GEOLOGY, DESIGN, CONSTRUCTION AND MAINTENANCE OF VARDØ SUB-SEA ROAD TUNNEL

Geologist T.E. Lynneberg	Public Road Administration	Norway
Eng. Geolog. A. Palmstrøm	Ing. A.B. Berdal A/S	Norway
Eng. S.Roska	County Roads Aut., Finnmark	Norway
Eng. K.J. Carstens	Ing. Thor Furuholmen A/S	Norway

Abstract

After extensive investigations the Vardø undersea road tunnel was built during the 4 years between 1979 and 1983. The locations of the adits were chosen with regard to short connection to the main road on the mainland and the course of the rock surface undersea, together with the required 32 m minimum rock cover.

The construction works, with probedrilling, excavation, rock support and sealing went on with acceptable advance.

Maintenance during the last 3 1/2 years has given valuable experiences with regard to constructions under sea. The corrosive environment leads to damages on metal constructions and machines. More surprising is the growth of bacteria in the drainage system.

1. Background

A small harbour and an unsatisfactory connection by ferries to the mainland were the reasons for the increasing propaganda pressure for a road link across Bussesund. This finally led to a parliament decision in 1977 to link the island of Vardø with the mainland, and a construction start-up scheduled for summer 1979.

At an earlier stage, two alternatives lower in cost, a road fill and a bridge founded on piles were considered. Both projects were found to be feasible. New planning left a stronger bridge design on table, together with an undersea tunnel project.

Preliminary studies for tunnel revealed the bedrock to be strongly folded and faulted and partly soft and permeable. The tunnel project therefore was considered too risky at that time.

By 1976, constructions of undersea tunnels had been reported both in Norway and abroad. In spite of rock bursts and water inflow it was possible to take necessary precautions. This information available led to a decision once again to study the tunnel alternative, now in sufficient detail. The results of this investigation, together with a cost estimate, had to be presented within spring 1978.

Only the summer 1977 was left to the field studies. These had to comprise all aspects to the tunnel project, including geology, installations, maintenance and service.

2. Studies of the Geology

Early planning, restricted to a bridge location, had led to studies by means of acoustical soundings, boreholes and seismic profiles. These investigations now had to be enlarged and cover other parts of the sound (4).

Field mapping was started to outline the geology in the predicted tunnel area. Frogman geologists were engaged in mapping the seabed geology. The rocks were classified and tentative stratigraphy was set up. The main structures were extrapolated into the sound, and a rough model of the geology was designed. To complete the outline of the geology, seismic measurements and boreholes were required.

In all about 9 km of seismic profiles were shot. In addition to offering a more accurate location than the acoustic measurements, the seismic measurements provided a distribution of the seismic velocities in the bedrock. From this distribution, the rocks were tentatively classified with respect to tunnelling. However, the seismic measurements are inconclusive, particularly as to the geometry and stratification of the rock. The detection of gaps in the bedrock, flatlying structures and structures parallel to the profiles are particularly difficult with this method.

In order to obtain a continous mapping of the bedrock in the tunnel alignment, horizontal holes were drilled from both sides of the sound, whithin narrow limits situated close to the crown of the tunnel project. Such holes were considered too expensive.

On the contrary, it was decided that horizontal cored holes should be drilled in front of the tunnel face during tunnelling to obtain an extra security.

The studies agreed in presenting a flat and even rock surface. The deepest bedrock surface was -33 mbsl., but up to 7 m greater depth within two low velocity zones revealed from boreholes.

Permeability tests indicated normal water leakage conditions. The bedrock seemed to consist of about 60 % clay shales and 40 % flaggy sandstones of low metamorphic grade. Bedding planes inclined approx. 50^d towards east on the island side, vertical or steeply towards west from the middle of the tunnel towards the mainland side, fig. 1.

The bedrock was partly jointed and some distinct faults were identified. One of them was found to have a seismic velocity as low as 2500 m/s, the half of ordinary values.

3. Making the decision

It was concluded that the tunnel project was feasible, and that a normal hard-rock tunnelling procedure could be used. The distinct fault zones might cause problems, but it was decided to postpone rock support measures until results from the cored holes had been achieved. The possibility of using the freezing method to stabilize the rock mass was held open.

Four items were recommended for safety:

- Long-core boreholes ahead of the tunnel face.
- Short percussion-drilled holes ahead of the face.
- Water- leakage tests in the holes.
- A continous state of preparedness for grouting, shotcreting and concrete lining.

The final project implied a tunnel 2618 m in length, with its deepest point 88 m below sea level.

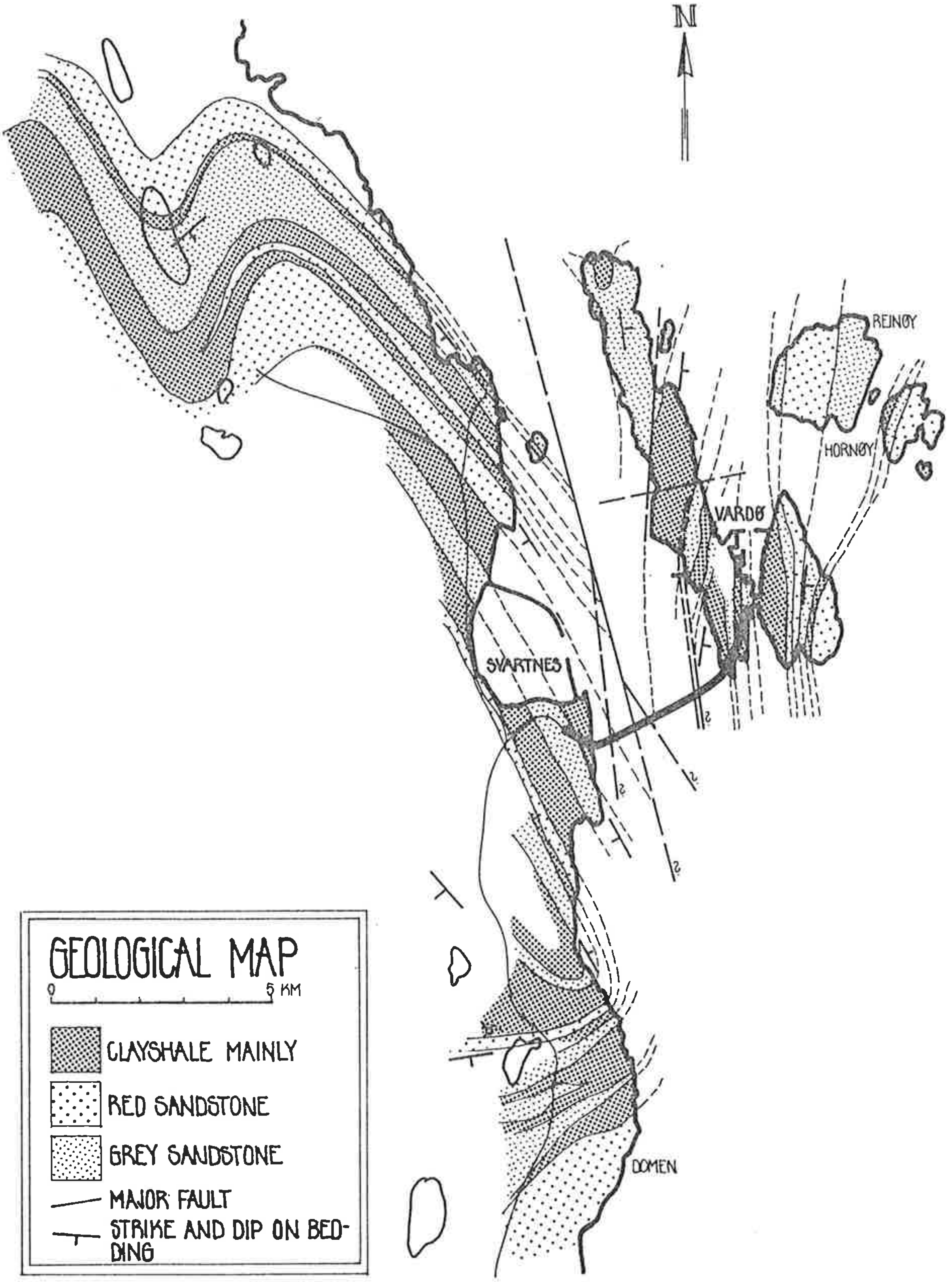

Fig. 1 Geological map showing major fault structures and rocks in the Vardø area together with the tunnel crossing. The original bridge alternative would have crossed the sound via the small island 7 km further north.

Early in 1978, the tunnel project was presented with a cost estimate between NOK 70 and 110 million, depending on rock conditions and permeability. The bridge project was estimated to cost NOK 120 million.

The annual maintenance and service costs were estimated to NOK 0,8 million for the tunnel, against NOK 0,2 million for the bridge alternative.

Several practical factors favoured the tunnel project. The bridge project would increase the road distance to the mainland by 4 km. During winter, arctic storms would expose bridge construction to the summertime only. These apparent advantages led to the final decision to build a tunnel.

4. Technical solutions

The locations of the adits were chosen with regard to short connection to the main road on the mainland and to the town on the island. The required 80 0/00 decent for the tunnel and the course of the rock surface in the sound together with the required 32 m minimum rock cover between tunnel roof and rock surface beneath the sound gave the 2618 m long rock tunnel, designed as shown in fig. 2. About 1700 m of the tunnel would be under the sea.

The figure shows two traffic lanes with pavements on both sides, and a cross sectional area of 50 m^2. Open cuts would give access to the portals.

The main features of the tunnel construction were:

- The tunnelling works with excavation, rock support, probe-drilling and sealing.

- Pumping of water from three pumping stations, one at each of the tunnel portals to remove rain and meltwater from the open cuts, and one at the low-point of the tunnel to pump out the permanent leakage of seawater.

- The electro-mechanical plant, i.e. the lighting-, ventilation-, control- and monitoring plants and systems. The power was to be supplied via a surface building on the mainland connected to the tunnel by a vertical shaft. A transformer would be placed in the tunnel at the pumping station at the lowest point of the tunnel, with the power- and control cables, located in a duct along the north side, given a cover to serve as a pedestrian pavement. Conduits for telecommunication cables was to be placed in a concrete block along the south side of the tunnel.

- Frost protection by insolated shield to prevent water seepage drips and ice formation during winter.

- Snow shelters in both open cuts was to be built as concrete culverts over a length of about 200 m on the mainland and about 70 m at the Vardø end.

- A 600 m dia, watermain, going to be Vardø's main supply of water, placed in a trench together with the drainage water pipe, with the necessary manholes for both pipes.

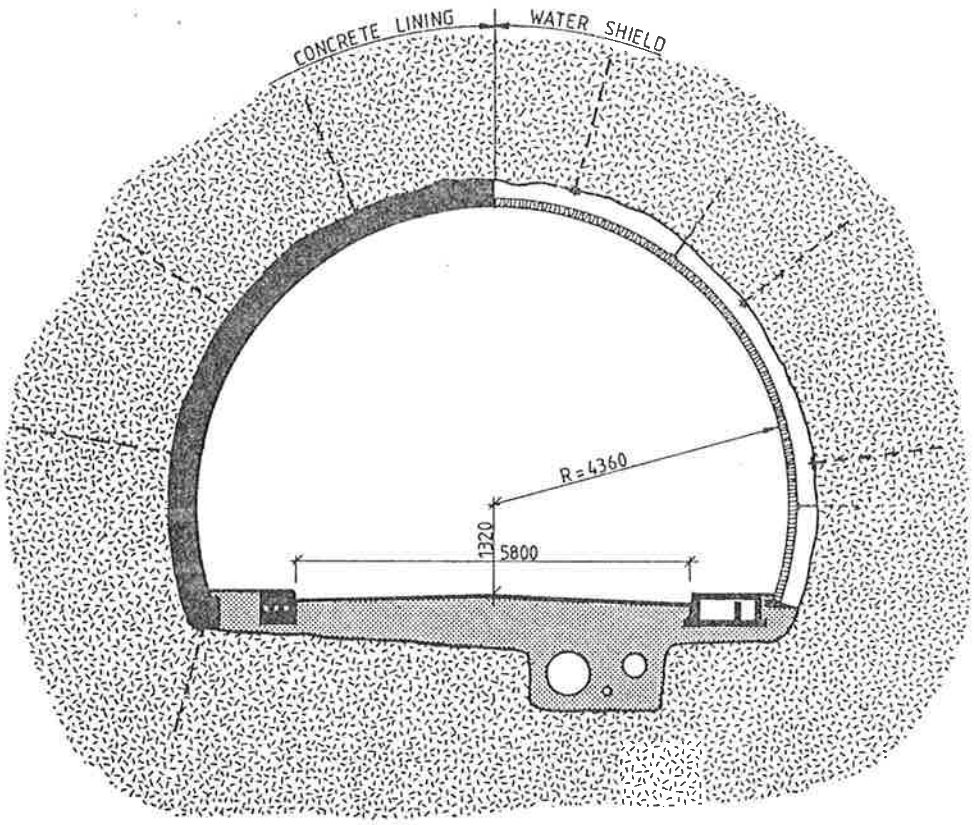

Fig. 2 Cross sections of tunnel showing rock support in poor rock on the left side and water shields on the right side.

5. Tunnelling works

The drill and blast method for excavating the shales and the harder abrasive sandstones along the tunnel was chosen as the most economical method. The expected fair to poor rock mass conditions caused special care to be taken in making out the drill and blast pattern using weak explosive in the contour holes.

The geological investigations and evaluations had showed that the rock mass conditions for tunnelling were fair to poor, with an overall Q-factor of 1-3.

The equivalent rock support types were evaluated as:

a) In fair rock mass conditions
   - rock bolts spaced 1-3 m and thin (5 cm) shotcrete or wire mesh.

b) In poor rock masses
   - rock bolts spaced 1 m and mesh reinforced shotcrete (15 cm thick).

c) In very poor rock masses
   - concrete lining.

Caused by the relatively thin rock cover (32-50 m) for the tunnel and the variable rock quality, a careful plan for the execution of the excavation works was worked out. The aim was to avoid large, unexpected water leakage to occur during excavation and to be prepared if unstable rock masses could cause cave-in to develop up to the sea bottom.

For these reasons, the extensive exploratory drillings were planned, comprising 3-4 percussive 25-30 m long probeholes drilled from the tunnel face with an overlap of 5-8 m leakage measurement in the holes to be carried out (fig. 3).

The aim of the percussive drillings was to obtain valuable information about the leakage conditions ahead of the tunnel face. Additional information about rock quality ahead of the tunnel face was planned to be found from 200 m long core drillings carried out as a part of the exploratory drilling programme (fig. 4).

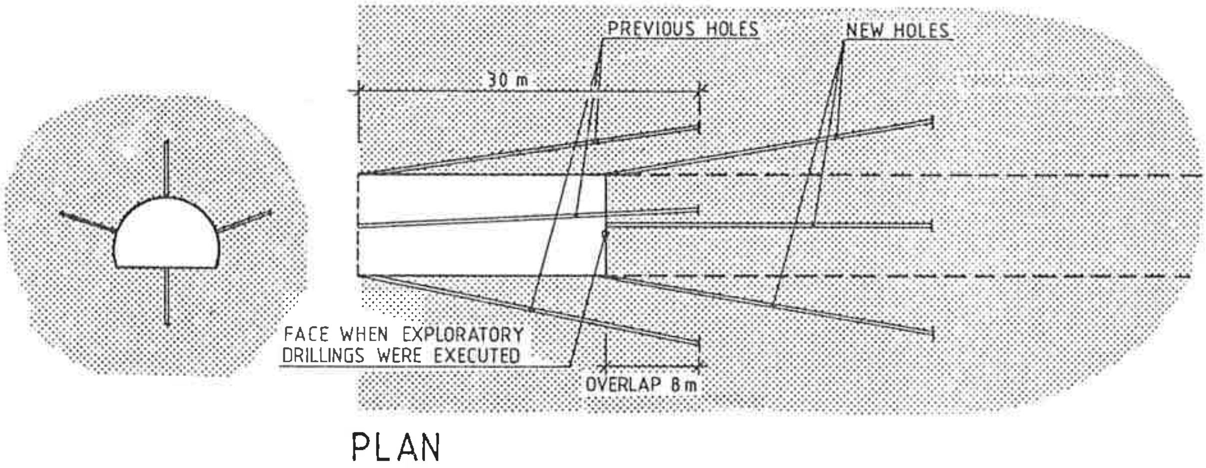

Fig. 3 Exploratory percussive drillings ahead of the tunnel face.

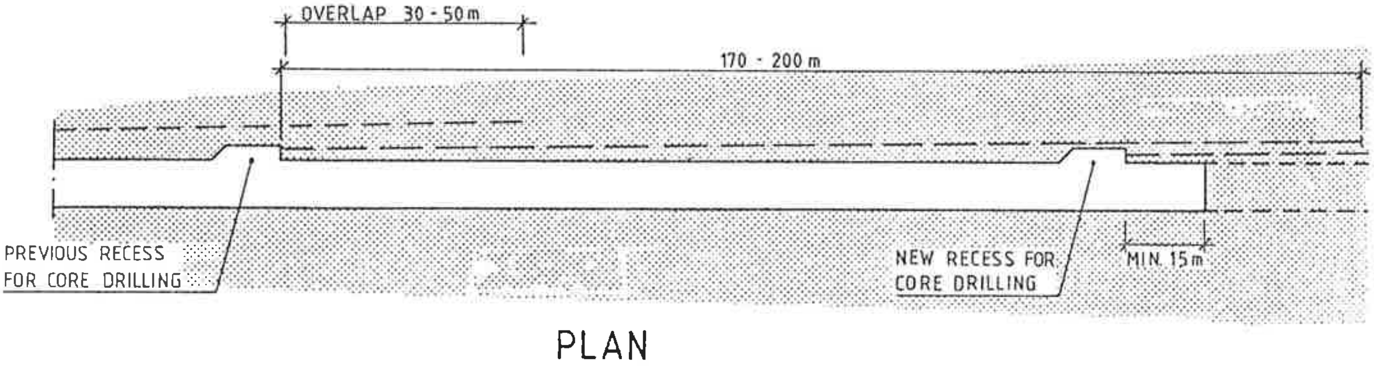

Fig. 4 Exploratory core drillings made from specially provided recesses.

Previous tunnelling experience in Norway had shown that the sealing of water bearing zones is most successfully done by pre-grouting ahead of the tunnel face. It was planned to carry out sealing works by pre-grouting when the exploratory drillings showed leakages in excess of 2.5 Lugeon (fig. 5).

From the results of the water pressure test in the investigation core drillings shown in fig. 6, it was roughly estimated that if leakages above 2-3 Lugeon were sealed by cement grouting, this had to be carried out over about 10 % of the tunnel length.

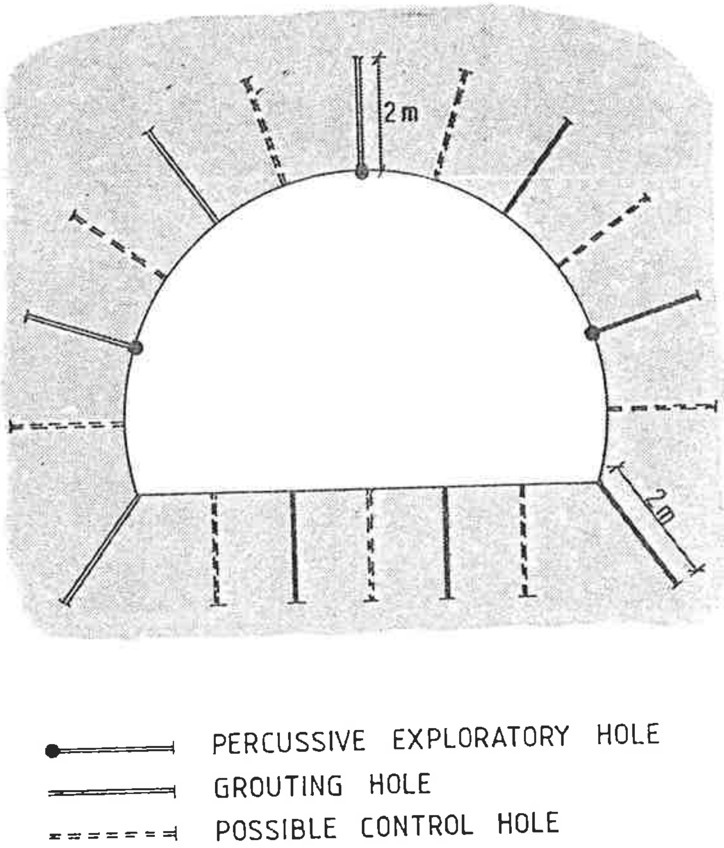

Fig. 5 Principles of exploratory holes, groutholes and control holes used for pre-grouting.

The aim of the sealing by pre-grouting was:

1) to avoid possible large inflows of water which could cause problems for the drill and blast work or under adverse conditions: to fill the decending tunnel.
2) To reduce the permanent leakage to an acceptable amount with regard to the pumping costs.

6. Technical installations

6.1 Pumping of water

Surface water (fresh) from the open cuts is collected at the two pumping stations placed just behind the portals. The volume of water varies with weather and season, and the stations are placed relatively high to give a small pumping head. In addition the pump sumps can be emptied for maintenance purposes by directing the run-off to the lowpoint of the tunnel.

The capacities of the pumps are 3000 l/min and 7000 l/min for the Vardø end and the mainland end respectively. The installation consists of 2 and 4 submerged pumps respectively, placed in guide-tubes. The water is pumped into PEH-pipes of 250 mm and 400 mm diameter respectively.

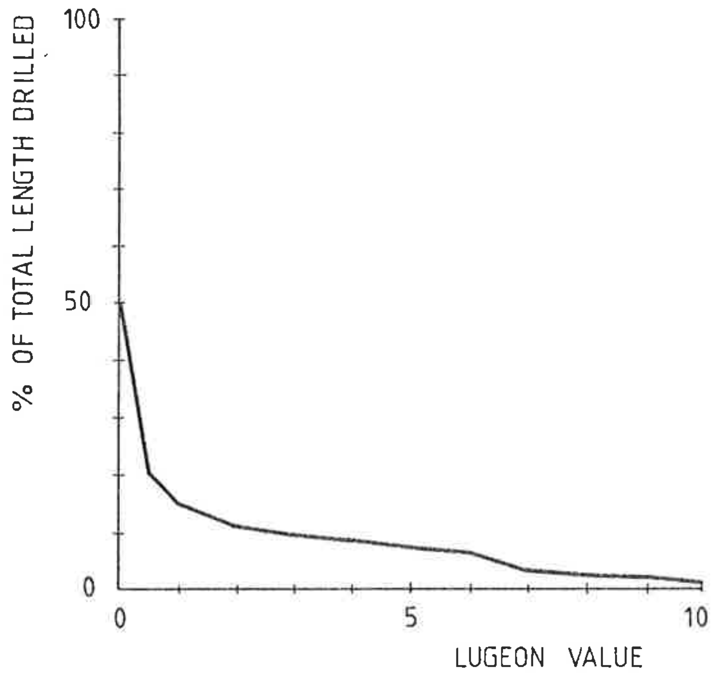

Fig. 6 Results from water pressure tests in core drilling holes.

The pump sump at the low point of the tunnel has a volume of 2400 m^3. Six identical submerged pumps are installed in two groups of three, connected to the normal and reserve discharge pipes. Both pipes are in PEH of 300 mm diameter and are 850 m long. The main pipe is placed in a trench and the reserved pipe in the pavement, up to the vertical shaft at 450 m from the mainland portal. In the shaft the pipes are placed in concrete.

It was difficult to estimate in advance the volume of seepage (mostly sea-water) into the tunnel between the pumping stations at the portals. Assuming that only major leakages in the rock were sealed, a total seepage of 1200 l/min was estimated. Because of uncertainties in the estimates, for a total seepage volume of 3000 l/min as the assumed upper limit.

Prior to construction consideration was given to which degree it was economically advantageous to seal the rock and install the corresponding pumping capacity with attendant running costs, during the useful life of the project. It became apparent that sealing minor leakages by grouting had no economic advantage and that pumping this seepage water out was the correct solution. This was valid providing the remaining leakage volume was not impractically large. Although the calculated (and actual) leakage volume is substantial (1000 l/min) and the pumping head high (90 m), the adopted solution is economical.

All ferrous components in the pumping plant have been specified to be stainless steel.

6.2 The electro-mechanical plant

The tunnel is supplied with power by a 22 kV cabel to the surface building at its mainland end which contains a 22/12 kV, 500 kVA transformer. In addition two 12/0,4 kV transformers have been installed, of which one is in the building and one at the lowpoint of the tunnel. The pumps at the lowpoint of the tunnel consume most of the power with their six 48 kW circuits.

The ventilation consists of ten 12 kW fans governed by CO-sensors.

Lighting consists of 55 W Nal fixtures at 18 m centres, augmented by four 250 W Nah fixtures in each entrance zone. Every fourth lighting fixture is equipped with an emergency power supply unit which will operate in the event of a power failure.

An alarm system monitors water level in the lowest pump sump and the CO-level in the tunnel. Should critical values be exceeded, red traffic light at both entrances will be switched on and an alarm will be transmitted automatically to the fire station in Vadsø.

A standby generator rated at 500 kVA in the surface building will cut in automatically in the event of a power failure.

6.3 Frost protection

The frost protection required was specified by the Norwegian State Roads Laboratory and was based on the following values:

- Frost value in free air in Vardø : 15000 h^0C/year

- Frost value in the middel of the tunnel : 3000 h^0C/year
- Ambient rock temperature : + 2 to + 5 ^0C
- Freezing point for salt water seepages : - 2 ^0C

This was interpreted to mean, for example, that dripping seepage water in the middel of the tunnel could freeze in the road surface in particularily cold periods and could create dangerous build-ups of ice as known from tunnels at higher altitudes. A double, insulated shielding was specified for all areas where water dripped from the tunnel roof or walls, to allow seepage water to reach the drainage system without freezing.

Measures required to prevent frost from penetrating the floor in the outer 500 m of the tunnel were 60 mm Styrofoam sheets placed on top of the drainage layer prior to placement of the roadbase. In the first 300 m of the tunnel this insulation was also placed under the pavements.

6.4 The snow shelters

The snow shelters are built as reinforcerd concrete culverts founded on rock, with a semicircular internal profile and a semicircular or square external profile. The thickness of the concrete in the crown is 300-400 mm. A sealing membrane is placed on the outside of the structures. The structures are covered approx. 1,5 m thickness of gravel.

7. Tunneling

Detailed information from the drill and blast tunnelling is not available in a suitable form. However, the following data give some ideas of the progress obtained and the problems encountered.

The tunnel was excavated from both sides. The average numbers were as follows:

From the island side pr. m advance:
2 1/2 hours scaling
4 pcs of rock bolts
0.7 m^3 shotcrete
0.10 m concrete lining
3.0 m exploratory holes
16 m advance pr. week.

From the mainland side:

2 hours scaling

2 pcs of rock bolts

1.0 m^3 shotcrete

0.23 m concrete lining

3.0 m exploratory holes

9.5 m advance per week.

2 episodes from the 2 years of tunnelling should be mentioned here:

After 3 months, while drilling for rock bolts at face, the tunnel roof caved-in and crushed the jumbo. The cost of this was 6 weeks with support work, mainly concrete lining and jumbo repair, to further advance.

A year later another cave-in occured in faulted rock. The fault zone was crossed by the use of forepoling, an extremely slow procedure in this case, 30 m tunnel advance in 3 months. Freezing was also considered.

During the 2 years of tunnelling the equipment and machineries suffered extremely. Corrosion damages, due to the salt water environment often led to thorough overhaulings.

8. Maintenance during 3 1/2 years

The tunnel has so far functioned as anticipated (fig. 7). In spite of operational problems people have hardly noticed anything of such kind.

Problems considered most important have been related to three areas of matter:

The drainage system,
Shotcrete damages in the salt water zone,
Corrosion on metal structures.

In addition to that, there has also occationally been problems with the pumping plant and the power supply respectively.

Fig. 7 Way down the tunnel from the mainland

8.1 <u>Shotcrete damages</u>

Damages on the shotcrete are limited to the salt water zone only. Bits and pieces of cracked shotcrete have fallen down to some extent, and can further on easily be broken loose by hand.

Ion wash-out and bacterial decomposition have been mentioned as possible causes. It is now certain that bacteria have been established in the drainage water and seem to use the acceleration agent water glass as substratum.

The content of cement measured in a single shotcrete sample had decreased from 500 kg per m^3 to 200 kg per m^3.

A committee has been appointed by the Norwegian Public Roads Administration to work on that particular problem, and a report is expected within the autumn of 1986.

8.2 Drainage system

Approximately 1000 litre mainly salt water leaks into the tunnel every minute. The leakages has been constant since 1981. The water salinity is highest at the lowest point, gradually decreasing towards the ends of the tunnel.

The problems consists in an insufficient permeability through parts of the drainage masses together with mud deposites and bacterial growth as well in the drainpipes, as in locally created water pools.

Photo inspections in the drainpipes has shown that the bacterial growth has reduced the cross section with 20 per cent at the most. A cross section of the pipe system is shown on fig. 8.

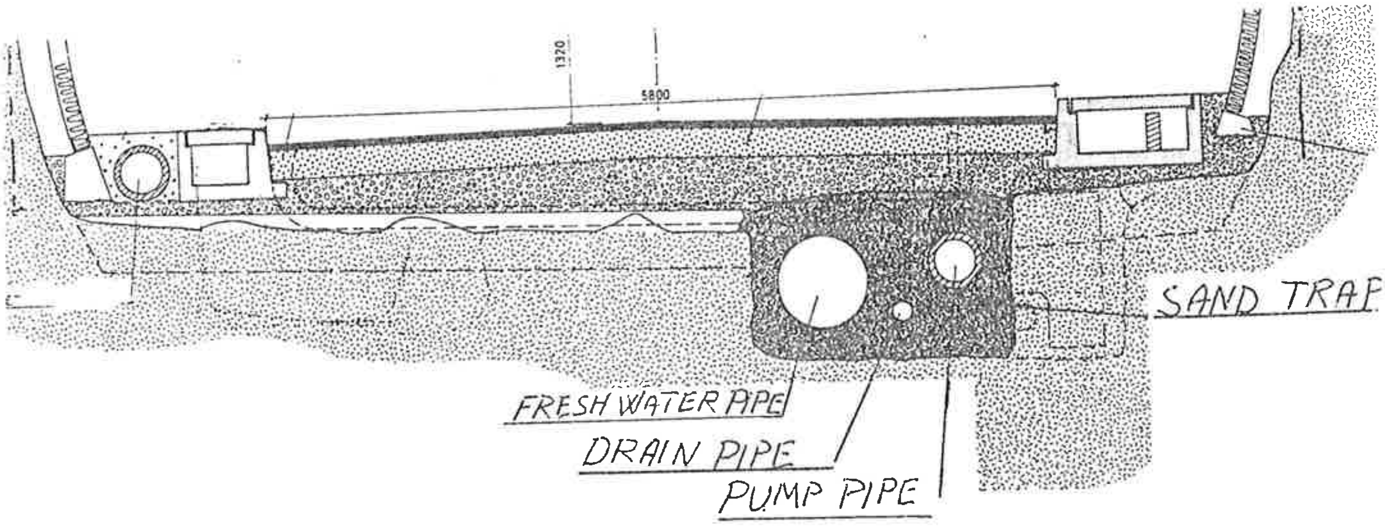

Fig. 8 Drainage ditch with location of the 3 different pipes

In order to remove the water pools, extra drains has been put down, and the bacteria removed by high-pressure flushing. It may be necessary to carry through flushing every second year.

The mud consists mainly of precipitated products of iron and aluminium oxides, together with the iron bacteria Leptotrix and Gallionella.

The precipitation of iron and the growth of bacteria are caused by the high level of Fe^{2+} ions in the drainage water. The aluminium hydroxide is deposited aluminium from the aluminium constructions due to corrosion. Great amounts of bacteria similar to Hyphomicrobium have also been established in the mud. Growth of Hyphomicrobium in seawater has not been reported before.

In case the remedial actions used so far turn out to be insufficient, the use of chemical remedies must be considered in order to reduce the growth of bacteria.

8.3 Metal corrosion

So far the corrosion problems in the tunnel are moderate, considering the corrosive environment established behind the aluminium shielding.

Corrosion has been traced on the aluminium rails and nails used for mounting the shielding. The residual product is $AL(OH)_3$, which can be found on the drainage masses.

However, corrosion distributed so far give no reason to believe that extensive replacements of the shielding will be necessary for many years to come.

Also valves of stainless steel (material quality 254 SLX) in the pump basin have turned out to be corroded. This can be observed as hollows in the material. The values have been repaired by the supplier.

Furthermore, corrosion has been found on one of the pumps. A circular hole 5 mm in diameter has been formed, but this may as well be due to material defects and needs further investigations on the matter.

The hot galvanized rock-bolts show no sign of corrosion, except where coating were damaged before mounting. However, this has happened to a very little extent.

8.4 The pumping plant

The two pumping stations just behind the portals have worked satisfactory.

However, the pumping station at the low point has caused some trouble.

The pumping station was built with a joint well for both systems (12.3 and 456), fig. 9.

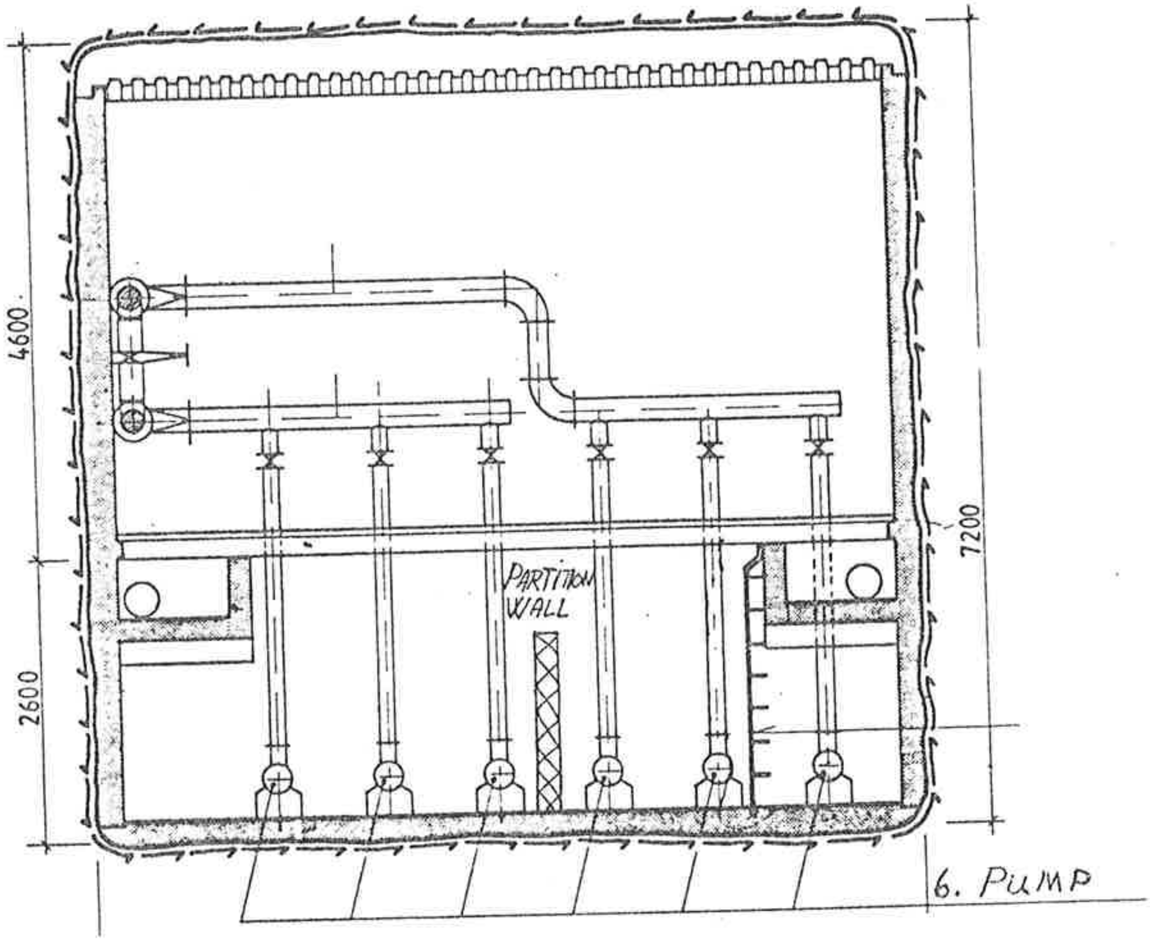

Fig. 9 The pumping station at the low point

This has entailed that pump repairs have to be done under water. This difficulty has been solved by the installation of a partition wall between the two systems, in such a way that one of the systems can be emptied while the other is in ordinary function.

The end of covers of both the manifolds were lateral and the material was too thin. This entailed that one of the manifolds cracked while running. The end covers are now substituted by curved ones with a thicker material.

Five break-downs of the pumps have taken place. Two of these happened due to incorrect assembling. The other three break-downs are probably brought about by a combination of corrosion and particles in the water.

8.5 The power supply

The tunnel is supplied with power from the local electric power plant. On account of the severe weather conditions and old power lines, there have been many power failors at times. 10-15 power failors per month, in time of a few seconds till several hours have occurred.

In order to keep the tunnel running, a stand-by 600 kW power unit has been installed. Occationally, the automation system of the aggregate has misfired. The main problems have either been a lack of start-up the aggregate, or stop when the main power supply appeared. Therefore, a manual running preparedness had to be set up 24 hours a day with extra expense. The problems have yet not been properly solved.

8.6 Running expenses

The running expenses for 1984 amount to a total of NOK 1.73 million and for 1985 NOK 1.62 million. The expenses for 1986 are calculated to a total of NOK 1.5 million, all current prices.

The reasons for the falling expenses seems to be that the initial weaknesses hopefully are about to disappear.

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