Paper presented at Bergmekanikdagen 2000, the Swedish national conference on rock mechanic, pp. 19 - 30

THE FRÖYA TUNNEL – A SUB-SEA ROAD TUNNEL IN COMPLEX GROUND CONDITIONS

Fröya-tunnelen – en undersjöisk vegtunnel i komplekse grunnforhold

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1 INTRODUCTION

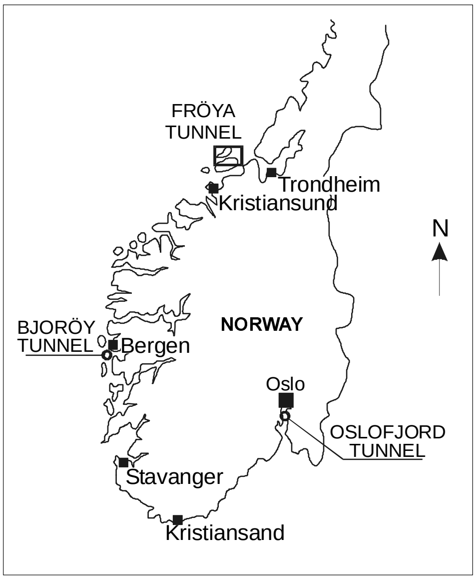

Figure 1 Location of the Fröya tunnel

The Fröya road tunnel is the second sub-sea tunnel of the Hitra - Fröya project, see Figures 1 and 2. After the 5.7 km long and 264 m deep Hitra tunnel was completed in 1994, the construction of the Fröya tunnel started in early February 1998.

The tunnel is 5.3 km long with its deepest point 155 m below sea level. 68 % of the length is below the sea, where the rock overburden varies between 37 m and 155 m. The two-lane tunnel has cross sectional area of 50 m^2 (T8 tunnel profile), and a maximum gradient of 8 %. A reservoir of 1150 m^3 in the lowest point is large enough to store 4 days of leakage water (if the supply of electricity fails).

When construction of the Fröya tunnel started, about 30 sub-sea rock tunnels had been successfully completed along the coastline of Norway. However, in the three following projects, large unexpected problems had occurred:

- 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. A very time-consuming procedure, involving stepwise grouting, drainage, spiling and shotcrete arches, was necessary to tunnel through it.

- The *North Cape* tunnel, where flat layered, weak sedimentary rocks (mainly shales and sandstones) unexpectedly caused very poor stability. Extensive shotcreting and concrete lining close to the tunnel face had to be installed, which reduced tunnelling progress to less than 20 m/week. The difficult conditions were not realised from the pre-investigations due to the relatively high seismic velocity of the rocks (5 km/s and more).

- 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.

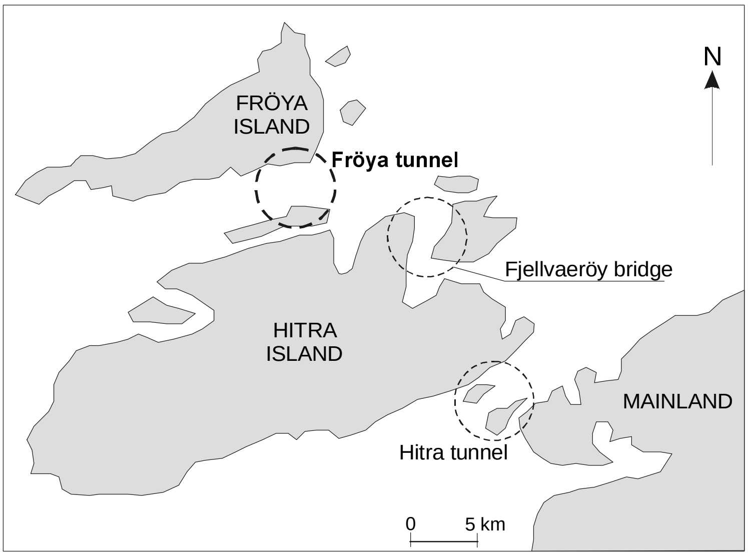

Figure 2 The Fröya tunnel of the Hitra – Fröya road project.

Due to the very difficult ground conditions of the Fröya tunnel, the Road Authorities wanted it to be thoroughly evaluated by two independent expert panels. In their two independent reports excavation methods and rock support were analysed and supplemented 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.

2 GROUND CONDITIONS

2.1 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 if 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 very old fault is assumed to have been reactivated during the Jurassic/Cretaceous, maybe also in the Tertiary time.

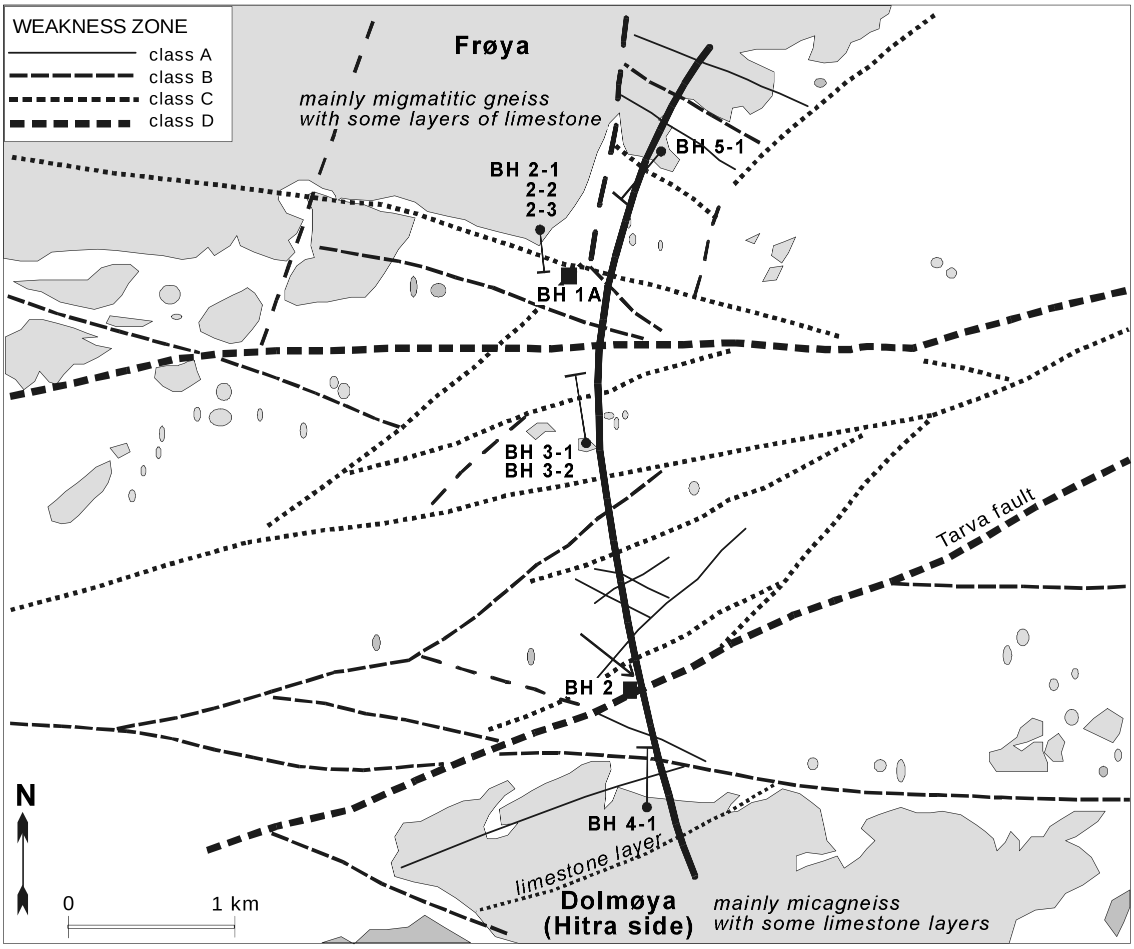

Figure 3 Assumed main weakness zones in the tunnel area, as interpreted from geological maps, aerial photos and field investigations. The zones were classified according to their estimated stability.

2.2 Field investigations

The field investigations for the project started in 1982 with initial 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 Fröy-fjord. 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 represent very difficult ground conditions. Therefore, part of the tunnel alignment was moved to the east in this area, 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.

2.3 Feasibility, risk and cost evaluations

The refraction seismic measurements show a larger portion 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.

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 actively used in the follow-up of the construction time and cost, as described in Section 3.2.

2.4 Evaluation of uncertainty in time and cost estimation

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.

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.

A commonly used alternative for quick and simplified uncertainty analysis, is the so-called Lichtenberg's method (Lichtenberg, 1978), see Figure 4.

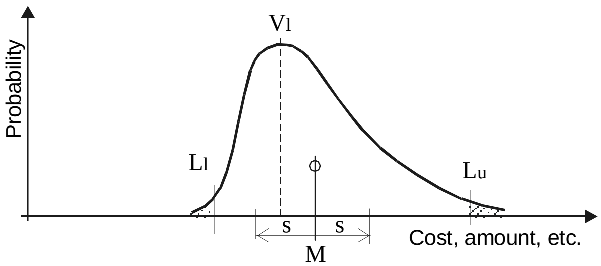

Figure 4 Parameters applied in the Lichtenberg uncertainty analysis.

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 reliable estimation.

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 1, 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 1 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.

Table 1 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

3 TUNNELLING EXPERIENCE

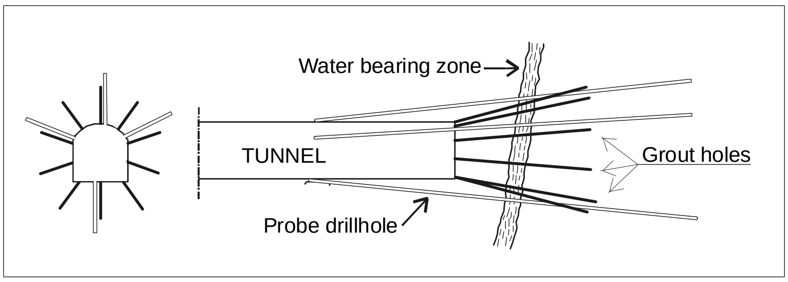

Figure 5 Principles of the basic probe drilling system. Where difficult ground was expected also core drilling was carried out

As the major uncertainties and risks were assumed connected to water leakage and unstable, collapsing ground, an extensive program for probe drilling and follow-up of the tunnel works were implemented. For every 20 m tunnel excavated, 3 to 5 exploratory holes were drilled ahead of the working face to gain information on the ground conditions. In this way, precautions could be taken in time before tunnelling into the difficult ground.

Some of the measures in difficult ground were:
1) To perform pre-grouting, i.e. grouting in 10 to 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 (fully grouted rebar) bolts spaced 0.2 to 0.5 m every 3 m, see Figure 6
3) To use short blast rounds and spraying of fibrecrete on roof, walls and face shortly after blasting.
4) Stepwise excavation and concrete lining in addition to 3) where stability was 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.

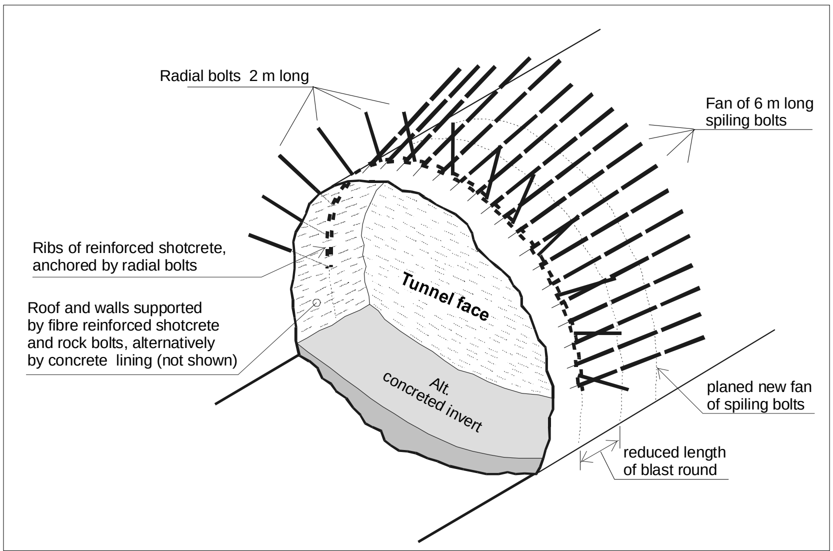

Figure 6 Principles of excavation through many stability weakness zones applying the spiling technique (revised from Nålsund et al., 1996)

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 had regular site meetings with visits every second month during the tunnelling period. Their task was to act as a discussion partner regarding safety measures, evaluation of ground stability, rock support assessment, etc.

The tunnel progress was carefully followed-up by engineering geologists and the ground features mapped after every blast round before the rock surface was covered by shotcrete. A characteristic ground feature is the occurrence of swelling clay in many of the seams and weakness zones. Special attention was paid to the composition and structure of the weakness zones, which sometimes were many tens of metres wide as result of crossing of two or more zones.

3.1 Example. Tunnelling through the weakness zone at chainage 3975 - 4025

In this area, the refraction seismic measurements showed 2 low velocity zones of 3.2 km/s and 2.9 km/s (see Figure 7), but experience and engineering geological mapping indicated that it probably was only one large weakness zone here.

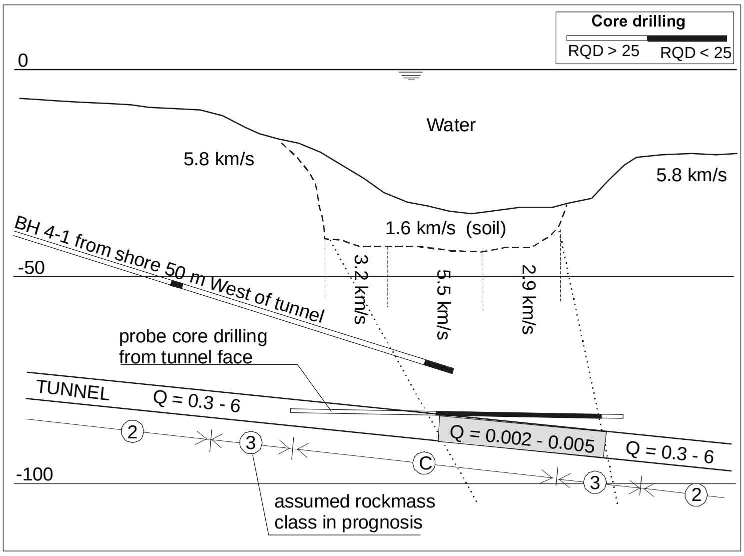

Figure 7 Details from the zone at chainage 3975 - 4025

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.

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 (see Figure 6):

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

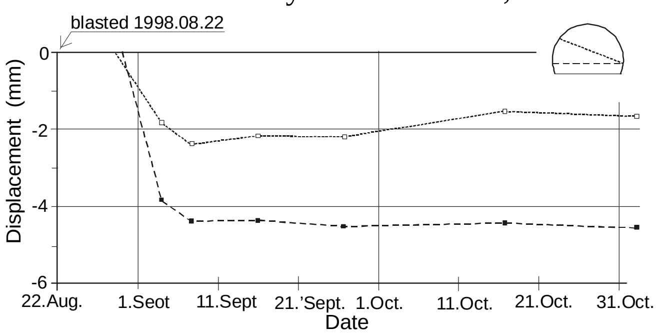

Figure 8 Convergence measurement at chainage 3992

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.

After the zone had been passed through, convergence measurements were started, see Figure 8. As shown the movements have ceased relatively quickly.

3.2 Comparison of prognosis and encountered conditions

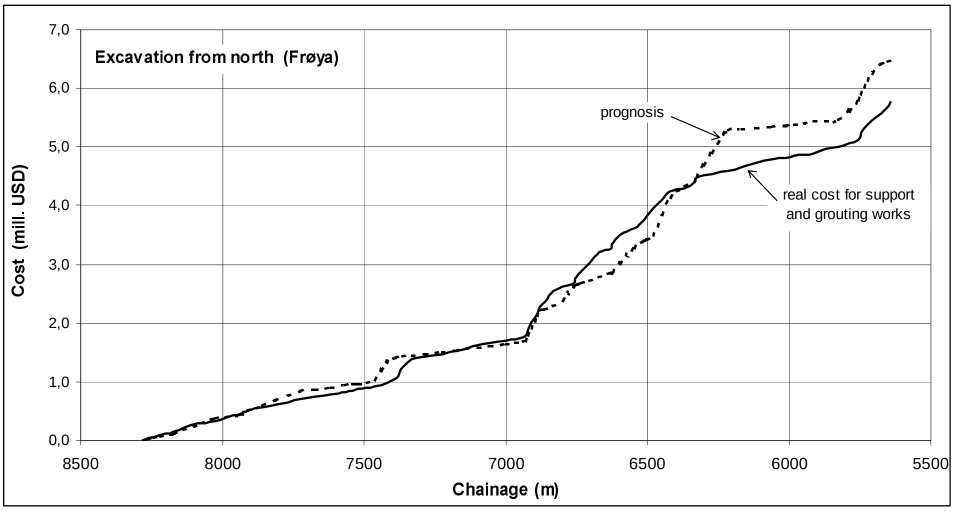

Figure 9 Comparison between assumed and real cost for the rock support and grouting works at the Fröya drive.

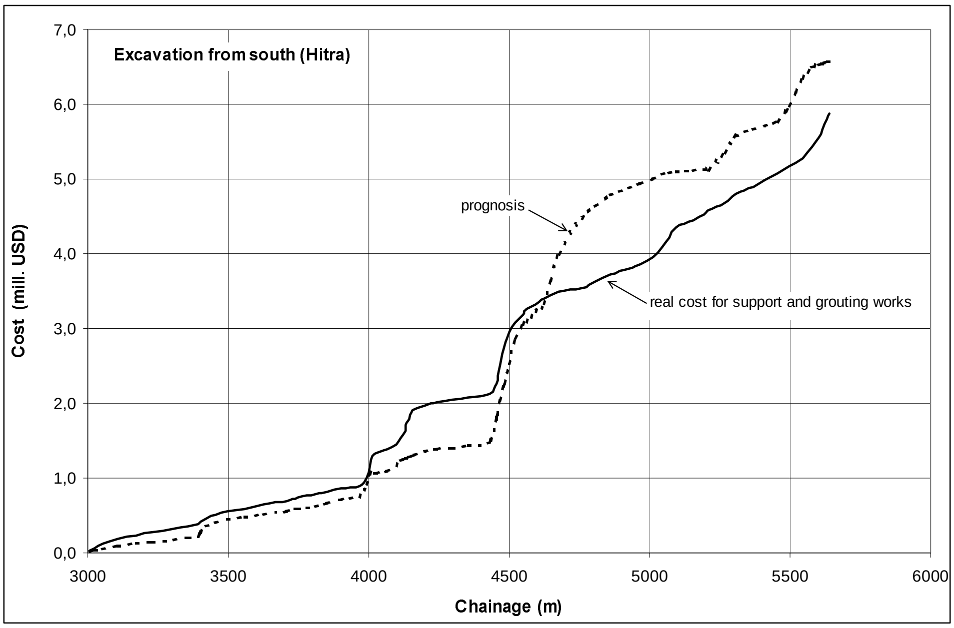

Figure 9 Comparison between assumed and real cost for the rock support and grouting works at the Hitra drive.

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 Figures 8 and 9. As seen there is a good accordance between the two for most of the drive from Hitra and 2/3 of the drive from Fröya.

The tunnelling works started in February 1998, and hole-through was in September 1999. This was about one year earlier than planned, mainly as result of:

- The amount of water sealing by pre-grouting was significantly lower than assumed, especially in the middle of the fjord.
- The official capacities for rock supporting works used in the prognosis are much lower than the real.
- The experienced contractor (Selmer asa) worked very efficiently with an enthusiastic crew of skilled tunnellers.

Average tunnelling progress was 39 and 35m/week on the south respective north drive. The average rock support was:

- 2.84 m^3/m tunnel shotcrete, mainly fibre reinforced. This is far more than in any other Norwegian sub-sea tunnel;
- 4.97 bolts/m tunnel. Also this is above average for other sub-sea tunnels;
- 270 m with concrete lining;
- 375 m of the invert is concreted;
- approx. 900 m of the tunnel was pregrouted in 50 grouting rounds. Consumption: 1000 tons of cement of which a large part was microcement.

The total cost for the tunnel is 52 mill. USD, including planning, field investigations, follow-up etc. This equals to 9,800 USD/m tunnel. The cost for rock support and sealing (pregrouting) works is 2,200 USD/m. The cost prognosis by expert panel Nilsen et al., (1997) was 2,460 USD/m, which is 11.9 % above the real. In Table 1 their calculated uncertainty is 10,5 %. Most of the "cost savings" are due to a lower amount of pregrouting than assumed and that the zones encountered could be treated easier by fibre reinforced shotcrete with the new alkali-free additive, which resulted in quicker hardening and that thicker layers could be applied.

The tunnel will be opened for traffic in June 2000, 11 months before planned.

4 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 tectonics, 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.

Acknowledgement

The authors want to thank the Norwegian Road Authorities, division Sör-Tröndelag for all help and permission to publish the paper, especially Svein Soknes, Jon Einar Lien and Arnstein Mæhlum.

5 LITERATURE

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ABSTRACT: The 5.3 km long and 155 m deep Fröya sub-sea road tunnel between two islands in Norway is located close to the continental rift. The area has been exposed to complex faulting. Compared to other, similar projects, comprehensive investigations were carried out for this tunnel, revealing complicated and - in some cases - rather uncertain geological conditions with several large, and probably difficult weakness zones to pass through. In addition, there were possibilities of encountering young, sedimentary rocks. Therefore, special measures were taken including quality control to ensure successful completion of the project. Detailed prognoses for cost and construction time were worked out, making use of the vast information collected during the field investigations. The prognosis showed good agreement with the real conditions encountered during tunnel excavation: many long sections of altered clay-rich rocks in connection with faults. The amount of rock support was larger than in the other sub-sea road tunnels in Norway. Due to efficient tunnelling works, however, the hole through was almost one year before planned.

SAMMENDRAG:
Den 5,3 km lange og 155 m dype Fröya tunnelen forbinder öyene Hitra og Fröya. Da området ligger nær kontinental-forkastningene, har det vært utsatt for kompleks forkastnings-tektonikk. Sammenlignet med andre undersjöiske tunneler er det utfört betydelig mer forundersökelser for Fröya tunnelen. Disse viste at det er komplekse og stedvis usikre geologiske forhold og at tunnelen ville krysse mange sannsynligvis vanskelige svakhetssoner. I tillegg var det mulighet for at den kunne påtreffe unge, löse sedimentære bergarter. På bakgrunn av dette ble det iverksatt spesielle tiltak, blant annet kvalitetskontroll og spesiell vurdering av gjennomförbarhet. Det ble også utarbeidet en detaljert prognose for inndrift og kostnader. Disse viste god overenstemmelse med hva som ble påtruffet under drivingen, nemlig lange partier med leirinfisert, omvandlet berg som krevet omfattende bruk av sikring; större enn i andre undersjöiske, norske vegtunneler. Takket være effektiv drift og bedre kapasiteter på sikringsarbeidene enn de "offisielle", hadde tunnelen gjennomslag nesten et år för planlagt.