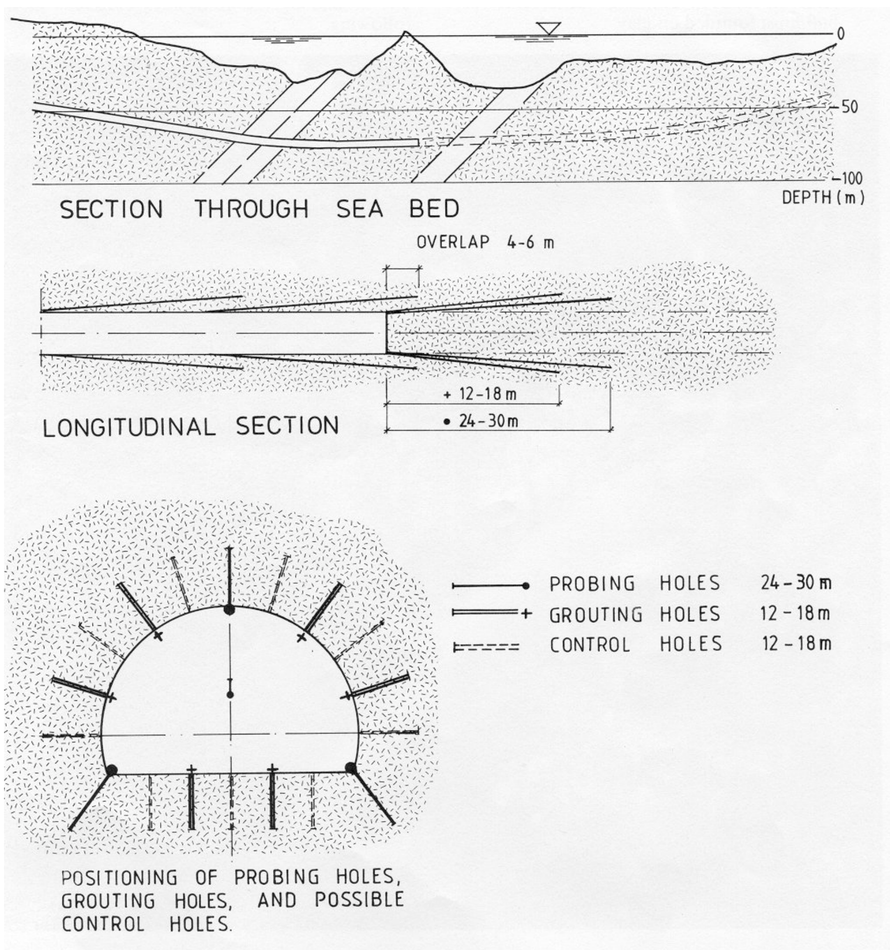

Fig. 17.1 Probing and pregrouting in sub sea tunnels

3. LEAKAGE CUT-OFF FOR LAKE TAP "PLUG "
Systematic probing ahead of the tunnel face is carried out to detect possible water leakages. Ordinary pregrouting is performed to sea! where required.
Seating of the remaining rock face for final blast (plug) can be difficult with ordinary methods of grouting. Therefore fast-setting grouts together with polyurethane (chemicals) are used in

limited amounts, and distributed over many holes. The grouting pressure must be as low as possible, but high enough to squeeze the grouting material into the cracks and fissures.
There is grouting experience for lake taps down to 116 meter below water level. More than 300 lake tap "plugs" have been sealed by grouting. More information about lake taps is given in Chapter 14.

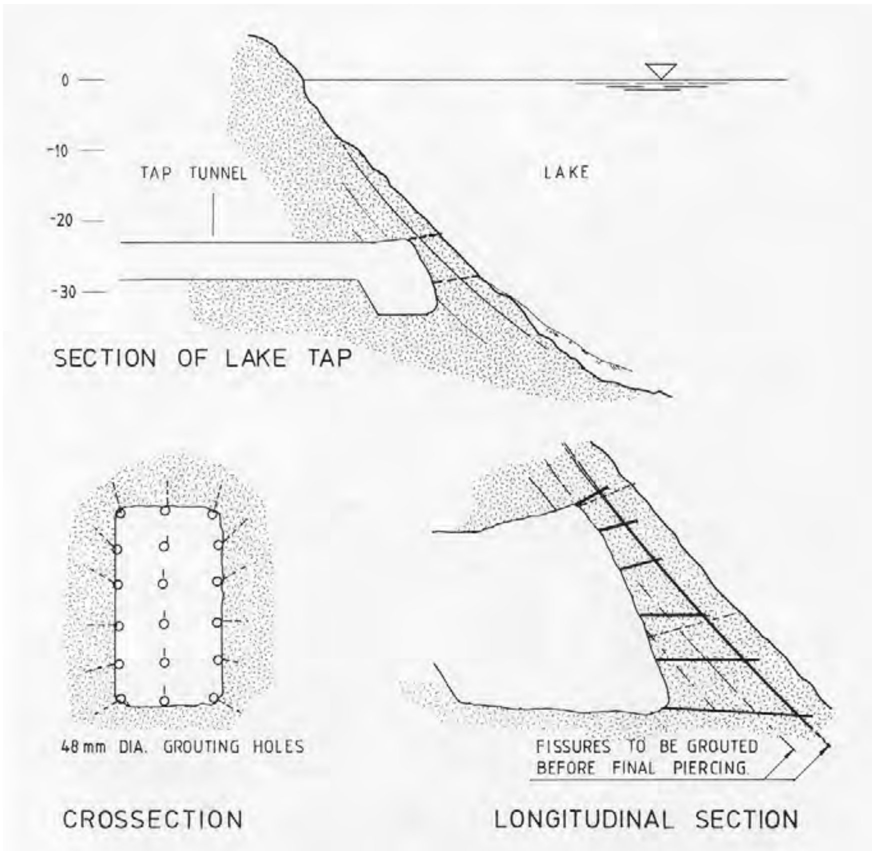

Fig. 17.2 Leakage cut-off before lake tap piercing

4. SEALING OF TUNNEL BELOW OVER-BURDEN SUBJECTED TO SETTLEMENTS

In sensitive ground where even minor fluctuations in the groundwater table might cause settlements on buildings above ground, probing and grouting in front of the tunnel is extremely important to ensure a high degree of impermeability in the rock mass adjacent to the tunnel.
The probing is performed with a double umbrella cover, and with a large number of holes.
Conventional pregrouting with cement is first performed, in a second round sodium silicates or similar materials are used. This to meet the limitation in water leakage of 1-5 l/min./100 m tunnel, measured at the tunnel face. The grouting pressure is generally high -20-50 bars -and hole length normally 15-20 m.
In the underground of Oslo City, more than 20 km of TBM-bored or blasted tunnels are pre-grouted according to this method.

5. GROUTING OF ROCK AND CONCRETE PLUGS EXPOSED TO EXTREMELY HIGH WATER PRESSURE (UP TO 1000 METER WATER HEAD)

A deep rock grouting is performed according to conventional methods, but with very high grouting pressures (90-110 bars). Hole lengths are 15-25 m, and grouting materials are cements, epoxies and polyurethane.

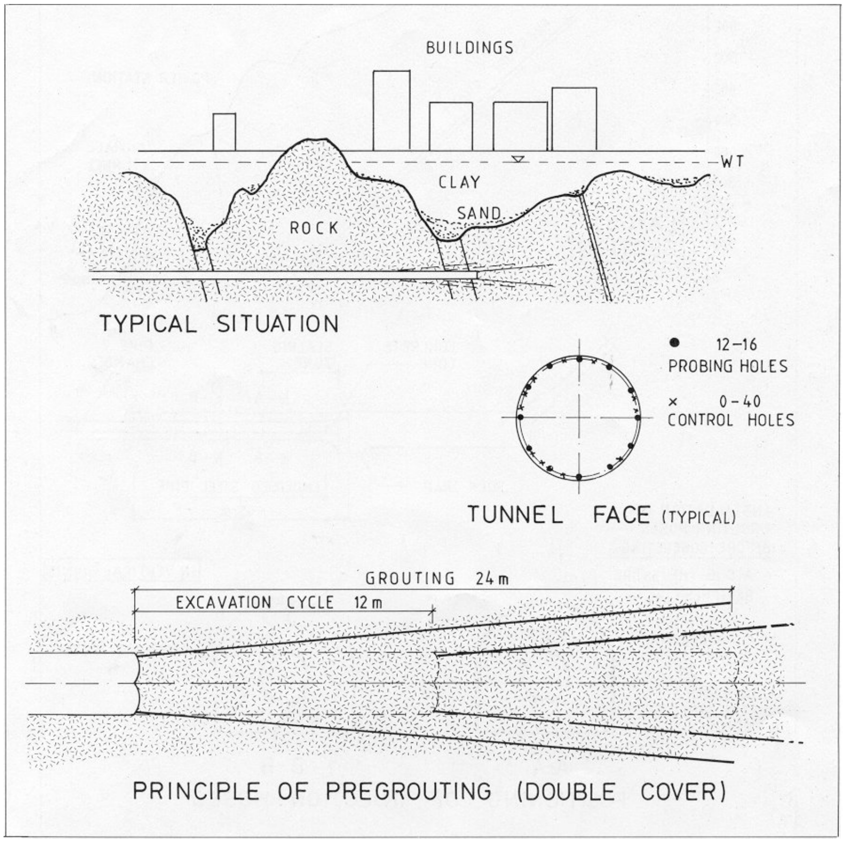

Fig. 17.3 Principles of probing and pregrouting to ensure minimum water leakage into the tunnel

For concrete plugs special grouting technique has recently been developed. Flexible grouting hoses are installed in all potential areas of leakage prior to concreting of the plug.
After concreting, when the concrete has "set" and the hydration temperature subsided, the entire construction, all joints and interfaces rock/concrete/steel are grouted through the hoses. Epoxy and polyurethane resins are normally used, and the grouting pressure is at least 10 bars higher than the final water pressure will be on the construction. In particular cases a secondary grouting operation is performed through newly drilled holes, in order to consolidate and test the construction.
Experience shows that leakage through such a concrete plug can be less than one l/sec., even with water pressures as high as 100 Bar (1000 meter water head).
At least 500 other concrete plugs exposed to high water pressure have earlier been sealed by standard grouting, but with somewhat larger water leakage than the result obtained by the new method.

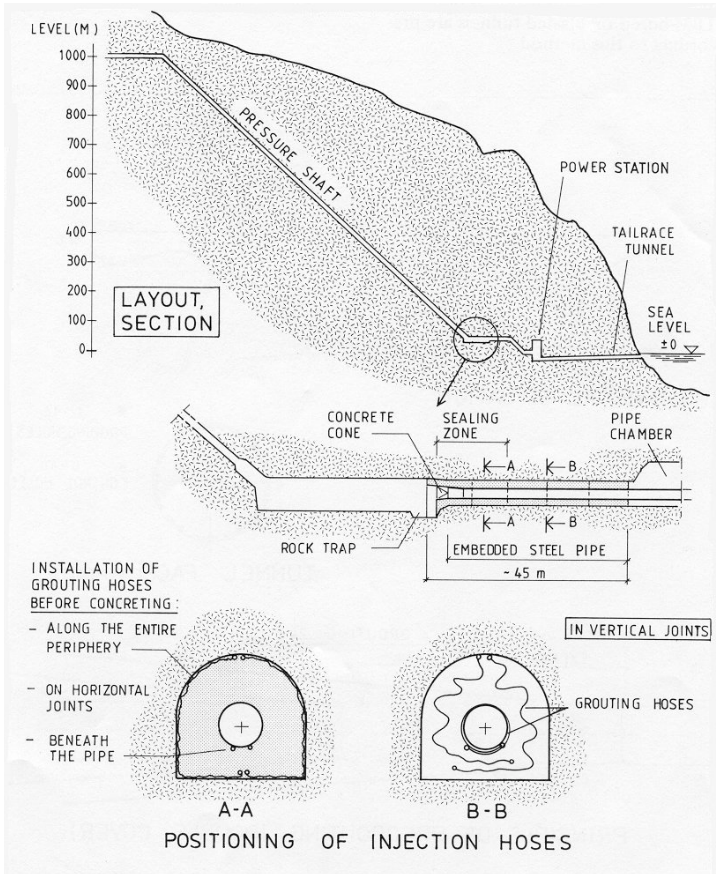

Fig. 17.4 Grouting of rock masses and concrete plug exposed to high water pressure

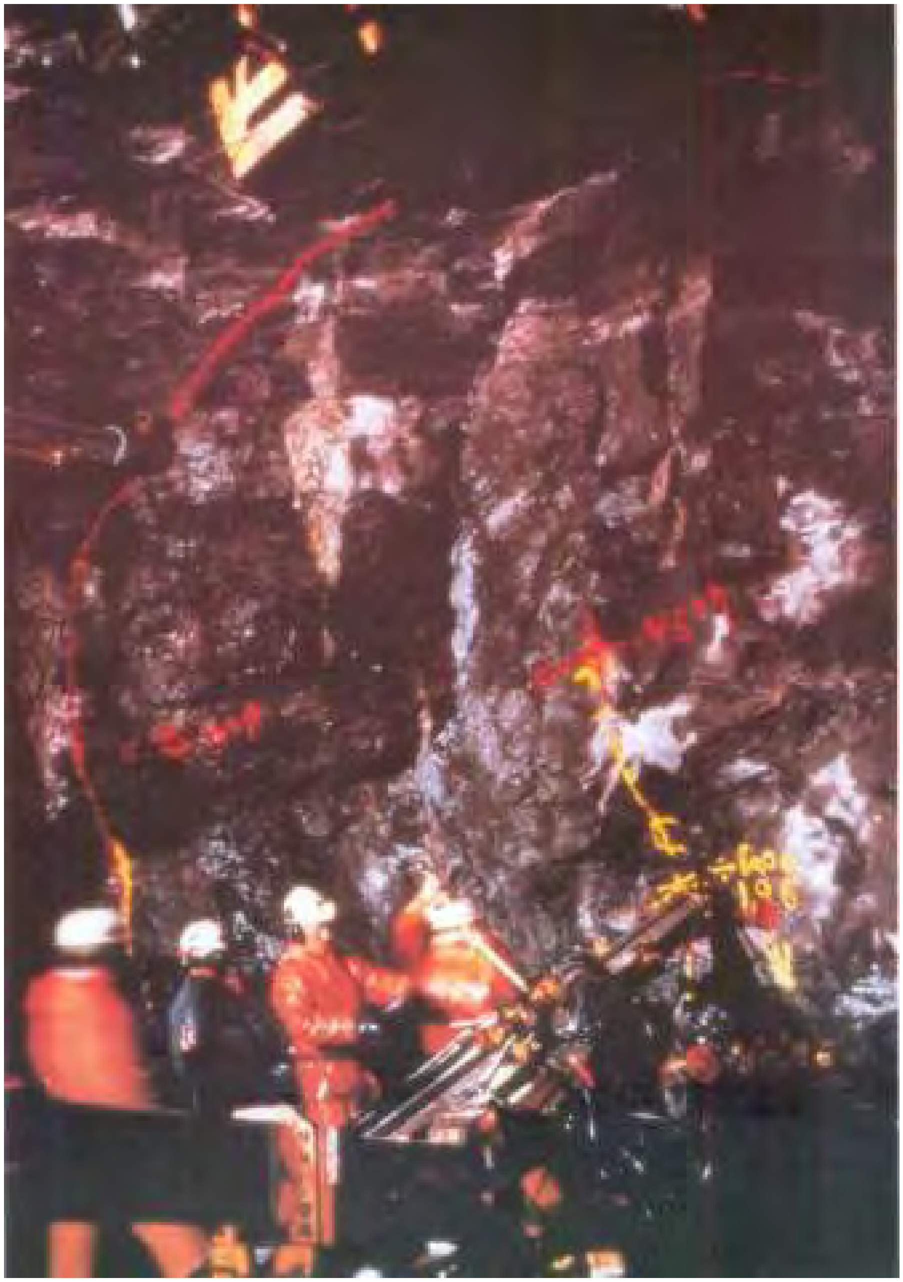

Photo 17.2 Drilling of grout holes to seal a water leakage in front of tunnel face. (O. Aasen)

18. HARD ROCK TUNNEL BORING IN NORWAY

Anders Borg
Selmer-Furuholmen Anlegg A/S

1. INTRODUCTION

Tunnel excavation with Tunnel Boring Machines (TBM), also called mechanical tunnel boring, was brought rather late into use in Norway. The reasons for this are the Scandinavian rock -which mostly was too strong and resistant for earlier available TBMs -and the fast excavation achieved by highly developed drill and blast techniques and skilled Norwegian miners. To compete with the Drill and Blast (DB) method, TBM-manufacturers were forced to design and build stronger machines.
A review of full face bored tunnels, Table 18.1, shows 150 km bored to date, increasing to approximately 200 km by 1989.
With further improvements and developments from both machine manufacturers and contractors, full face boring will not only remain as an alternative to the drill and blast method, but will in the future increase its share in the tunnel building industry.

Table 18.1 Some TBM tunnels and shafts in Norway

Project location	Machine Manufacturer	Size (m)	Project purpose	Boring period	Tunnel lenght (m)	Rock type
Trondheim	Demag	Ø 2,3	Sewer tunnel	1972 – 1974	4300	Greenstone, greenschist
Oslo	Robbins	Ø 3,15	Sewer tunnel	1974 – 1976	4300	Shale, limestone, dolerite dykes
Kjøpsvik	Wirth	Ø 3,32	Transport tunnel	1977 – 1978	1150	Limestone, amphibolite
Oslo	Bouygues	Ø 3,0	2 sewer tunnels	1977 – 1981	10800	Shale, limestone, dolerite dykes
Oslo	Wirth	Ø 3,35	Sewer tunnel	1977 – 1981	7600	Shale, limestone, dolerite dykes
Oslo	Atlas Copco	2,1 · 3,2	Sewer tunnel	1979 – 1980	1000	Shale, limestone, dolerite dykes
Oslo	Robbins	Ø 3,5	2 sewer tunnels	1978 – 1981	14200	Shale, limestone, dolerite dykes
Aurland	Robbins	Ø 3,5	Hydropower tunnel	1977 - 1978	6200	Phyllite
Sørfjord	Robbins	Ø 3,5	Hydropower tunnel	1980 – 1982	5840	Micashist
Brattset	Robbins	Ø 4,5	Hydropower tunnel	1980 – 1982	8150	Micashist, diorite
Ulla Førde	Robbins	Ø 3,5	Hydropower tunnel	1981 – 1984	8020	Granite, gneiss
Mosvik	Jarva	Ø 3,5	Hydropower tunnel	1982 – 1983	5700	Micashist, greenstone, gneiss
Yset	Robbins	Ø 4,5	Hydropower tunnel	1982 – 1984	7300	Micashist
Lysefjord	Jarva	Ø 3,2	Hydropower shaft	1983 – 1984	1250	Gneiss
Lysefjord	Jarva	Ø 3,5	Hydropower tunnel	1983 – 1984	5100	Gneiss
Kobbelv	Robbins	Ø 6,25	2 hydropower tunnels	1983 – 1986	5150	Granite, gneiss
Glomfjord	Robbins	Ø 6,25	Road tunnel	1984 – 1985	4333	Micashist, quartzite
Kobbelv	Robbins	Ø 3,5	Hydropower tunnel	1984 – 1987	11000	Granite, gneiss
Bergen	Robbins	Ø 7,8	Twin road tunnel	1984 – 1986	6850	Granite, gneiss
Årdal	Jarva	Ø 3,2	Hydropower shaft	1985	1370	Gneiss

2. ASPECTS OF TBM BORING

Most of TBM bored tunnels in Norway are built for sewer, water supply and hydro power plants. Advantages of using TBM are :

- Rock excavation by TBM creates less noise, vibration and pollution to the environment than the DB method. These are the main reasons for choosing TBM on sewer and water supply tunnels because they are generally closer to or even below populated areas (See next page about Trondheim and Oslo sewer tunnels).
- For similar reasons some road tunnels are being excavated by TBM (See next page about Fløyfjell twin road tunnel in Bergen).

* Due to less hydraulic loss in full face bored tunnels (35- 40% reduced cross section compared to DB) and faster excavation, i.e. shorter construction period, TBMs were and are mostly used in connection with the building of new hydro power plants and the renovation and increase in capacity of old plants like Nedre Vinstra hydropower plant. Also, quite often the number of adits (incl. roads and construction camps) are reduced since longer tunnel sections can be driven with the TBM-method.
* TBM bored tunnels require much less rock supporting work -if any at all -compared with those excavated by the DB method, and they are also easier to clean.

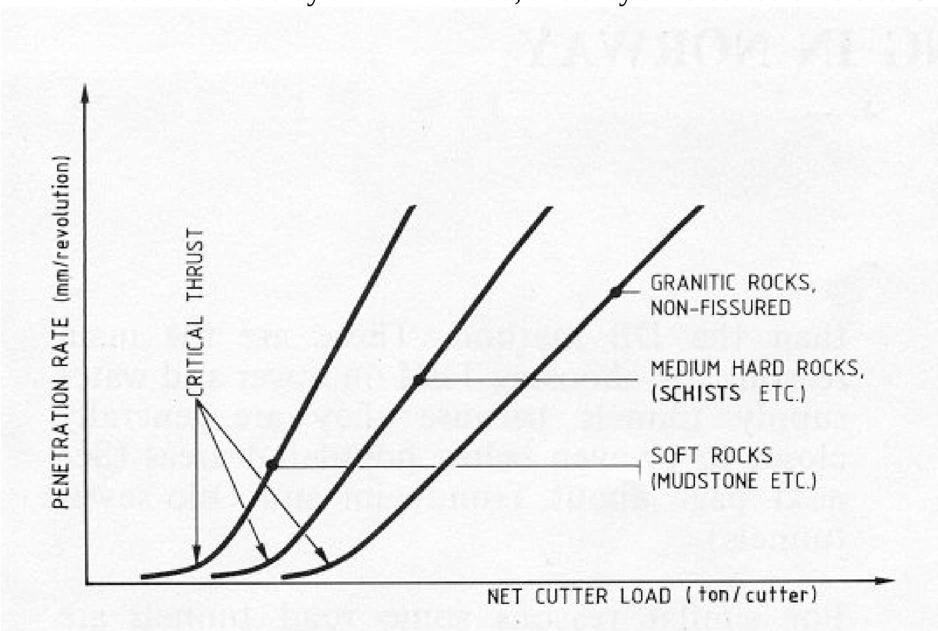

Fig. 18.1 Relation between cutter head and penetration rate

Penetration rate, expressed in advance per cutter- head revolution, is the most important factor and measure in tunnel boring.
Observations show nearly a linear relation between penetration rate and cutter load.
A certain minimum cutter load, so-called critical thrust in above Fig. 18.1, must of course be given to overcome the rocks own resistance against breaking, as below this no advance at all can be obtained. There are as well upper limits. These are not connected to the boreability of the rocks, but are limits in the capacity of the muck hauling system, available cutter head torque etc.

3. SOME NORWEGIAN TBM PROJECTS

Sewer Tunnels in Trondheim and Oslo
Full face boring started in rather weak rock types, like mud- and sandstone on the European continent.
Later, as stronger machines became available, a TBM was introduced in Norway in 1972. It was a Demag TBM of 2.3 m diameter which bored a 4300 m long sewer tunnel through greenstone and schist in the city of Trondheim. A little later the first Robbins TBM was put in operation for the large Western Oslo Fjord Regional Sewer Plant. A total of 7 machines bored altogether 38 km on this ambitious project. Excavation was permitted only with TBM because of the previously mentioned environmental problems expected with the DB method. In addition, this was the first project where probe-drilling and pregrouting ahead of the tunnel face were carried out

systematically over the entire length. The decision on whether to use chemical or cement grouting was to be taken on the basis of water loss testing in the probe holes.
This extensive and ex pensive pregrouting procedure was executed to prevent lowering of the ground water table and ground settlements in the area above the sewer tunnel. Pregrouting as a new technique in connection with TBM worked
very well, see Chapter 17. This new technique will be used increasingly in the future in tunnels to limit their drainage effect where this would cause ground settlements and damages to structures.

TBM-bored road tunnels
Under the city of Bergen a Robbins hard rock TBM bored two parallel tunnels of 7.8 m, diameter with a total length of 6.9 km through gneiss and granite.
The tunnel would have cost less by the drill and blast method, but environmental problems (populated are as above the tunnel alignment and with hospitals close to both portals) were sufficient reasons for the owner to use TBM. Specific for this project was the enlargement of the bored section after finished boring. The cross section had to be fitted to the roadways demand as to both width and height. This was achieved by benching on both sides, starting above the tunnel centre line. See Fig. 18.2.

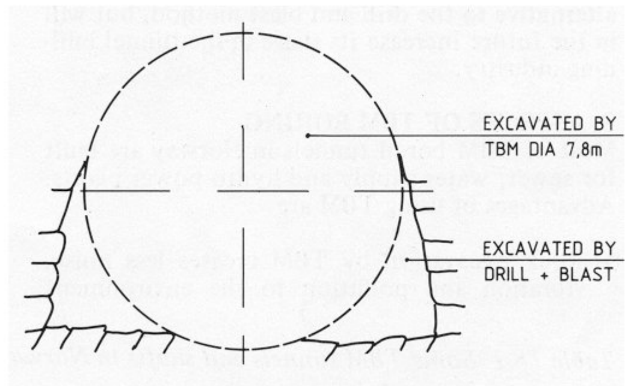

Fig. 18.2 TBM bored road tunnel in Bergen

4. EXPERIENCES
The most significant about the use of TBM in Norway is the excavation in extremely hard basement rock.
The attached list of some TBM projects shows that the large majority of the tunnels were driven under rock conditions which have not been equalled in other countries.
The fresh and strong gneisses and granites are very resistant and abrasive to machinery and cutter tools. The TBM-manufacturers needed to change their design to meet these new conditions. Not all changes were crowned with success. They were costly both to manufacturers and contractors and also to the owner when operating as his own contractor .
The lesson is clear :

1. In the Norwegian underground, expect tough conditions.
2. The equipment should be as strong and reliable as possible and not too sophisticated.
3. Changes in equipment design should be made as required on site. For changes to bring real improvements, they should be implemented step-by-step, based on practical underground experience.

A much higher cutter head thrust was a must to bore in granite. Design changes of the TBM like bigger cutters, followed by increased installed cutter head torque, stronger main bearing, gears, supports etc. have taken place, Fig. 18.3 and 18.4.

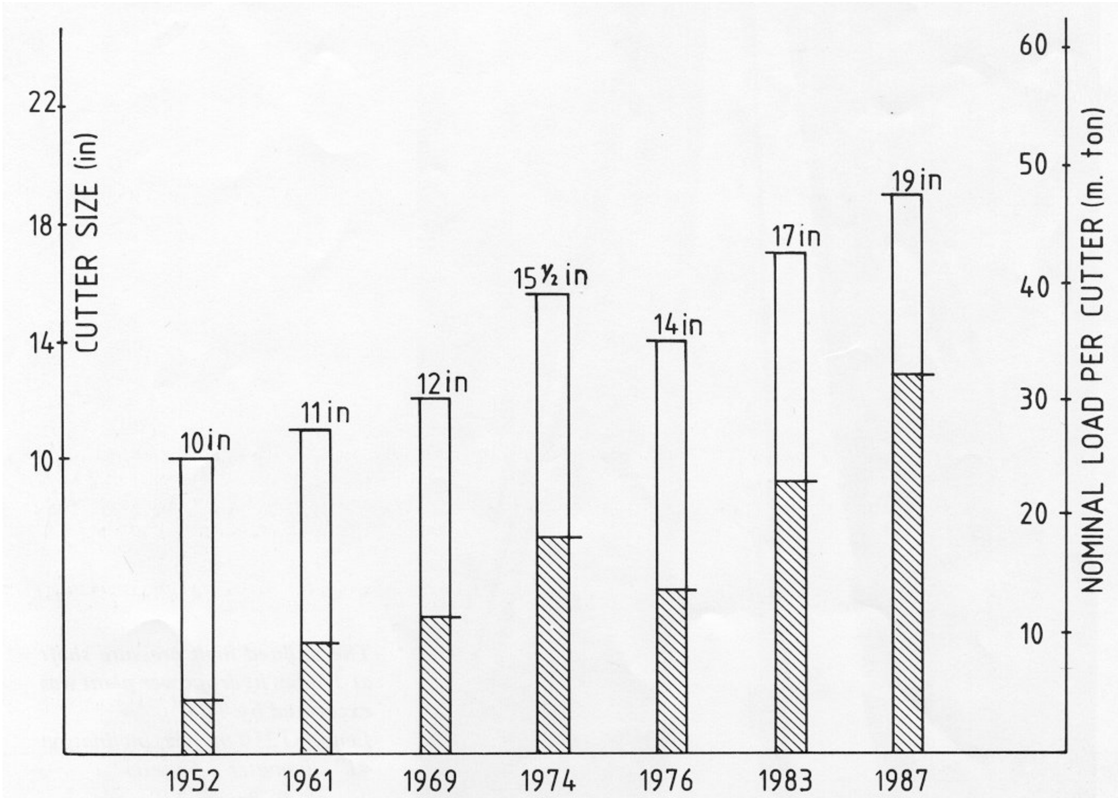

Fig. 18.3 Development towards larger disc cutter size and cutter head

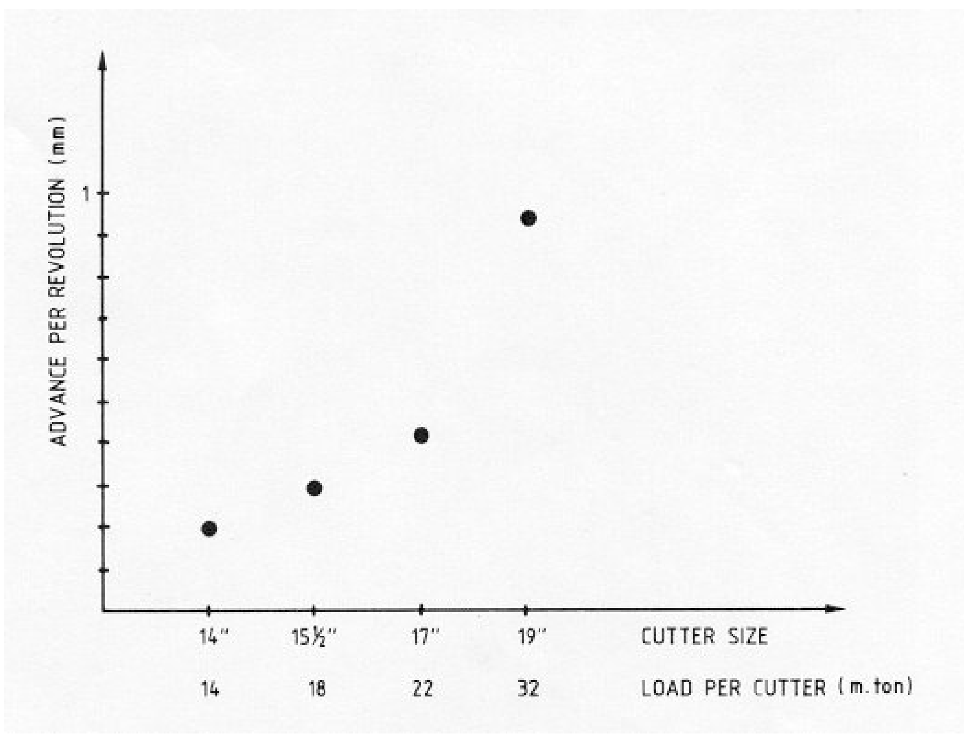

Fig. 18.4 Advance per revolution in relation to cutter size in granitic gneiss
(Advance with 19 inches cutter is assumed based on experience with other cutters)

Photo 18.1
The unlined high pressure shaft at Tjodan hydropower plant was excavated by TBM.
Length 1250 meters. inclination 41°, diameter 3.5 meter
(ing. A. B. Berdal A/S)

LARGE DUAL PURPOSE INSTALLATIONS IN ROCK.

Jan A. Rygh
Fortifikasjon A/S

1. INTRODUCTION

Norway, a country large in area but small in population, has put large effort in protection of its inhabitants in case of war. The Civil Defence has provided shelters for more than 50% of the population. Some of these shelters are placed in rock.

Since rock installations provide good resistance to weapon effects and also with advantage can be used for other purposes in peacetime, an interesting development has taken place in the last decades.

In towns and cities can be found sport halls, swimming pools, bowling halls and shooting ranges, etc. in rock, installations that easily can be converted into civil defence shelters.

Some typical installations are highlighted in the following.

2. ODDA SPORT HALL IN ROCK

The breakthrough in using rock installations for untraditional use came in 1972 with the completion of the Odda sport hall in rock. The main hall is 25 by 60 metres, adequate for playing international handball games. The hall is equipped with the normal installations for instructing and training of gymnastics. It has a gallery stand for 500 spectators, locker rooms and showers.

The rock conditions were favourable with Precambrian granites and gneisses.

The rock overburden is more than 50 m (fig. 19.1).

The sport facility has been in continuous use since 1972 and has performed well both in use and in low maintenance cost, e.g. the extremely low heating costs in the winter period.

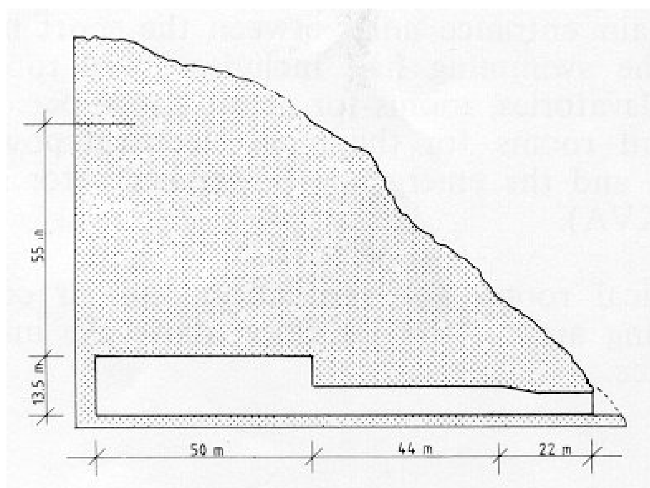

Fig. 19.1 Section of the Odda sport hall

Since 1972 a large number of such installations have been materialized. Some typical examples are described in the following.

3. SKÅRER SPORT HALL AND SWIMMING POOL IN ROCK

The situation was this :

The community of Lørenskog (just south of Oslo) had a great need for public shelter. In addition they also needed 4 gymnasiums for schools and swimming pool for beginners. They decided to see all those needs together and built a rock installation close to the schools. The rock excavation started in October 1978 and all construction work was finished in October 1980. The situation plan is shown on fig. 19.2.

The installations are situated in a small hill, where the rock is a relatively homogeneous gneiss. Three core drillings were done.

The axes of main halls have a N-S direction which is the best in the region. Greatest span width is 25 m and the smallest overburden is 15 m. The excavation was carried out without any particular problems. The initial rock support in the blasting period was done mostly with spot rockbolting (length 3 -4 m) and some shotcreting. For the permanent support about 400 bolts (2- 4- 5 m long) were used. The supporting work can be characterized as moderate.
The total rock volume was: 25 600.m^2 Fig. 19.3 gives more details.

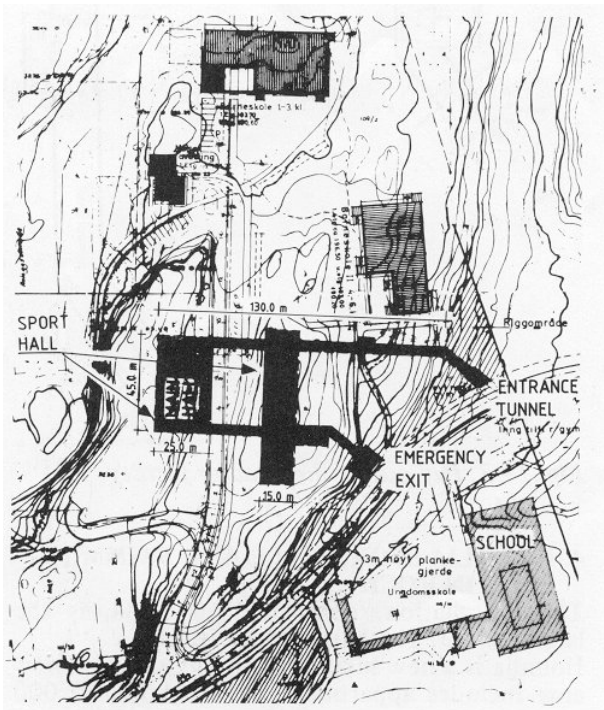

Fig. 19.2
Location of the Skårer sport hall and swimming pool

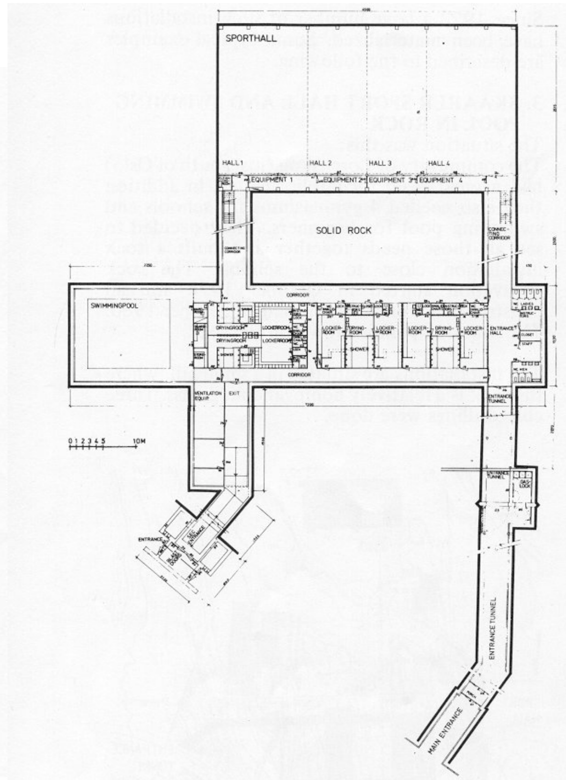

Fig. 19.3
Lay-out of the Skårer facility

4. HOLMLIA SPORT HALL AND SWIMMING POOL IN ROCK

This installation, completed in 1984, is the largest so far. The back-ground is this:
Holmlia is a new suburb south of Oslo. This new area includes apartment buildings for 10 000 people, schools, a shopping centre etc.
At an early stage in the preplanning phase for Holmlia, it was decided to build a modem centre for sports activities including a swimming pool, which could also be utilized for other social activities. The suburban

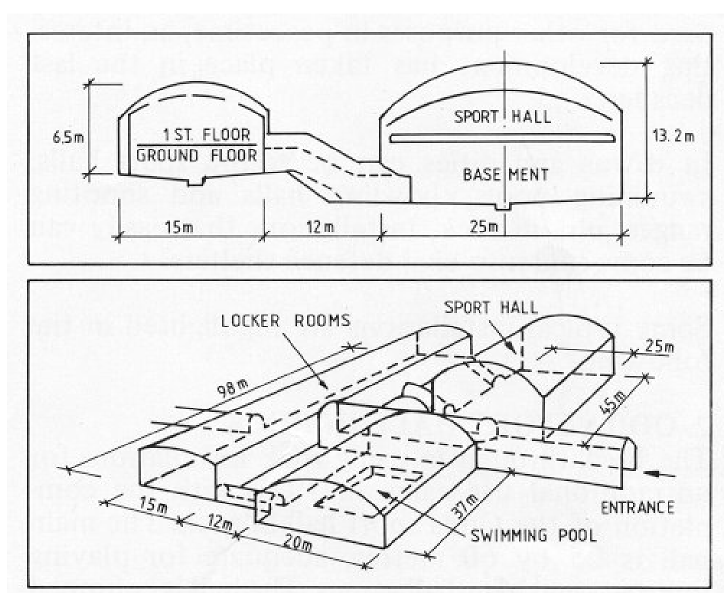

Fig. 19.4
The Holmlia sport hall and swimming pool

inhabitants should also need blast resistant and gas tight civil defence shelters in case of war for about 7000 people.
Large amounts of rock material was needed for construction of roads, parking areas etc. All this could be solved simultaneously if the civil defence shelter were built as an installation in rock, designed and equipped for swimming and other sports activities. Outside areas could be save d for other purposes. As the performance of similar projects over a decade was good, with energy consumption as well as running and maintenance costs being very low, it was decided to place the sports activity centre of Holmlia in rock.
The installation consists of a sport hall 25x45 m (fig. 19.4) for various kinds of ball games and for athletics (gymnastics) and a swimming hall, 20 x 37 m, with swimming pool, 12,5 x 25 m (6 tracks), both halls with auxiliary facilities such as wardrobes and showers. The saunas with rest rooms are built in connection with the swimming hall wardrobes. A central hall inside the main entrance and between the sport hall and the swimming hall includes guard room, shop, lavatories, rooms for maintenance personnel and rooms for the main electrical power panels and the emergency power generator set (300 KVA).
Technical rooms for ventilation and air conditioning are on the 1st floor above the main entrance.

Split of cost. Typical installation (Holmlia)

Excavation (drill & blast)	13,0%
Rock support and mucking out	5,5%
Civil works	34,8%
Electrical installations	7,2%
Lifts	0,3%
Emergency power system	1,0%
Ventilation/ cooling	5,0%
Sanitary and heating	4,5%
Water cleaning system for swimming pool	0,7%
Mapping and geological investigations	0,5%
Engineering	7,5%
Management, administration	2,2%
Taxes	17,8%
Total	100%

5. VÅGAN SPORT HALL IN ROCK

The last installation to be described is the Vågan Sport Hall, completed in 1984.
It is situated in the middle of the fishing town Svolvær far north in the country. As seen on the situation map, the rock formation (monzonite) is rather small. After excavation, rock walls and roof of only 10 -15 m thickness remain. Fig. 19;5.
In addition to the more or less standard sport hall there are club rooms, showers, saunas and locker rooms and even a shooting range (30 m) situated inside. A special feature in this hall is a training wall for rock climbers, as se en on photo page 115.
Quantities and capacity: Excavated volume: 21 100 m^3 Floor area: 2 970 m^2 Sport hall: 24 x 42 m Shelter: 3 330 persons

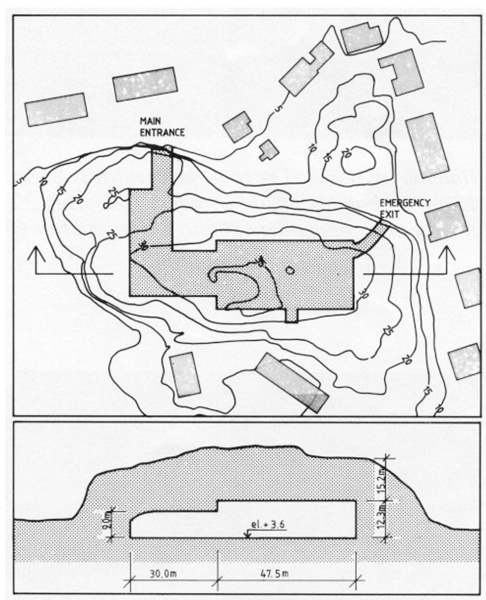

Fig. 19.5
Location of Vågan sport hall

Table 19.1 Ten typical dual purpose rock installations in Norway

Name of project	Finished year	Max span	Volume (m^3)	Rock type
1. Odda, sport hall	1972	25	27 000	granite
2. Lykkeberg, swimming pool	1970	16	4 000	granite
3. Gjøvik, swimming pool	1975	20	17 000	gneiss
4. Vaasøyholtet, shooting range & bowling alley	1979	12	5 500	gneiss
5. Skårer, sport hall & swimming pool	1981	25	25 600	granite
6. Holmen, sport hall (enlarged)	1981 1988	25	35 000	shale & limestone
7. Tærud	1982	25	27 500	gneiss
8. Vågan, sport hall	1983	25	21 000	monzonite
9. Holmlia, sport hall & swimming pool	1983	25	53 000	gneiss
10. Klemetsrud, sport hall*	1988	25	35 000	gneiss

**) Average cost: blasting, support and mucking out, (Klemetsrud 1987), 27 US$/$m^3$*

6. FUTURE PROJECTS

Several dual-purpose installations in rock are under planning and/or construction by 1987.
Where the rock is of good quality, span widths up to 60 m are being planned.
The saving of energy for heating in the harsh Norwegian climate as well as saving of land lots are also major factors when the rock solution is chosen.

Photo 19.1
Holmlia sport hall The sport hall which is 25 m x 45 m with a maximum height of 13 m, includes a hand-ball court measuring 20 m x 40 m. The gallery and grand stand can accommodate 300 to 350 spectators (Fortifikasjon A/S)

Photo 19.2
Vågan sport hall An area for rock climbing can be seen in the wall (Fortifikasjon A/S)