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GEO-INVESTIGATIONS AND ADVANCED TUNNEL EXCAVATION TECHNIQUE IMPORTANT FOR THE VARDØ SUBSEA ROAD TUNNEL

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Abstract

The first subsea road tunnel in Norway connects the island of Vardø with the mainland. The tunnel, with a cross section of 53 m2 and a length of 2,600 m, had its breakthrough in July 1981. The lowest point of the tunnel is 88 m below sea level and the minimum rock cover 32 m.

From the field investigations, consisting of detailed geological mapping, seismic refraction measurements and core drilling, three major faults and several minor ones were predicted below the sea bottom. Only few severe water leakages were to be expected. Caused by the relatively thin rock cover over the tunnel and an overall poor rock mass quality an advanced plan for the excavation works were made consisting of exploratory drillings and grouting ahead of the tunnel face.

The tunnel is excavated in a flaggy sandstone of Late Precambrian age. In some fault zones the rock mass stability was so poor during construction that highly advanced excavation and tunnel supporting technique had to be carried out. A special rapid system for concrete lining at the working face described in the paper was invented.

As a result 561 m length of the tunnel was concrete lined in addition to 2,500 m3 of shotcrete and 18,000 rock bolts. The rock supporting works amounted to 220% compared to the costs of blasting and mucking out. This is much higher than normal in Norway.

The average tunnelling progress was 17 m/week on each of the two working faces and the average construction cost amounted to NOK 40,000 per m corresponding to USD 7,000 per m (1982).

1. Introduction

Vardø is a small island off the far north-eastern point of Norway, latitude 70° 20' North (Fig. 1) inhabited by approx. 4000 people who work mainly in the fish industry. The island, which is situated 1.5 km from the mainland (Fig 2), borders on the Barents Sea. The climate is strongly influenced by the arctic conditions with average winter and summer temperature of -3°C and +6°C respectively.

The planning of a permanent bridge connection with the mainland was looked upon after the town of Vardø was being rebuilt after the Second World War. In 1977 when money was allocated for the project, it was found that a tunnel beneath the sound was feasible and even cheaper than a bridge (Ref. 2). The reason for this was that considerable experience had been gained during the preceding 5 years from construction of 2 other submarine tunnels.

The tunnel descends at 8.0 per cent from both sides, producing a 2.6 km long tunnel with a minimum rock cover of 32 m under the eastern part of the sound. Some 1,700 m of the rock tunnel is beneath the sea (Fig. 2 and 3). The two-lane tunnel has a cross sectional area of 53 m2 (Fig. 3), and a width of 9.4 m.

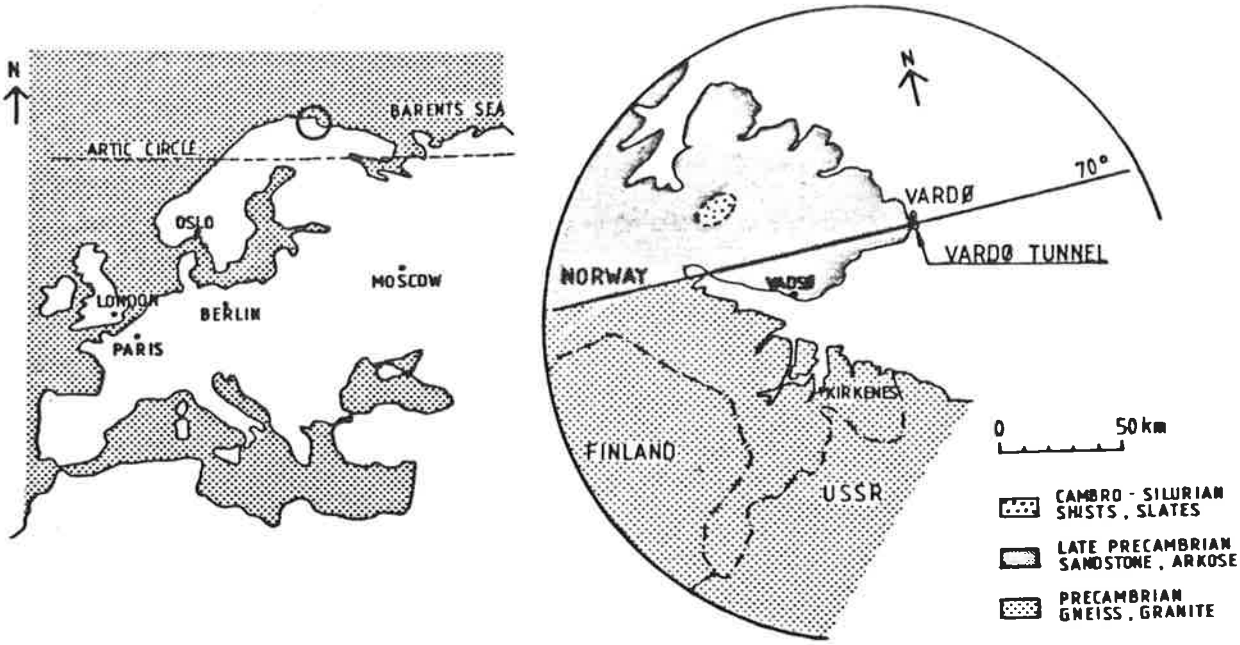

Fig. 1 Key map

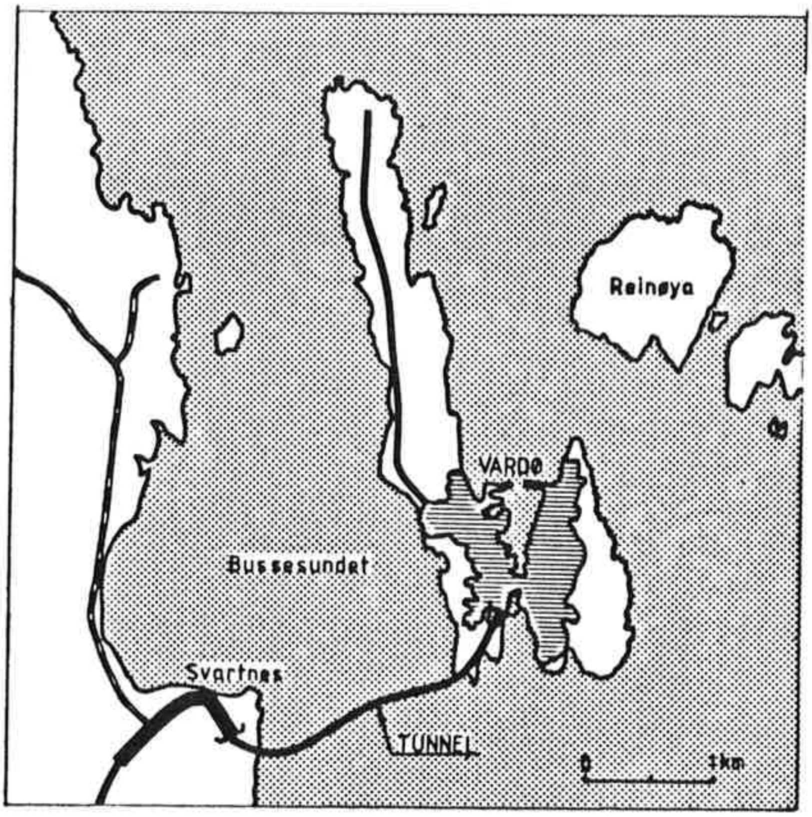

Fig. 2 Location of tunnel

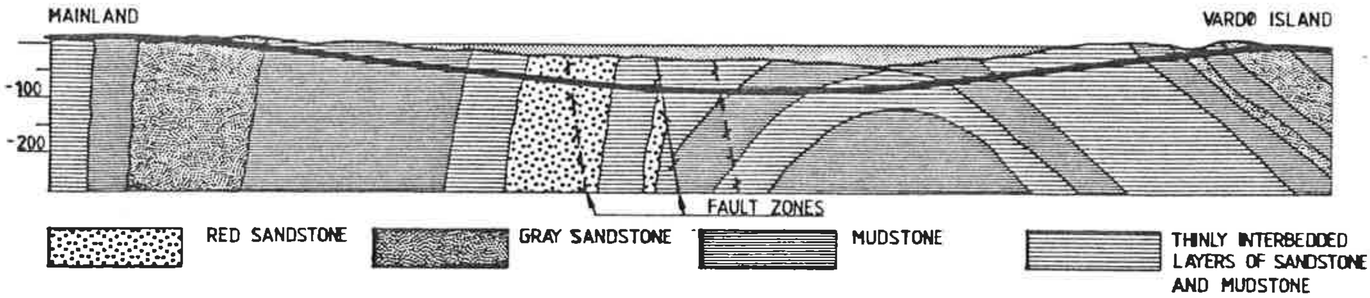

Fig. 3 Geological section along the tunnel

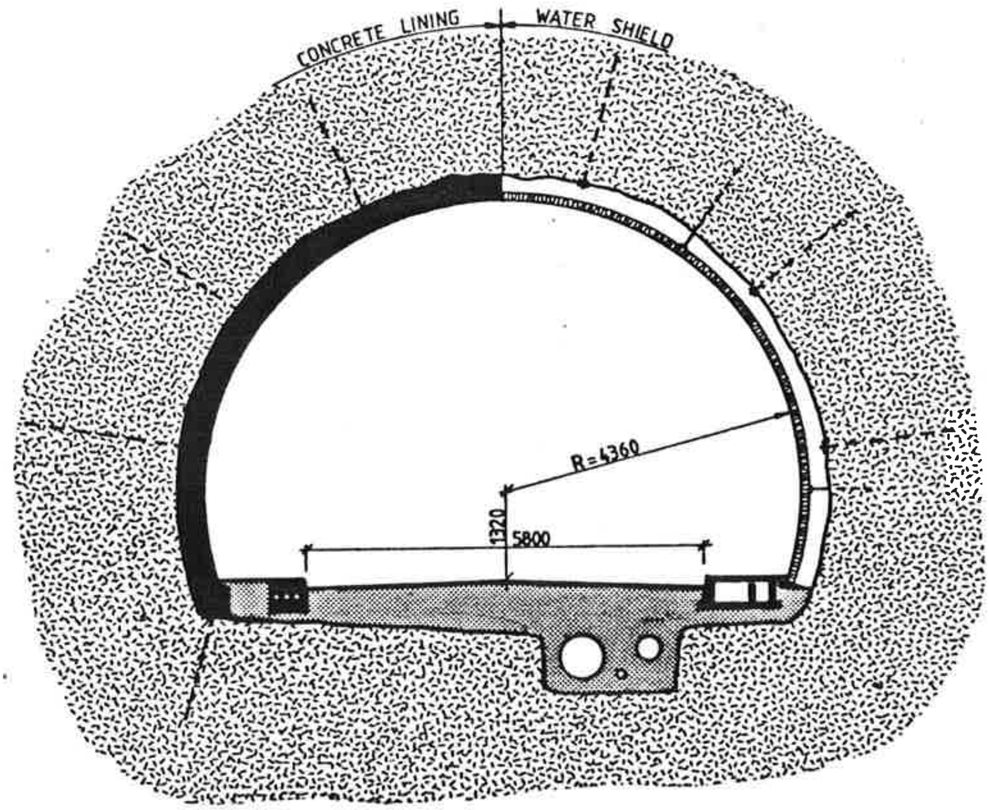

Fig. 4 Cross section of tunnel showing rock support in poor rock (on the left side) and water shields (on the right side)

The excavation of Vardø Tunnel started in June 1979 and the breakthrough occurred two years later. The tunnel excavation was attacked from both ends, using normal drill and blast method. Average tunnelling progress was 17 m/week with a maximum progress of 55 m/week working 75 hours a week (10 shifts).

2. Field investigations

As basis of the feasibility study and detail engineering the following field investigations were carried out, (Fig. 5).

Detailed geological mapping both on the island and on the mainland with results presented in maps scaled 1:1000. Loose deposits over large areas on the mainland reduced the accuracy of the geological interpretation here.

Boomer-sparker profiling, which made it possible to work out a map in scale 1:1000 of the seabed, indicating the thickness of the loose deposits.

25 seismic refraction profiles with a total length of 12.7 km. The profiles cover a 500 m wide zone across the sound. The measurements gave data about the thickness of the loose materials as well as seismic rock mass velocities, indicating weakness zones or faults.

36 borings in the sound to detect the exact position of the rock surface in the largest depressions above tunnel alignment. The main reason for this was to control the rock cover over the tunnel. Short core drillings were carried out in the bottom of some of the holes to obtain samples and water pressure tests of the rocks. These borings showed that the rock surface found by the preceding seismic method had an accuracy of ± 0.5 m, not counting clefts along weakness zones.

7 core holes totalling 660 m drilled from the coast. The holes were submitted to water pressure tests and the water leakages recorded as Lugeon-values.

From these field investigations the rock surface was determined relatively accurate. This was significant for the evaluation of the tunnel alignment with respect to necessary rock cover and hence total length of the tunnel. The costs of the field investigations were 0.8 mill.USD (1982).

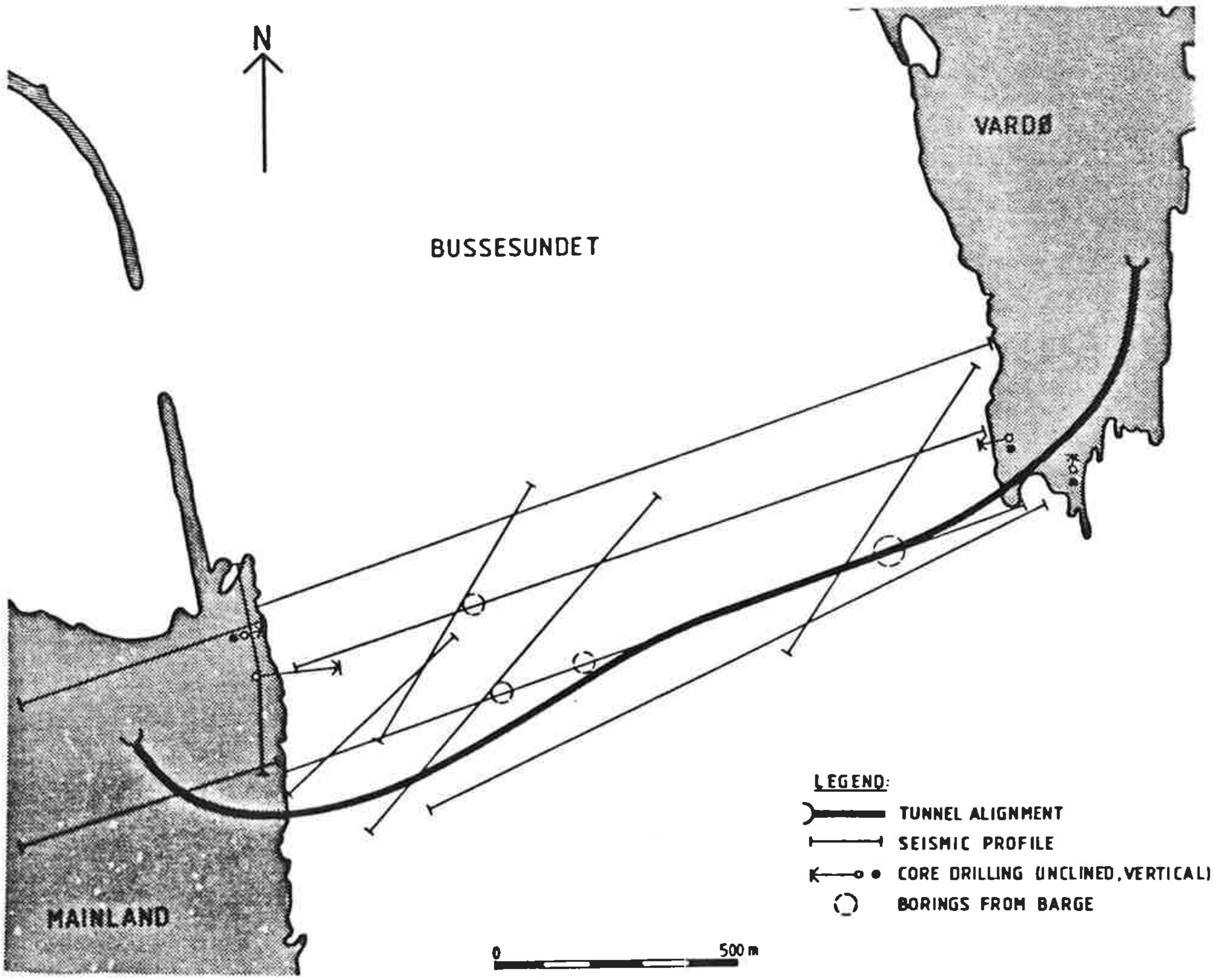

Fig. 5 General view of the field investigations

3. Geology and rock mass conditions

The bedrocks of Late Precambrian Age consist of slightly metamorphic quartzitic sandstones, siltstones and clayschists (Fig. 3). The rocks are folded along a N-S fold axis. In the middle of the sound Bussesundet a faulted and unsymmetric anticline occur. On the mainland the bedding dips steeply towards West and on the island Vardø the dip is 40-50^{o} East.

Also, several 0.5-5 m thick dolerite veins occur, striking NE-SW with rocks composed mainly of unaltered metadolerite.

In addition to frequent bedding joints spaced 0.1-1 m, three other steep-dipping joint sets occur with spacings as follows:

Vertical joints, strike N-S	spacing	1-2 m
Vertical joints, strike E-W	"	0.2-1 m
Flat-dipping joints	"	0.5-2 m

The degree of jointing can be classified as moderate to high with a volumetric joint count (Jv) of 5-19 joints per m3 of rock mass, and a mean value of 8 (Ref.7). Most of the joints are planar and have thin clay coatings.

The geological investigations and evaluations showed that the rock mass conditions for tunnelling were fair to poor. Using the Q-factor classifying system the overall Q-factor, Ref. 1, outside weakness zones was calculated as:

$$Q_1 = \frac{RQD}{Jn} \times \frac{JR}{Ja} \times \frac{Jw}{SRF} = \frac{90}{12} \times \frac{1}{4} \times \frac{1}{1} = 1.9 \text{ (poor)}$$

assuming a normal stress situation and only minor water leakages into the tunnel.

The Q-factor in weakness zones was found between

$$Q_2 = \frac{25}{12} \times \frac{1}{4} \times \frac{1}{2{,}5} = 0.2 \text{ (very poor)}$$

and

$$Q_3 = \frac{10}{15} \times \frac{1}{6} \times \frac{1}{7{,}5} = 0.015 \text{ (extremely poor)}$$

4. Rock support and water leakage evaluations

According to the Q-system the equivalent dimension for a 9.4 m m wide road tunnel will be:

span/ESR = 9.4/1.0 = 9.4

where ESR = Excavation Support Ratio. Its value depends on the permanent use of the tunnel. (Ref.1)

The rock support classes and estimated types of support are given in Table 1.

Rock mass quality (Q-value)	Support class	Type(s) and amount of support to be used
Q_1 = 1.9	22	Rock bolts spaced 1 m Shotcrete 5 cm
Q_2 = 0.2	31	Rock bolts spaced 1 m Shotcrete, mesh reinforced 10 cm
Q_3 = 0.015	38	Shotcrete, mesh reinforced 0.7 - 2 m thick

Table 1. Rock Support in different rock mass qualities.

The core drillings all showed small leakages except for a few zones. This indicated that the permanent leakage in the tunnel would be low when the main water bearing zones were sealed by grouting. A permanent water leakage of 1.0 m3/min. was predicted based on the water pressure tests in the bore-holes.

5. Safety measures during excavation

Caused by the relatively thin rock cover (32-50 m) for the tunnel and mostly poor geological conditions 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 extensive exploratory drillings were carried out. Along the entire length of tunnel under the sea, 3 - 4 percussive probeholes were drilled from the tunnel face. The holes were 25-30 m long with an overlap of 5-8 m, (Fig. 6). During the drilling operation, variations in the drilling rate were roughly recorded, yielding information about possible fractures ahead of the tunnel face. The leakage out of the holes was recorded and water pressure tests were performed. It was sometimes very difficult to drill through clay seams or highly fractured zones.

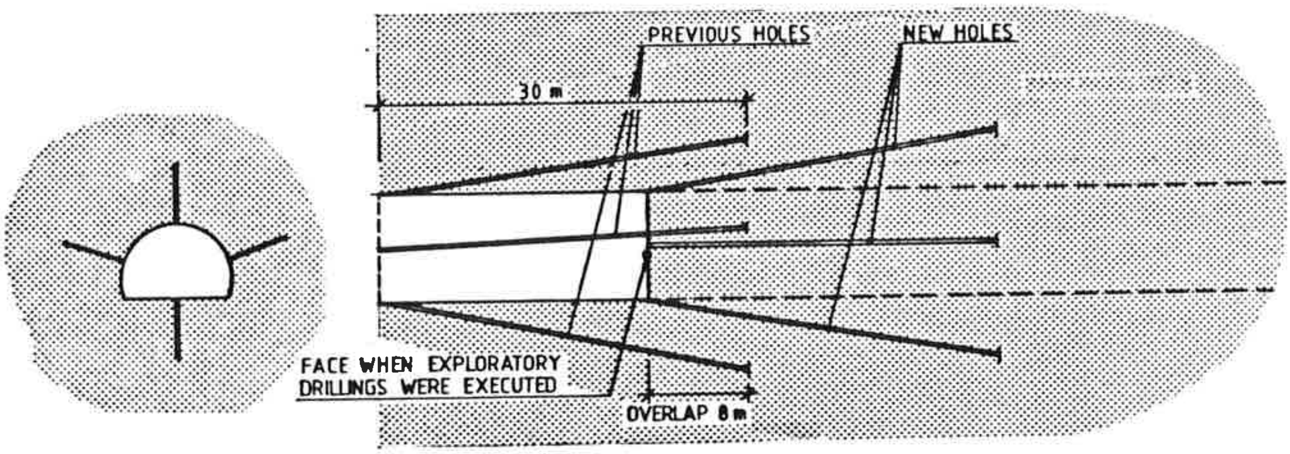

Fig. 6 Exploratory percussive drillings ahead of the tunnel face.

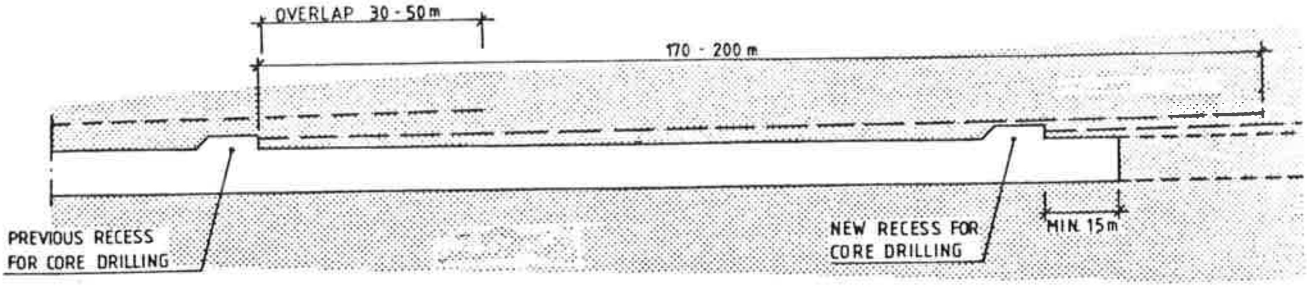

Fig. 7 Exploratory core drillings made from specially provided recesses

The percussive drillings gave valuable information about the leakage conditions ahead of the tunnel face. Additional information about rock quality ahead of the tunnel face was achieved by core drillings carried out as a part of the exploratory drilling programme. The core holes were drilled from specially provided recesses, and performed during the ordinary tunnel excavation (Fig. 7). The length of the holes was approx. 200 m and the distance between recesses 170-180 m. For each 6-12 m length of the core hole, water pressure tests were performed and the Lugeon-values calculated.

6. Tunnelling experience

The expected fair to poor rock mass conditions caused special care to be taken in making out the drill and blast procedure. Weak explosives were used in the contour holes.

The degree of jointing proved to be somewhat higher than expected. There was also a greater number than expected of joints with clay coating or clayey fillings. The immediate support in these rock mass conditions was carried out by scaling works, rock bolts and shotcrete. Later this support was strengthened mostly by one more layer of shotcrete, and/or some more rock bolts. As shown earlier in Table 1, this was in fact more or less the types and the amount of rock support expected.

The bedding joints were often very closely spaced in the clay schists. Where in addition other joints or fractures occurred, the stability was severely reduced, and extensive supporting works resulted. This was the situation in large parts of the tunnel excavated from the mainland. The unfavourable direction of the tunnel with regard to jointing during the first 400 m here had also a great impact on the amount of the supporting works which here was somewhat higher than expected.

The predicted large size and moderate size weakness zones were mostly found in the tunnel (Fig. 8). Some of the zones had, however, a poorer stability than expected because of the overall prominent claycoated bedding joints. An example of this is shown on Fig. 9 where the necessary linings of the predicted weakness zones were longer than expected from evaluations of the seismic refraction velacities. The same features were found also in other low seismic velocity zones.

All the large and moderate weakness zones found in the tunnel were stepwise lined with in situ cast concrete. Concre lining was used in many other parts of the tunnel where unfavourable fractures or smaller clay-containing zones cut through the flaggy clayschists.

On two occasions the stability of weakness zones was so poor that special precautions had to be taken during excavation in order to prevent cave-in on the working face. A special excavation and supporting procedure was invented for these situations. The principles of this method, which was successfully applied, are shown in Fig. 10. Because of the short stand-up time (10-30 minutes) the experience was that the shotcreting should be carried out soonest possible after blasting before mucking out. The spiling bolts should be installed from stabilized rock masses or from the concrete lining. The final and most important rule is that the length of the round must be adjusted to the actual rock mass quality conditions. In a few instances the length was reduced to 0.8 m with stepwise in-situ concreting before the next round could start.

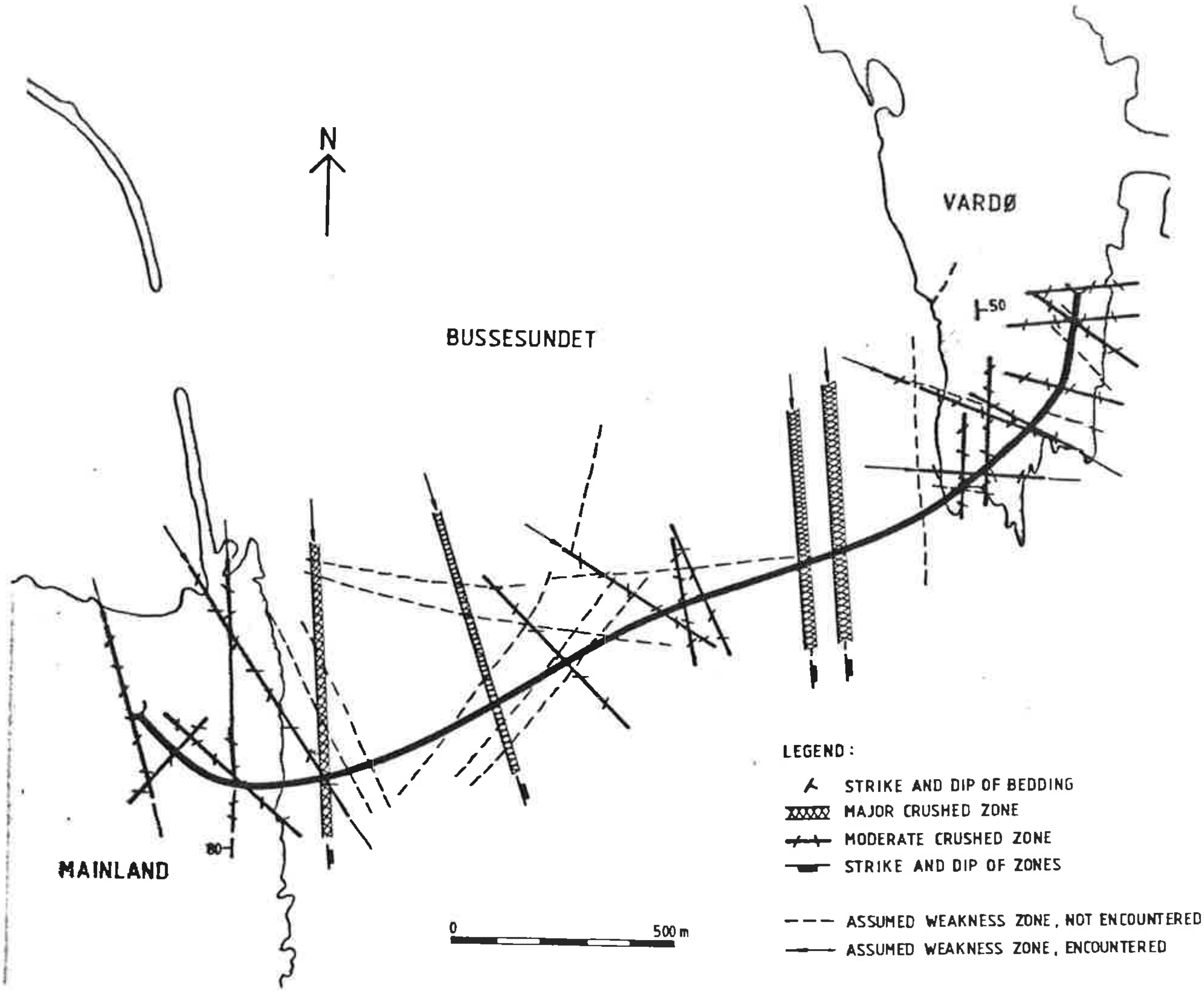

Fig. 8 Weakness zones encountered in the tunnel

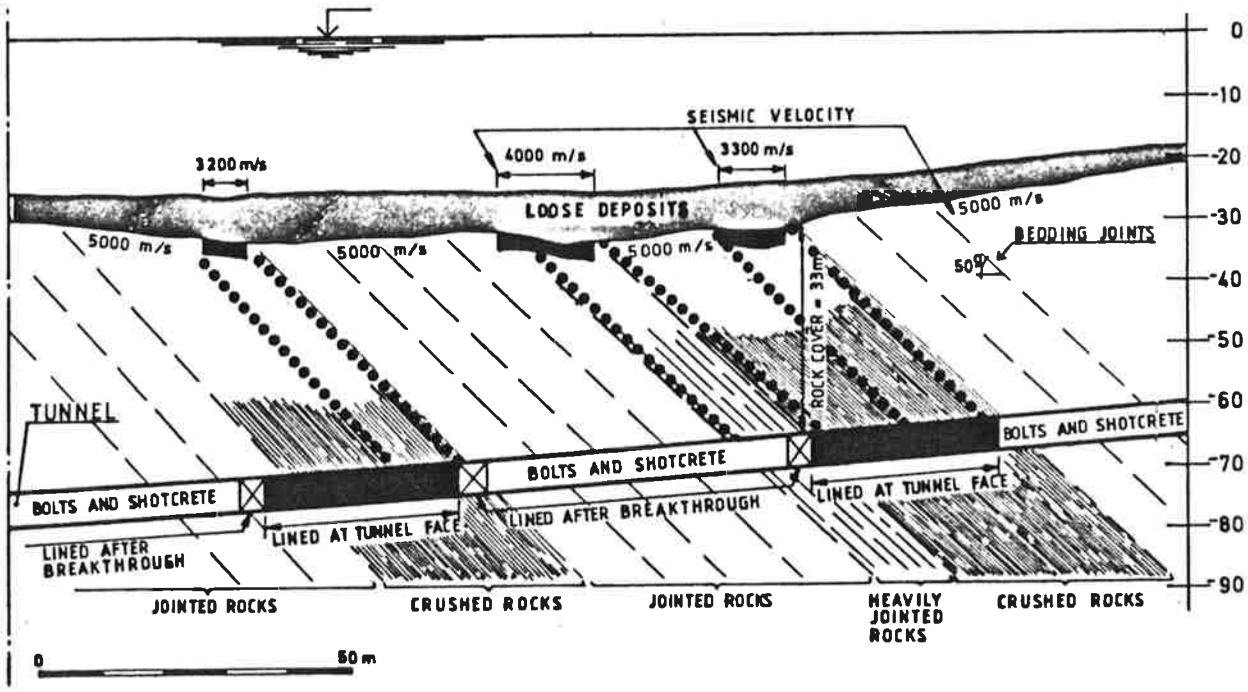

Fig. 9 Correlation between seismic velocities and rock support carried out in the tunnel

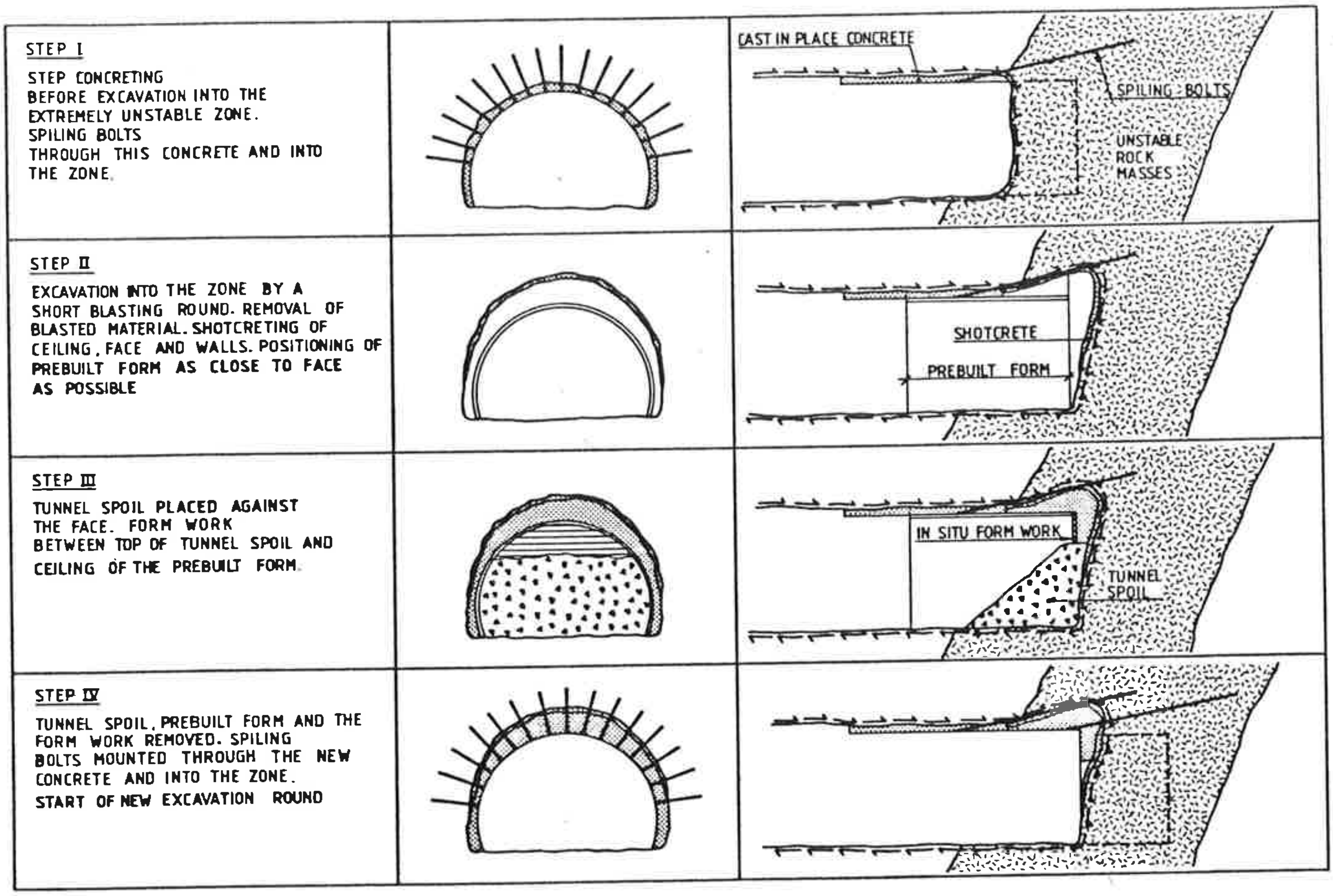

Fig. 10 Principles for excavation and rock support in extremely poor rockmass conditions

It ought to be mentioned that the rock mass quality was even poorer here than the lowest Q-factor class. In addition to the special concrete lining technique the quick use of shotcrete after blasting, spiling bolts and short blasting rounds adjusted to the rock mass conditions, were important for the safe and relatively quick advance through the zones which were 25 and 28 meters wide.

7. Sealing works

Previous tunnelling experience in Norway has shown that the sealing of water bearing zones is most successfully done by pre-grouting ahead of the tunnel face. An estimate based on the pumping costs of permanent leakage water gave as result that leakages in excess of 2.5-3 Lugeon were economically sealed by such grouting. The plan was therefore to carry out sealing works by pre-grouting when the exploratory drillings showed leakages in excess of 2.5 Lugeon. The additional 6 - 8 grouting holes were drilled (Fig. 11), and cement grouting of 25-30 bars pressure was performed.

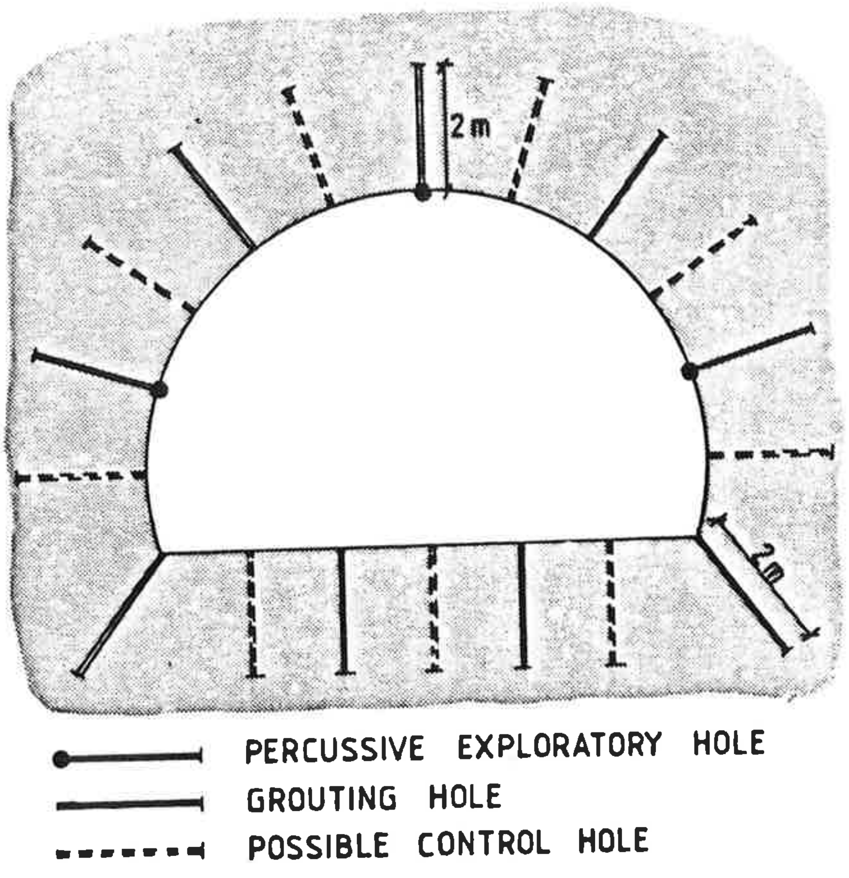

Fig. 11 Principles of exploratory holes, groutholes and control holes used for pre-grouting

Pre-grouting was used in 6 areas along the tunnel and 76 tons of cement was applied. 7% of the tunnel length was sealed in this way, which was some 2 - 3% less than estimated.

In one occasion, however, grouting problems occured when the water leakage ahead of the face was detected after the tunnel had been excavated into poor, clay-containing rock masses, a too weak for fastening of the grouting packers. The tunnel face therefore had to be strengthened with concrete before a successful pregrouting could be done.

The permanent water leakage in the entire tunnel is 1.0 m3/min., which happens to coincide with the estimate.

8. Concluding remarks

A total of 561 m of the tunnel was concrete lined, 350 m at the working place. 2,500 m3 of shotcrete was placed, most of if without reinforcement, 18,000 rock bolts (which equals a little less than 7 bolts per m tunnel), and some 9,500 m of steel straps plus 7,500 m2 of nylon nets were used. The water leakages are distributed along most of the tunnel as drips and approx. 2,000 m length had therefore to be water protected by isolated aluminium shields as described by Ref. (4) and (5).

The price per m tunnel was USD 8,000 (1982). The excavation costs amounted to USD 3,000 per m, which was close to the estimate. Salt leakage water caused some difficulties for the drilling and loading machinery and special care had to be taken to protect the electrical equipment.

Even though the rock mass tunnelling conditions were of poorer quality than normally encountered in Norway, the Vardø road tunnel has been a success, and has given valuable experience in Norwegian subsea tunnelling technique. Three other subsea tunnels totalling 12 km have later successfully been excavated, and there are plans to carry out several more in the forthcoming 10 - 15 years.

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