Quality control of a sub-sea tunnel project in complex ground conditions

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ABSTRACT: Underground projects today are often characterised by difficult ground conditions, complex contracts and environmental focus. For sub-sea tunnels in particular, the consequences of a tunnel collapse can be enormous. Quality control therefore is an essential part of such projects. Key factors for success are adequate geological investigation and good planning of the tunnel work. In this paper some general aspects of new trends in geotechnical planning and control are described, as well as the investigation and planning of the 5.3 km long and 155 m deep Fröya sub-sea tunnel between two islands in Norway. The area has been exposed to complex faulting, resulting in extreme tunnelling conditions. Special precautions, extensive investigations, and measures for quality control have therefore been taken to ensure completion of the project within time and at budget.

1 INTRODUCTION

The main background of this paper are quality control and evaluation of feasibility, risk and cost carried out by the authors on behalf of the Norwegian Road Authorities for the Fröya sub-sea road tunnel. The approximately 5.3 km long, 50 m^2 tunnel is presently under construction on the north-west coast of Norway, see Figure 1.

About 30 sub-sea rock tunnels have previously been successfully completed along the coastline of Norway. Thus, valuable input from a number of comparable projects could be benefited from in the planning of the Fröya tunnel. When the Road Authorities still wanted this project to be thoroughly evaluated, in fact by two independent panels of experts, this was based on the anticipated very difficult ground conditions of the Fröya tunnel, and the unexpected problems of several recent sub-sea road tunnel projects, such as:

- The *Bjoröy* tunnel, where a more than 10 m wide Jurassic, tensional fault zone filled with clay, sand and coal fragments, quite unexpectedly was encountered in the Precambrian bedrock. This was a zone of extremely high permeability and very poor stability, and a very time consuming procedure involving stepwise grouting, drainage, spiling and shotcrete arches was required to get through it.
- The *North Cape* tunnel, where flat laying, weak sedimentary rocks (mainly shales and sandstones) caused very poor stability, requiring comprehensive shotcreting and concrete lining at the face, reducing tunnelling progress to about 20 m/week. The difficult conditions were not realised from the pre-investigations due to the relatively high seismic velocity of the flat laying layers (5 km/s and more).

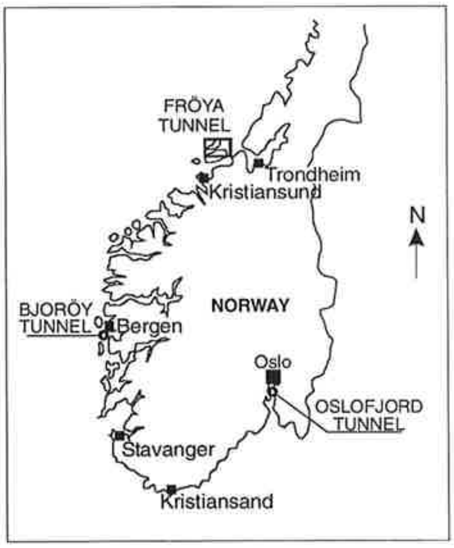

Figure 1 Location of the Fröya tunnel and two of the other sub-sea tunnels where unexpected problems have been experienced.

- The *Oslofjord* tunnel, where a deep cleft filled with Quaternary soil was encountered, necessitating ground freezing to get through. The cleft was not detected prior to tunnelling, despite very comprehensive pre-investigations including traditional refraction seismics as well as directional core drilling and seismic tomography at the actual location.

The Fröya tunnel is the second sub-sea tunnel of the Hitra-Fröya project. The 5.7 km long and 264 m deep Hitra tunnel was completed in 1994.The pre-investigations for both tunnels started in 1982, and for the Fröya tunnel more and less continuously continued until construction started in early February 1998. (Horvli, 1992; Heggstad and Nålsund, 1996). Compared to other, similar projects, very comprehensive investigations were carried out, revealing complicated and, in some cases, rather uncertain geological conditions. Thus, very challenging tunnelling conditions were anticipated, with several large, and probably difficult, weakness zones to pass through, and in addition, possibilities of encountering young, sedimentary rocks.

As a basis for final decision, and as a quality assurance of all previous investigation and planning, the Road Authorities decided that a final, independent evaluation of feasibility, cost and project risk was desirable. This is described in this paper, together with some general aspects on geotechnical planning and control.

2 NEW TRENDS IN GEOTECHNICAL PLANNING AND CONTROL

Underground projects are almost unique as the conditions and demands vary from one project to another. A high degree of complexity is also common as the projects are characterised by difficult conditions, complex contracts and environmental focus. Society also becomes more involved. History has shown that damage events originating from geological hazards often have a significant impact on the project time and cost.

The new trends in geotechnical engineering have their origin in a wish to better understand and control the complexity of underground projects.

New codes and guidelines for design, development of modern quality system, better understanding of the complexity by using risk analysis, better system to describe the uncertainties in cost and time estimation are all examples of new trends.

The basic principle for defining necessary extent of investigation and exploration should always be related to the decision to be taken which depends on the type and complexity of the project, and to the prevailing geological conditions (Sturk, 1998). This is reflected by the guidelines for geotechnical planning and design given in several countries. As illustration, some aspects of the guidelines according to the Norwegian Council for Building Standardisation (NBR) and the European Committee for Standardisation (CEN) are described in the following.

In the light of the above it is obvious that the need for quality control is large. A review of damage events indicates quite clear that a dualistic quality system is needed. (Stille et al 1998). The two problems "doing the right thing" as well as "doing things right" must both be handled by the quality system. The trend up to today, has been to develop quality system based on ISO 9001. Such system can easily handle the question "doing things right" while the question "doing the right thing" is much more difficult to cover with such system. Other quality tools like risk analysis, technical audits and team qualification can be used for the second question.

The tools used in risk analysis like for example fault tree and event tree have shown to be very valuable in order to, in a more systematic way, describe how damage can be developed from damage event, initiating event and the nature of hazard. With such tools the importance of good organisation and communication as well as clear responsibility can more easily be recognised.

2.1 Guidelines for design

Norwegian Standard NS 3480

The Norwegian Standard NS 3480 "Geotechnical planning", covering rock as well as soil, gives guidelines for geotechnical and engineering geological investigation, planning, supervision and control. A basic principle is that project owner and designer jointly, based on evaluation of so-called damage consequence class and degree of difficulty define the geotechnical project class as shown in Table 1.

Potential damage consequences to be evaluated according to NS 3480 relate to life as well as property, including long term economical consequences.

Table 1 Definition of Geotechnical Project Class according to Norwegian Standard (from NBR, 1988).

DAMAGE CONSEQUENCE CLASS	DEGREE OF DIFFICULTY		
	Low	Medium	High
Less serious	1	1	2
Serious	1	2	2
Very serious	2	2	3

Degree of difficulty is to reflect uncertainty in planning and construction, and depends mainly on:

- The in-situ engineering geological conditions.
- To what extent the ground conditions will influence on the planned project.
- Whether reliable methods exist for defining the ground conditions and the input parameters for analyses.
- Whether reliable methods exist for design of the project.
- Whether experience exists from similar projects.

Hydropower tunnels and conventional, low-traffic road tunnels in rural areas are examples of projects often belonging to geotechnical project class 1, while sub-sea tunnels and large caverns in urban areas often belong to class 3.

Geotechnical Project Class defines the efforts to be put on:

- Collection of information on ground conditions.
- Analyses and planning.
- Design supervision and control.
- Construction supervision and control.

Eurocode 7

The relevant European standard, Eurocode 7, exists as a so-called European Prestandard, or “ENV”, (CEN, 1994). After having gained experience from practical application, it will be modified if required, and, most likely, converted to European Standard (EN) in 2000. In Norway it will then replace NS 3480 (and the respective relevant national standards in other European countries).

The basic approach to geotechnical design of EC 7 in principle is the same as for NS 3480. However, while NS 3480 mainly gives the framework for geotechnical design, EC 7 gives more detailed rules. Here, three so-called Geotechnical Categories, 1, 2 and 3 (corresponding to the Geotechnical Project Classes in NS 3480) are introduced. Preliminary classification of a structure according to Geotechnical Category normally should be performed prior to the geological investigations.

The following factors, according to CEN (1994), shall be taken into consideration when determining the geotechnical design requirements:

- Nature and size of the structure and its elements, including any special requirements.
- Conditions with regard to its surroundings (neighbouring structures, traffic, utilities, vegetation, hazardous chemicals, etc.).
- Ground conditions, ground water situation, and regional seismicity.
- Influence on the environment (hydrology, surface water, subsidence, seasonal changes of moisture).

The various design aspects may require treatment in different geotechnical categories. It is not necessary to treat the whole of the project according to the highest of these categories.

Supervision of design

In NS 3480 and the Norwegian application document for EC 7 (NBR, 1977), the following guidelines for supervision of the design, depending on geotechnical category, are defined:

- For category 1 the supervision can be carried out by the person who has carried out the design (“simple supervision”).
- For category 2, the supervision shall be carried out by a person who is appropriately qualified and experienced and who has not taken part in the design (“normal supervision”).
- For category 3, it is recommended that in addition to normal supervision an extra supervision is carried out by a person or organisation who is independent of the geotechnical designer (“extended supervision”).

This is also in accordance with the principles of Eurocode 7, although supervision of design is not expressly mentioned here. It is also interesting to notice that the ”extended supervision” can be a part of the technical audit in order to check that ”the right things are done”.

2.2 Risk analysis

The aim of a risk analysis is to evaluate potential damage and factors that may lead to damage.
A risk analysis may be divided into the following steps:

1. Identification of hazards and damage events.
2. Assessment of probabilities for hazards and damage events identified.
3. Description and valuation of consequences including analysis of initiating events.
4. Calculation of risk.

Within a project, risk analysis and quality control/assurance should be focused upon:

- Identifying and elimination or reducing hazards.
- Reducing the probability of getting initiating events.
- Finding barriers to stop damage events.
- Reducing the consequence of possible damage events.

All events included in the process where hazards evolve into damage are associated with uncertainties. The uncertainties are not only related to the soil or rock, but also with people involved in a project, and the relation between those people. Consequently, one major task within risk analysis is to understand, describe and handle uncertainties.

2.3 *Evaluation of uncertainty in time and cost estimation*

Estimation of cost and time is an important part of any underground project. All uncertainties involved in such projects will contribute to uncertainties in the estimation of cost and time. One way to evaluate the uncertainties in the estimation is to describe the estimation as a stochastic process. Several solutions of this problem exist, see Einstein et al. (1987) or Isaksson (1998). All are relatively complex.

A commonly used alternative for quick and simplified uncertainty analysis, is the so-called Lichtenberg's method (Lichtenberg, 1978), see Figure 2.

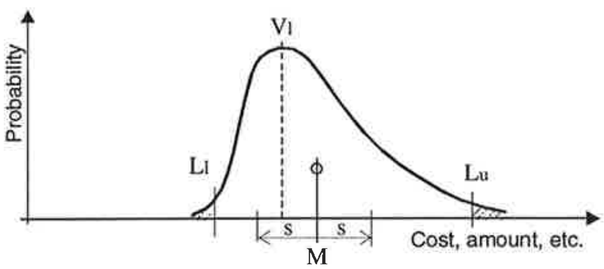

Figure 2 Parameters applied in the Lichtenberg uncertainty analysis.

In brief, the basic principle of the method is that for each factor (an amount or a price) a lower limit (L_l, to be underpassed only by 1 %), an upper limit (L_u, to be exceeded by only 1 %) and the most likely value (V_l, representing the best estimate) are estimated. Based on this, the average value (M) and standard deviation (s) are calculated as follows:

$$M = 1/5(L_l + 3V_l + L_u)$$
$$s = 1/5(L_u - L_l)$$

The Lichtenberg method is pseudo-statistical, and is valid strictly only if the factors are mutually independent and can be described by an Erlang distribution. Applying the method for establishing the cost contributions of a tunnel project, therefore, is in general a gross simplification. Still, the method may give a good indication on distribution of uncertainty, and on what are the main uncertainties. However, the very base is the variables describing the capacity or cost per basic unit. A well established database for tunnelling capacities and costs is the key to every type of estimation. An example of the method is described in Section 3.4.

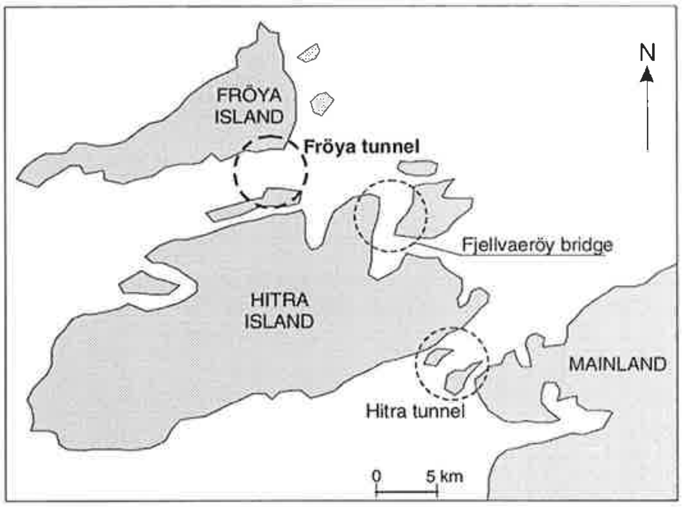

Figure 3 The Fröya tunnel connecting the two islands, Hitra and Fröya

3 DESCRIPTION OF THE FRÖYA PROJECT

The Fröya tunnel is 5.3 km long with its deepest point 155 m below sea level. It has a major part (3.6 km) below the sea, where the rock overburden varies between 37 m and 155 m. The two-lane tunnel has cross sectional area of 50 m² (T8 tunnel profile). The maximum gradient is 8 %. A reservoir of 1150 m³ will be excavated in the lowest point, large enough to store 4 days of leakage water (if the supply of electricity fails). The tunnel cost is estimated at 55 mill. USD (exchange rate 1 USD = 7.7 NOK) which equals 10,400 USD/m tunnel. The tunnelling works started in February 1998, with a planned hole-through in August 2000, and opening of the tunnel for traffic in June 2001.

3.1 Geology

The metamorphic rocks in the area are of Precambrian age with gradual transitions between various gneisses, such as granitic gneiss, micagneiss, and migmatite. A few bands or layers of limestone/marble have been observed in the actual area. The strike of the rocks is mainly ENE-WSW with steep dip towards NW.

The area is located close to the main continental faults, and the sedimentary rocks of the continental shelf are only 20 - 30 km off Fröya. One important task was therefore to find a local sedimentary basin could occur on the sea bottom along the tunnel.

The area has been exposed to major faulting in Precambrian as well as the Caledonian and the Alpine orogenesis. Several depressions and valleys representing faults and thrusts can bee seen in the topography. Similarly, also the map of the sea bottom showed a topography with marked depressions indicating the presence of fault or other weakness zones. The refraction seismic measurements confirmed this.

A main geological feature is the Tarva fault (see Figure 4) which can be followed more than 150 km towards NW on the Norwegian mainland. This probably old fault is assumed reactivated during the Jurassic/Cretaceous, maybe also in the Tertiary time.

3.2 Field investigations

The field investigations for the project started in 1982 with construction of maps, collection of available geological material, and the initial seismic measurements, consisting of shallow reflection seismic (acoustic) measurements and the first refraction seismic profiles. In 1992, the tunnel alignment was chosen, for which cost estimate and detailed design was performed.

In 1995, during the final design, core drillings were performed on both sides of the Fröyfjord. Unexpected, exceptionally poor ground conditions were then discovered in the northern side of the fjord. It was found that a more than 30 m thick zone consisting of silt, sand and gravel material, and with direct connection to the sea bottom, would be very difficult to pass with a 50 m² tunnel. Therefore, the

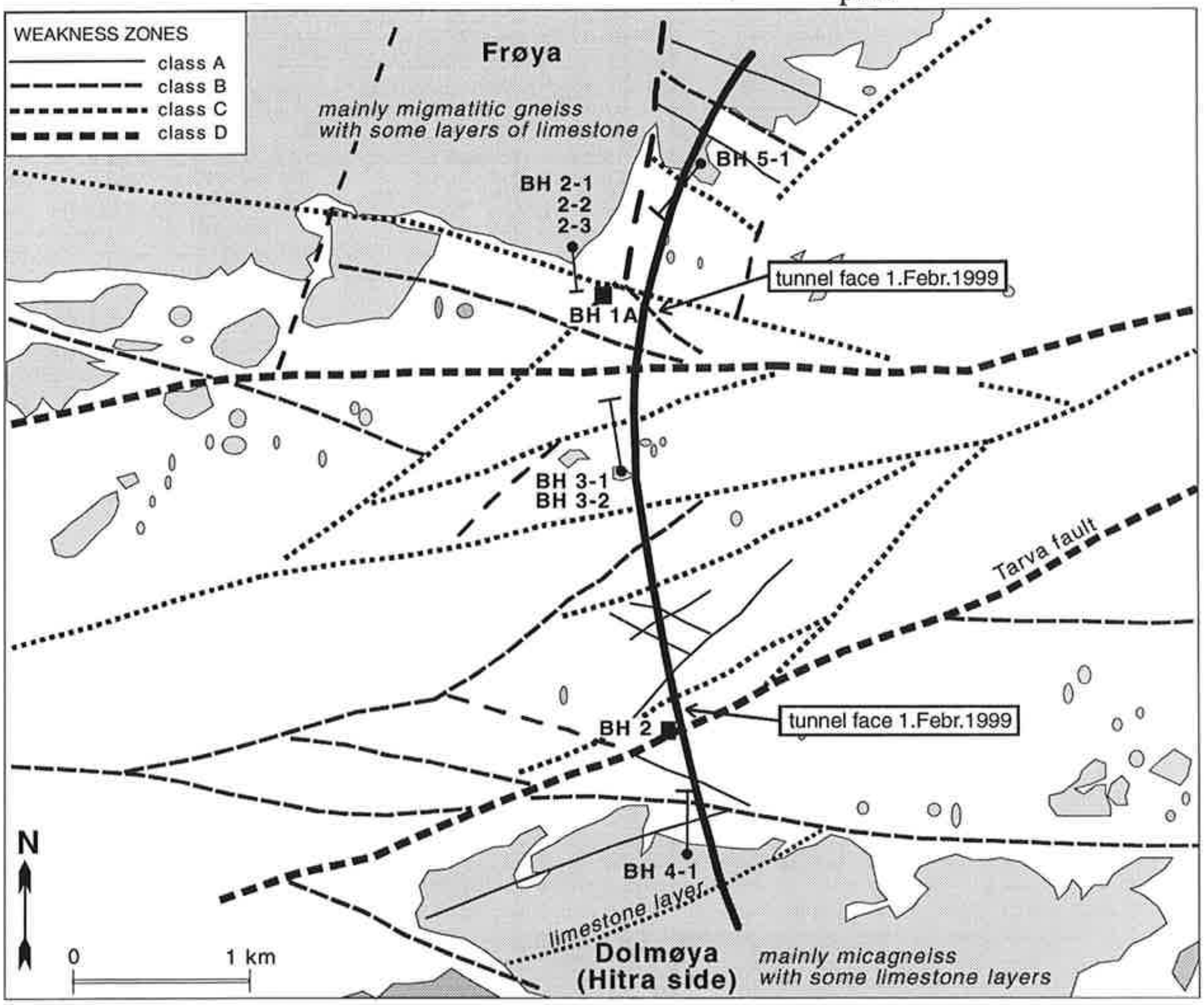

Figure 4 Assumed main weakness zones in the tunnel area, as interpreted from geological maps, aerial photos and field investigations.

Tunnel alignment was adjusted to the East in this part, where also the following, additional field investigations were performed:

- Refraction seismic profiles along the tunnel alignment with several cross profiles
- Inclined core drillings both from land, from small islets in the Fröyfjord. Many of these had great drilling problems caused by the difficult ground conditions.
- In addition, two holes in the fjord were performed from a drill ship.
- Special studies of the tectonic setting in the region.
- Detailed core logging and laboratory testing

New cost calculations were performed based on the additional information of the geology and ground conditions by two groups of engineering geological experts, as described in the next section. A risk/uncertainty analysis was also made before final decision to construct the tunnel was taken in 1997.

3.3 Feasibility, risk and cost evaluations

The refraction seismic measurements have shown more of low velocity (weakness) zones than for any of the other sub-sea tunnels constructed in Norway. In addition, the core drillings have penetrated long sections of rocks with weakness zones having a higher degree of alteration than is normal in Norwegian hard-rocks. Thus, the material in many zones consists of soil-like materials (clay, silt, sand and gravel). Often, the clay material shows high degree of swelling with low strength and friction properties.

The results of the investigations proved that the Fröya tunnel would require thorough evaluations of all investigations to assess its feasibility, and that special routines and control should be implemented during planning and construction.

As a part of this, two groups were established to evaluate the feasibility of the tunnel. In their two independent reports excavation methods and rock support were analysed supplied by a cost estimate and risk assessment, (Nålsund et al., 1996; Nilsen et al., 1997). Both reports concluded that the tunnel could be constructed within economical limits using the drill and blast method for excavation, provided thorough quality control in planning and construction.

In the report prepared by the authors of this paper, the construction time and cost estimates were based on a detailed prognosis of the expected ground conditions. For this, the ground was divided into 8 different classes; 4 classes for the expected ground quality between weakness zones, and 4 classes for the main types of weakness zones. For each class the appropriate types and amount of rock support were given. In addition, the leakage conditions with the possible amount of grouting works were assumed along the tunnel. This prognosis has been used in the follow-up of the construction time and cost, as described in Section 4.2.

3.4 Evaluations of risk and uncertainty

For the Fröya tunnel, due to the limited time available, the approach described in Section 2.3 was applied. The calculations were based on estimated extents and costs of the various rock mass classes and works ahead of the tunnel face. As shown in Table 2, the lengths of the most difficult rock mass classes (C and D) and the extent of grouting are the most uncertain factors (highest variance). The standard deviation in Table 2 corresponds to a coefficient of variation of 10 %, corresponding to what is regarded as a low uncertainty in "conventional cost estimation" based on anticipated geological complexity and extent of probe drilling, grouting and rock support.

4 RESULTS FROM TUNNELLING

As of present (February 1999), 60% of the tunnel has been excavated, and many of the expected difficult parts have been encountered and passed through. Only 2.1 km in the middle remains. The construction is 4 months ahead of the schedule.

Major uncertainties and risks have been, and are, connected to water leakage and unstable, collapsing ground. As a part of the quality control an extensive program for probe drilling and follow-up of the tunnel works have been implemented. For every 20 m tunnel excavated, 3 - 5 exploratory drill holes are being made ahead of the working face to gain information on the ground conditions. In this way, necessary measures can be made in time before tunnelling into the difficult ground.

Table 2 Uncertainty analysis (“max/min-estimation”) based on Lichtenberg’s method.

CLASS		UNIT	Ll	Vl	Lu	M	M (NOK)	s	s (NOK)	s^2 (NOK)
GROUND QUALITY	1	m	700	1235	2000	1 281		260	2 162 472	4,67629E+12
		NOK/m	7000	8362	9500	8 317	10 654 333	500	640 500	4,1024E+11
	2	lm	1200	1720	2500	1 772		260	2 611 492	6,81989E+12
		NOK/m	8400	10107	11500	10 044	17 798 322	620	1 098 640	1,20701E+12
	3	m	600	1060	1700	1 096		220	2 913 416	8,48799E+12
		NOK/m	11000	13338	15200	13 243	14 514 109	840	920 640	8,47578E+11
	4	m	50	330	500	308		90	1 658 430	2,75039E+12
		NOK/m	14500	18545	22000	18 427	5 675 516	1 500	462 000	2,13444E+11
	A	m	50	125	300	145		50	868 110	7,53615E+11
		NOK/m	13500	17437	21000	17 362	2 517 519	1 500	217 500	47306250000
	B	m	150	240	400	254		50	2 443 710	5,97172E+12
		NOK/m	39000	48457	60000	48 874	12 414 047	4 200	1 066 800	1,13806E+12
	C	m	150	380	600	378		90	6 749 190	4,55516E+13
		NOK/m	56000	73985	97000	74 991	28 346 598	8 200	3 099 600	9,60752E+12
	D	m	10	140	280	142		54	8 862 869	7,85504E+13
		NOK/m	135000	155212	220000	164 127	23 306 062	17 000	2 414 000	5,8274E+12
FREEZING		m	0	0	50	10		10	3 100 000	9,61E+12
		NOK/m	250000	300000	400000	310 000	3 100 000	30 000	300 000	90000000000
PROBE DRILLING	I	m	3500	4310	4500	4 186		200	32 400	1049760000
		NOK/m	150	160	180	162	678 132	6	25 116	630813456
	II	m	250	320	1000	442		150	41 430	1716444900
		NOK/m	240	267	340	276	122 080	20	8 840	78145600
	III	m	300	600	1100	640		160	208 032	43277313024
		NOK/m	1000	1167	2000	1 300	832 128	200	128 000	16384000000
PRE-GROUTING	i	m	400	1195	3200	1 437		560	4 095 168	1,67704E+13
		NOK/m	5000	7188	10000	7 313	10 508 494	1 000	1 437 000	2,06497E+12
	ii	m	300	745	1500	807		240	5 140 464	2,64244E+13
		NOK/m	15000	19031	35000	21 419	17 284 810	4 000	3 228 000	1,042E+13
	iii	m	10	70	200	84		38	5 283 528	2,79157E+13
		NOK/m	60000	145067	200000	139 040	11 679 377	28 000	2 352 000	5,5319E+12
						SUM =	159 431 528		$(\Sigma S^2)^{0.5}$ =	16 484 868

Some of the measures in difficult ground are:

1) To perform pre-grouting, i.e. grouting in 10 - 20 m long holes drilled ahead of the tunnel using high pressure injection, see Figure 5.
2) To stabilise the ground over and on both sides of the next round by 6 m long spiling bolts spaced 0.2 - 0.5 m every 3 m
3) To use short blast rounds and spraying of fibrecrete on roof, walls and face shortly after blasting.
4) To use stepwise excavation and concrete lining in addition to 3) where stability is very poor.
5) Availability of equipment to quickly and fully concrete the tunnel face, in case of dangerous situations, such as cave-in, progressive sliding, etc.
6) High pumping capacity and modern equipment for rock support operating at short notice.

As a part of the control system, a reference group of 7 experts was established, consisting of 5 from the Road Authority, 1 external, and 1 from the site management. The group has regular site with visits and meetings with the site management every second month during the tunnelling period. Their task is to act as a discussion partner regarding safety measures, evaluation of ground stability, rock support assessment, etc.

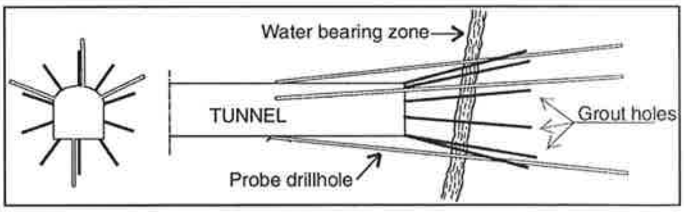

Figure 5 Principles of the basic probe drilling system. In addition, where difficult ground is expected and additional information required, core drilling is carried out.

An example on the works performed for tunnelling through one of the weakness zones (see Figure 6) is described in the next section.

4.1 Example from the tunnelling works in the weakness zone at chainage 3975 - 4025

The refraction seismic measurements showed 2 low velocity zones, 3.2 km/s and 2.9 km/s in this area, but experience and engineering geological mapping indicated that it probably was only one large weakness zone here.

Probing by core drilling performed from a recess in the tunnel showed that the zone consisted of altered gneiss containing clay seams with thickness 5 cm to 75 cm. The poorest stability was at the end of the zone. Here, just before the sharp boundary to the adjacent rock, the drilling had 2 m core loss.

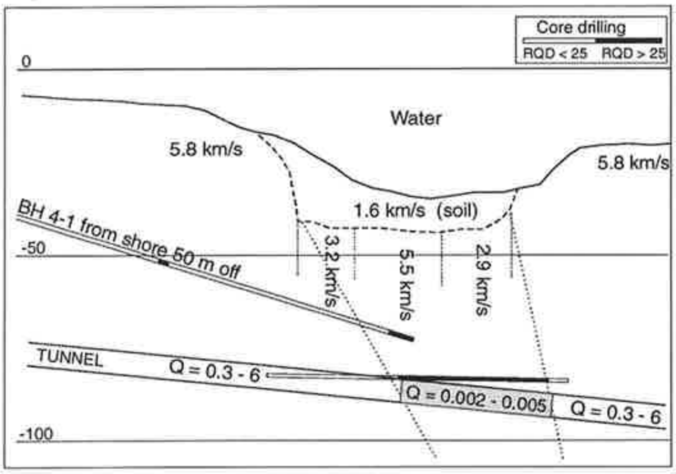

Figure 6 Details from the zone at chainage 3975 - 4025

Caused by a small leakage in a few of the 30 m long probe holes, a 30 m long grouting sequence was performed using 20 holes. The grout take was 14,500 kg cement and 22,200 kg microcement. The grouting successfully stopped the leakage, in addition it also resulted in increased stability. The following steps were implemented in tunnelling through the zone:

- reduced excavation round, only 3 m (instead of 5 m);
- 6 m long fully grouted spiling bolts with 0.25 - 0.4 m spacing (36 - 64 bolts). Steel straps and shotcrete are used to fix the outer end of the bolts to the rock;
- 1 - 2 layers of fibre reinforced shotcrete (fibrecrete) 6 - 12 cm thick in roof and walls, immediately after blasting;
- 4 m long bolts, in average spaced 1.5 m; and
- additional 2 - 3 layers of fibrecrete, total shotcrete thickness 19 - 31 cm.

At the end of the zone a 5 m long section was concrete lined as it was impossible to install rock bolts here. In the poorest ground quality the excavation was carried out using the excavator.

Later, the floor along the zone was concreted over a 35.5 m long section.

Some time after the zone had been passed through, convergence measurements started. The results presented in Figure 7 show that the movements have ceased.

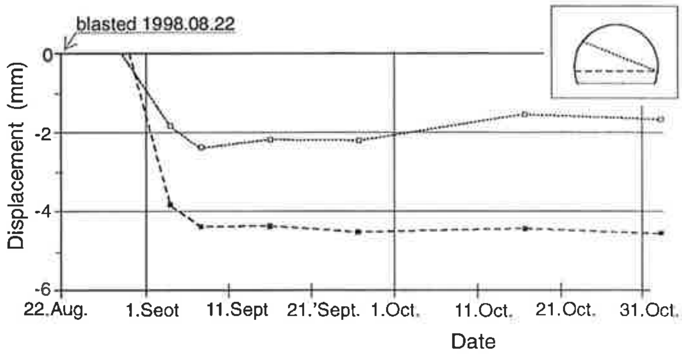

Figure 7 Convergence measurement at chainage 3992

4.2 Comparison of prognosis and encountered conditions

The detailed prognosis of the expected ground conditions, rock support, and construction cost have been used to compare the real ground conditions encountered with the assumed, as presented in Figure 8. As seen there is a very good accordance between the two. This is also the case for the northern (Fröya) part of the tunnel.

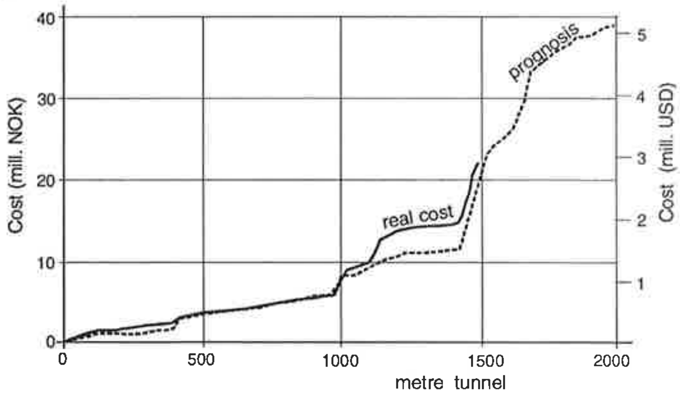

Figure 8 Comparison between assumed and real cost for the rock support and grouting works at the Hitra side.

5 SOME RECOMMENDATIONS FOR FUTURE PROJECTS

Undoubtedly, the quality of site characterisation, engineering geological reports and tunnel contracts can often be improved. Based on review of the Fröya tunnel and other projects in complex rock conditions, the following lessons of general relevancy for the planning of future projects are particularly worth mentioning:

- The extent of ground investigation and planning should always reflect the complexity of the geology and the type and of the project. The results from the investigations should be properly documented and their use in calculations and assessments shown.
- The geological setting, including understanding of the tectonic, is vital for all large tunnel projects.
- Ground investigations where the extent is based on bidding, may cause vital information to be lost, and should never be accepted. Sufficient time must be allocated to do all necessary investigations and testing.
- The ground investigations should not stop when the tender documents are completed, but continue through the entire construction period. Tunnel mapping and following up should be done by experienced engineering geologists representing owner as well as contractor.
- Risk analysis and assessment of uncertainties are important.
- The tender documents, including geological reports, should be thoroughly prepared, with quality control of all descriptions and quantities.
- To ensure proper review and satisfactory quality control of complex projects, an independent reference committee should be established at the earliest convenience.
- For the construction period, strict requirements should be put both on the engineer's and contractor's competency, qualifications and routines.

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