Introduction

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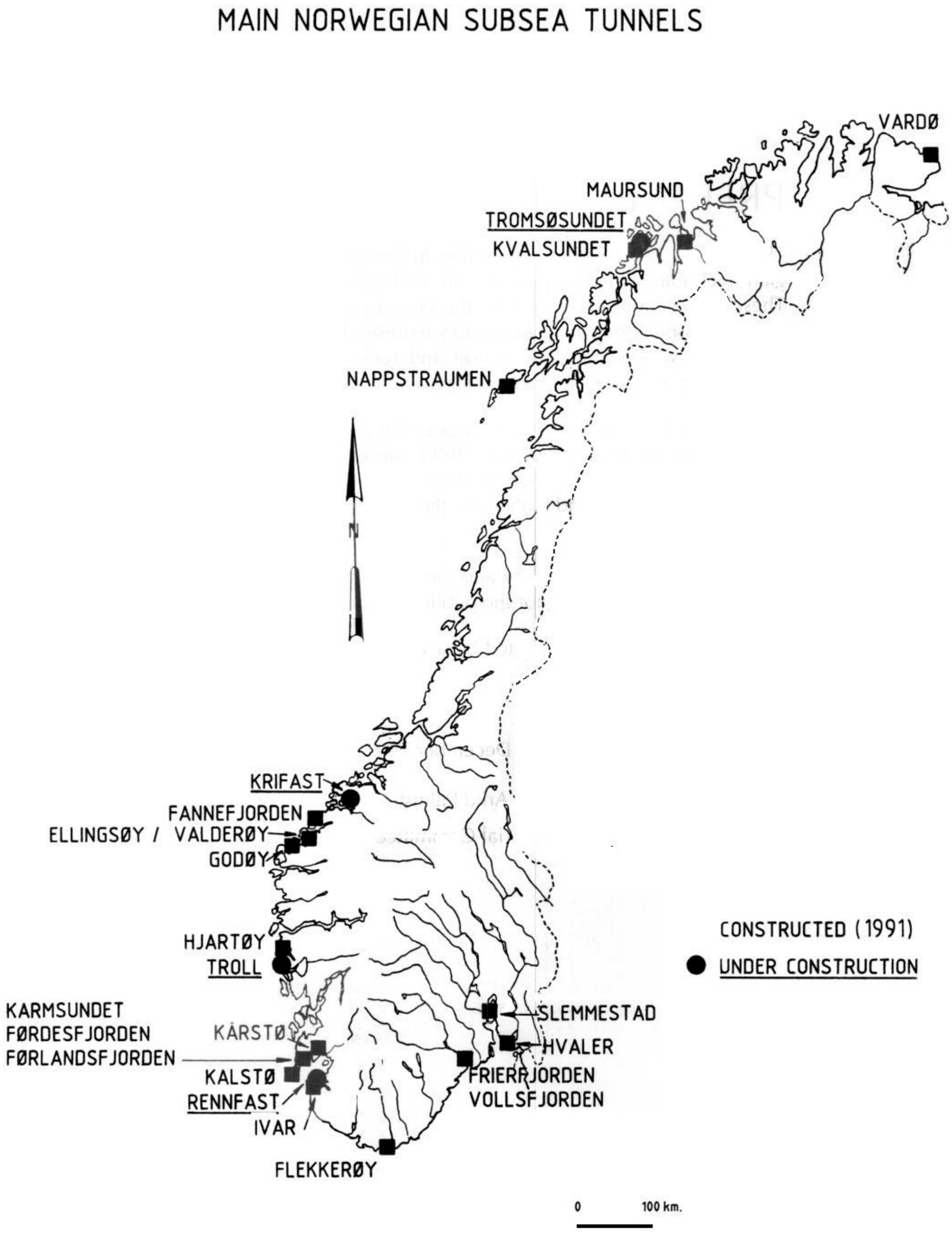


The Challenge of Subsea Tunnelling

Subsea tunnels are those tunnels which pass beneath the sea or a lake bottom; where the geology is hidden by water. They are more affected by geological uncertainties and risks than most other tunnel projects because of the limited geological information and the close proximity of large amounts of water.

The articles in this publication summarize how these challenges are met in Norwegian sub sea tunnel construction. In addition to outlining procedures and reporting experience from some more interesting tunnel projects, the articles deal with the following aspects of sub sea tunnelling: field investigation methods, tunnel design and excavation experience, safety aspects related to road tunnels, future sub sea tunnel projects, and installations in sub sea road tunnels.

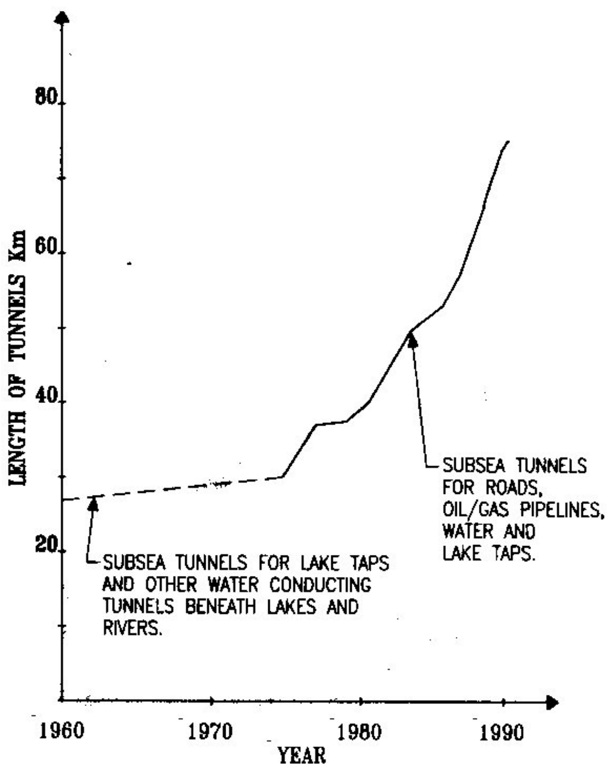


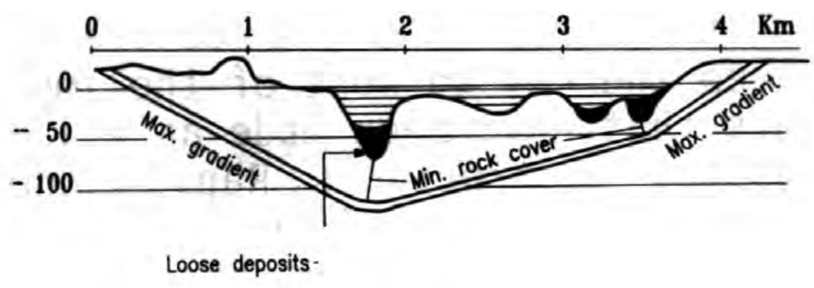
Fig. 1 Development of Subsea Tunnel construction in Norway

History

The first sub sea road tunnel in Norway was constructed in 1982. Since then, over 45 km of sub sea tunnels have been excavated, see Table 1 at the end of this paper.Looking back on earlier tunnel projects in Norway, there are many tunnels, especially in conjunction with hydro- power projects, which pass under rivers and lakes. Intakes to reservoirs consisting of submerged bottom piercings or "lake taps", a specialty in Norwegian tunnel construction., number more than 500, of which 70 have been made since 1980.

The total length of all sub sea tunnels constructed in Norway during the last 75 years is not known. In Fig. 1 it has been estimated at approximately 80 km.

The deepest sub sea tunnel in Norway was constructed in 1974. It has a 50 m rock cover at its deepest point: 253 m below sea level.



Experience

All sub sea tunnels in Norway have been excavated by the drill and blast method. Piercings/lake taps have also been performed by blasting the final rock plug except for

Fig. 2. Main features detrmining the alignment of a subsea tunnel.

the latest piercing performed at Kalstø (see Table 2 at the end of this paper) where the final hole was made by the reaming method. Alignment of a sub sea tunnel is determined by geological and topographical conditions as well as the tunnel's maximum gradient requirement (see Fig. 2). The minimum distance for safety between the tunnel roof and the rock surface under the sea, otherwise known as the rock cover, is a crucial dimension for locating a sub sea tunnel. Fig. 3 shows the minimum rock cover used in Norwegian sub sea tunnels.

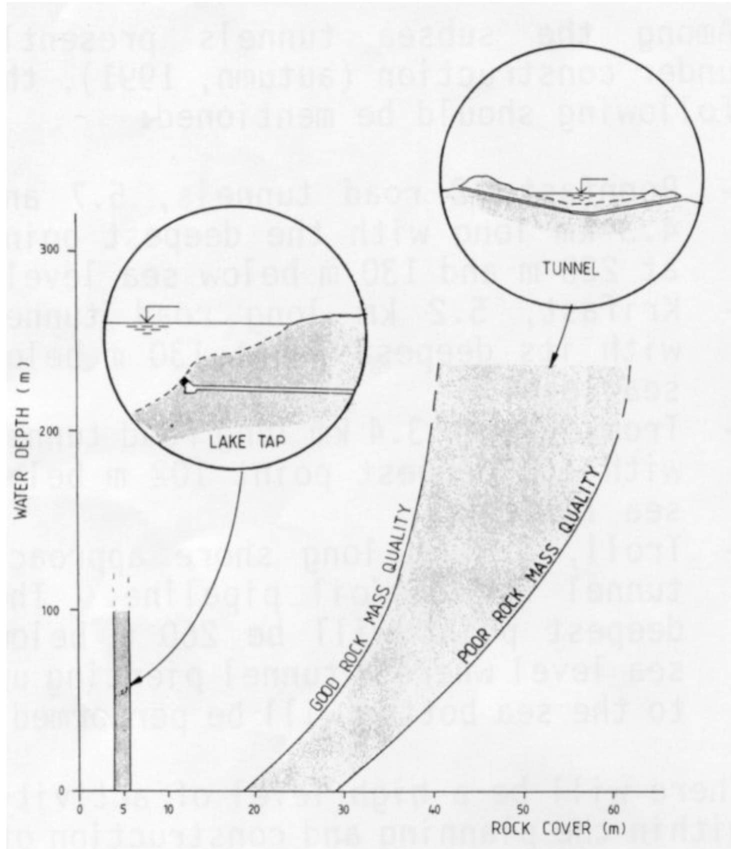


Fig 3. Norwegian practice regarding minimum rock cover. More information on tunnel overburden is presented in other articles in this publication

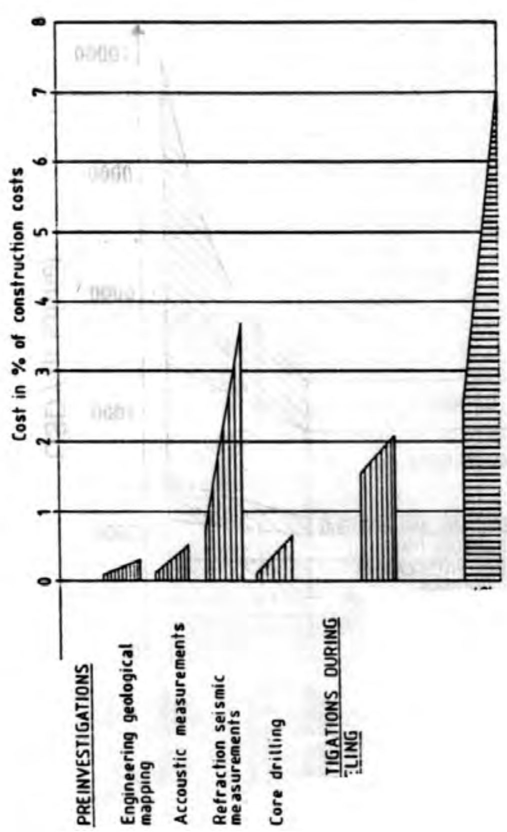


Fig. 4 Approximate distribution of investigation cost for Norwegian subsea rock tunnels.

Although there has been a continuous development in sub sea tunnel construction since the start of lake taps in 1905, more systematic improvements have taken place during the last 10-15 years due to the increase in sub sea tunnelling activity. Many of these improvements regard geophysical site investigation techniques represented by refraction seismic measurements and acoustic profiling. Results from these field measurements are vital for tunnel alignment planning. A map of the sea bottom is obtained from the acoustic profiling which gives the distribution and thick- ness of loose deposits. The refraction seismic measurements give additional information on the rock mass quality and a more accurate location of the rock surface. Developments in equipment have also resulted in a faster execution of field investigations, better data pro- cessing, and consequently, a reduction of cost, which now, for sub sea tunnels, amounts to 2.5-7% of the total construction cost, see Fig. 4.

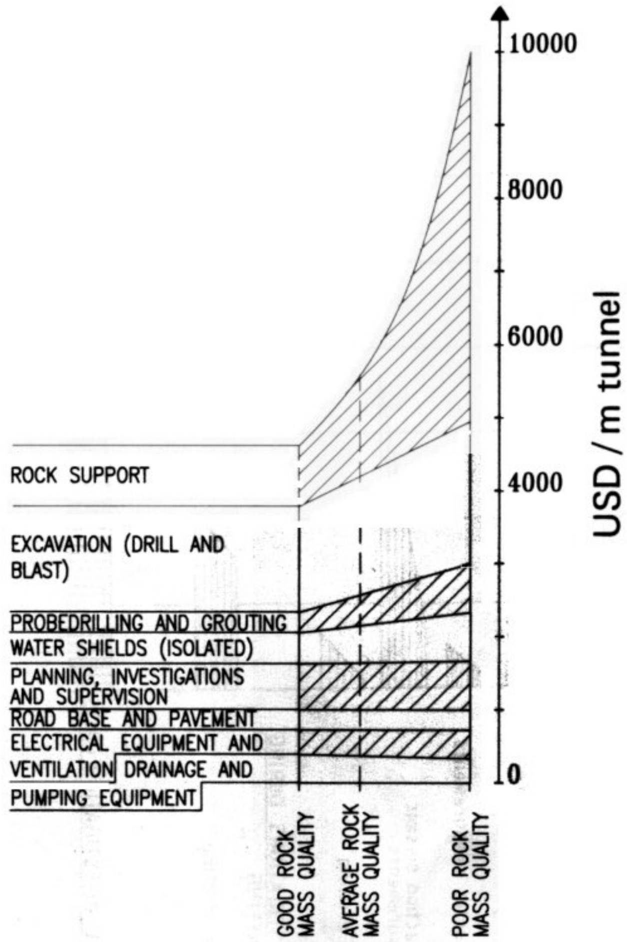


Fig. 5. Approximate distribution of cost for a two-lane 50m^2 subsea road tunnel.

New and promising geophysical techniques under development are the radar and cross-hole methods. A couple of papers presented in this publication describe the geophysical investigations carried out for sub sea projects.

In addition to the use of advanced field investigation methods, the special challenges of sub sea tunnelling require thorough planning and execution of the excavation works. The following safety measures, important for safe tunnel construction, are standard today in sub sea tunnelling:

-Systematic 20-30 m long exploratory drill holes ahead of the tunnel working face.

-Additions, longer exploratory core drill hales where possible poor quality rock masses can be expected.

High pressure pre-grouting if water bearing zones, and/or poor rock mass qualities, have been detected in the exploratory holes.

-A high pumping capacity for de-watering the tunnel in case of unforeseen water ingress.

-High capacity application of fibre-crete quickly after blasting in order to support poor stability rock masses of short stand-up time.

All these measures reduce the possibility of tunnelling problems caused by unforeseen ground conditions.

A continuous exchange of experience and a close cooperation between engineering geologists, planners and contractors has been the key to most of the improvements that have been made as well as successful constructions. Rapid tunnel- ling has resulted in lower excavation costs for sub sea tunnels, see Fig. 5.

Present and Future Sub Sea Tunnel Projects

A good number of studies have been made for possible sub sea tunnels in the last 10 years, amongst which are 60 km long tunnels from the Norwegian mainland to some of the nearer offshore oil fields and a 45 km long railway tunnel beneath a deep fjord. Several other sub sea projects are at the planning stage and are referred to in one of the articles.

Among the sub sea tunnels presently under construction (autumn, 1991), the following should be mentioned:

Rennfast, 2 road tunnels, 5.7 and 4.3 km long with the deepest point at 220 m and 130 m below sea level. Krifast, 5.2 km long road tunnel with its deepest point 130 m below sea level.

Tromsøysund, 3.4 km long road tunnel with its deepest point 102 m below sea level.

Troll, 3.8 km long shore approach tunnel for gas/oil pipeline. The deepest point will be 260 m below sea level where a tunnel piercing up to the sea bottom will be performed.
There will be a high level of activity within the planning and construction of sub sea tunnels in Norway in the coming years.

Table 1. Sub sea tunnels constructed in Norway after 1980

TUNNEL	TYPE	YEAR	LENGTH	DEEPEST POINT	CROSS SECTION	ROCKS
Slemmestad	W	1980	1.0 km	- 93 m	10 m^2	claystone, limestone
Vardö	R	1982	2.6 km	- 88 m	46 m^2	slate and sandstone
Kårstö	W	1983	0,4 km	- 58 m	20 m^2	phyllite
Karmsundet	O	1984	4.7 km	-180 m	26 m^2	gneiss, phyllite
Fördesfjord	O	1984	3.4 km	-160 m	26 m^2	gneiss
Förlandsfjord	O	1984	3.9 km	-170 m	26 m^2	gneiss, phyllite
Ellingsöy	R	1987	3.5 km	-140 m	68 m^2	gneiss
Valderöy	R	1987	4.2 km	-137 m	68 m^2	gneiss
Hjartöy	O	1987	2.3 km	-110 m	26 m^2	gneiss
Kvalsund	R	1988	1.5 km	- 56 m	43 m^2	gneiss
Godöy	R	1989	3.8 km	-153 m	48 m^2	gneiss
Flekkeröy	R	1989	2.3 km	-101 m	46 m^2	gneiss
Hvaler	R	1989	3.8 km	-120 m	45 m^2	gneiss
Nappstraumen	R	1990	1.8 km	- 60 m	55 m^2	gneiss
Maursundet	R	1990	2.3 km	- 93 m	43 m^2	gneiss
Fannefjord	R	1990	2.7 km	-100 m	43 m^2	gneiss
IVAR, Jaeren	W	1991	1.9 km	- 80 m	20 m^2	phyllite
Kalstö	O	1991	1.2 km	-100 m	38 m^2	greenstone

R = SUBSEA ROAD TUNNEL
W = SUBSEA WATER TUNNEL
O = SUBSEA TUNNEL FOR OIL/GAS PIPELINE

Table 2. Lake taps/tunnel piercings performed in Norway after 1980

PROJECT	TYPE	YEAR	AMOUNT	WATER DEPTH	ROCKS
Aurland	H	1980	4	15-22 m	gneiss
Kjela	H	1980	1	48 m	gneiss
Holen	H	1980	1	45 m	gneiss
Vangen	H	1980	2	21-22 m	gneiss
Oksla	H	1980	1	85 m	gneiss, granite
Eidfjord	H	1980	5	9-52 m	gneiss
Slemmestad	W	1980	1	40 m	claystone
Reppa	H	1981-83	2	10-15 m	phyllite
Aurland II	H	1981-84	10	10-30 m	gneiss, phyllite
Sörfjord	H	1982	1	70 m	mica schist
Lomen	H	1983	2	20 m	phyllite
Mosvik	H	1983	1	40 m	amphibolite, mica gneiss
Tjodan	H	1984	4	15-25 m	gneiss
Bergsbotn	H	1984	1	12 m	granitic gneiss
Ulla Förre	H	1986	8	36-101 m	gneiss, phyllite
Skarje	H	1986	2	6-20 m	gneiss
Eikelandsosen	H	1986	1	60 m	granitic gneiss, phyllite
Kobbelv	H	1986	7	5-120 m	gneiss, mica schist
Jostedal	H	1986-89	6	16-73 m	gneiss
Hjartöy	O	1987	1	80 m	gneiss
Mel	H	1989	4	30-90 m	gneiss
Nyset-Steggje	H	1986	2	10-17 m	gneiss
IVAR, Jaeren	W	1991	2	40-80 m	phyllite
Kalstö	O	1991	1	60 m	gneiss

H = LAKE TAP/TUNNEL PIERCING FOR HYDROPOWER DEVELOPMENT
W = TUNNEL PIERCING FOR SEWERAGE OUTLET
O = TUNNEL PIERCING FOR SHORE APPROACH OF GAS/OIL PIPELINE

Safety aspects of Norwegian road tunnels.

(Invited paper.)

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ABSTRACT:
This paper start with an evaluation of the standard of Norwegian road tunnels today. The problem of accidents in tunnels are analysed and two tunnels with accident black spots are described. Lastly a special calculation model for safety, incidents and fires in tunnels are evaluated.

1 .TUNNEL STATISTICS

Tunnel statistics on the Norwegian national road network are revised every second year. The data are published by the Public Roads Administration, the latest edition is dated January 1989.

The statistics tells us that Norway has 529 tunnels with an accumulated length of 344 km out of a national road network of 26031 km. Even if the number of tunnels are large, their length are usually short. This is shown in Table 1.

	Number		Total length km	
Less than 100 m	125	24%	7	2%
101 - 500 m	222	42%	55	16%
501 - 1000 m	94	18%	63	18%
More than 1000 m	88	16%	219	64%

Two thirds of the tunnels are shorter than 500 m and have an accumulated length of 18%, while the 16% longer than 1 km have an accumulated length of 64%.

If this table is compared to earlier editions of the statistics, it is shown that the newer tunnels are longer. The increase in tunnel length is far larger far tunnels longer than 3 km, than for tunnels shorter than 500 m. Typical for the Norwegian road network are low traffic volumes. The average annual daily traffic for the main network is less than 2000. Of the tunnels 310 have an AADT of less than 1000. Only 54 tunnels have traffic over 5000. This is also changing today. More and more tunnels are built in or close to cities. The Oslo tunnel is planned for an AADT of 90000, while the Vålerenga tunnel in Oslo today has more than 27000 cars a day.

A large proportion of the tunnels are old and have therefore been built to other design standards than today. 363 of the tunnels have a hard surface that is 6 m wide or less. Most of these tunnels will not pass the new requirements. Because of the low traffic in most of the tunnels, this low standard does not constitute a special problem.

The free height in most of the tunnels constitutes a more serious problem. Only 27 tunnels have a free height more than 4,6 m. As many as 43% of the tunnels have a free height less than 4,0 m.

As you all probably know by now, all Norwegian tunnels with artificial ventilation use the longitudinal system. Because of the low traffic volume, only about 7% are ventilated. Of the same reason artificial lighting is only used in 40% of the tunnels. Even in most of these tunnels the luminance is less than 1 cd/m^2. Both for ventilation and lighting the standards are increasing every year. We will probably never reach a high standard for all our tunnels, but the more than 20 km of tunnels that are opened every year conform to our new design guideline.

2. ACCIDENT STUDIES

It has not been possible to do a special accident study of sub aqueous tunnels in Norway. Three studies of ordinary tunnels will therefore be reported here.

In 1980-81 a student at the

Technical University at Trondheim made a study of 362 tunnels on main highways. As for all Norwegian tunnels, the main part of the tunnels was short (79% shorter than 500 m) and with low traffic volumes (85% with AADT less than 1500). In the study all personal injury accidents in tunnels and 100 m on both sides were analysed. In the 10 year period 1970-79 221 accidents were recorded by the police. More than twice as many accidents occurred in the entrance zone as in the middle section. The accident frequencies were as follows:

Middle section 0,16 acc $\cdot 10^{-6}$

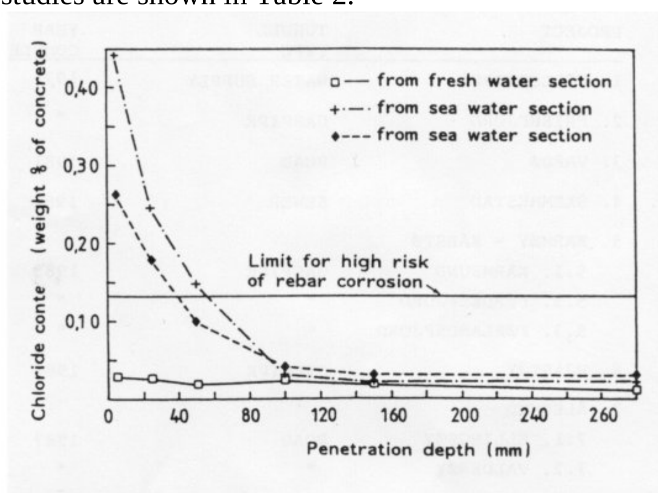
Entrance zone 0,51 acc $\cdot 10^{-6}$

Mean tunnel 0,28 acc $\cdot 10^{-6}$

Tunnel + 2 $\cdot$ 100m 0,31 acc $\cdot 10^{-6}$

A tendency towards increase in accident frequency for tunnels with little traffic, less than 500 AADT, and for tunnels longer than 1000 m and shorter than 200 m.

The second study was also made by a student in 1984, and consists of 145 personal injury accidents in 145 tunnels. This study covered tunnels with some more traffic than the first study. Accidents in the tunnels are compared with accidents on the public roads in Norway. Results from the two studies are shown in Table 2.



Single vehicle accidents occur more often in these bi-directional tunnels than on the open road.

Tailing accidents increase with road width and AADT.

Meeting accidents decrease with road width, but increase with AADT.

Single vehicle accidents increase with AADT, but this type of accident is most common on roads with low AADT .

A special study was made for 36 tunnels in the county of Hordaland. The accumulated tunnel length was more than 31 km. 57 accidents recorded from 1977- 1986 were analysed. The following accident frequencies were:

Entrance zone 0,78 x 10-6 Middle section 0,14 x 10-6

The general conclusion on these studies must be that tunnels are as safe as roads in the open, even for tunnels of low standard. The entrance zone is the accident prone place with accident frequencies as high as 3-5 times that in the middle section.

This is, however, not to say that we do not have an accident problem altogether. Only last year we recorded two tunnels with accident black spots, and we had to witness two very serious accidents, both in the county of Hordaland.

3. BLACK SPOTS

The two tunnels with an accident black spat are the Vålerenga tunnel in Oslo and the Ellingsøy tunnel in Ålesund.

The Vålereng tunnel in Oslo is a two tubes tunnel. The northbound tube with three lanes and a positive gradient of about 6% was opened early 1988. Except for a problem with speeding cars, no accidents or other traffic irregularities were recorded. The problem however started w hen the southbound tube opened early 1989. In the first five months of operation 10 accidents were recorded, of which 5 had personal injury. All accidents (except one) had happened between 9 am and 2 pm. Half of the accidents were single vehicle accidents while in the other half, 5 involved two or more cars in the same direction.

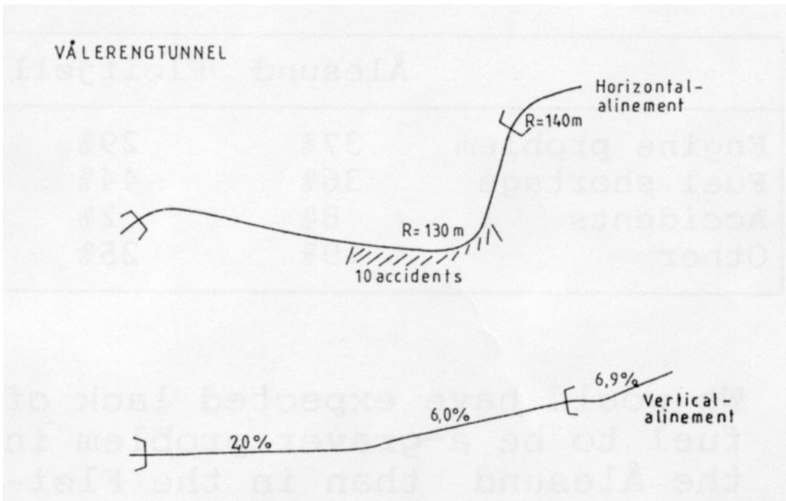


Fig 1. Geometry of the Vålerengtunnel

After preliminary traffic studies the problem was pin- pointed to a curve with radius 130 m on a downgrade lane. Even if the cars approached the tunnel with a safe speed (60 km/h), they accelerated in the tunnel to about 70 km/h in a curve with a design speed of 60 km/h. Measurements of friction coefficient of the concrete pavement showed extremely low volumes. Many accidents occurred on a wet surface due to insufficient drainage of surface water outside the tunnel.

After rutting the pavement, installation of traffic signs and adoption of an automatic traffic surveillance system, the driving speed decreased by about 7 km/h, from an average of 66 km/h to 59 km/h. For the last 7 months no accidents have be en recorded in this specific curve.

The Ellingsøy tunnel is the first of the Ålesund tunnels when driving from the city of Ålesund. The tunnel has one tube with three lanes, it is 3,5 km long and has a gradient of 8,5%. The AADT is about 3050 and the speed limit 80 km/h. From April 1988 to April 1989 six accidents were recorded in the tunnel (4 accidents with personal injury).

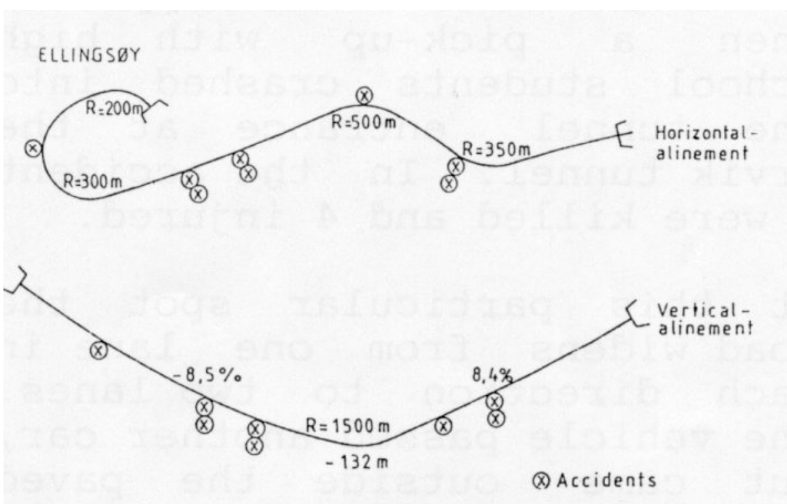


Fig. 2 shows where the accidents happened. 4 of the accidents were single vehicle accidents.

The accident rate for the Valderøy tunnel is 0,7 x 10^{-6} acc/vehl km year, which is w hat one should expect from a tunnel with this combination of gradient and curvature.

The figure shows that the tunnel has no particular black spot, as the accidents are distributed throughout the tunnel. We have not, so far, been able to tell why the accidents occur in this tunnel and not in the Valderøy tunnel. Some of the theories that are being developed is that diesel smoke makes the pavement very slippery, that salt water is seeping in, that the drainage system is not adequately designed for the residue w hen cleaning the tunnel etc.

The most tragic tunnel accident in Norway happened on August 15 1988. A bus with 23 school children and 11 adults lost its braking power in the Måbø tunnel and crashed into the concrete entrance portal which protruded more than one metre from the tunnel wall. All inside were injured and 12 children and 4 adults lost their lives.

The tunnel is 540 m long and has a radius of 200 m where the bus hit the tunnel wall. The tunnel has a gradient of 7,8%. The bus driver had tried to slow down the bus in the downgrade by steering with two wheels on the loose gravel shoulder, but hit the concrete wall that was hidden behind the lining of plastic materials.

The other accident happened when a pick-up with high school students crashed into the tunnel entrance at the Ervik tunnel. In the accident 5 were killed and 4 injured.

At this particular spot the road widens from one lane in each direction to two lanes. The vehicle passed another car, but came outside the paved surface. The driver, who did not have a drivers license, could not manage to steer the car back on to the road and crashed into the tunnel entrance

4. RECORDED INCIDENTS

Of all the Norwegian tunnels, only the Oslo tunnel hasclosed circuit television Data on incidents or break downs are based on the SOS-telephone system. Compared to tunnels with CCTV we must expect to record less incidents in Norwegian tunnels. For tunnels with SOS-telephone systems we have data from the Ålesund tunnels (Valderøy and Ellingsøy) and the Fløifjell tunnel in Bergen. Data from the tunnels are shown in Table 3.

	Length	Tubes/lanes	AADT	Max gradient
Ellingsøy	3,5 km	1/3	3050	8,5%
Valderøy	4,2 km	1/3	2390	8,5%
Fløifjell	3,2/3,7 km	2/2	20000	Small

Data given from the tunnel authorities show a recorded incident rate of $9{,}2 \cdot 10^{-6}$ (cars-km-year) in the Ellingsøy tunnel, $5{,}5 \cdot 10^{-6}$ in the Valderøy tunnel and $3 \cdot 10^{-6}$ in the Fløifjell tunnel. The reduced rate in the Fløifjell tunnel compared to the two other tunnels has probably something to do with people managing by their own or not reporting incidents or because of the higher gradient. Table 4. shows why the cars have stopped in the tunnels.

	Ålesund	Fløifjell
Engine problem	37%	29%
Fuel shortage	36%	44%
Accidents	8%	2%
Other	19%	25%

We would have expected lack of fuel to be a graver problem in the Ålesund than in the Fløifjell tunnel because of the grades. This does not appear to be the problem we had expected.

The difference in incident rates and incident types does probably have something to do with the Ålesund tunnels being sub aqueous and the Fløifjell tunnel situated in the middle of Bergen. We have experience from other city tunnels that shows that the use of 808- telephones is very low.

5. EVALUATION OF SAFETY

Based on a great number of studies we can conclude that tunnels are as safe or safer than roads in the open. This conclusion is, however, mostly derived from tunnels that have none or large radius curves and a vertical gradient of less than 4%. Short curves can however occur. From studies of roads in the open air we have learnt that steep gradients, short radius and the combination of the two increases the accident rate.

To be able to evaluate sub aqueous tunnels with gradients as steep as 10% and curves with radius less than 100 m, we have developed a special calculation model. The model is calibrated from known studies throughout the world and PIARC statistics on tunnels and data from roads in the open air. Mostly Norwegian and Swedish material for ordinary roads have be en used.

To use the model the following data are necessary:

- AADT (10 years after opening)
- % heavy traffic
- Speed limit
- One or two-way traffic
- Length of tunnel
- Length of extra passing lane
- Gradient
- Curvature and curve length

If the gradient or the curvature changes within the tunnel, the tunnel must be divided into sections with similar alignment. The model can handle nine different sections.

The model then calculates the following parameters:

- Accidents with personal injury per year
- Accidents without personal injury per year
- Break downs per year
- Fires per year

As more tunnels come into operation, the model will be further refined. So far the model seems' to work best for personal injury accidents and break downs. As the ratio between material damage accidents and personal accidents in the open air is between 15- 25, the ratio in tunnels seems to be as low as 1-5. So far the model has been used to evaluate and compare different design alternatives far more than ten tunnels.

To show how the model works an example from the proposed subaqueous Hitra tunnel is calculated. The two alternatives are made up for this purpose, but are based on the real plans.

Alt 1

AADT = 1200
Speed limit = 80 km/h
% heavy traffic = 12
Bi-directional traffic
Tunnel length = 6000 m
Max gradient = 8%
Climbing lane = 6000 m

Curvature

Radius	200 m	400 m	500 m
Length	800 m	200 m	500 m

Accidents with injuries
Frequency = 0,57
Per year = 1,5
Material damage only
Per year = 7,5
Break downs
Per year = 21,0
Fires
Per year = 0,03

Alt 2

AADT = 1200
Speed limit = 80 km/h
% heavy traffic = 12
Bi-directional traffic
Tunnel length = 8000 m
Max gradient = 6%
Climbing lane = 0 m

Curvature

Radius	400 m	500 m
Length	1000 m	600 m

Accident with injuries
Frequency = 0,50
Per year 1,8
Material damage only
Per year = 8,8
Break downs
Per year= 24,5
Fires
Per year = 0,04

Even if the gradient is 2% less and the curvature better, the tunnel length makes the longer tunnel less safe than the shorter. This is, however, only the safety part of the cost/benefit analysis that also has to cover vehicle costs, tunnel operation costs, construction costs etc.
Use of the model will help designers make tunnels as safe as possible, and serious design problems may be avoided.

6.CONTINGENCY PLANS
For every tunnel longer than 500 m and with AADT more than 500 a contingency plan will be made. In general the plan is an agreement between the tunnel operator, the police, the fire department and the ambulance people on how to work together in case an emergency situation should develop.

The plan consists of the following three parts:

A
Information on the tunnel, design, alignment, possible routes for detours, technical equipment, safety equipment etc.
B
Evaluation of possible hazards like accidents, break downs, fires etc.

C

Measures taken when accidents, break downs, fires etc occur. Manpower as well as equipment must be defined.

So far plans have been made for the tunnels in Oslo and the sub aqueous tunnels.
The intentions and the contents of such plans have be en discussed with the Norwegian Directorate for Prevention of Fires and Explosives.

Future design of sub-sea road tunnels based on cost and technical experience

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Abstract

Since the construction of Norway's first sub sea road tunnel under the straits at Vardø in 1979, more than 23.5 km of road tunnels have been completed under several fjords in this country. In addition many new tunnels are already on the drawing-board or under construction.

In this paper, cost, technical conditions, and general experience from the completed tunnels, are discussed. Among other conclusions, a substantial reduction in sub sea tunnelling costs is registered over the last decade.

Experience, particularly from the Ålesund tunnels, indicates a negative development as far as traffic accidents are concerned. It would appear that this is related to the steep gradient in these tunnels.

The paper further suggests that 3-lane tunnels should not be utilized where traffic density dictates more than a 2-lane tunnel. In such cases dual 2-lane tunnels are to be preferred.

In conclusion, it is stated that there remains a significant amount of research and development work to be done in connection with traffic safety in sub sea tunnels.

1. Introduction

Before the construction of the Vardø sub sea tunnel, straits crossing in Norway had been traditionally accomplished either by bridge or by ferry. The Vardø tunnel introduced a new dimension to straits crossings in this country. After the Vardø tunnel, several new sub sea tunnels have either been built, are under construction, or are on the planners' drawing boards.

Fig. 1. Map showing subsea tunnels in use.

2. Experience from the first 8 tunnels.

2.1 Completed Sub Sea Tunnels.

At present (May 1990) 8 sub sea tunnels are open to traffic, these are summarized in Table 1, and five other tunnels are under construction.

TUNNEL	LENGTH	GREATEST DEPTH	MAX. GRADIENT	CROSS SECTION
Vardø	2,620	- 88 m	8.0 %	46 m2
Ellingsøy	3,481 m	- 140 m	8.5 %	68 m2 *
Valderøy	4,176 m	- 137 m	8.0 %	68 m2 *
Kvalsund	1,530 m	- 56 m	8.0 %	43 m2
Godøy	3,835 m	- 153 m	10.0 %	48 m2
Flekkerøy	2,321 m	- 101 m	10.0 %	46 m2
Hvaler	3,751 m	- 120 m	10.0 %	45 m2
Nappstraumen	1,776 m	- 60 m	8.0 %	55 m2 **

* The Ålesund tunnels (Ellingsøy and Valderøy) have 3-lanes.
** Nappstraumen tunnel has a 2-lane highway + a sidewalk.

Table 1. The 8 sub sea tunnels currently in use (May 1990)

2.2 Costs

The costs for the completed tunnels are shown in Fig. 2. The figures for Nappstraumen, Flekkerøy and Hvaler are somewhat uncertain as yet, because the construction accounts are not yet closed. However any errors can be assumed to be small and will have no significant influence on the final result.

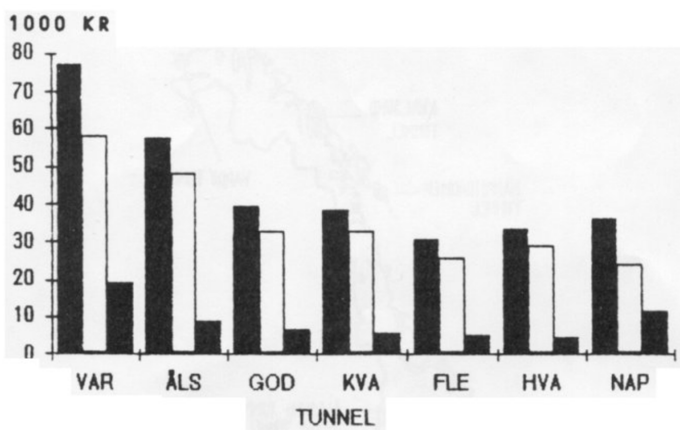

Fig .2. Tunnel costs pr metre

The Ellingsøy and Valderøy tunnels were driven simultaneously and data for these two tunnels has been combined. All cost have been converted to 1989 price levels. The costs are divided into three columns. Column #1 shows the total costs, whilst column #2 shows the amount paid to the major contractor. Column #3 includes the Public Road Authorities expenses to control, direct labour, other minor contracts, value added tax and price-rises during construction.

The total cost for planning and site investigation are not included in all of the projects. These costs can be as high as 2000-3000 kr pr metre tunnel. Incentive costs for early opening of the tunnels are included where they were paid.

The total tunnel-costs vary from ca. 78 000 kr/m for the Vardø tunnel to ca. 32 000 kr/m for the Flekkerøy tunnel. The earlier tunnels cost much more than the later ones.

2.3 Site Investigations

Before a tunnel can be constructed there has to be a thorough geological investigation. The geological factors are quite decisive for the location of the tunnel. Development has of late tended towards simpler and less comprehensive investigations. Investigations have often been limited to geological mapping

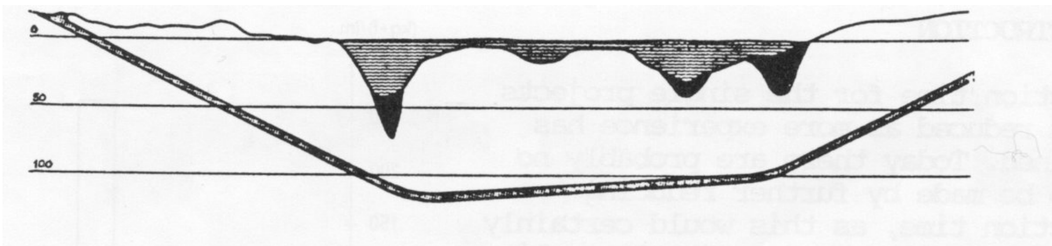

Fig. 3. Typical section of a Norwegian sub sea tunnel.
(Hvaler tunnel, length = 3751 m, depth = -120 m. Rock and sedimentary deposits are shown.)

supplemented with acoustic measurements and a few seismic profiles. These elementary investigation procedures have not so far resulted in construction problems for any of the tunnels yet completed.

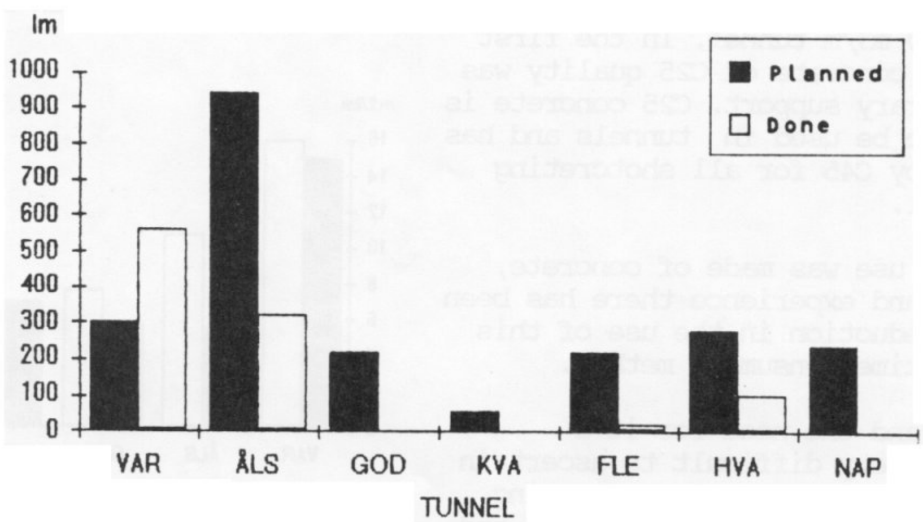

Fig. 4. Concreting, planned and executed.

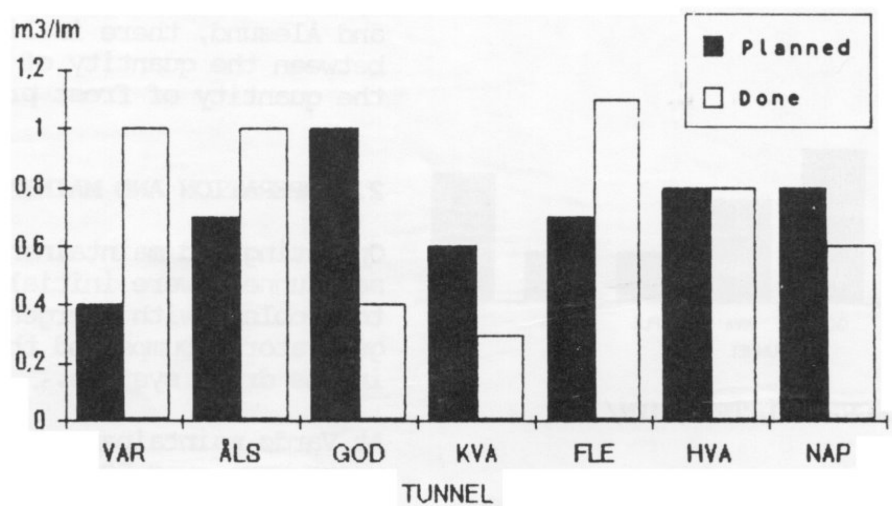

Fig. 5. Shotcreting, planned and executed.

2.4 Construction

Construction time for the single project has been reduced as more experience has been gained. today there are probably no gains to be made by further reducing construction time, as this would certainly be at the expense of the projects quality. The most common method of support is bolting, sometimes combined with netting and strips. Shotcrete or traditional concreting are used when circumstances demand it.
Bolting is the most popular method in Norway, and represents a large percentage of the support costs, particularly at Vardø, Ålesund and Godøy.
The use of shotcrete has stabilised at about 0.7-1.0 m^3/m tunnel. In the first tunnel, Vardø, concrete of C25 quality was used for temporary support. C25 concrete is to inferior to be used in tunnels and has been replaced with C45 for all shotcreting below sea level. Initially much use was made of concrete, but with time and experience there has been a noticeable reduction in the use of this expensive and time-consuming method. Water Ingress and the need for it's prevention are very difficult to ascertain prior to construction. Factors governing leakage are; rock type, crack pattern and the amount of clay in the cracks. Fig 6. shows the amount of leakage from each tunnel at the time of the opening.

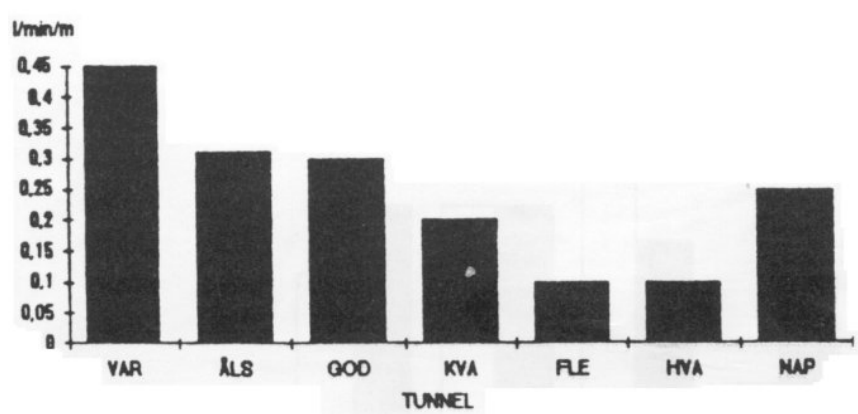

Fig. 6. Water ingress, liter/min. pr meter.

This can be compared with the amount of injection that is shown in Fig. 7.

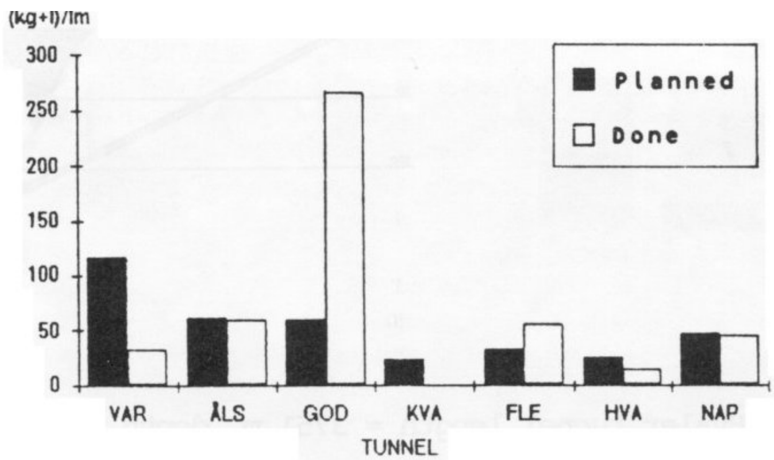

Fig. 7. Injection.

So far Vardø tunnel has the most leakage, at the same time it is the tunnel with the least amount of injection in it!

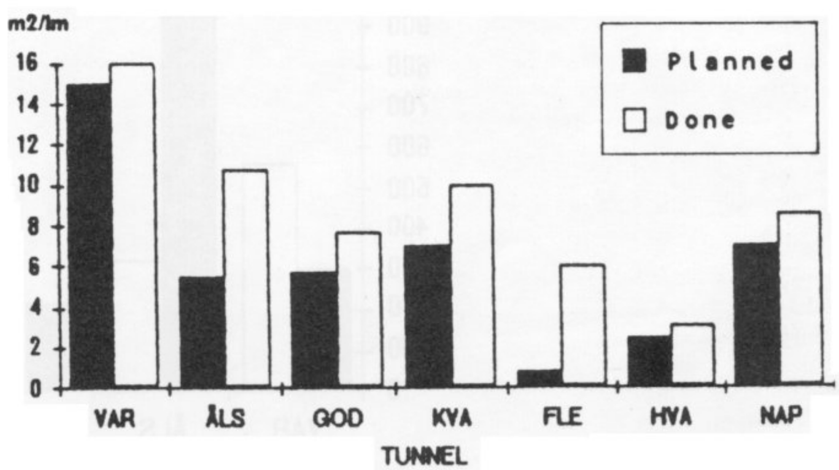

Fig. 8. Frost protection.

Fig. 8 shows the amount of frost protection installed in Norwegian subsea tunnels. Apart from the large quantity used in Vardø and Ålesund, there is little correlation between the quantity of water-ingress and the quantity of frost protection.

2.5 Operation and Maintenance

Operating and maintenance costs for sub sea tunnels were initially very high, due to problems with emergency power generators, pumps and the growth of algae in the drain systems.

At Vardø maintenance costs for the first years were over 500 000 kr/km a year. They are now reduced to about 450 000 kr/km a year. For Ålesund the costs are 420 000 kr/km a year. A breakdown of costs shows the distribution:

General costs	527 000. NOK
Drains and pumping	308 000 NOK

Lighting and ventilation	2,382 000 NOK
Winter costs	33 000 NOK

2.6 Other operating problems

The most extraordinary problem in tunnels is algae. This phenomenon exists at Vardø, Valderøy and Hvaler. There appears to be no connection between rock type and the growth of algae.

Experience from Vardø suggests that the algae population expands to a certain population level before collapsing in order to start all over again.

Sea-water leakages make asphalt quite slippery, perhaps because of algae.

Shotcrete is broken down very quickly by seepage, particularly by salt water. Poor quality shotcrete is more susceptible than high quality shotcrete. Consequently new and more stringent rules have been made for the use of shotcrete in tunnels. The Norwegian Ministry of Transportation has also initiated research into the field of shotcrete in salt water.

So far corrosion has not resulted in any particular problems for subsea tunnels.

2.7 Traffic conditions

Whilst the Vardø and Ålesund tunnels represented a breakthrough for tunnelling techniques one has now become more aware of problems that are concerned with traffic management, safety and traffic psychology. As yet our experience in these fields is rather meagre. there are three factors which have a conclusive effect on traffic operations in tunnels:

- -gradient and curvature
- -number of traffic lanes
- -safety facilities

In Norway these factors are governed by the "Road Standards" Among these, the following clause is relevant for tunnels: "Choice of gradient can be an economic question which can be measured against the cost of overtaking lane."

3. Future design of sub sea tunnels

As a result of experience from the first sub sea tunnels, The Norwegian Public Roads Authority has set up standards for future construction.

3.1 Gradients

The following standards have been drawn up for gradients in sub sea tunnels.

Annual traffic flow.	Maximum gradient.
<1500	10%
1500-5000	8%
>5000	6%

A series of incidents in the tunnels has shown that high gradients result in higher accident rates. In the Ellingsøy tunnel the accident rate is 0,9 accidents/million vehicles km, and this is much higher than normal for other tunnels.

Earlier research has shown that the accident rate increases with steeper gradients from about 6-8%. 70% of the accidents occur on the downhill stretches, were speed, usually combined with overtaking is the usual accident cause. one can assume that the same factors also apply to tunnels, perhaps even more so.

The Public Road Authorities have earlier accepted gradients of up to 10% in tunnels with little and local traffic, with short gradients. The development of the accident rate for these tunnels is now under scrutiny.

RISK CATEGORY						
EQUIPMENT	A	B	C	D	E	COMMENTS
LIGHTING	@	@	@	@	@	
EVACUATION LIGHTING		0	0	@	@	EVERY 50 m
FIRE EXTINGUISHERS	@	@	@	@	@	CLASS A,B EVER 250m CLASS C,D EVERY 125m CLASS E EVERY 50m
FIRE HYDRANT		0	0	0	0	CONFER WITH FIRE BRIGADE
EMERGENCY TELEPHONE	0	@	@	@	@	CLASS B EVERY 500m CLASS C,D EVERY 250m CLASS E EVERY 100m
ACCESS BETWEEN TUNNEL TUBES		O	@	@	@	750m FOR TRAFFIC 250m FOR PEDESTRIANS
TURNING POINTS	0	0	0	0	0	DEPENDENT ON TUNNEL LENGTH
EMERGENCY SIGNS	@	@	@	@	@	EVERY 50m
INFORMATION SIGNS NEAR ENTRANCE	0	@	@	@	@	
VARIABLE TEXT SIGN			0	@	@	
BLINKING RED LIGHT AT TUNNEL ENTRANCE		0	@	@	@	CONTROLLED BY CO MEASUREMENT
TURNPIKE FOR CLOSING TUNNEL	0	@	@	@	@	MANUAL OPERATION <20,000veh./d y
EMERGENCY LAYBYS						
TV SURVEILLANCE				0	0	HIGH TRAFFIC
EMERGENCY RADIO CHANNEL	0	0	@	@	@	FOR USE ONLY TO GIVE TRAFFIC INFO
MOBILE TELEPHONE	0	0	0	0	0	
EMERGENCY GENERATOR	0	0	0	0	0	
WARNING TO HIGH VEHICLES	0	0	0	0	0	SHOULD BE USED CLOSE TO LOW EQUIPMENT

@ = Equipment must be installed; 0 = Equipment may be installed.

Table 2. Safety measures, by risk class.

3.2 Factors affecting safety

In Norway tunnels are divided into 5 different risk categories. The Road Standards define which safety measure must be met for each risk category. The risk categories are defined in Fig. 9.

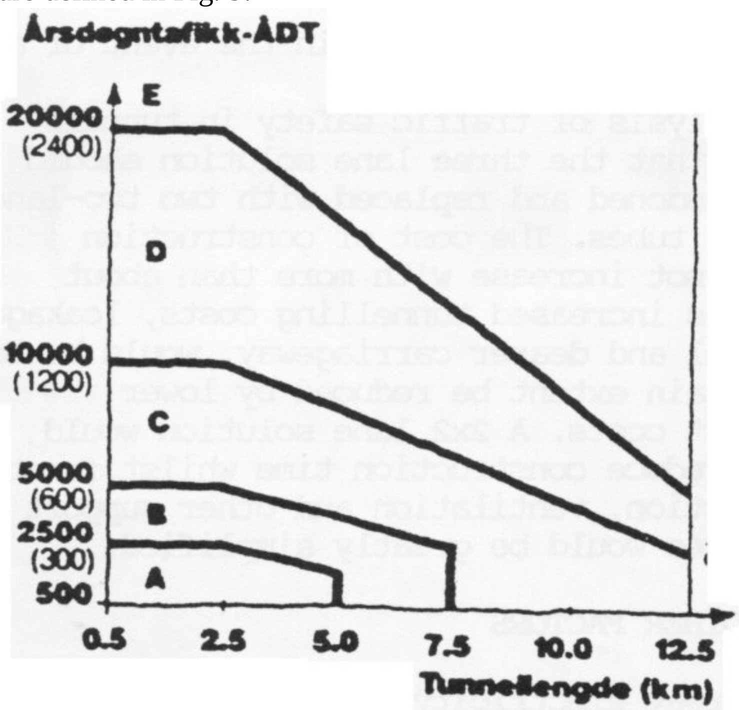

Fig. 9. Risk classes. (As defined by the Road Standards.)

3.3 Possible and probable projects in the next 3-5 years

County	Tunnel	Length in metres	Max.depth in metres
Finnmark	Magerøy(Fatima)	6600	210
Troms	Maursundet	2300	95
	Tromsøysund	3360	102
Sør-Trøndelag	Hitra - Frøya	4800	155
	Hitra - Mainland	5650	275
Møre & Romsdal	Fannefjorden	2730	105
	Freifjorden(Krifast)	5200	130
	Averøy	5850	245
	Norøyvegen Harem	3500	107
	Eiksund	6500	315
	Hareid	13000	630
Hordaland	Sveio - Stord	10430	335
	Alt.Bømlo - Føyno	3000	145
	Føyno - Sveio	6400	275
	Håkonshella-Bjorøy	1860	88
Rogaland	Byfjord(Rennfast)	5700	220
	Mastrafjorden(R.fast)	4300	130
Akershus -Buskerud	Indre Oslofjord	7400	130
Østfold - Vestfold	Ytre Oslofjord	14000	300

Table 3. Sub sea tunnels that will be, or can be constructed during the next 3-5 years.

As the list shows, there is a tendency towards longer and deeper tunnels. There are still many unanswered questions concerning the longest and deepest of these tunnels and these will be appraised thoroughly before construction can begin.
If the concept of tunnels is to develop as a viable alternative for crossing straits, three factors must be paid close attention.
These are:

Construction costs. These can still be reduced.
Maintenance costs. These must be reduced.
Traffic conditions. These must be taken more seriously.

In many of the projects that are on the drawing-board the above factors will have to be balanced against one another; the shorter the tunnel, the cheaper construction and maintenance will be, but the resulting steeper gradients will worsen traffic conditions.

3.4 Cover

Another condition for tunnel length is the necessary rock cover required to enable construction. The lesser the cover, the shorter the tunnel. The tunnels built so far has shown that leakages are more dependent on rock type than cover depth.

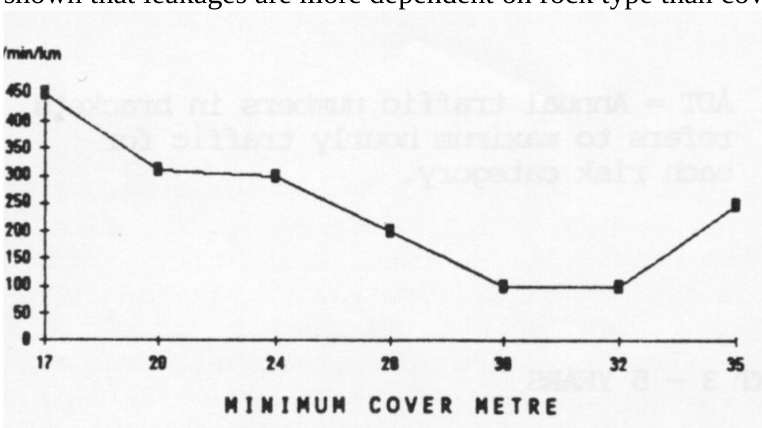

Fig. 10. Water ingress / cover.

Methods must be found to evaluate possible leakages early in the planning to evaluate possible leakages early in the planning phase so that both geological and topographical features can be thoroughly utilized to give the most economical result.

3.5 Site investigation

New methods of site investigation are being developed to try to solve these problems. Common for these methods is that they give much more accurate information on faulting, leakages and clay content in the rock.
Of the more interesting techniques, the following are worth mentioning:
Refractions seismic or “surface tomography” that can make 3D models of the tunnel area.
Improved seismic signal analysis.
Resistance tomography; a measurement of electrical impediment in a borehole that can be measured either from sea or land.
“Down the hole” radar.
It is probable that methods will be developed that can be used by tunnellers on the face, so that the geophysicists need only be contacted when the measurements give cause for alarm.

3.6 Choice of number of lanes

The number of lanes in a tunnel is not only a question of traffic capacity, but also the result of factors such as safety and evacuation possibilities in the event of an accident. An analysis of traffic safety in tunnels show that the three lane solution should be abandoned and replaced with dual two-lane tunnel tubes The cost of construction would not be increased with more than 25% since increased tunnelling costs, leakage control and dearer carriageway would be to a certain degree be offset by lower support costs. A 2 · 2 lane solution would also reduce construction time, whilst frost protection, ventilation and other support measures would be greatly simplified.

3.7 Other factors

In Norway electricity costs are dependent on the maximum load used over a period of 15 minutes, measured twice a year. In order to hold these costs to a minimum, it is essential to coordinate the use of lighting, ventilation and pumping as much as possible. One way to do this is to build a reservoir for seepage water with a capacity for 4-5 days of leakage. In this way it would be possible to empty the tunnel when the use of lighting and ventilation is low. A large reservoir makes it also possible to reduce or eliminate the need for emergency generators and reserve drainage.
Lighting conditions can be greatly improved if concrete paving is used and the tunnel walls are painted white in the entrance zones.
The road must have sufficient gradient and camber to ensure a rapid drainage of the road surface so that vehicles will avoid driving in “brine”.

Stability and rock cover of Norwegian hard rock sub-sea tunnels

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ABSTRACT:
Initially, this paper summarizes some major results from a recent state-of-the-art review of Norwegian sub-sea tunnels. The review primarily concentrated on discussing the reliability of preinvestigations in predicting tunnelling conditions and the long term behaviour of rock support materials. Key data from completed Norwegian projects are used as a basis for discussing the optimum rock cover for sub-sea tunnels. Cases of instability and cave-in are documented. For most Norwegian subsea tunnel projects, the rock cover appears to be relatively conservative, but experience from some of the projects indicates that this apparent conservatism may be a good investment.

1. INTRODUCTION
The Foundation for Scientific and Industrial Research at the Norwegian Institute of Technology (SINTEF) has studied sub-sea rock tunnels since 1986. Initially, a major state-of-the-art review was conducted in order to evaluate experience from completed Norwegian sub-sea tunnels. Owners, contractors and consultants contributed actively so that a broad overview was possible.
The state-of-the-art review included all Norwegian sub-sea rock tunnels which were completed until 1988. General details of the tunnels are given in Table 1.

PROJECT	TUNNEL TYPE	YEAR COMPLETED	CROSS-SECTION	TOTAL LENGTH	SUBSEA LENGTH	LOWEST LEVEL
1. VOLLSFJORD	WATER SUPPLY	1977	16 m^2	9.4 km	0,6 km	- 80 m
2. FRIERFJORD	GASPIPE	"	16 "	3.6 "	3.1 "	- 252 "
3. VARDØ	ROAD	1981	53 "	2.6 "	1.7 "	- 88 "
4. SLEMMESTAD	SEWER	1982	10 "	0.9 "	0.7 "	- 95 "
5. KARMØY - KÅRSTØ						
5.1. KARMSUND	GASPIPE	1983	27 "	4.8 "	2.1 "	- 180 "
5.2. FØRDESFJORD	"	"	"	3.4 "	1.6 "	- 170 "
5.3. FØRLANDSFJORD	"	"	"	3.9 "	1.0 "	- 160 "
6. HJARTØY	OILPIPE	1986	26 "	2.3 "	1.8 "	- 105 "
7. ÅLESUND						
7.1. ELLINGSØY	ROAD	1987	68 "	3.5 "	1.1 "	- 140 "
7.2. VALDERØY	"	"	"	4.2 "	2.2 "	- 137 "
8. KVALSUND	ROAD	1988	∿ 50 "	1.6 "	0.8 "	- 56 "
9. GODØY	"	1989	52 "	3.8 "	1.6 "	- 153 "
10. HVALER	"	"	∿ 50 "	3.8 "	1.9 "	- 121 "
11. FLEKKERØY	"	"	"	2.3 "	1.0 "	- 102 "
12. NAPPSTRAUMEN	"	-	"	1.8 "	0.9 "	- 63 "

Table 1. List of sub-sea rock tunnels in Norway

Most of the existing tunnels are fjord crossings on the west coast of Norway, used as road tunnels or tunnels for oil and gas pipelines. The greatest depth below sea level so far is about 250 m, and the greatest sub-sea length is approx. 3.1 km (Frierfjord gas pipe tunnel). Ordinary drill and blast techniques have been used for all tunnels.

The Norwegian sub-sea tunnels are situated in a variety of geological structures ranging from typical hard rock such as Precambrian gneiss to less competent phyllite and poor quality schists and shales. All tunnels cross significant zones of weakness under the sea.

The state-of-the-art study concentrated mainly on the following topics:

- Preinvestigations.
- Tunnelling results.
- Behaviour of rock support materials.

Particular emphasis was placed on the usefulness of the preinvestigations in predicting tunnelling

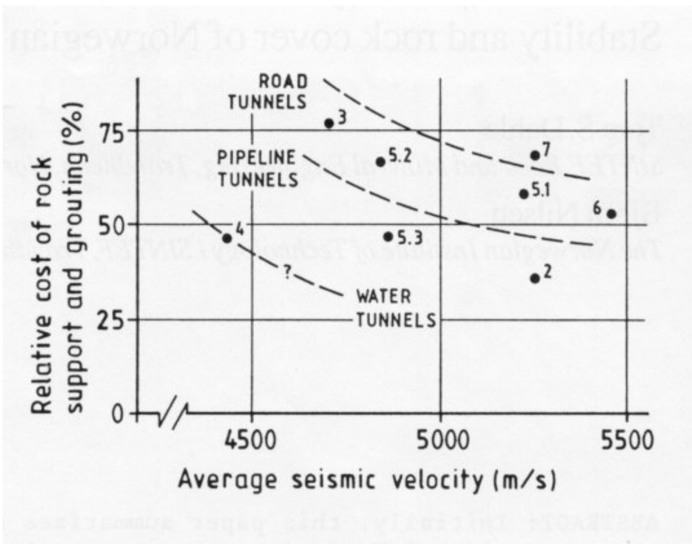

Fig. 1. Relative cost of rock support and grouting as a function of average seismic velocity (from Nilsen, 1989).

conditions, and on studying the effect of saline environments on the condition of rock support materials. While the total costs of preinvestigations for conventional tunnels under land in Norway are often less than 1% of the cost of excavation and rock support, the preinvestigation costs for subsea tunnels are normally between 5 and 10%. Refraction seismic profiling normally represents the most ex pensive part of the investigations, but is also the key investigation method for predicting subsea rock quality. A relatively good correlation was found between tunnelling conditions and rock quality prognoses based on the preinvestigation results. It was also concluded that all major discrepancies have geological explanations. Fig. 1 illustrates the correlation between average seismic velocity and the relative cost of rock support and grouting

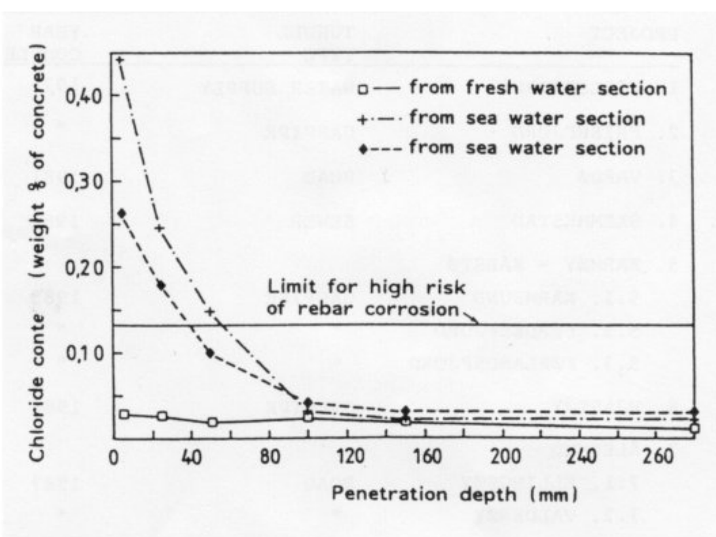

Fig. 2. Chloride penetration profiles in cast concrete, examples from the Vardø tunnel (from Nilsen, Maage, Dahlø, Hammer & Smeplass, 1988).

(in percent of total tunnelling cost). As can be seen, the average seismic velocity gives a good indication of the degree of complexity and cost level of the project, but the type and the cross sectional area of the tunnel also have a major impact. A typical total cost per meter of two-lane (approx. 50 m^2) subsea road tunnel in Norway today is from

NOK 30,000 to 40,000 (US $ 4,300 - 5,700 approx.).

No alarming corrosion or deterioration of rock support materials was identified. In sections with seeping seawater on the concrete surface, however, the chloride content was found to be higher on the exposed surface than the generally accepted limit for a high risk of rebar corrosion (see Fig. 2). For a more detailed presentation of the SINTEF state-of-the-art review, see Nilsen, Maage, Dahlø, Hammer & Smeplass (1988) and Nilsen (1989).

Since 1989, the Royal Norwegian Council for Scientific and Industrial Research (NTNF) has supported the research activities on sub-sea rock tunnels. Particular emphasis has been put on evaluating stability and optimum rock cover. In the following sections, some preliminary results will be discussed.

2. DESIGN PRACTICE

Subsea rock tunnels have become increasingly popular in Norway, particularly for road purposes. Since 1988, four tunnels have been completed (see Table 1), two are under construction, and almost 20 are under planning or consideration. The plans include several very challenging projects. The extremity is represented by the Hareid tunnel, which has a planned length of 132 km and a maximum depth of 630 metres below sea level.

As a result of national regulations, the maximum gradient for a road tunnel in Norway is in most cases limited to 1:12.5. Hence, the parameter which will mainly define the length of a subsea tunnel, is the minimum rock cover. If the rock cover decision is too conservative, considerable extra costs due to extra tunnel length will be the result. For a typical Norwegian subsea road tunnel (cross sectional area 50 m^2, declination 8%), a reduction of the minimum rock cover of only 1 meter may represent a cost reduction in the order of NOK 1 mill.

(US $ 140,000 approx.). However, if the rock conditions are poor and the rock cover too small, severe stability problems and large water inflow may be the result as discussed in section 3.

An important basis for the planning of new tunnels is represented by the results and experience from completed projects. Because of the good results from sub-sea tunnelling so far, the general trend for new projects has been a gradual, but slow reduction of minimum rock cover as indicated by the diagram in Fig. 3.

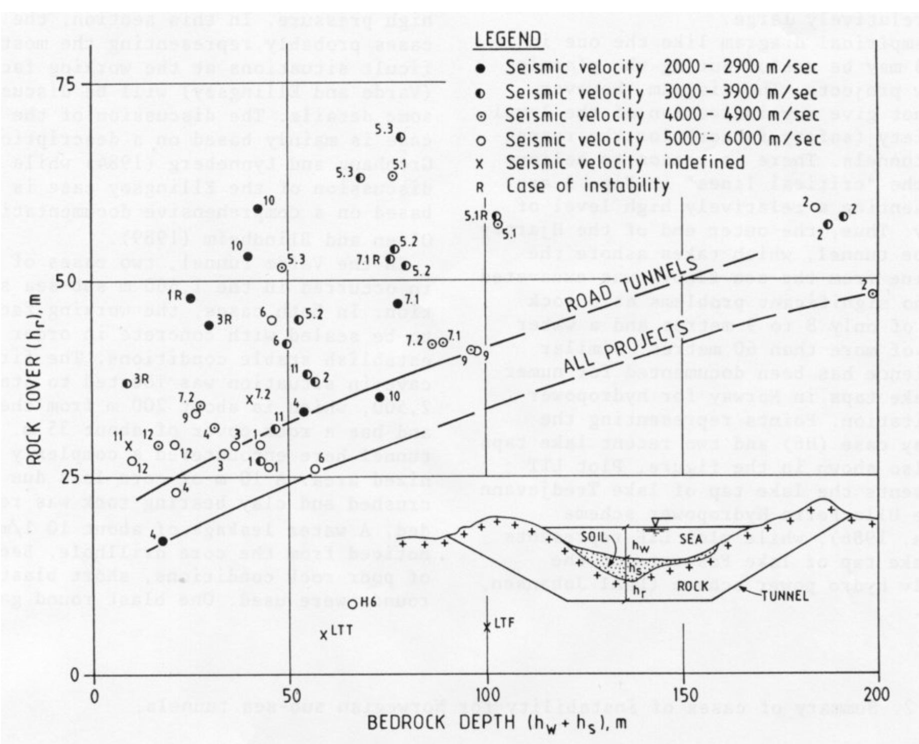

Fig. 3. Minimum rock cover under sea as function of the bedrock-depth

In Fig. 3 each point represents a section of critical rock cover for the respective tunnel. The numbering refers to the list of tunnels in Table l. The term "Critical rock cover" refers in most cases to tunnelling underneath clefts or other depressions in the sub-sea bedrock, and does not necessarily coincide with the deepest parts of the fjord. The "Bedrock depth" represents the sum of water depth and soil thickness on the sea floor. The plots are all based on reports from geo- physical and geological investigations for the respective project.

Based on the plots, lines representing the minimum rock cover as a function of the bedrock depth may be defined as shown in Fig. 3. As can be seen, the minimum rock cover for completed road tunnels is generally 6 to 7 metres greater than it is for the smaller pipeline and sewer tunnels.

Fig. 3 clearly demonstrates that very often the location of critical rock cover coincides with zones of poor quality rock (seismic low velocity zones). This is a logical result of the geological pre-history of the fjords and straits, and is certainly a fact which should be kept in mind when planning future projects. As can also be seen from the diagram, the rock cover in cases of instability so far has been relatively large.

An empirical diagram like the one in Fig. 3 may be useful during the planning of new projects. The diagram, however, does not give any indication of the level of safety (safety factor) for the respective tunnels. There is reason to believe that the "critical lines" in Fig. 3 are representing a relatively high level of safety. Thus, the outer end of the Hjartøy oil pipe tunnel, which takes ashore the pipeline from the sea floor, was excavated with no significant problems at a rock cover of only 8 to 9 metres and a water dept. of more than 60 metres. Similar experience has been documented for numerous lake taps in Norway for hydropower exploitation. Points representing the Hjartøy case (H6) and two recent lake taps are also shown in the figure. Plot LTT represents the lake tap of lake Tredjevann (literally: Thirdwater) of the Ulla Førre Hydropower scheme (Krogh, 1986), while plot LTF represents the lake tap of lake Fossvatn of the Kobbelv hydro power scheme (Dahl Johansen, 1987).

3. STABILITY PROBLEMS

SINTEF research has clearly shown that stability problems due to faulted and crushed rock represent a threat to hard rock sub-sea tunnel projects.

TUNNEL	CROSS SECTION m^2	WATER DEPTH m	ROCK COVER m	COMMENTS
Vollsfjord	16	14	40	Collapse after water filling
Vardø	53	10	35	Cave-in at working face
Vardø	53	20	45	Cave-in at working face
Ellingsøy	68	70	45	Cave-in at working face
Karmsund	27	80	55	Cave-in avoided
Slemmestad	10	50	35	Rock fall/cave-in tendency

Table 2. Summary of cases of instability for Norwegian subsea tunnels

As shown in Table 2, two cases of cave-in at the working face occurred in the Vardø tunnel, and one case in one of the Ålesund tunnels (Ellingsøy). One case of minor instability occurred in the Slemmestad tunnel (SRV), and in the Karmsund tunnel stability problems were encountered which might have led to severe problems. The Vollsfjord tunnel, which is the water supply tunnel to a petrochemical plant, collapsed in several locations shortly after it was taken into use.

The major problems have in all cases been caused by faulted rock carrying clay minerals and water leakages of relatively high pressure. In this section, the two cases probably representing the most difficult situations at the working face (Vardø and Ellingsøy) will be discussed in some details. The discussion of the Vardø case is mainly based on a description by Grønhaug and Lynneberg (1984) while the discussion of the Ellingsøy case is mainly based on a comprehensive documentation by Olsen and Blindheim (1989).

In the Vardø tunnel, two cases of cave- in occurred in the 1,660 m sub-sea section. In both cases, the working face had to be sealed with concrete in order to establish stable conditions. The first cave-in situation was located to station 2,500, which is about 200 m from the shore and has a rock cover of about 35 m. The tunnel here encountered a complexly tectonized area. A 10 m of core loss due to crushed and clay bearing rock was recorded. A water leakage of about 10 l/min was noticed from the core drill hole. Because of poor rock conditions, short blast rounds were used. One blast round gave an over break at the upper part of the working face, and leakages from the roof resulted in no attachment between the shotcrete and the rock. After mucking out some of the rock debris from the blast, the casting steel shield was shoved towards the tunnel face. A pile of rock was placed in the inner part of the tunnel close to the tunnel face. This was done to reduce the time and efforts necessary to seal up the cave-in area with a concrete plug. The cave-in developed before the casting could start, and crushed rock was constantly dropping onto the steel shield during the operation.

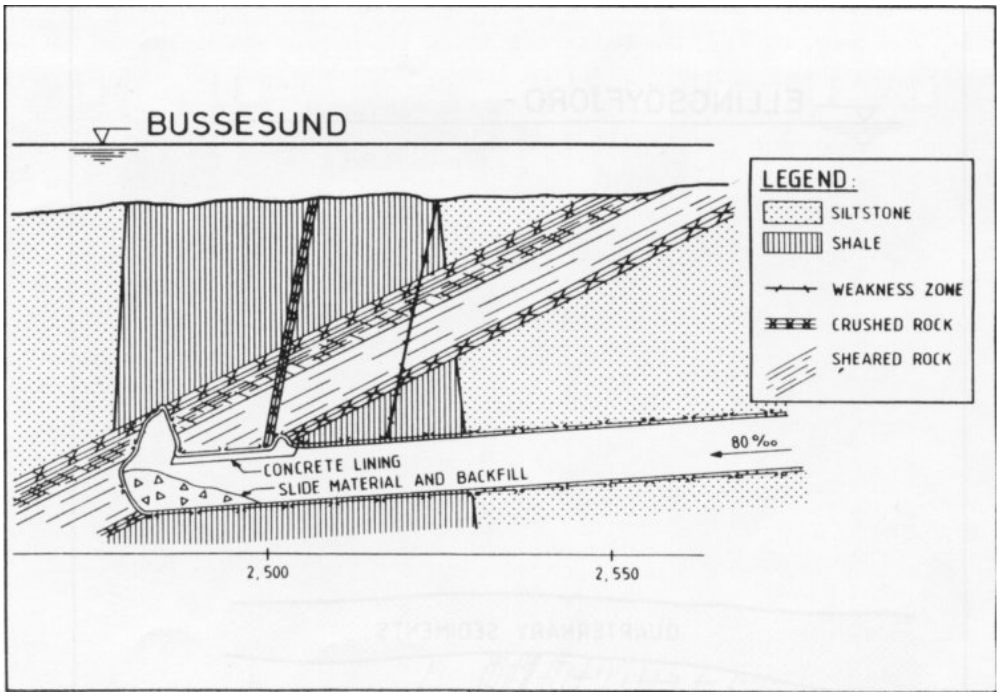

Fig. 4. Cave-in at Vardø, station 2,500. Sketch mainly based on Grønhaug and Lynneberg (1984).

The problem was even worse during excavation through the main fault zone (station 2,090 to 2,115). The tunnel was about 1,1 km from Vardø island, which implies a rock cover of about 45 metres and a sea depth of 20 metres. Because of cave-in, the inner part of the steel casting shield was lost as it became necessary to seal up the tunnel about 1 meter from the inner end of the shield. In this zone, the rock conditions were so extreme that the loader got stuck during loading of the trucks. At the Ellingsøy tunnel, a cave-in at the working face occurred in the main fault area. In this area, several weakness zones were identified during the pre- investigations.

The working face was located at station 9,900, about 700 metres from the shore of Ellingsøy. The rock cover was about 45 m, and the sea depth was about 70 m.

A 2.5 meter blast round was done to shape up the tunnel face before casting. Within a short time however, rock fall from the working face developed. Shotcreting was not successful because of seeping water in combination with clay, which gave no attachment to the rock. The weakness zone was beyond reach for spiling. Within 6 hours, a cave-in was developed to about 7 metres above the tunnel roof. It was then decided to seal the working face with a concrete plug from the inner part of the casted section. The resulting plug was approximately 7 m long, containing about 700 m^3 of concrete.

Excavation through the concrete plug and the weakness zone, a total of 20 metres, was carefully done within 5 weeks.

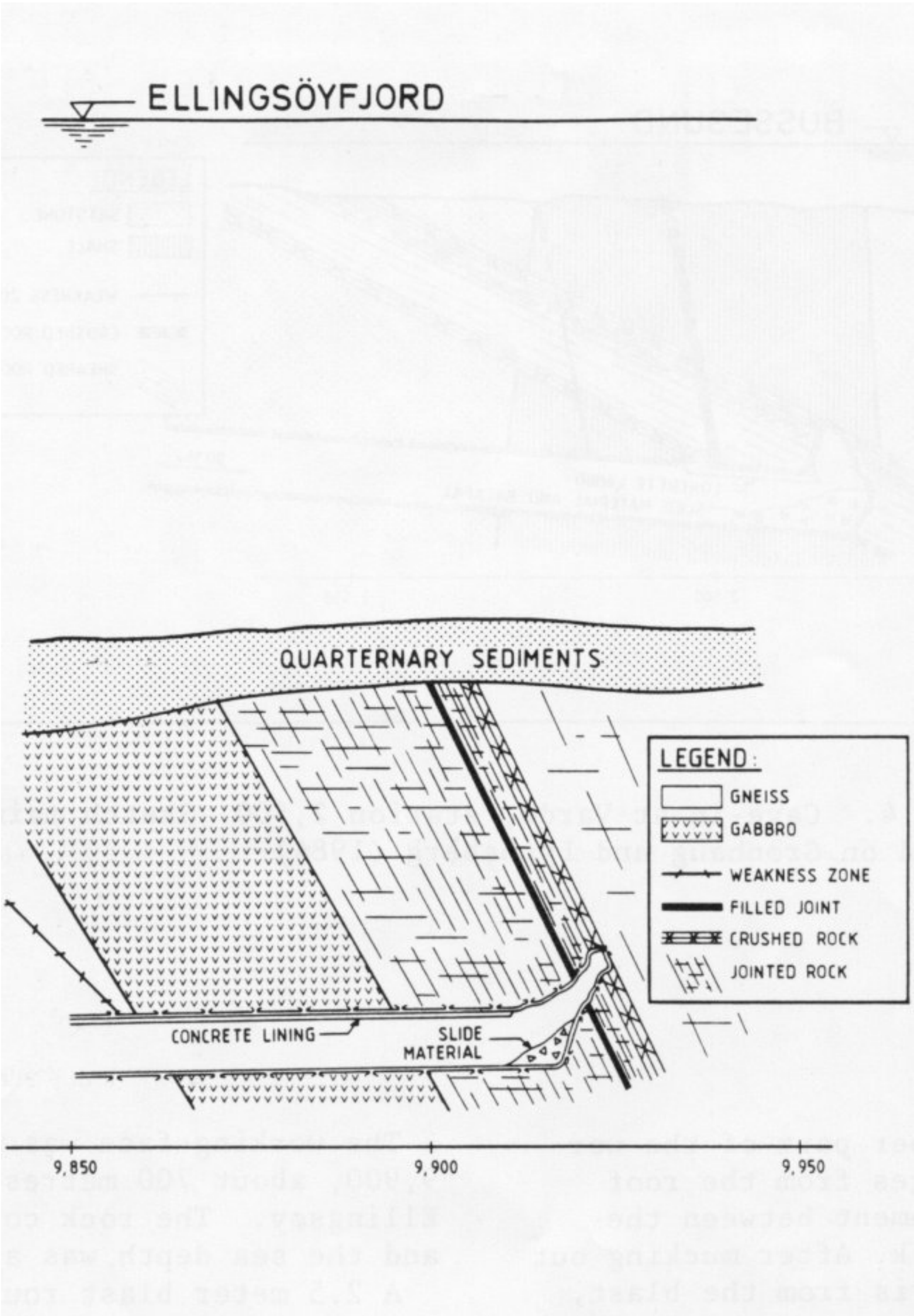

Fig. 5. Cave-in at Ellingsøy tunnel (Based mainly on Olsen and Blindheim 1989).

4. EXCAVATION AND REINFORCEMENT METHODS

As a consequence of the poor rock conditions associated with sub-sea tunnelling activities, the procedures for tunnelling through extremely poor rock conditions are constantly improving. A stepwise procedure of the following character is followed:

1. Exploratory drilling program: The drilling programme includes both core drilling as well as percussive drilling.

2. Grouting: For leakages over a certain level, grouting is performed.
3. Drainage: For cases with a combination of poor rock conditions and water, drilling for drainage of the rock ahead of the tunnel is considered. At Karmsund, this system was established by 8 to 10 drill holes with length from 15 to 20 metres.
4. Spiling: Spiling is done with a bolt spacing of 0.3 to 0.5 m, length 6 to 8 metres. The boreholes are drilled fromsome metres behind the working face, with an angle of about 15 to 20° to the tunnel axis. The bolts are grouted if possible.
5. Short blast round: Blast round lengths down to 0.8 m have been used. The tunnel cross section may be divided into separate blasts.
6. Rapid reinforcement: All exposed rock surface at the working face might be shotcreted immediately after blasting (before the mucking operation is started). Steel fibre reinforcement is used.
7. Mucking out.
8. Additional rapid reinforcement: When necessary, the rock surface that is exposed after the mucking out operation is shotcreted. This may be supplemented by radial rock bolting of the roof. At Vardø, tunnel spoil was piled up against the working face.
9. Cast concrete lining: The last operation is reinforcement by means of concrete lining, in which the upper part of the working face may be included (Vardø).

The philosophy is typically "design as you go", continuously adjusting from one meter of tunnel to the other, according to the rock quality that is encountered.

5 DISCUSSION AND PLANS FOR FURTHER RESEARCH

The most unstable rock conditions are re- presented by the major faults which often are located approximately in the middle of the fjord. Therefore, the most severe problems are generally encountered in a late stage of the tunnelling and shortly before tunnel breakthrough. Partly because the tunnelling conditions so far might have been relatively good, the problems may come as a surprise. Even though problem areas are expected from the geological preinvestigations and core drilling ahead of the tunnel is carried out, the problems may be underestimated. If instability occurs, it may take several days before the situation is stabilized, although it is a matter of hours to establish a proper sealing of the cave-in area by piling up with rock and/or using a mobile steel shielding that is specially constructed for the purpose.

A diagram such as Fig. 3 may be useful as input to a general cost analysis study for subsea tunnels. For the actual de- sign, however, w hen optimum rock cover is addressed, this should be

TUNNEL Station No	Vollsfjorden 775	Vardø 2,500	Vardø 2,100	Ellingsøy 9,900	Karmsund 1,600	SRV 650
Rock	gneiss	shales	sandst	gneiss	meta.s.st	gneiss
Seismic velocity (m/s)	2,700	≥3,300	2,500	2,800	3,500	≥2,200+
Thickness of zone (m)	20	10	30	5	80	≤10+
Core loss (m)	0	0	-	-	0	-
Water loss (Lugeon)	0-8	0-0,4	-	-	0-3	-
RQD	+	0-20	-	-	25-80	-
Minerals	1,2,4,5,7	2,6,7	-	-	1,2,3,5,6,7	-

+ Uncertain data.
- Core drilling not performed.
Minerals; 1: swelling clay 2: chlorite 3: talc 4: mica 5: altered siderock 6: quartz 7: calcite.

Table 3. Data from preinvestigations

supplemented by more advanced rock mechanical analyses in which knowledge of the rock conditions in the critical area are implemented.

For the cases of instability in Norwegian subsea tunnels which have been studied, the main facts about the actual problem zones which were known before tunnelling are listed in Table 3.

As shown, the weakness zones in question normally have a thickness less than 15 m, and a seismic velocity lower than 3,500 m/s. Core drilling had indicated zones of relatively moderate permeability, as well as RQD values down to zero.

TUNNEL:	Vollsfjorden	Vardø	Vardø	Ellingsøy	Karmsund	SRV
Station No	775	2,500	2,100	9,900	1,600	650
CORE DRILLING AHEAD OF TUNNEL						
Core loss (m)	*	9	10	2	*	*
Water inflow (1/min/borehole)	*	≤10	*	≤30	*	*
DESCRIPTION OF WEAKNESS ZONE						
Thickness of zone (m)	≤10	≤15	15	≤10	≥100	≤10+
Dip (°)	60	35	60	70	75(?)	30+
Q-value	*	≥0.015	≥0.015	*	*	≥0.05+
Minerals	1,2,4,5,7	1,2,6,7	1,2,6,7	1,2,3	2,5,6	1,2,5+
Stand up time (hours)	*	≤0.5	≤0.5	≤2	*	≤60+
CAVE-IN DESCRIPTION						
Max height above roof (m)	*	7	*	≤10	-	-
TUNNELLING THROUGH THE ZONE						
Length of blast round (m)	*	≥0,8	*	≥2	≥1	*
Excavation rate (m/week)	*	5	2,5	3,5	10-15	*

* Information not known.
\+ Uncertain data.
\- No cave-in developed.
Minerals; 1: swelling clay 2: chlorite 3: talc 4: mica 5: altered siderock 6: quartz 7: calcite.

Table 4. Data from tunnelling.

Pertinent data from tunnelling through cave-in areas are listed in Table 4. As can be seen, the Q-values indicate "extremely poor" rock mass. The stand up time is very short according to norwegian standards. In our continuing research on hard rock sub-sea tunnels, we will focus on methods for optimising rock cover. As indicated by Fig. 3, the rock cover for sub-sea tunnels generally seems to be conservative. There have been several hundred lake taps in Norway throughout the years. Large water leakages have occasionally represented a problem for drilling and charging a lake tap blast round w hen the distance to the water reservoir has been small (10 metres or less). To our knowledge, however, severe stability problems due to rock cover have never occurred. Apparently, minimizing rock cover is not so much of a technical problem for solid rock, but rather a question of economics as discussed in the paper by Lli Ming and Nilsen on this symposium.

Further SINTEF research on sub-sea tunnels will also concentrate on developing models appropriate to the type of problems that are associated with faulted zones. Classical geotechnical methods will be applied, with emphasis on the ground water flow regime. Such analyses, however, have to be combined with engineering geological analyses of geological structures and tectonic elements.

For future subsea tunnels, a major challenge is also to improve the techniques for identifying the locations as well as the extensions and the orientations of faulted zones in complex tectonic areas. More advanced geophysical methods may become an important contribution to the solution of this problem.

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Water control in sub sea road tunnels in rock

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ABSTRACT:

The paper describes the principles in design and the experiences with water control in the sub sea road tunnels that have be en completed up to now in Norway, totalling more than 20 km. The deepest tunnel has been 155 m.b.s.l. Recent developments in the preinvestigation methods are discussed, and the systematic methods for probe drilling and pregrouting described. In all tunnels large water inflows have been avoided, and the remaining leakages have be en reduced to an acceptable level for pumping.

1. INTRODUCTION

1.1 Completed tunnels

Up to now 7 sub sea road tunnels in rock have been completed in Norway. All of them have replaced ferries over fjords or straits, and are now a common part of the public road system. The first, the Vardø tunnel opened in 1982 in the NE part of the country, was financed over the ordinary public budget. The others are financed in total or partly by private loans, paid back by toll systems. The basic data for the completed tunnels are given in table 1.

Tunnel	Length km	Max. depth m	Max. grade %	Compl.
Vardø	2.6	87	8.0	1981
Ålesund 1)	3.5	140	8.5	1987
Ålesund 2)	4.2	137	8.0	1987
Kvalsund	1.5	55	8.0	1988
Godøy	3.8	155	10.0	1989
Hvaler	3.7	120	10.0	1989
Flekkerøy	2.3	105	10.0	1989

1) Ålesund-Ellingsøy
2) Ellingsøy-Valderøy

Table 1. Completed tunnels

The Ålesund tunnel project soon became a reference project due to the many method developments. This included systematic refraction seismic surveys in grids for the tunnel route optimisation, extensive use of probe drilling by percussive drilling ahead of the tunnel face, very short completion time due to installation work done before breakthrough, etc. Problems were encountered with face collapse in very difficult rock conditions, and because of change of fire-protection standards during construction. The experiences have be en widely published, see reference list. For all tunnels except the first the construction period has be en 2 years or less. Except for the Ålesund tunnels, that have 3 lanes in the slopes (75 m^2), the other tunnels are all build in low traffic connections.
Because of this they only have two lanes and a cross-section of about 50 m^2. The latest 3 are not a part of the main road system, but are rather acting as side connections, and have a vertical slope of

maximum 10% (1:10) This quite austeric standard is used to reduce the necessary tunnel length and by this the total cost of the project. For these tunnels the real challenge has been the further development of cost effective construction methods, and to find optimal solutions regarding maintenance.

1.2 Tunnels under construction

Construction has now started on both longer and deeper tunnels, see table 2.

Tunnel	Length km	Max. depth m	Max. grade %	Exp. compl.
Nappstraumen	1.8	60	8.0	1990
Fannefjord	2.7	105	10.0	1991
Freifjorden*	5.2	130	9.0	1992
Maursundet	2.3	95	10.0	1992
Byfjorden*	5.8	220	8.5	1992
Mastrafjord*	4.3	130	8.5	1992

Table 2. Tunnels under construction
** 3 lanes in the slopes*

The tunnelling market in Norway is very competitive. The Public Roads Administration is monitoring and continuously improving the bidding documents for all new contracts.
The exchange of news about techno- logical improvements is also quite open. A further rapid development of efficient methods for investigations, design and construction can therefore be expected. A number of tunnels are under systematic planning, with lengths up to 15 km and depths down to 600 m. It may be estimated that before the decade is over 25 tunnels will be completed, totalling more than 100 km.

2. DESIGN PRINCIPLES

2.1 Drained constructions

All tunnels are designed to operate as a drained construction. Where concrete lining is used, it is not designed for full water pressure. Considerable effort is however spent to reduce the quantity of in-leaking water to a level acceptable for pumping. This level has usually been set at 300 l/min per km tunnel. This is achieved by probe drilling and pregrouting and will be described later.

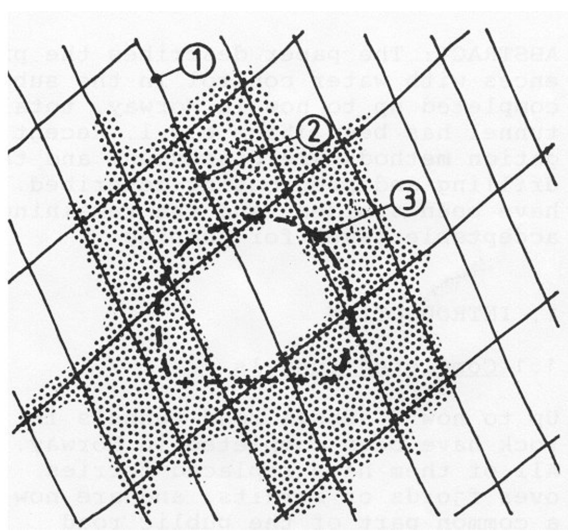

Figure 1. Schematic cross section
1. Jointed rock mass with full water pressure
2. Pregrouted zone
3. Tunnel profile

The full water pressure in joints etc in the rock mass is taken up by a grouted zone outside the tunnel, typically 1-5 m from the tunnel contour, depending on the conditions.
Figure 1 shows the basic principles of this design.

Usually an inner water shielding, collects and leads the remaining in-leaking water to the drainage pipes, see Figure 2. Any water leaking up in the unlined floor will follow the drainage layer until cut off by cross drainage ditches. Finally it ends up in the pumping station, from where the salt but usually clean water are pumped up and into the fjord where it came from.
Typical elements in this system are

1. 45 mm polyethylene sheets mounted on short rock bolts, or corrugated plates of heat galvanized epoxy-coated steel or burn-painted aluminium
2. Crushed rock filter (15-30 mm) in ditches and in a 300 mm layer below the road pavement.
3. 1-2 150-200 mm PVC pipes in drainage ditches on one or both sides
4. Pumping station with pump sump buffer for 24 hours leakage in case of power loss
5. One stage pumping (down to 200 m) in 160 mm PE-pipes, and most often through a grouted and re-drilled hole from the shore

The water leakages are mostly reduced to dripping leakages by pregrouting. Any shotcrete lining is therefore usually applied directly on the rock without
any drainage channels. Because of this, in spots with heavy dripping, the water will either penetrate the fresh shotcrete and make it locally permeable, or a hole is later drilled through the shotcrete to release water pressure and concentrate the influence. By this the shotcrete could locally and slowly develop a reduced quality. If the rock otherwise are considered stable, a limited crumbling from the shotcrete could be allowed on the sheets. If the shotcrete is applied as a thicker lining in order to carry load from loose rock in crushed zones etc, drainage channels are mounted and further layers of shotcrete added.
Where it is necessary to use in-situ cast concrete lining up to the face due to fault zones with swelling clay, the same approach is usually applied. Only concentrated leakages are gathered in flexible drainage channels before concreting. The shrinkage of the concrete usually leaves a void or at least some channels behind the lining, giving some drainage. Water stops are sometimes used in the section joints, but as this .is time consuming it has become more seldom. Grouting hoses are also in use, but more often the concrete lining is now considered as purely for stability (support) reasons, and joints between sections are left as possible drainage channels. This increases the area of the inner shielding, but the total costs may be reduced.
It should however be mentioned that this rather unsophisticated approach has a background in the fact that for most tunnels the fully concrete lined sections are usually less than 10%, typically 2-5% of the total length. Thus, the increase in water shielding, be- cause of the concrete lining not being made water-tight, is usually insignificant compared to the cost and time savings by a simpler construction of the concrete lining in the demanding working conditions at a face in unstable ground.
This approach is however not used, when a water tight concrete inner lining is used as a mean of avoiding pore pressure reduction under settlement sensitive clay etc under cities.

3. GEOLOGICAL CONDITIONS AND PREINVESTIGATIONS

3.1 Rock types

The geological conditions have varied a lot. Even if most of the tunnels have been built in hard rock formations, significant sections have been excavated in rock mass of very low quality. The obvious reason is that most of the fjords have be en formed by glaciation along major fault systems.

3.2 Weakness zones

Weak zones are often present in the form of crushed zones with swelling clay and broken rock material. The tectonic pattern can be rather complicated along the coast, with horst & graben systems or folding and over thrust zones. Many of the rock types involved, granites, gneisses, metamorphic schists etc are rather stiff. The tectonic displacements that have taken place, especially during the 2 km upheaval of the crust-plate in southern Norway, have therefore in many areas left joint sets with a quite open character.

3.3 Occurrence of water

Quite often the largest water leakages are found on relatively limited sections along the tunnels. This may be in zones with open cracks along large steps in the coastal topography, believed to be representative of quite recent faulting, too young for having permitted enough mineral precipitation to fill the joints. Such joints may have openings of up to 2-5 cm, capable of carrying catastrophic volumes of water, if intersected by a tunnel before grouting has be en performed.

Moderate water leakages may some- times be present over long sections of a tunnel, and most often are the dominating leakages then coming from one joint set. This could typically be a sub vertical joint set intersecting the fault zones in the area at medium angles. They can extend over long distances. Mostly they are open with clean surfaces, clearly indicating tensile or extension failures.

For such joints the leakage is little or not at all dependant on the depth below surface. Generally small seepages from other joints show a clear tendency to decrease in leakage volume for overburdens of 30-50 m or more.

Along some joints, that are partly filled by mineral precipitation, (calcite is quite common) the leakages often fallow channels along the joints, typically with 1-5 mm openings and widths 5-20 times as much.

The influence of rock overburden has been much discussed, but it is clear that for the joint sets with potential of giving large water inflows, the depth below surface is of little significance. From hydro- power developments it is also known that similar open joints have flooded tunnel faces at 500 m depth below surface.

Soil deposits above the rock over-burden have frequently shown an influence. For some tunnels it has been more important whether it has been a moraine deposit above the rock or not, than if the tunnel has be en below sea or land. This was the case for the Ålesund tunnels, where some of the tunnel sections were covered by moraine deposits.

Here the water leakages often stopped after some days, and were mostly reduced to slight dripping, indicating a local drainage of the rock mass below the moraine. The Godøy tunnel proved however that even an up to 70 m thick moraine can have more permeable material at the contact to the rock. It is also possible that even if the moraine is dense, the joint sets may be open enough to gather water sideways over long distances.

3.4 Preinvestigation methods

Besides the obvious purpose of gathering information about the rock surface, location and extension of weakness zones etc, a systematic grid of refraction seismic profiles may give a warning about at least large water leakages. Concentrated zones with seismic velocity less than 3000 m/s have often proved to consist of rather good rock but with several water bearing joints. This is in contrast to 5000-6000 m/s in massive rock and 3500-4500 in broken and jointed rock. Such indications can however only been taken as a warning.

Refraction seismic have been the most important geophysical preinvestigation method, but other methods are under development also Geoelectric measurements on weakness zones, where they cross up to dry land have been tried. Then readings of resistivity and induced polarization can give indications whether the weakness zones are typical fault zones with clay or are dominated by water bearing fissures. This was done first at the Ålesund project. Used in close cooperation with the geological preinvestigations this method can give useful warning about water bearing zones.

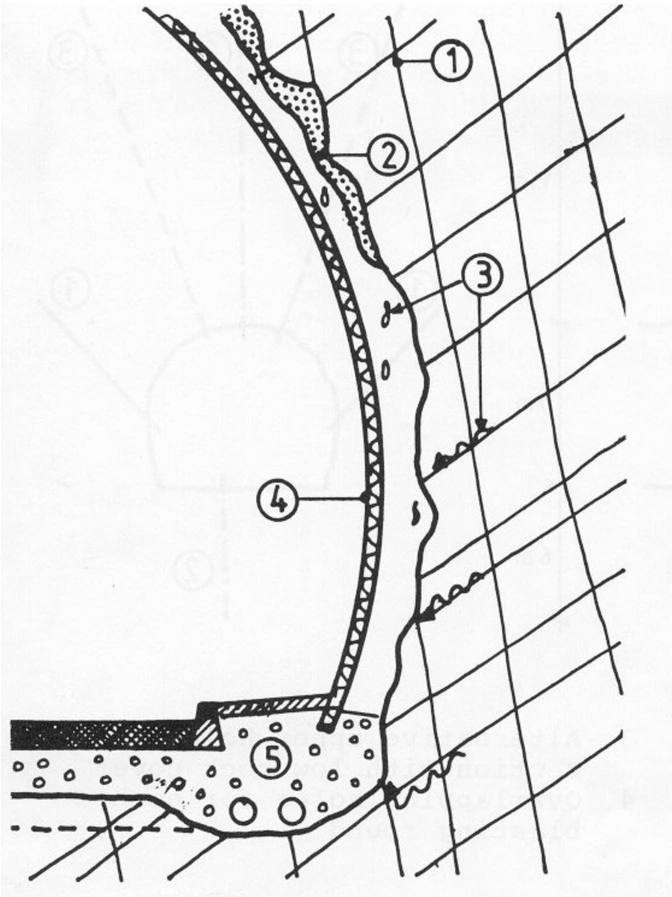

Figure 2. Shielding of remaining leakages
1 .Grouted rock mass
2. Shotcrete, usually without drainage channels
3. Remaining leakages
4. Shielding plates
5.Drainage ditch with pipes Mounting and rock bolts not shown

But all zones can obviously not be measured on land, and core drilling from floating platforms has proven time consuming, expensive and quite often difficult to interpret, even for large diameter cores from drill ships. Directional core drilling has therefore been tried. For the Maursundet project a 292 m long core hole has been drilled from one shore out below the sea bottom approximately along the tunnel axis. From this core hole to bottom seismic and resistivity tomography have been tried and may be developed into methods of interest also to locate possible water bearing zones.
However, the preinvestigations have up to now not been earned especially at the potential water leakages. This has in fact not be en consider- ed as feasible or cost effective, and has mostly been left for the probe drilling from the tunnel face to take care of.

4. CONSTRUCTION METHODS

4.1 Excavation method

All the tunnels have been excavated by drill & blast methods. Except for contour and floor holes, the charging is done with bulk ammonium nitrate-fuel-oil explosives. They are sensitive to water, and must be replaced by cartridge explosives it water bearing joints cross the blast holes.
Tunnel boring, machines, although frequently in use in other tunnels, have not been used for sub sea tunnels yet. The method are under consideration for at least one project, the connection of the North Cape plateau to the main
land. Here the rocks are broken and faulted sedimentary rocks, partly with a high leakage potential.

4.2 Probe drilling and grouting

The extensive use of probe drilling ahead of the tunnel face and efficient pre-grouting are really the most important factors for the successful completion of most of
the tunnels.
The basic principles are shown in Figure 3 and 4.
The probe drilling is performed routinely typically for each 5th blasting round by 2 to 5 percussion drill holes (length 30 m, diameter 45 mm) .They are drilled by the ordinary drilling jumbo, under careful observation of water in-leakage for each drill rod extension of 3 m.

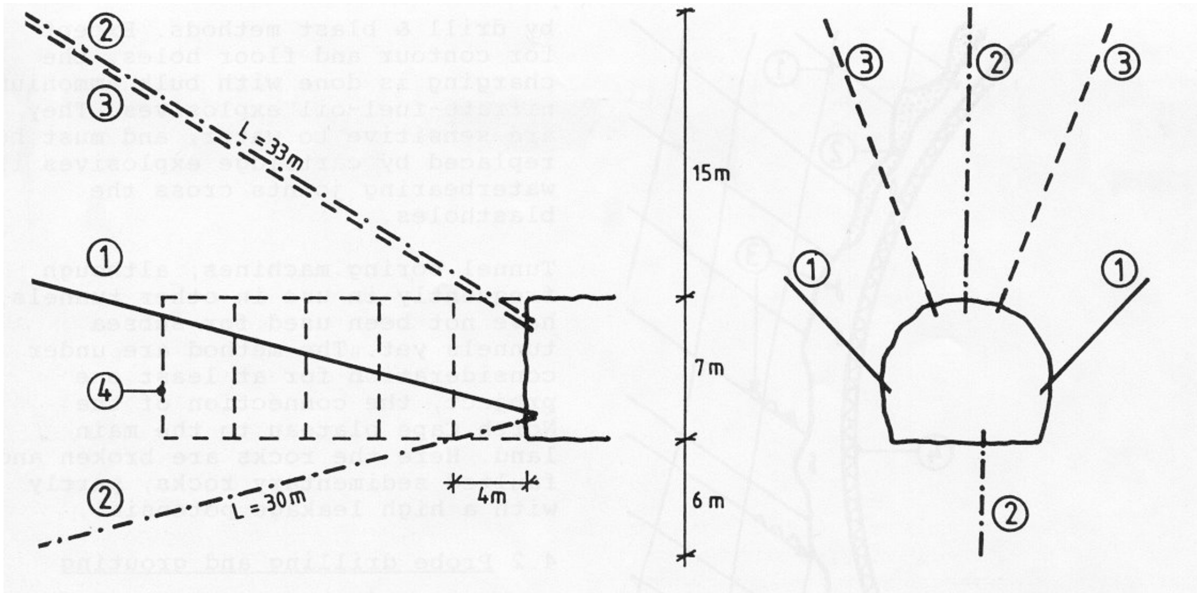

Figure 3.(above) Probe drilling ahead of the tunnel face
1. Probe holes at all sections covered by loose deposits on sea
2. Additional holes in weak zones
3 Alternative upper holes in section with low rock cover
4 Overlapping holes for each blasting round

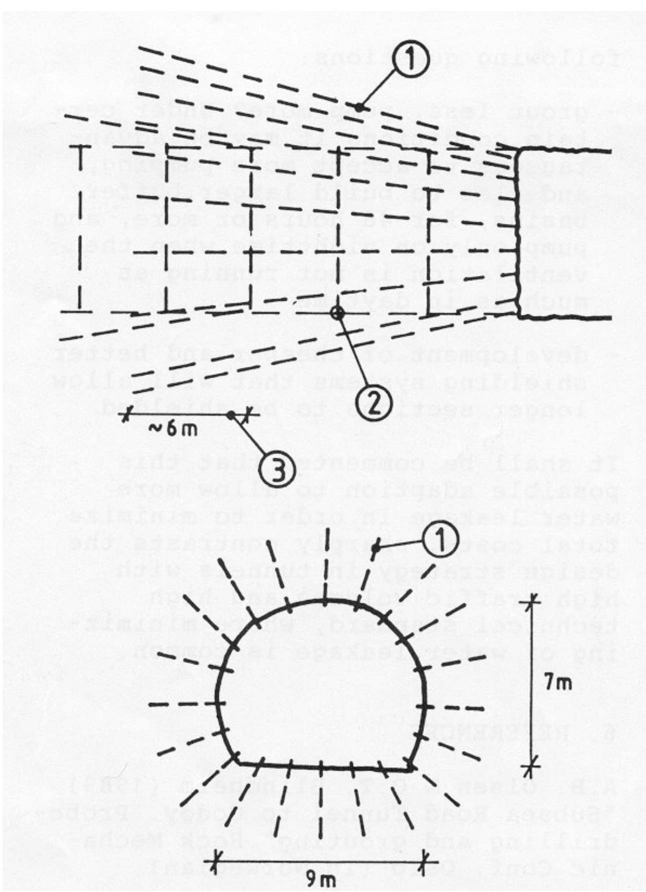

Figure 4.(left) Pregrouting, full cone of 18 m holes
1. 18 m percussive 45 mm holes
2. Blasting rounds
3. Typical overlap between cones

By this, open joints carrying large volumes of water are easily located with reasonable accuracy and quite of ten also zones with smaller leakages.
Water pressure tests are now seldom performed, as small leakages are of no significance for these tunnels. The decision procedures for pre-grouting are carefully developed and adapted to the situation. It is usual to decide pregrouting if the leakage from one or more probe holes exceeds 6 l/min and the distance from the leakage to the face is less than 10 m. If experience for the particular tunnel section shows that the leakages may be reduced over time, the limit may locally be set higher. Depending on the typical joint orientation the probe holes may

be angled more to the side to get a better intersection and earlier warning of possible leakages.
When pregrouting is decided it is usually first performed with a full cone of drill holes, as shown in Figure 4.
The lengths of the tunnel sections that have needed pregrouting have varied much. In the Kvalsundet tunnel, with no weakness or fault zones, no grouting was necessary at all. In the Godøy tunnel, with a lot of open tension cracks, 40% of the whole tunnel length was pre- grouted.
The necessary amount of grout and the total time for each round varies, in the Godøy tunnel between 1-50 t cement and 2.548 hours total time per round.
The grouting is usually performed with thin grout first (w/c =2), but if pressure is not developed, thicker grout is used. The cut off pressure is adapted to the situation, but 40 bar is usually used for ordinary cement (Rapid) .To allow earlier blasting, the grouting is of ten completed by use of a special fast hardening cement (Cemsil), and with cut-off-pressure of 60 bar. A limit for each hole is usually set at approx. 2t. If the joints obviously are very open, admixtures to enhance clogging are tried.
It is not possible to go into all the details in the procedures, but the overall target is to limit the grouting time, and avoid excessive grout consumption. Usually the costs of downtime will be about 2/3 of the total cost for grouting. It is therefore important with good pumping capacity, but also competent decision-taking at the tunnel- face during grouting.
Except for the first tunnel, where little pregrouting was used, and the remaining leakage is 1000 l/min per km, the remaining leakage is 100-300 l/min per km. Where data is available yet there is a tendency to a very slow reduction of the leakage.
The total amount of grout consumed has been from 0 to above 100 kg pr m tunnel in average, with the Godøy tunnel as the exception with more than 400 kg pr m tunnel.

4.3 Important results

It is an important conclusion that by systematic probe drilling, competent decisions and efficient pregrouting, it has been possible to completely avoid large and hazardous water inflows. Several zones with open joints have been passed that could have flooded the tunnels if they had not been pre- grouted.
Other special problems have been connected to the immediate stability of the tunnel face, in water bearing, clayish, crushed zones. Such problems have been treated
by grouting, drainage holes between the grouted zone and the tunnel contour, in combination with reduced blasting rounds and spiling with groutable anchors.
Geophysical probing with seismic tomography has proven as an extremely useful tool under such circumstances. It can give a 3-dimensional overview of the rock mass ahead of the face, allowing a tailoring of the necessary measures, to save time and costs and reduce risks.
Corrosion has up to now not given problems except for minor details Fibre reinforcement in shotcrete has shown a good durability where the shotcrete is not permeable.

5. FURTHER DEVELOPMENTS

5.1 Improved methods

For the preinvestigations the new and more reliable methods for directional core drilling are promising. They will allow a better evaluation of the geological conditions in general. They also offer the possibility of hole-to-bottom seismic, and even tomographic profiles allowing possible detection of zones with especially low seismic velocity, indicating water bearing fractures.
Geoelectrical measurements to classify the type of weakness zones may be developed further.
Core holes should probably be utilized more for water pressure test in order to verify geological/ tectonical interpretations regarding which joint set that is most open.

Research and analyses of the experiences, are well in progress, and have mostly been directed towards:

-durability of support exposed to salt water, especially concerning shotcrete application and quality specifications
-clogging of drainage system due to algae growth on sections with little slope
-work face collapse preventions in clay and water bearing zones

Regarding design strategies, consideration is now given to the following questions:

-grout less, pump more? Under certain conditions it may be advantageous to accept more pumping, and also to build larger buffer basins, for 48 hours or more, and pump only on night time when the ventilation is not running as much as in daytime.
-development of cheaper and better shielding systems that will allow longer sections to be shielded.

It shall be commented that this possible adaptation to allow more water leakage in order to minimize total costs, sharply contrasts the design strategy in tunnels with high traffic volumes and high technical standard, where minimizing of water leakage is common.

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Earthquake hazard at strait crossings.

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ABSTRACT:

Earthquake loading guidelines for onshore construction are not yet established in Norway, although such guidelines have been valid since 1984 for offshore Norway. From a seismological point of view, the costal areas of Norway are as exposed to earthquake loading as the offshore areas, and the different requirements on- and off-shore presently therefore represent conflicting views with regard to safety requirements at critical facilities like strait crossings. While the seismic activity in Norway normally is characterized as intermediate-to-low, there still have been some quite strong historical earthquakes, and several contemporary ones with magnitudes above 5. Such data in conjunction with well-established methodologies for earthquake hazard assessment offshore Norway provide therefore a sound basis for earthquake load assessments for earthquakes at annual exceedance probabilities of 10^{-2} and 10^{-4}. These loads are caused normally by the presence of shear waves and surface waves originating at distances up to a few hundred kilometres. Strait crossings, however, are also potentially affected by more long period ground motions from large distant earthquakes, which often are associated with standing waves in fjords and lakes. This particular phenomenon may be hazardous to long bridges and submerged tubes with large natural periods, and should be the subject to more in-depth research efforts.

1 Introduction

The need for the stringent safety measures offshore Norway has been recognized as important by regulatory agencies as well as operators also in the context of earthquake hazard. This led to the definition of earthquake load as being an environmental type of load, and official guidelines have consequently been established by the Norwegian Petroleum Directorate (NPD 1987) for the determination of ultimate limit state loads on fixed offshore constructions.

The regulations prescribe earthquake hazard to be considered at annual exceeding-probabilities of 10-2 and 10-4. The design load for the highest probability should be absorbed by the construction without structural damage, while at the lower probability, progressive collapse of the structure should be prevented under the design load. The annual probabilities of exceedance (often expressed in terms of return period) of the design loads are key parameters determining the acceptable risk with respect to structural performance and loss of lives, respectively.

The damages from the October 17, 1989, San Francisco {Loma Prieta) earthquake is one of the most recent examples of the vulnerability of advanced communication (lifeline) facilities like highways and bridges, which experienced major structural collapse and loss of lives.

Onshore construction in Norway lacks the regulatory basis for earthquake resistant design. The present building code (Norsk Standard 3479) classifies earth- quake load as accidental, and this type of load is therefore normally not considered during the design, even of critical facilities. Since earthquakes occur onshore as well as offshore, with about equal strength, and because oil and gas pipelines already cross from offshore to onshore areas, the present situation with different design requirements on- and offshore is regrettable from the point of view of the society's safety needs. Future planning of critical facilities within the field of communications, the energy sector, and high rise buildings, should therefore include earthquake resistant design in accordance with current offshore practice. An initiative to this effect has recently been taken by The Norwegian Society for Earthquake Engineering, to encourage the discussion and possible implementation of such practice.

The current work with Eurocode 8 will hopefully stimulate the development of codes and practice for onshore earthquake resistant design in Norway.

2 Earthquakes in Norway

Large historical earthquakes in Norway are confined almost exclusively to the coastal waters of Norway. The locations of some of the largest ones are shown in Fig. 1 (Wood and Woo 1987). With exception of a magnitude 6.1 earthquake in the Lofoten Basin in 1929 (felt strongly in northern Norway), the largest historical earthquakes in Norway are below magnitude 6. Of these, the 1819 Lurøy earthquake of magnitude 5.8 on the Helgeland coast is the largest near shore event known to occur in north-western Europe in the past two centuries. The earthquake caused damage to houses, landslides, and heavy rock fall from the mountains, and was widely felt throughout Scandinavia. Another large event in northern Norway, with magnitude 5.4 off the Lofoten Islands in 1894, caused the Danish 40 ton schooner Henrietta to spring a leak and finally to sink.

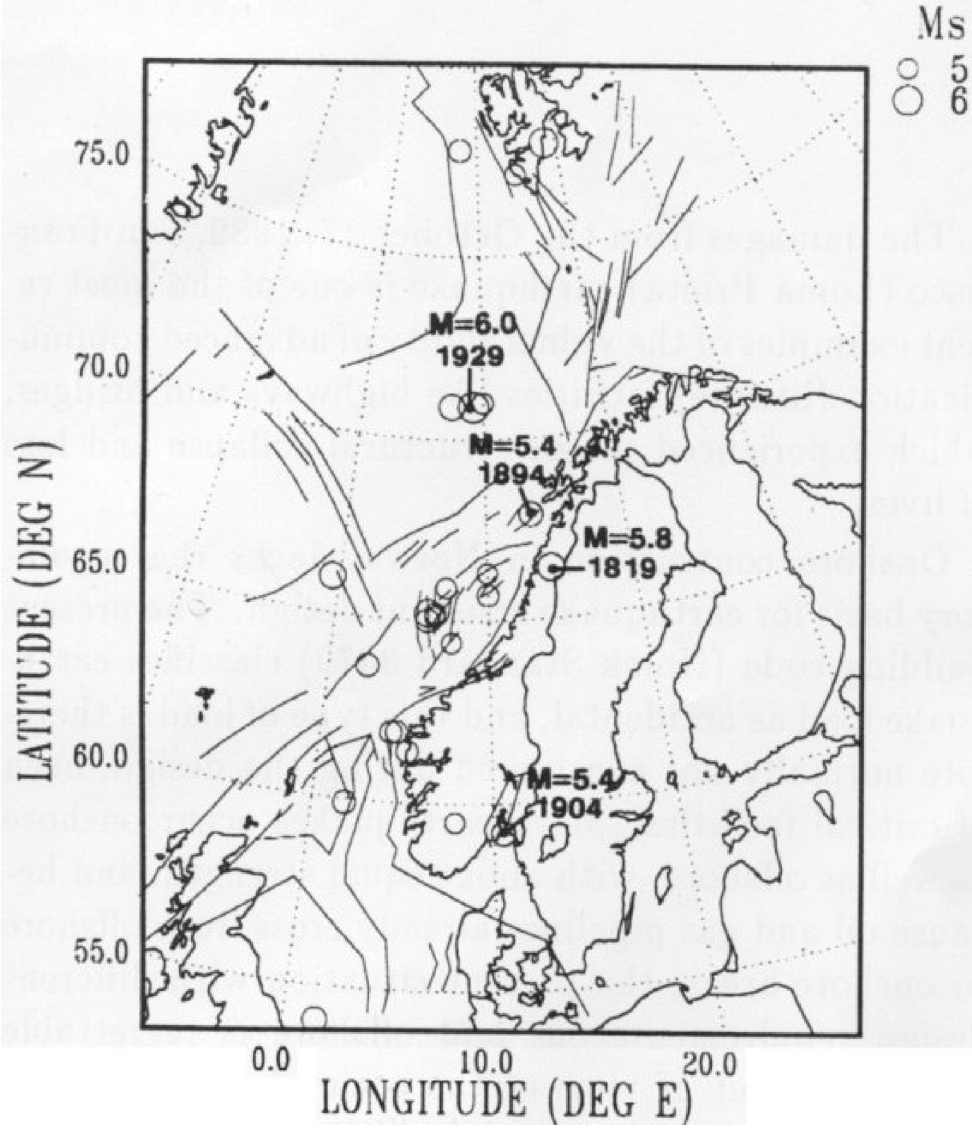


Figure 1: Historical earthquakes in Norway and surrounding areas (time period 1800-1989, magnitudes above 5.0).

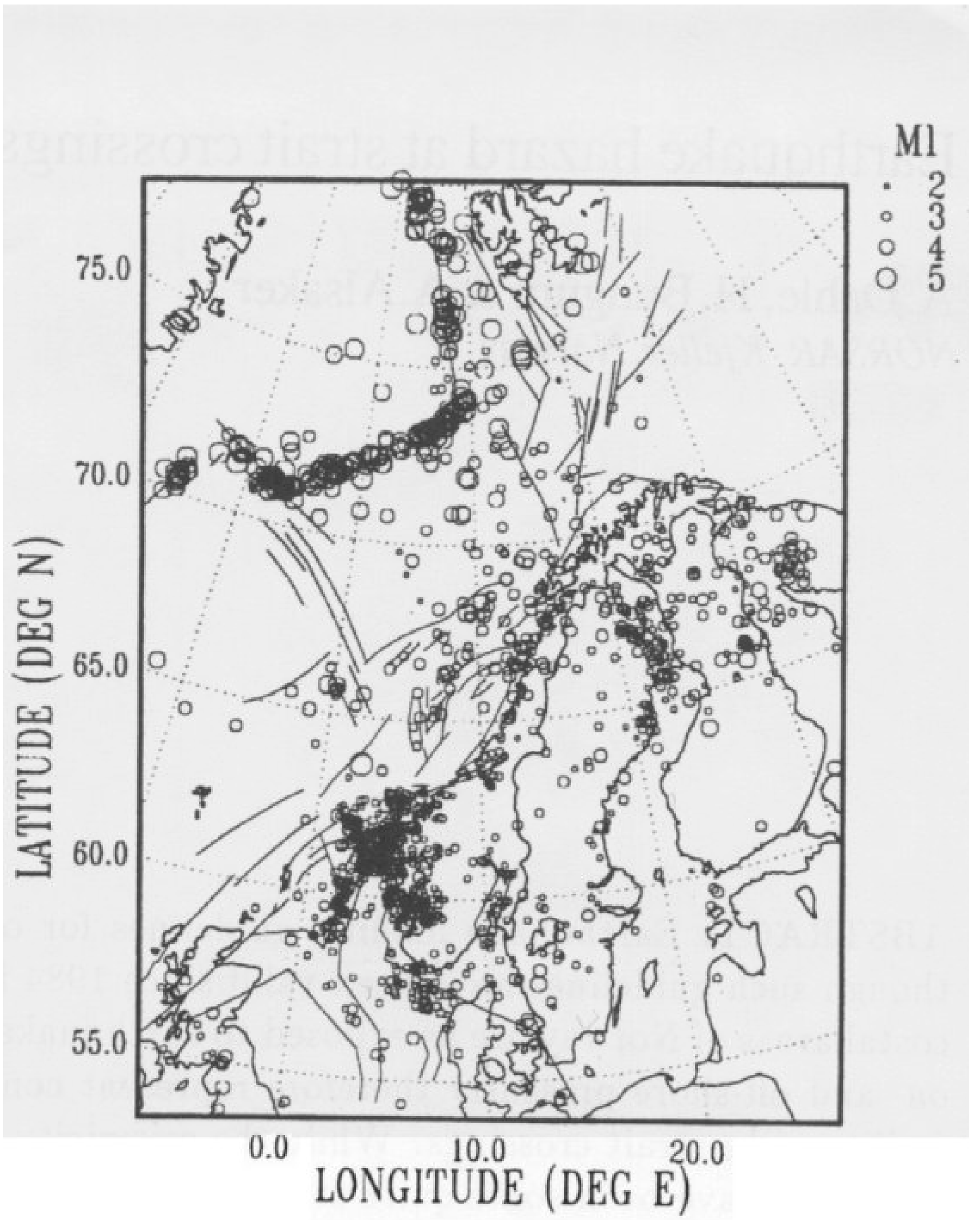


Figure 2: Contemporary seismicity in Norway and surrounding areas (time period 1955-1989, magnitudes above 2.0).

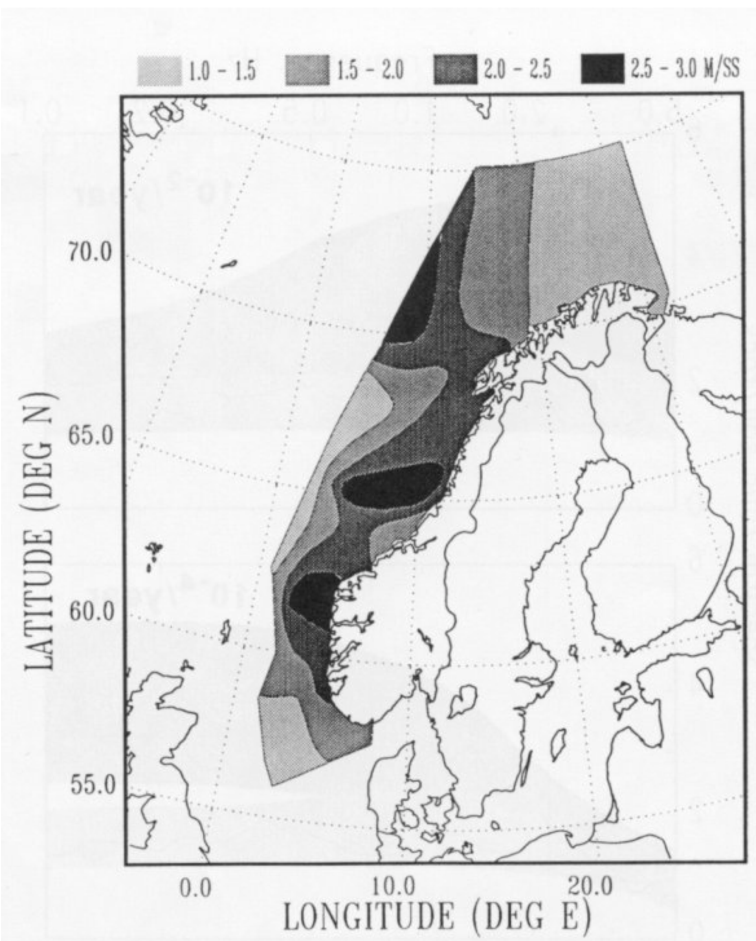
Another significant event was the often-remembered magnitude 5.4 earthquake occurring in the Oslofjord area in 1904. This earthquake, which occurred on a Sunday morning, caused widespread panic in several churches, with several people injured, as well as extensive damage to houses in and near Oslo. Several chimneys fell into the streets, and houses were abandoned due to cracks from the base floor to the roof.

The historical seismic activity of Norway has been thoroughly reanalysed during the last few years, in parallel with significant improvements in seismic instrumentation. In Fig. 2 we show in this respect the locations of contemporary and therefore predominantly instrumentally recorded earthquakes (Bungum et al 1989), demonstrating the concentration of earthquakes in the coastal areas of Norway. It needs only a glance at this map to accept that the seismic exposure in areas of Norwegian strait crossings will be in the range of the highest levels to be expected anywhere in Norway.

3 Earthquake hazard in Norway

In Norway, the majority of earthquake hazard and load analyses have been performed in connection with large fixed offshore installations, while studies for on- shore critical facilities include constructions such as pipelines, oil refineries, and large dams and water reservoirs.
The key elements in most of the hazard analyses performed are :

1. An earthquake source model, describing the occurrence and recurrence characteristics of earthquakes within distances of engineering significance from the site.
2. A wave attenuation model, describing the near- field excitation as well as the more far field decay characteristics (geometrical spreading, anelastic attenuation), as functions of magnitude and wave frequency.
3. An earthquake hazard analysis, including a probabilistic estimation for rock sites and the expected soil response (amplification) resulting in design response spectra for both rock and soil sites.



The methods, models and results of this procedure were studied in considerable detail (Woo et al 1988) in the joint industrial research project ELOCS (Earthquake Loading on the Norwegian Continental Shelf), which included 15 different sub-projects. Even though this study was confined to the offshore areas, it is clear that the results are interesting also from the point of view of onshore earthquake excitation potential, at least for coastal areas. One of the results of this study was a seismic zoning map as shown in Fig. 3, published in a summary report from the project (Bungum and Selnes 1988).

Figure 3: Seismic zoning map for the Norwegian Continental Shelf, exceedance probability 10-4 per year , (Bungum and Selnes 1988).

The zoning map shows PGA (Peak Ground Acceleration), for the main horizontal component of ground motion at a rock site, at 5% of critical damping corresponding to an annual probability of exceedance of 10-4. The zoning map shows that with the exception of The Trøndelag area, the

coastal areas of Norway from Stavanger to Tromsø could expect PGA levels in the range 20-30% of the acceleration of gravity (1 g), at the probability 10-4/year. W hen combined with the appropriate response spectrum (Woo et al 1988), these excitation levels correspond to 4-6 cm dynamic displacement in the period range 2-5 seconds.
The zoning map is developed for rock sites, and does not account for the amplification due to soils above bedrock. Such amplification was also investigated in the ELOCS project (Rognlien 1988), with results as shown in Fig. 4 for soft and stiff/hard soil, respectively. According to these factors, soft soil sites may experience dynamic displacement in the range 8-30 cm at the probability 10-4/year. Foundation or anchoring in soft soils could therefore be one of the critical tasks from an earthquake engineering point of view. It should be noted here that natural periods of motion for strait crossing constructions may be longer than those normally accounted for in earthquake engineering for offshore constructions. The development of earthquake loading criteria at such long periods is consequently not well established. In general, uncertainties increase with increasing natural period.

4 Effects from large, distant earthquakes

The observation of long period motions (10-100 seconds) from large distant earthquakes is well established in the seismological community. These motions are due to the Rayleigh-and Love-type waves propagating along the surface of the earth. Due to the disperse character of the wave propagation, these waves appear as fairly monochromatic wave trains with very long durations (several minutes), depending on magnitude and the distance to the epicentre.

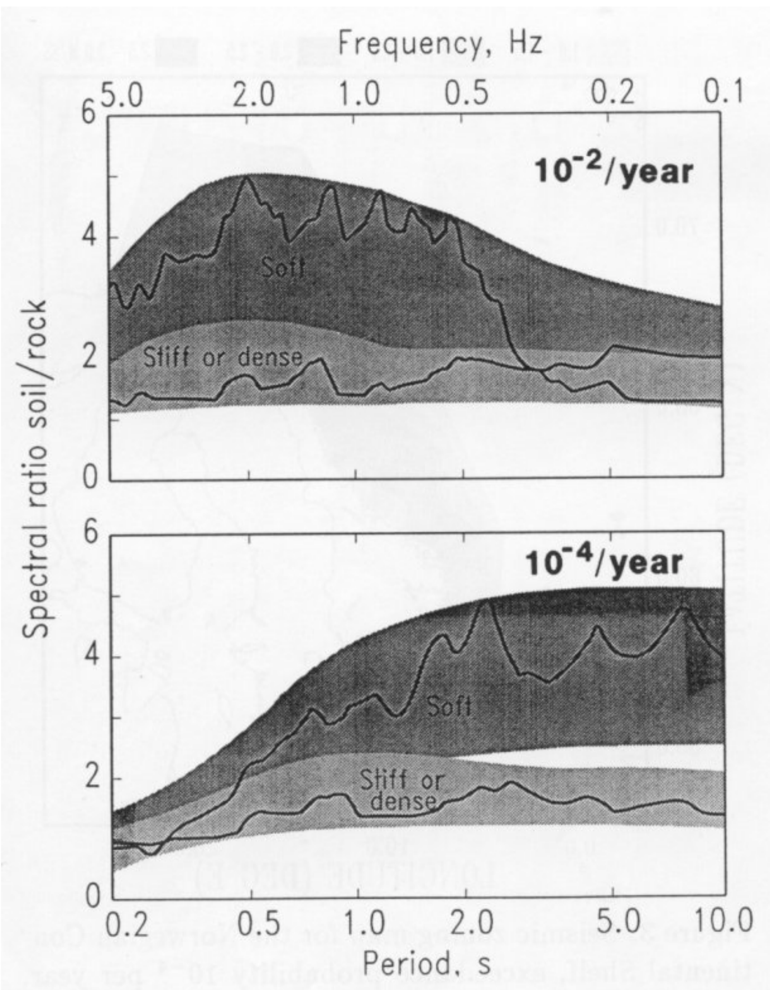


Figure 4: Amplification factors for soil site relative to rock site, (Bungum and Selnes 1988).

The impact of these long period earthquake motions on constructions with long natural periods should therefore be taken into account when designing strait crossings like bridges and submerged tubes.
These long period wave motions may under certain conditions create standing waves in fjords and lakes. Such waves, also called 'seiches', were observed in Norway after the 1755 Lisbon earthquake, the 1920 Kansu earthquake in China, and the 1950 Assam earthquake in India. The phenomena observed after these earthquakes (Kvale 1955) indicate water level changes with amplitudes up to one meter and periods between 30 and 120

seconds. The character and severity of these standing waves obviously depend on the azimuth, the depth and the length of the water basin exited by the earthquake motions.
The fact that seiches has been observed and described at least for three earthquakes in three hundred years, indicates a probability' of occurrence per year which is high enough to be seriously considered as a hazard for strait crossings. The present knowledge of the loading potential for seiches is limited. However, empirical data as well as theoretical modelling methods are available and the problem can therefore be studied by a combined deterministic and probabilistic approach.

5 Conclusions

The earthquake loading potential in the coastal areas of Norway is comparable to that of Norwegian off-shore areas. From a safety point of view, critical constructions onshore should therefore be designed under similar careful considerations as required offshore.

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Integrated and extended use of geophysical methods for investigation of near shore subsea projects

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ABSTRACT:
Investigation procedures for strait crossings have changed during the last years from traditional sampling by drilling to extended use of geophysical methods. The project types have enforced the change; subsea tunnels, pipelines and submerged tubes on deep water. But the use of geophysics have increased as the methods have improved in accuracy and productivity. Reflection seismics and refraction seismics are the dominant methods, but the side scan sonar and the penetration echo-sounder have also been very useful in special project types. Most methods are cheaper today than they were 5 years ago. Optimum use of geophysics is obtained w hen it is used in combination with geological knowledge. There is no single geophysical method which can "give the complete picture of the underground". Every project is dependent on a successful and integrated use of many different methods and geological knowledge.
This article gives a short description of available methods and type of projects in which geophysical methods may be used. Three different strait crossings are presented as case histories to show application of the different methods. Further development of the geophysical methods applied on strait crossing projects especially, is briefly described.

1. GEOPHYSICAL METHODS IN USE

All sub-sea constructions are dependent on accurate information about the topography and the geology within the construction area. In many cases this information is out of reach from traditional geotechnical investigations such as drilling etc. Never the less we need information about the topography, soil conditions and the bedrock quality and we are forced to reveal the se conditions by interpreting the physical condition of the sub-sea and not the mechanical conditions.
For this purpose we may use simple physical properties such as the resistivity or the sound velocity of a material to evaluate the mechanical properties of the material. Since this activity has an element of evaluation it calls for an interpreter that can transform the se physical recordings to measurements usable for the construction engineer.
Different ways of treating sound propagation is most commonly used to evaluate the sub-sea conditions. Other methods such as magnetic properties are not in use at the project level, but may be used in large scale regional surveying. Electrical methods used in marine environments are on the R&D level.
Echo sounding with precision echo sounders or multibeam echo sounders are used to obtain the sub-sea topography, which is the basis for all evaluation. High frequency transmitters with a narrow spread angle is a necessity to obtain accurate images of the seabed.
Side scan sonar is a tool that returns a visual image of the seabed based on the reflectivity of the seabed- High frequency sound is sent via two transmitters to reflect from the seabed. The two transmitters move sideways like a pendulum while the side scan sonar fish is towed through the water. The result is an aerial coverage of seabed at both sides of the towed route, see fig. 1.
When we need information further down into the sediments, we use such methods as penetration echo sounders, reflection seismics or refraction seismics.
The penetration echo sounder is a low frequency, high energy echo sounder that penetrates the sediments on the seabed. Penetration depth is dependent on the type of sediment, but the depth increases with increasing softness of the sediments.

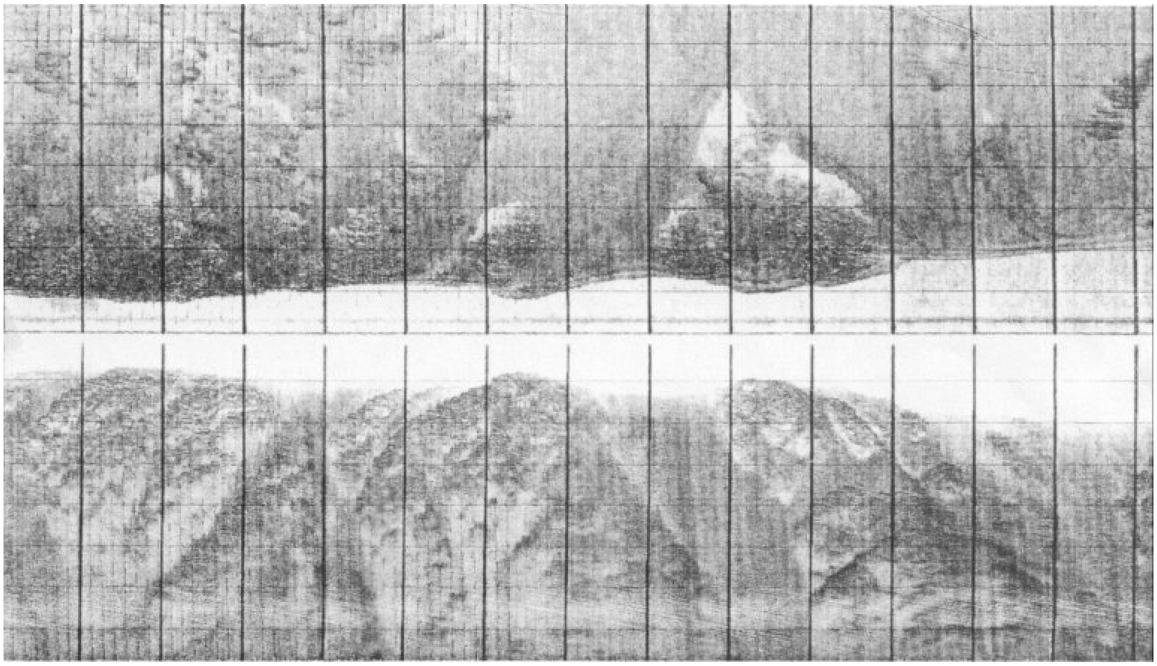


Fig. 1 Side scan sonar "photo" Vertical scale appr. 1:5000. Horizontal scale appr. 1:6500

One may as an example detect bedrock below 5- 10 m of soft clays. The penetration echo sounder is used mostly in connection with pipeline studies.
Reflection seismics is used to penetrate deeper into the sediments and to produce maps of top bedrock. These bedrock maps are of utmost importance for the design of sub-sea tunnels and piercings. Near shore projects are usually surveyed with modified analogous equipment based on boomer/sparker energy sources, see fig. 2.

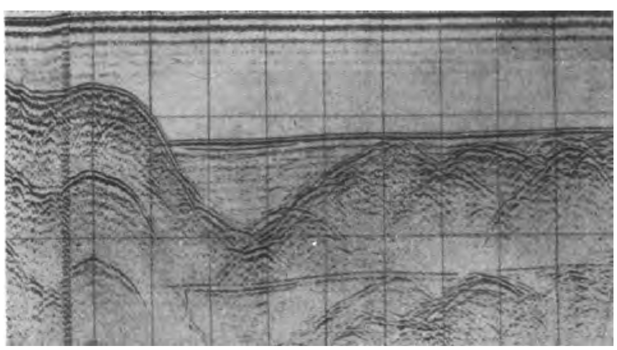


Fig. 2 Sparker recording. Sub-bottom profile Vertical scale 1 cm = 45 m

The bedrock may be detected below 100-200 m of sediments. Interpretation is based on recognition of special reflection patterns in connection with know ledge of the regional geology. On special occasions mini airguns and digital recording may be chosen.
Refraction seismics is used for the detailed survey where high precision determination of bedrock level and sediment/bedrock quality is needed. Refraction seismics are performed by dropping a 48 channels hydrophone cable on the seabed a cross the area of interest. Small charges of dynamite are detonated to send pulses through the sediments and along the bedrock surface. The resulting product is thickness of the different layers and their velocities, see fig. 3.
Finally, visual inspection of special areas with Remote Operated Vehicles may be used. ROV's are usually equipped with video cameras, still photos, sonar and high precision echo sounders for detailed bathymetry .

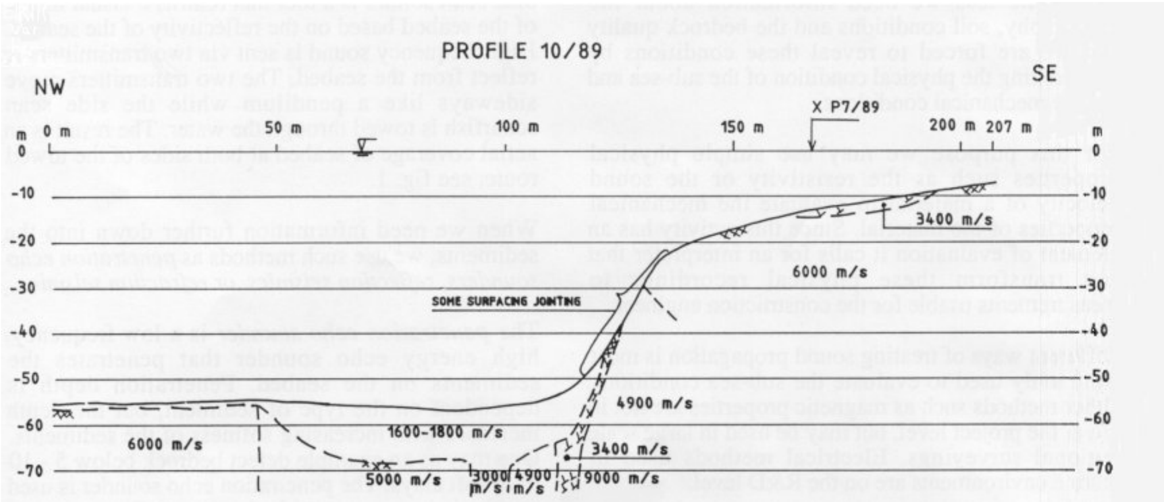


Fig. 3 Refraction seismic profile 48

2 APPLICATIONS OF THE DIFFERENT METHODS

Bach project type calls for different investigation methods. In fig. 4 we have listed different project types and indicated when and how much the geophysical methods are used.

GEOPHYSICAL METHOD / PROJECT	Echo-sounding	Reflection seismics	Refraction seismics	Side scan sonar	Penetrat. Echosound.	ROV
Subsea tunnel	B	B	E	D	–	–
Piercing points	B	D	E	D	–	E
Pipelines	B	B	D	E	E	E
Bridges	B	D	D	D	–	D
Submerged tubes	B	B	D	B	D	D
Fillings	B	D	D	D	D	–
Quays	B	D	D	–	D	–

B – Basic information for the project
E – Extended use
D – Dependent on the specific form of the project
– Not used

Fig. 4 The use of geophysical methods in different projects

Echo sounding

As a basic information for all sub-sea projects, a sea bottom map is required. The equidistance may vary according to the requirements in the projects, but are usually between 1 and 5 m.

It is essential to bear in mind that the accuracy of the map is totally dependent on the collected soundings. In general (for single beam echo sounders) it is a rate of 10 for the distance between profile lines and equidistance; 10 m between lines can give 1 m equidistance.

Without using any other geophysical method, information about the geology, both sediments and rock, can be improved extensively by improving the bathymetric map.

Reflection seismics
In the first stage of an investigation, together with the echo sounding, boomer or sparker equipment (reflection seismics) is often used. The main target for this type of survey is to get an overall view of the soil distribution of the area.
Penetration is dependent on the soil conditions and it may sometimes be hard to distinguish compacted moraine from bedrock. Sound velocity in each medium has to be estimated. Analogue recorded seismics are interpreted based on recognition of reflection patterns, and are totally dependant on the interpreters skill. Refraction seismics (see below) can then be used for the "calibration" of the reflection seismic data, since they produce accurate information about velocity distributions.
As can be seen in fig. 2, the boomer/sparker registrations of ten also give an indication of sediment type and the surface of rock. Especially soft sediments of clay or silt can give good readings of the layering and distinct recording of the bedrock surface. In areas with thick moraines, interpretations of the recordings may of ten give a too low estimation of the soil thickness.
Distance between profile lines will also decide the quality of the map, which can be given as a bedrock surface map or a bathymetric map showing the different soil thickness as hatchuring. The different use of the boomer/sparker is indicated in fig. 4 for some adequate project types.
Penetration echo sounder
Geophysically speaking, we are here talking about an echo sounder, but the results are more easily compared with the boomer/sparker. Penetration is lower, but resolution is much better. This makes it very useful for investigating pipeline routes and anchoring locations where the top soil conditions are of utmost importance.
Refraction seismics
The refraction seismic method is mostly used for a more accurate determination of the rock surface, and it also gives indications on soil type and rock quality based on the sound velocity. Extended use of the method takes place for sub sea tunnels and sub sea piercing points for tunnels (Aagaard, 1986) and drilled holes.
The conditions (soil thickness and rock quality) for bridge foundations are also investigated with geophysical methods; echo sounding, reflection seismics and refraction seismics.
Side scan sonar
The side scan sonar is mostly used as a surface towed fish which limits good registrations to around 150 m water depth. When the soil covers only parts of the sea bottom, the "picture" clearly shows the distribution of soil between rock outcrops. Scree material becomes visible and the size of large boulders can be calculated. Structural features in rock can also be registered.
Side scan sonar is used as a basic or extended investigation method for pipelines. For sub sea tunnels and piercing points it has become useful where soil cover only parts of the sea bottom.
ROV. Typical use of ROV's is inspection of piercing points and foundation sites for bridge piers. Pipeline routes are also surveyed with ROV's. In the detail ed planning of anchoring sites for platforms, ships and submerged tubes the ROV is of ten used.

3 INTEGRATED INTERPRETATION

The main object with this article is to give the reader an idea of the best combination of geophysical investigation methods. It is also important to be aware of the limitations in geophysical mapping and the advantages of integrated interpretation.
As in all parts of a project, a cost /benefit analysis decides necessary costs in all phases of a project. So also for the surveying part. The use of echo sounding, reflection seismics and side scan sonar are relatively cheap pr m profiling, w hen ship and equipment is mobilized. This fact calls for extended use of the methods in early stages of the projects. The lack of information or wrong

(misinterpreted) information may lead to a sub optimal project or unnecessary expensive investigations to be carried out in a later (or too late) project phase.

In the tables below we have categorised the survey in three phases according to the design phases; pre-feasibility , feasibility and final design.

For *sub sea tunnels* the basic investigations are echo-sounding and reflection seismics. The possible misinterpretations especially of reflection seismics, makes it advisable to measure at least one refraction seismic profile in the first phase of the project. If the sea depth is moderate and rock is at least partly exposed, it is recommended also to carry out side scan sonar surveying early in the project, see fig. 4.

The geophysicist must be in close contact with the engineering geologist, who as well should make a field visit during the survey. The integrated analysis, based on collaboration between geologist and geophysicist, will gather much information at an early stage, from which the project will gain at later stages.

The main part of the refraction seismics will be carried out during the second phase of the investigations. The results from the previous phase are used and revised according to the new findings.

When five or more refraction seismic profiles are measured, it allows statistical analyses of the seismic velocities. In addition to giving a good overview, the statistic material is used for stratigraphical analyses of the geology. Figure 7 shows the distribution from all profiles in a project. Similar distributions can be made for different profile directions and different areas. Together with other information; geology on land and the first phase geophysical data, the stratigraphy sub-sea can fairly well be indicated.

Survey of *piercing points* forms part of the subsea tunnel investigation. W hen promising piercing areas have been found, detailed bathymetric mapping are carried out and a ROV survey takes place.

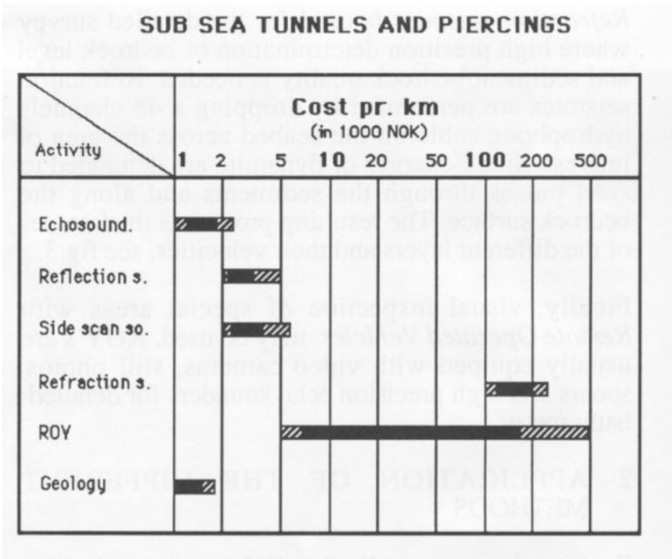
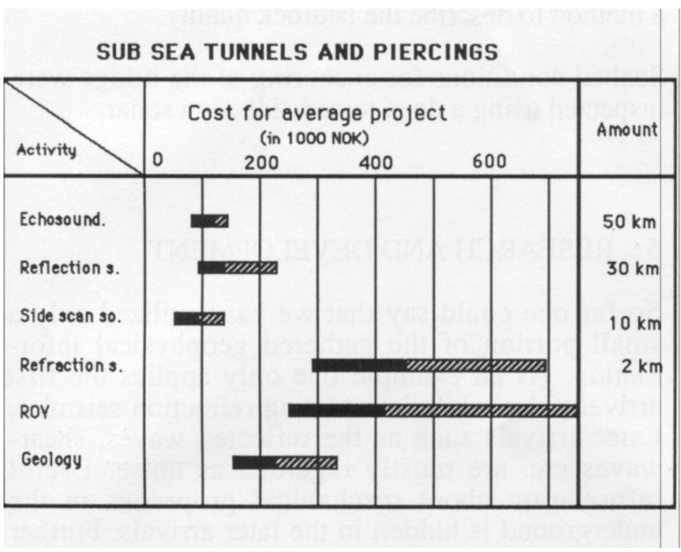


Fig. 5 (above)Cost pr km. Mob/demob not included

Fig. 6 (below) Total costs pr survey



Pipeline surveys starts with echo sounding and reflection seismics. The side scan sonar should be used extensively in the first phase of the project, and the results should be used for the detailed planning of echo-sounding and penetration echo-sounder.

Foundations of *bridge* piers calls for another strategy for the investigations. Echo-sounding is basic, but other geophysical methods are dependent on the specific form of the project. Side scan sonar and reflection seismics can be used for reconnaissance and planning of geotechnical drilling. The refraction seismic method can be used instead of or in addition to drillings.

Investigation of anchoring areas for *submerged tubes* are very similar to pipeline investigations because the top soil layer is of most interest. The sub-sea piercings are investigated like the sub-sea tunnel piercings.

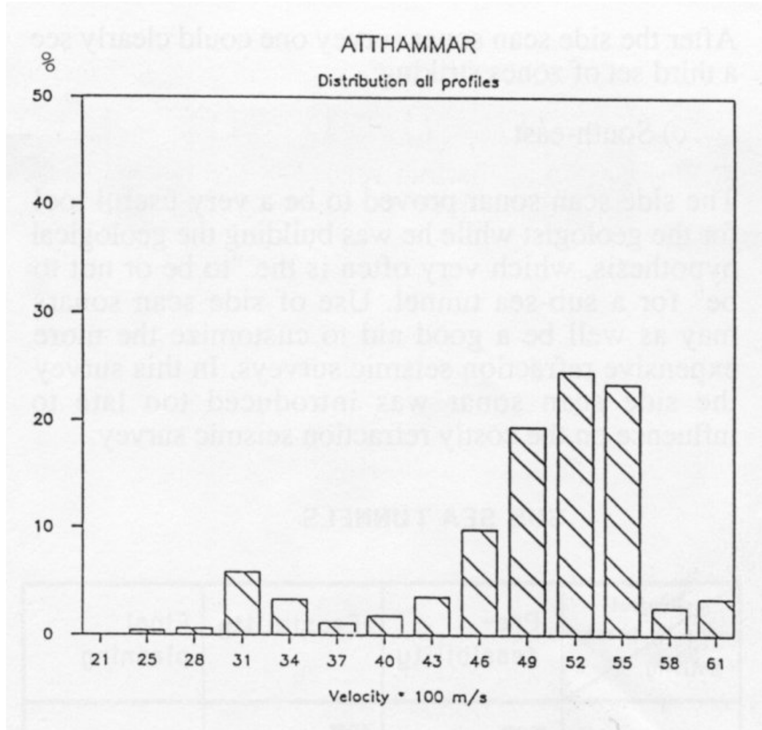


Fig. 7 Velocity distribution from refraction seismics

4 CONCEPT FOR FIELD INVESTIGATIONS SOME CASE STORIES.

4.1 FRØYFJORDEN SUBSEA TUNNEL

The Frøyfjord sub-sea tunnel will cross a number of faults and weakness zones between Hitra and Frøya. All traditional investigations such as engineering geology mapping on shore, bathymetric surveys, boomer/sparker surveys and a number of refraction seismic profiles have been carried out. The refraction seismics were carried out in two separate surveys in search for the optimum tunnel routing.
Since aerial photos are in common use amongst geologists, w hen they look for faults and weakness zones, we decided to try the side scan sonar .We hoped that the side scan sonar would give us a "sub-sea aerial photo" to be used in interpretation of the fracture patterns and the orientation of the weakness zones.

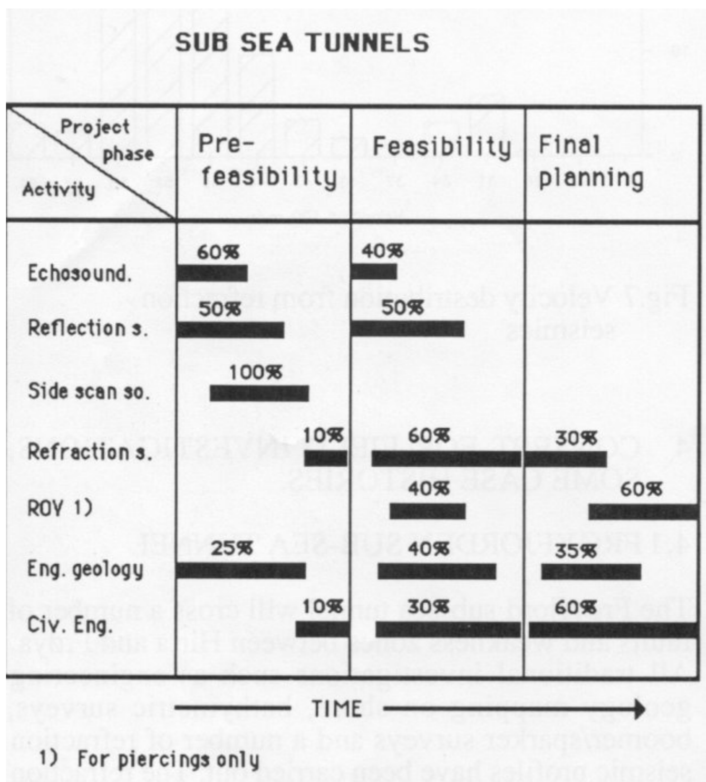


Fig. 8 Execution of geophysical surveys in different project phases

An example of a sonar recording is given in fig. 1. The recording shows exposed bedrock and one can clearly see the orientation of the weakness zones.
A map was compiled showing exposed bedrock, sediments and orientation of weakness zones. The result was compared with the previous geological interpretation based on bathymetry , reflection seismics and refraction seismics. Prior to the side scan survey the geologists had anticipated two sets of weakness zones with strikes:
a) North-east
b) East
After the side scan sonar survey one could clearly see a third set of zones striking:
c) South-east
The side scan sonar proved to be a very useful tool for the geologist white he was building the geological hypothesis, which very often is the "to be or not to be" for a sub-sea tunnel. Use of side scan sonar may as well be a good aid to customize the more expensive refraction seismic surveys. In this survey the side scan sonar was introduced too late to influence on the costly refraction seismic survey.

4.2 TROMSØYSUNDET

The Tromsøysund tunnel is part of the new road system within Tromsø city. A sub-sea tunnel system of two parallel road tunnels will release the traffic tension on the old bridge connecting Tromsø city with the mainland. At the time of the symposium a number of entrepreneurs will compete for the building contract.
In the pre-feasibility stage, the reflection seismic investigations revealed a soil cover and smooth seabed along the whole sub-sea tunnel corridor. The first phase was carried out without refraction seismics.
The second phase (feasibility), with both reflection and refraction seismics revealed a bottom soil layer of compacted moraine which had not been clearly identified during the first phase.
A final survey for the detailed planning was successfully carried out with 13 spreads of refraction seismics (spread length = 235m). A few bedrock control borings have been carried out in shallow waters to control the interpretations. No core- drillings have been carried out, and the whole tunnel design is based on geological and geophysical interpretations.
Based on the geological and geophysical surveys, we managed to optimise and shorten the tunnel length and reduce the tunnel inclination.

4.3 HØGSFJORDEN DEEP-WATER CROSSING

The Høgsfjord deep-water crossing has been described at the last Strait Crossing symposium. During the early stages of planning, both a sub-sea tunnel and a high level bridge on submerged pontoons have been evaluated.
The problem from a surveying point of view was the enormous glacial deposits located at the bottom of the fjord and the very steep sides of the fjord. Huge boulders are spread out on the seabed causing problems with anchoring of the pontoons. The steep sides reflect the signals from the boomer/sparker in such a way that it masks the bedrock reflections.
After previous less successful attempts with the regular analogous equipment (due to side reflections and poor penetration), a larger survey boat with digital equipment and deep towed side scan sonar were chartered. With digital recording of the signal one may reprocess the data and avoid the side reflections. The survey was carried out with a 294 m hydro-phone streamer and a mini airgun as energy source. The low frequency airgun was able to detect the bedrock below 150 m of glacial deposits, which really closed the discussion about a sub sea tunnel. With such sediment

thickness it is almost impossible to use conventional refraction seismics as a method to describe the bedrock quality.
Seabed conditions for anchoring of the bridge were inspected using a deep towed side scan sonar.

5 RESEARCH AND DEVELOPMENT
So far one could say that we have utilized only a small portion of the gathered geophysical information. As an example one only applies the first arrival pulse while interpreting refraction seismics. Later arrivals such as the reflected waves, shear- waves etc. are mostly regarded as noise. Useful information about mechanical properties in the underground is hidden in the later arrivals. Further more today's collection and interpretation are based on viewing the underground as a 2-D problem.
A/S GEOTEAM is developing a new concept with combined measurements simultaneously using two or more hydrophone cables on the seabed. Both refracted and reflected data are collected and run through a specially designed computer algorithm. The resulting product in a "3-D" image of the under- ground compiled by one tomographic image of top bedrock in the x,y-plane, two reflected sections in the x,z-plane and two ordinary refracted profiles also in the x,z-plane.

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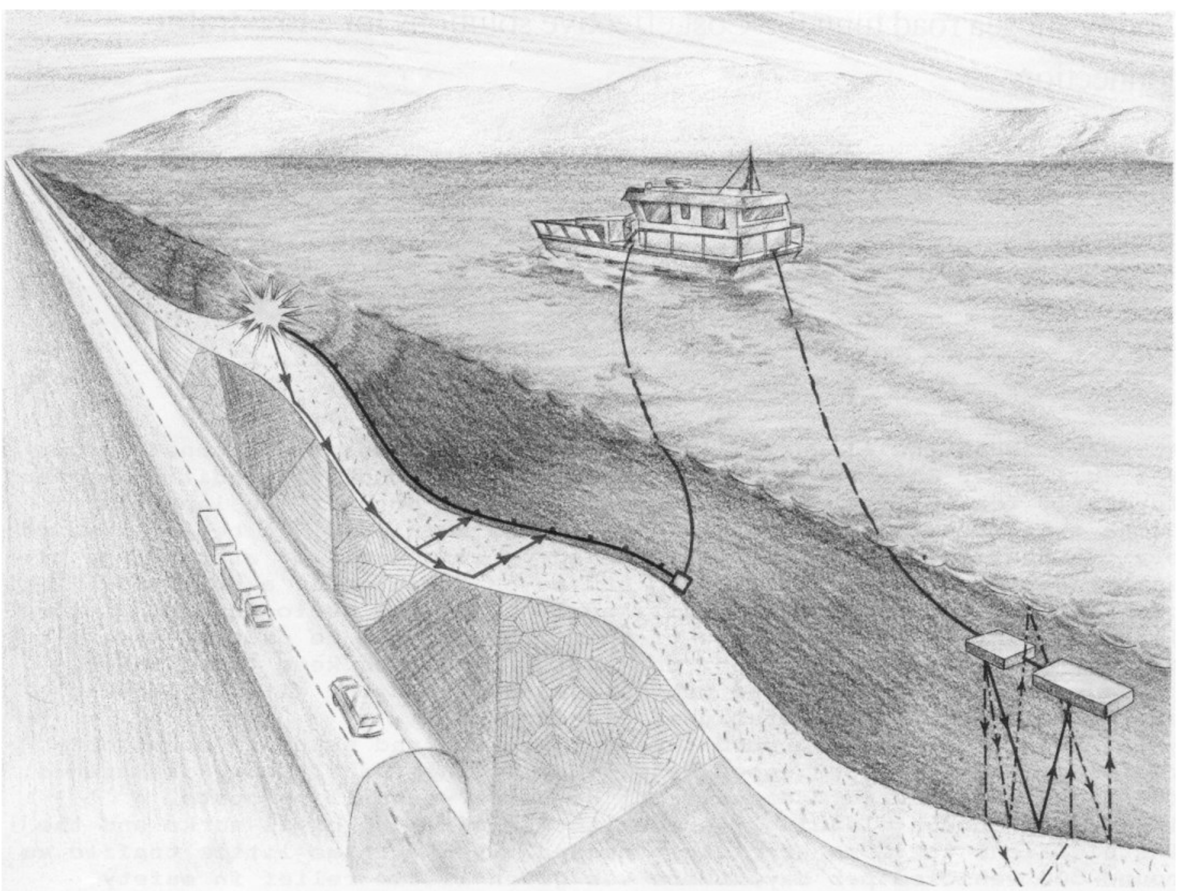

Godøy sub sea road tunnel-
Cost effective solutions for a low-traffic connection

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ABSTRACT:
When the Ålesund-Ellingsøy-Valderøy sub sea road tunnel project was well under construction, the economic and technical conditions seemed favourable for an extra 3.8 km sub sea tunnel to connect the last one of Giske municipality's 4 major islands to the mainland. It became however obvious that the funds from toll financing were limited, at the same time as the decision to start building had to be taken after a very short planning period, and without full knowledge of the geological conditions. In this lay the very challenge of this project: How to protect the project from the risk of high, unexpected costs and keep the planned costs much lower than by the previous sub sea tunnels, without loosing anything in safety or operational standards. Thoroughly optimised procedures for grouting and support, constantly redesigned to the experienced conditions at the tunnel face, diminished the risks and ensured "adequate but no extra" tunnelling costs. A likewise thorough planning was done to minimize the civil works and the installations to those absolutely needed. Even with as little traffic as round 500 vehicles per day, there was given little relief in safety demands, and thus it is quite an achievement that the overall cost per km was reduced by 1/3, and particularly that the installation cost was reduced by 2/3, compared with those of the two tunnels first completed in the same project.

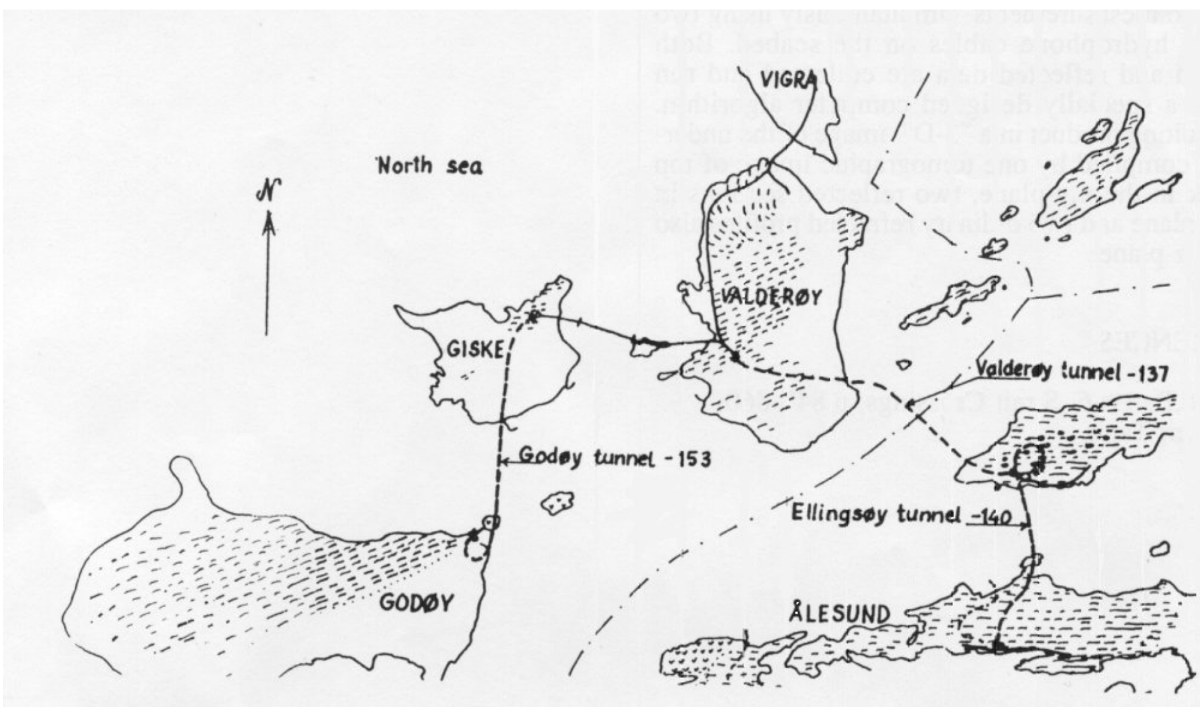

Fig. 1. Tunnel routes and gneissic outcrops on the islands

1 INTRODUCTION

The Giske municipality outside the town Ålesund at the western coast of Norway consists of 4 major islands, 3 of which were to be connected with the mainland by 2 sub sea road tunnels and one bridge in a toll financed project that was under construction 1985- 87. By this a network of car ferries was reduced to only one ferry route between the town and the last island Godøy. This is the home for about 500 inhabitants who are more or less dependent on the town for work, school, hospital and other facilities. For the municipal administration, efficiency gains by the new connections were limited as long as this one island remained dependent on ferries. Local politicians therefore enforced a rapid planning and decision process towards a road connection for Godøy as well.

2 DEEPEST SUB SEA TUNNEL SO FAR

Both bridge and rock fill alternatives were soon rejected as far too expensive. Geological investigations revealed deep quaternary deposits in the strait between Godøy and Giske. This gave a maximum tunnel depth of more than 160 m, but it was the most favourable route both in length and in road connection. Only one rock outcrop on Giske gave room for a tunnel entrance. Within short time refraction seismic were carried out along most of this one actual route to give a fairly good survey of the rock surface. The rocks belong to the igneous basement rock group of north-western Norway, but apart from this fact, little was known about petrographic details or fracture systems along the tunnel route. Diamond corings would have been very expensive if possible. The economical outlooks for the big mainland project indicated by 1986 that an extra 100 mill NOK. could be financed within a payback period of no more than 20 years. Reduced ferry subsidies could release an extra 10 mill NOK from the county transport budgets. The tunnel could also trigger a desirable development in housing and industry within the municipality. By the summer 1987 the Godøy tunnel was authorized by the Parliament, and construction started late that autumn. The winning bid was given by Selmer- Furuholmen, who at that time was about to complete the main project. The gain by continuous construction period was assessed to some 5 mill NOK.

3 MODERATE ALIGNMENT, STEEP GRADE

The cost estimation at an early stage indicated 100 mill NOK for the Godøy tunnel. According to experiences from the two previous tunnels this soon was adjusted to same 120 mill NOK. It then became obvious that the margins vs. fund limits would be small, and efforts were taken to keep the costs down as much as possible. The minimum curve radius was set as small as 125 m, according to a speed of 50 km/h. Maximum grade is 10º, giving the shortest possible length of 3835m. The cross section is $50 m^2$, and the width of the two- lane road is 6 m (6.5 m in the steep curve).

4 COST SAVING DESIGN

At that time discussions had started towards even stricter safety measures in road tunnels. For instance, the two first tunnels built for 4000 AADT, were given extra fire protection to a cost of more than 20 mill NOK. Even with as small design traffic number as 500 vehicles daily, much of the same safety measures were demand ed in the Godøy tunnel. Some of the designed solutions were however more simple and cost effective. Instead of an extra lane in the steep uphill parts, emergency parking niches were put at every 250 m. Instead of a double pump- and tube system, a 2 days pumping storage was excavated. The high pressure tube was connected to a borehole in rock under land at the shortest possible distance from the pumping station. Drain tube was designed single sided in connection with a layer ox coarse gravel in the road foundation. On the road shoulder there is no threshold or pavement, only coarse, drain-open gravel that can be removed ix it is choked with asphalt dust.

5 SAFE AND RAPID TUNNELLING

Typical for Norwegian tunnelling experience is the principle of "design as you go". This is also fully adapted for the sub sea tunnels, only here the skilful and experienced manpower is even more vital. An engineering geologist followed the drill and blast tunnel excavation intensely, and he took part in every round of exploratory drilling ahead of the tunnel face. On this base it was decided whether to start cement grouting and the procedures for such. The tunnellers had to light potential heavy leakages from open joints in approx. 40% of the tunnel length, but water inflow was kept down on the design value of 300 litre/min, km, and there was no delay or hazardous situations due to water inflow. Likewise, tunnel support was designed at the face. To save both money and time, most of the support was completed to permanent standard at once. Rock bolts combined with steel fibre reinforced shotcrete in weak zones did the job, and no full concreting was needed. An important element in this achievement is an accurate and careful contour blasting. The tunnelling made breakthrough after 13 months, and only 3 months later the first civilians could drive through the tunnel and forget all about waiting time and stormy weather on the ferry. On completion one had to admit that the expectations of a dry tunnel went far above the suppositions, and shielding panels had to be installed in more than 60% of the tunnel length. This called for development of a simple aluminium shield to cover only the roof. A test length of 300 m was very successful and saved money compared to the PE-panels. Also the thoroughly planned technical installations contributed much to the short completion time, and as the solutions seem to represent a "step forward" a more detailed description is presented:

6 NEW ACHIEVEMENTS IN TECHNICAL INSTALLATIONS

In the Ålesund-Ellingsøy-Valderøy tunnels, the cost of all technical installations was NOK 4.350 per m of tunnel. This includes cost of the high voltage systems, diesel generator unit, and the operation monitoring system at Ålesund fire station. Comparable figures for the Godøy tunnel are NOK 1.680 per meter of tunnel. Particular emphasis has been put on the energy costs for operation of the tunnel. The operation control system controls the ventilators and pumps so that power peaks are avoided. When the ventilators have to run at full capacity, the drain pumps will stop, and when the ventilators are running at 2/3 capacity, one of the drain pumps is allowed to run and so on.

6.1 Power supply

The total power demand for the tunnel is approximately 340 kW with an annual energy consumption of 500.000 kWh. This power consumption comprises ventilators, approx. 210 kW, drain pumps approx. 118 kW and tunnel lighting approx. 13 kW.

The main power supply is fed from a transformer at Giske. In addition it is possible to supply the system from the high voltage distribution system at Godøy.

The drain pumps at the tunnels low points, may easily be connected to mobile diesel generators if all other power supplies should fail. Three transformer stations and distribution substations are located approx. 900 m apart throughout the tunnel. Uninterruptible power supplies (UPS) with battery back-up, have been installed at each distribution substation. The UPS supplies the priority consumers such as fire alarm system, traffic signal system, emergency lighting, control- and monitoring-system etc.

6.2 Lighting

To provide good driving conditions in the tunnel, 156 35 W lamps are installed at 25 m intervals. The lighting has been strengthened with 4 250 W lighting fittings at the tunnel outlets. The lighting is controlled by photocells (day/night). Every fifth lamp has a built-in uninterruptible power supply which will allow it to function w hen the regular power supply fails.

6.3 Ventilation

The tunnel ventilation is longitudinal, without any vertical shafts. Total force is 7.200 N, which comprises 14 asymmetric impulse ventilators mounted in the tunnel ceiling. The fans are step-controlled, and step selection is based on exhaust (CO) measurements. Normal direction of ventilation is towards Giske. The ventilation system is designed to maintain good air quality at maximum traffic loads. If the estimated maximum traffic load is exceeded, the system is prepared for installation of two additional ventilators. In the event of a fire, the fire brigade may change the direction of ventilation to ease evacuation and fire fighting.

6.4 Control-, monitoring, and alarm-systems

All the vital functions of the tunnel systems are controlled and monitored by a PLC system. The master unit and a PC are located in the control room at Ellingsøy. The system monitors exhaust (CO), water level in the drain sumps, lighting level and the power supply system. The ventilation-, pump-, and traffic signal- systems are controlled based on these inputs. Operator terminals are located in the control room and at Ålesund Fire station’s operation centre which is manned 24 hours a day. The signals between the tunnel and the control monitoring system are transferred on radio.

6.5 Fire alarm, fire-extinguishing equipment and emergency telephones

Emergency stations with manual fire alarm push buttons and extinguishers are located approx. every 250m. Emergency telephones are located every 500 m. At the instance the emergency station cabinet door is opened, an alarm will sound if the manual fire alarm push button is activated or it the fire-extinguisher is removed from its stand. When the emergency telephone receiver is lifted off the telephone, contact is immediately established with the fire station's operation centre. This system is judged to give more reliable and informative fire alarms than any automatic system available.

6.6 Traffic signal system/information signs

At the tunnel entrance, on both sides of the road, traffic lights are installed. These lights will appear flashing red when it is necessary to close the tunnel. The lights will be activated automatically by the monitoring system w hen the exhaust level exceeds preset limits. These shutdown limits are based on health danger and traffic safety.
If the fire station's operation centre receives notice of an accident or fire in the tunnel, the red flashing lights can be manually switched on to close the tunnel.

6.7 Material selection/ Method of installation

The tunnel environment is usually very corrosive due to carbon dioxide and sulphuric acid from the vehicle exhaust gases. In addition the tunnel contains saltwater. The environment can therefore be compared to that of an offshore oilrig in the North Sea. The environmental conditions have always be en considered w hen selecting equipment and components. Hot dip galvanized and acid-proof steel have been used throughout, and all electrical equipment is waterproof and protected as best possible so that it may function with a minimum of failure for years to come.

6.8 Comment

The close follow-up during the construction of the Giske -Godøy tunnel and the optimisation of all systems, have contributed to reduce costs for the Flekkerøy tunnel and the Nappstraumen tunnel. Comparable figures for these tunnels will be between NOK 1.400 to NOK 1.500 per meter of tunnel.

7 EXPERIENCES

The tunnel ended up with a total cost of nearly 140 mill NOK (1989) .The difference from the estimate was mainly due to heavy grouting, rise in prices and administration costs. Nevertheless, the price per m of tunnel came out 30% lower than the two first tunnels, which is substantially better than w hat the reduction from 3 to 2 lanes should bring under similar geological conditions. Both traffic convenience, safety and technical operation conditions of the Godøy tunnel is so far reported to be very satisfactory. The maintenance costs after only one year of operation are of little interest, but they seem to be promisingly low.

The main concern today is however that the Godøy tunnel should not have been built. The economic foundation in the main project was too weak and is still getting worse due to high loan interests and reduced traffic volume in the toll station. The financing firm is bankrupt and the banking group now (march 1990) face a loss of maybe 200 mill NOK.

This may be of no major concern to the people who enjoy the safe and convenient connections every day, but it has meant a severe setback for many other similar toll-financed projects that were promoted with great optimism only a couple of years ago.

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Geological and geophysical investigations for the Hitra and Frøya sub sea rock tunnels

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ABSTRACT
The project for a strait crossing connecting Hitra and Frøya requires two sub-sea rock tunnels, several fillings and two bridges. The paper presents the step by step geological and geophysical investigations carried out for the project.
The investigations were in principle carried out in four main steps :
1. geological mapping
2. acoustic investigations of several corridors
3. seismic investigations of the most relevant corridors
4. core drilling at critical points

1 INTRODUCTION
The pattern of communications is in a constant state of change and development. In the past, long fjords were typical transport routes in coastal areas of Norway. Today, they represent a hindrance for the most common means of transport the car. Fjord crossings are an interesting challenge for Norwegian road builders. Key words in this connection are suspension bridge, floating bridge, submerged floating tube and sub sea tunnel. Our first sub sea road tunnel was the Vardø tunnel opened in 1982 and the second the Ålesund tunnels in 1987. Since then several other sub sea tunnels have been put into service and several more are being built or are in the planning stage. The constructional work for the mainland connection for Hitra, Frøya and Fjellværøy is planned to start in 1991. In 1987 the main plan was presented and we are currently in the final stages of detailed planning.

2 KEY DATA
Some key data for the project are as follows:

-rock type:	Precambrian gneisses
-max. water depth:	
Trondheimsleia (Hitra)	185 m
Frøyfjorden (Frøya)	55 m
-max. thickness of unconsolidated sediments in the investigated area:	
Trondheimsleia	70 m
Frøyfjorden	32 m
-length of tunnel:	
Trondheimsleia	5.3 km
Frøyfjorden	4.9 km
-max rock depth (tunnel trace):	
Trondheimsleia	210-215 m
Frøyfjorden	90 m
-max depth of tunnel:	
Trondheimsleia	267 m
Frøyfjorden	145 m
-total construction costs (1990 cost level in million kroner):	
Hitra -mainland (Trondheimsleia)	340

Hitra -Frøya (Frøyfjorden)	270
Hitra –Fjellværøy	115
Total	**725**

-start of construction work:

1991 fill Fjellværøy

1992 tunnel Trondheimsleia

-constructional time for the whole project: 6-7 years

3 ROAD LINE

Connection with the mainland is planned for the islands of Hitra, Frøya and Fjellværøy by a unified road system. This requires two sub sea tunnels, one under Trondheimsleia between the islands of Hemnskjel and Hitra (5.3 km) and one under Frøyfjorden between the islands of Hitra and Frøya (4.9 km). In addition, the land connection will be established by filling in the channel between Hemnskjel and the mainland. Fjellværøy will also be linked to this system by fill/bridge towards Hitra. Possible areas for tunnelling were first determined by traffic conditions and topographic features of the sea-floor. These areas were then reduced progressively depending on information provided by geophysical studies. In the following pages a description is given of the problems which arose on the way with regard to the geological and geophysical studies. In addition, a considerable amount of work has been done on road planning. Traffic and transport economic analyses have also been carried out.

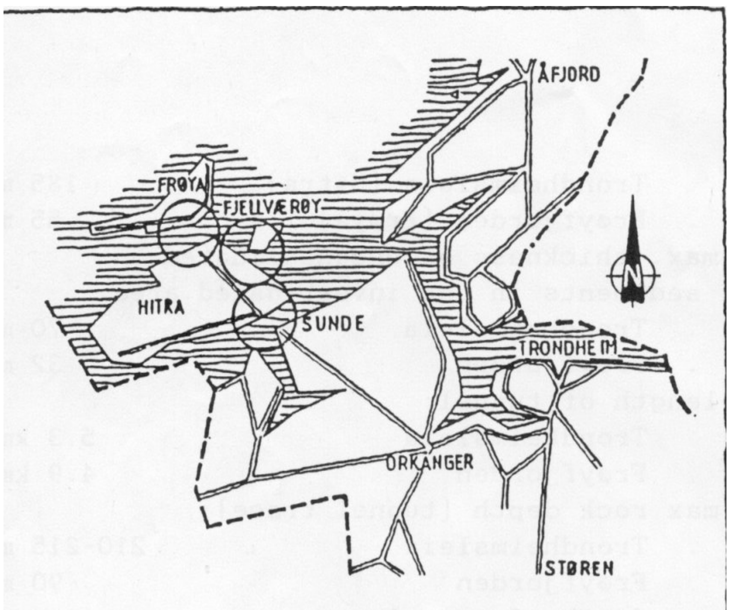


Fig. 1 General Map.

4 WORK PROGRAMME

In 1979 a programme for a pilot project was worked out. This was revised in 1983 when planning was intensified. The investigations can be divided into 4 phases

1 Geological mapping (G)
2 Acoustic investigations of actual corridors (A)
3 Refraction seismic investigations of promising corridors (S)
4 Core drilling and Rock control drilling (G)

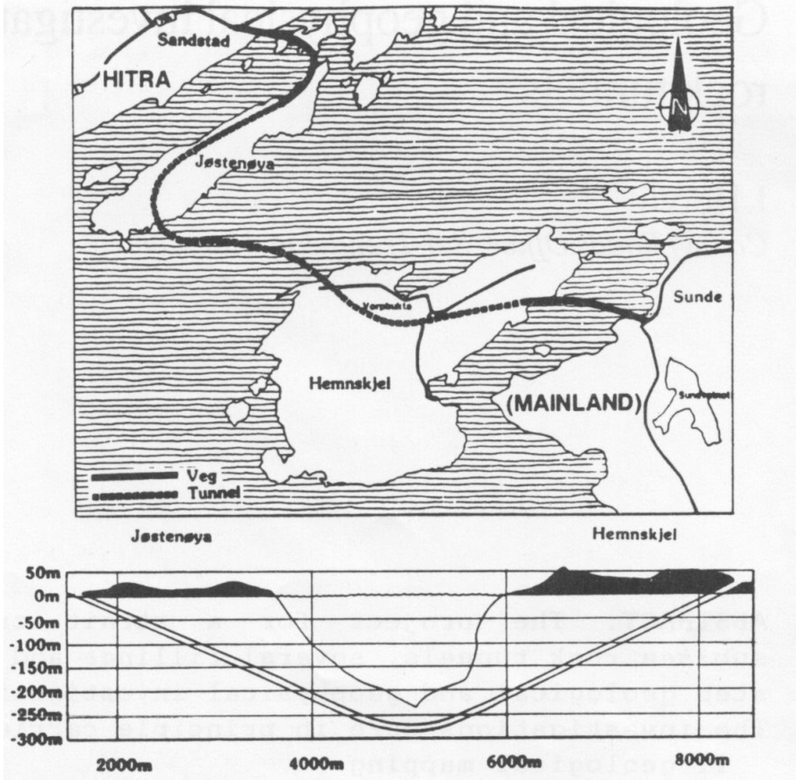


Fig. 2 The Hitra (Trondheimsleia) tunnel

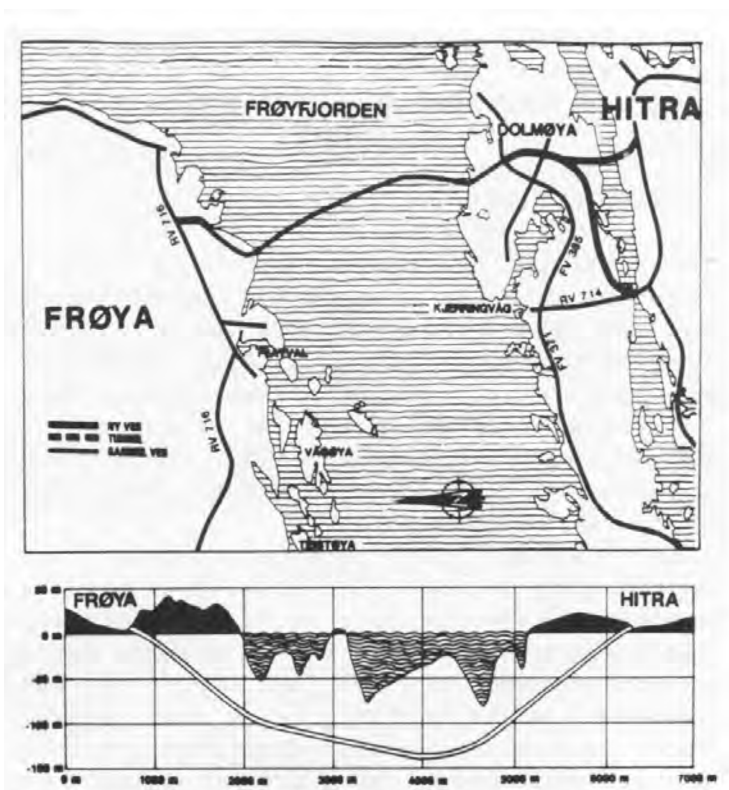


Fig. 3 The Frøya (Frøyfjord)tunnel

The main work plan indicates a time schedule of 3-4 years for the step by step investigation for the main plan. This is in good agreement with the time we have used so far up to the main plan (1983-87). Table 1 shows how the work has been carried out in practice. We see that both the acoustic and refraction seismic investigations have been carried out in several phases, acoustic measurements 5 times in all, refraction seismic 3 times twice for the main plan and once for the detailed plan.
Table 1. Geological investigations carried out and geophysical

1) Code
P = work programme
A = acoustic(reflection seismic)
S = seismic (refraction seismic)
SS= side scan sonar
G = geological investigations/prognoses
UNSPECIFIED: Several geological/ constructional technical evaluations and compilations
T/F= Trondheimsleia/Frøyfjord

CODE	INVESTIGATION	COMPLETION DATE
	MAIN PLAN	
P I	-Work programme	Feb.-79
A I	-Reflection seismic I (boomer Trondheimsleia)	July/Aug.-82
P II	-Work programme	Apr. -83
G I	-Geology preliminary investigations	Oct./Nov. -85
A II	-Reflection seismic(boomer/sparker T, F)	Sept. -84
S I	-Refraction seismic I (T and F)	March/Apr. -85
G II	-Geol. planning. Rock stability prognoses and costs	May \-85
A III	-Reflection seismic III (Frøyfjorden W and E tracks)	Nov.-85
	-Study of constructional costs	March -86
	-Current velocity measurements	Jun./Jul.-86
S II	-Refraction seismic II (T and F)	Jun./Jul.-86
A IV	-Reflection seismic IV (T)	Aug.-86
	-Compilation of shallow refraction seismic and refraction seismic in Trondheimsleia	Oct.-86
	-Evaluation of tunnel depths and track lengths	Oct.-86
	-Study of cover criteria	Oct./Nov-86
A V	-Reflection seismic V (T and F)	Des.-86
	-Compilation of data from reflection seismic and refraction seismic	Jan./Feb.-87
	-Optimising of tunnel track/tunnel cross-section/costs	Oct.-86
	Summing up	Oct.-87
	-Summary of geological conditions	July-87
	DETAIL PLAN	
	-Study programme for detail plan	
S III	-Refraction seismic III (F)	Sept.-87
S IV	-Refraction seismic IV (T) and side-looking	Aug.-88
SS I	sonar SS I (T,F)	Aug.-88
G III	-Rock control drilling / care-drilling	July/Aug.-88
G III	-Geological detailed planning	Feb.-88/Jan. 90

5 GEOLOGICAL INVESTIGATIONS

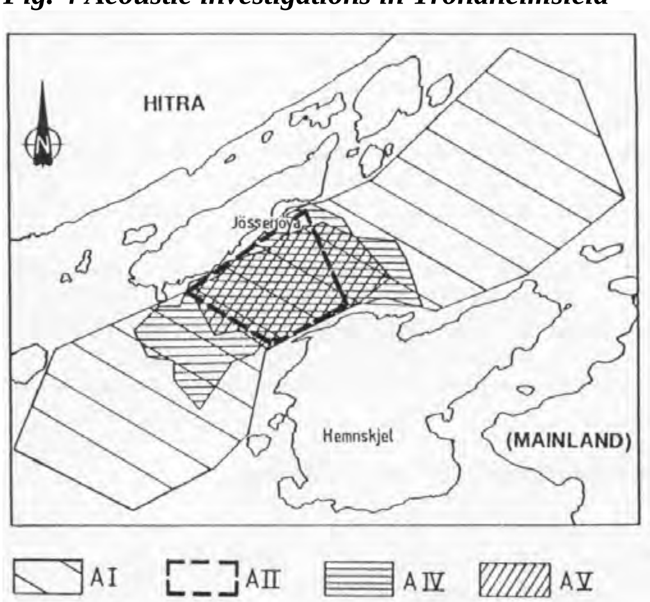
G I: These were carried out for the project in October November 1983. The work comprised geological compilations and mapping of rock distribution. jointing and zones of weakness. Evaluations were also made of the degree of difficulty and

precautions needed to ensure rock stability. The suggested course of the tunnels was also evaluated and commented upon.
G II: Later, after carrying out a first round of acoustic and refraction seismic studies, a more detailed geological investigations of the tunnel projects was performed. More accurate cost analyses were also carried out.
G III:
The last phase of geological investigations is for the detailed planning stage. detailed mapping of the tunnel portal areas and critical zones on land was carried out. drilling at one place. The most favourable locations for rock striking were now decided and cost analyses with statistical evaluation of the accuracy (minimum, average, maximum) carried out.

6 ACOUSTIC REFLECTION SEISMIC

A I:
The first acoustic investigation was carried out in Trondheimsleia during the summer of 1982. Light Seismic Equipment (ELMA) with an energy level of 25 J was used. This gave merely a first orientation of the general conditions. The measurements indicated clearly the existence of thick deposits of unconsolidated material (sediments) in Trondheimsleia.

Fig. 4 Acoustic investigations in Trondheimsleia



A II:
The first investigation with rather heavier equipment was carried out in late 1984. The areas investigated in Trondheimsleia and Frøyfjorden are shown in fig. 4. Measurements showed the presence of great thickness of unconsolidated deposits and depth to hard rock of -215 to -220 m. Recordings by boomer were, however, uncertain. Sparker gave more reliable data (greater penetration). An area towards the NW in the relevant corridor was not covered by sparker measurements. In Frøyfjorden the thickness of the unconsolidated deposits was less (max. 32 m) and penetration by boomer sufficient to give good data about rock conditions. The measurements were presented in the form of bottom contoured maps and maps of the unconsolidated deposits. A map showing the lines along which measurements were made was also included. From the last the measurement density and therefore accuracy could be judged

A III:
After further planning it was decided to look more closely at an alternative corridor further to the west in Frøyfjorden. In the autumn of 1985 acoustic profiling was carried out in this alternative corridor and in an extension to the NE in the original corridor. These results were also presented in maps showing the measurement lines,

the contoured surface of the seafloor and the unconsolidated deposits. Between phases A II and A III refraction seismic measuring was carried out. An geological compilation was made together with prognoses concerning precautions needed to en sure rock stability and cost analyses.

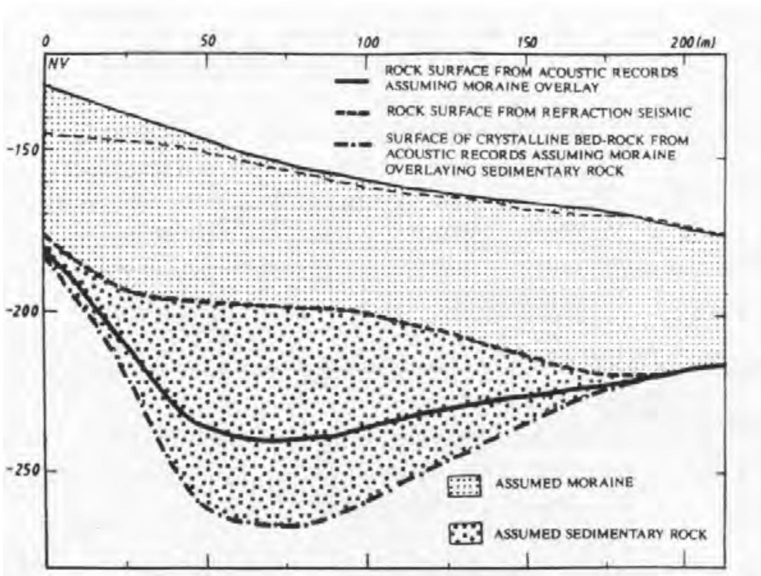


Fig. 5 Acoustic and seismic investigations in Trondheimsleia, alternative interpretative models.

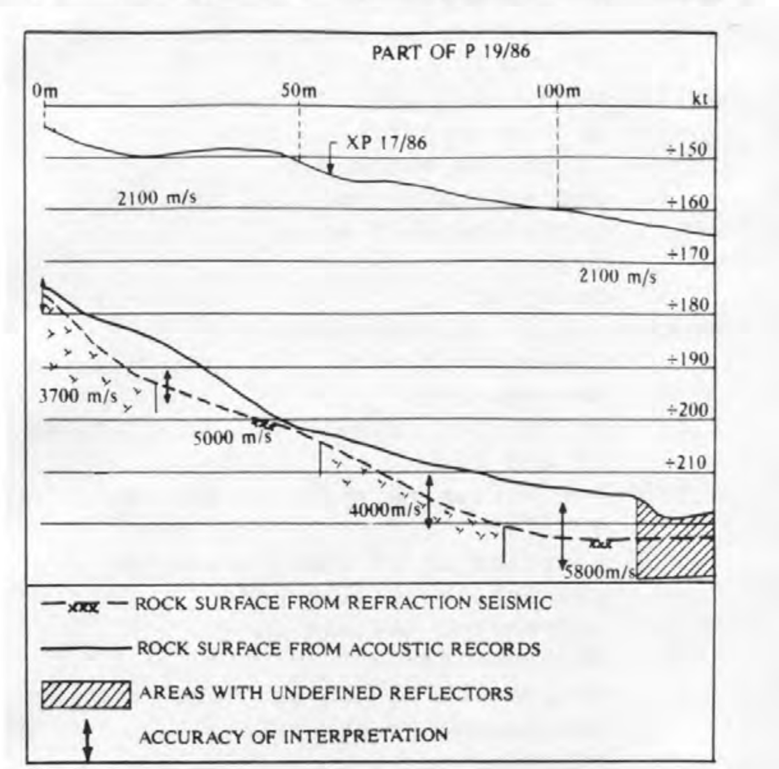


Fig. 6 Acoustic and seismic investigations in Trondheimsleia. alternative interpretative models

A IV:
In the ensuing work discussion centred on the possibility that younger sedimentary rocks were present in the relevant areas due to down faulting. This was particularly discussed for Trondheimsleia. It was therefore decided to attempt to clarify this using NGU's air cannon in a limited programme, A IV. This showed it was possible to identify reflectors in w hat was originally interpreted as unconsolidated, i.e.

superficial, deposits. These can be explained as stratigraphic layering in the moraine, a layer of strongly shattered bedrock, overlying younger sedimentary rocks, or side reflections. Assuming an acoustic velocity of 3500 m/s in possible sediments and 2100 m/s in moraine, the depth to crystalline bed-rock shown by the two interpretative models can differ by as much as 35m, see fig. 5-6.

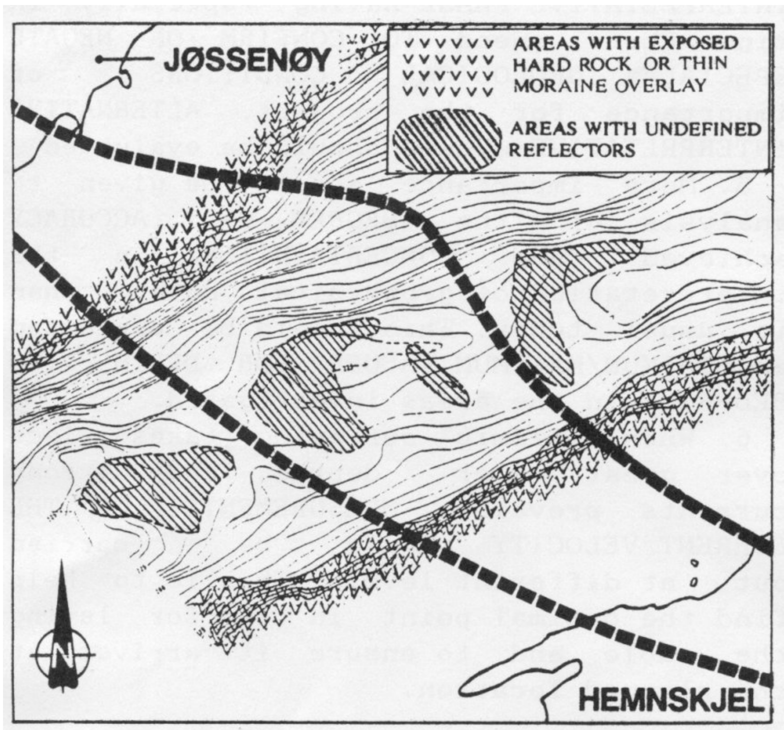


Fig. 7 Acoustic and seismic investigations in Trondheimsleia. Areas with undefined reflectors

A V:
As a result of these conclusions it was decided to carry out supplementary acoustic recording in both Trondheimsleia and a control area in Frøyfjorden. Data from these surveys could then be compared with previous data and possible discrepancies brought to light. In addition, there was a desire to enlarge the earlier areas with sparker registrations in Trondheimsleia somewhat. The results of the investigation were presented as lines of measurement maps, contoured maps of the rock surface and shading to indicate areas with deeper-lying horizons evident on sparker profiles "undefined" reflectors. It was shown that there is disagreements regards to the depth to bed-rock in certain limited are as in Trondheimsleia. However, it is possible to avoid these areas by placing the tunnel line between them, see fig. 7

7 REFRACTION SEISMIC
Refraction seismic studies were carried out in two periods. Twice for the main plan and once for the detailed plan.
S I:
The first measurements were carried out by Noteby A/S in the spring of 1985. In Trondheimsleia attempts were made to measure along 2 lines transverse to the deep channel, see fig. 8. Unfortunately strong currents and great water depths (ca. 180 m) made it very difficult to drop on the seismic cable at the planned location. Accuracy in the position was from 30-100 m except at one place where a drift of 200 m was recorded. Nor was it possible to lay or take up the cable except during times of current change. In Frøyfjorden water depths were less with a maximum of 55 m in the deep channel towards Hitra. A strong current was present here too. Frøyfjorden has, in

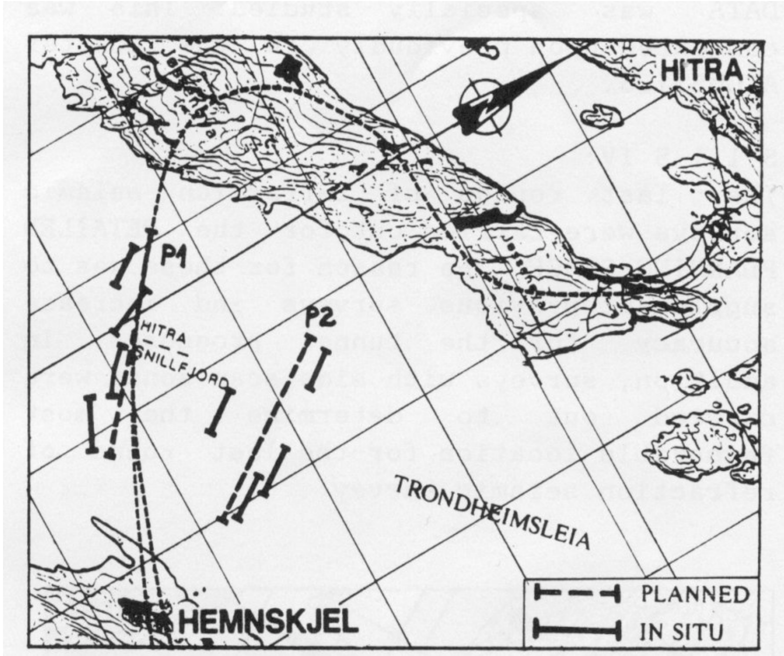


Fig. 8 Seismic investigation in Trondheimsleia

contrast to Trondheimsleia, several deep channels. Seismic profile shooting took place in the deep channels along a chosen line a cross the sound. In the deepest part, towards Hitra, 2 profiles were shot along a parallel line about 200 m to the NE. The results were presented in the form of detailed plotting of the location of the geophones and seismic profiles (map 1:1000) in addition to planar and length profiles at a scale of 1:5000. In the length profiles the seismic velocities were shown in the usual way. The degree of accuracy in positioning of the geophones was also expressed as maximum deviation from different recordings. The results showed that Trondheimsleia is characterized by two marked low velocity zones while the line over Frøyfjorden crosses 14 low velocity zones.

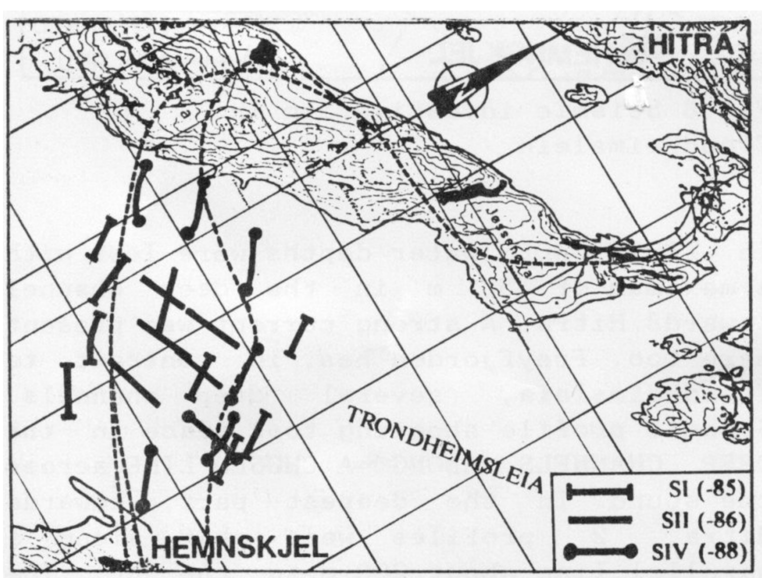


Fig. 9 Seismic investigations (SI, SII, SIV) in Trondheimsleia

S II: In the summer of 1986 refraction seismic work was carried out in an alternative profile further west, in Frøyfjorden. The measurements were limited to the deep channels along a relevant profile line. supplementary measurements were also made in the original profile in Frøyfjorden (east) and in Trondheimsleia. Both profiles in

Frøyfjorden gave a similar picture as regards crush zones, depth to bed-rock and lengths. Finally, reflection seismic measurements were compiled and interpreted along with the acoustic data. The degree of accuracy and discrepancy between seismic/acoustic data was specially studied. This was commented up on previously in the chapter *acoustic*.
S III-S IV: The last rounds of refraction seismic surveys were carried out for the detailed planning stage. The reason for these was to supplement previous surveys and increase accuracy for the tunnel prognoses. In addition, surveys with side scan sonar were carried out to determine the most favourable location for the last round of refraction seismic survey.

8 EXPERIENCE

Briefly speaking the project bas provided the following experiences regarding the use of geological-geophysical methods:

1. Thorough geological mapping at an early stage is very useful.
2. Acoustic surveying must cover a relatively large area which is then reduced for seismic measuring later.
3. The first refraction seismic measuring should be limited to the most critical parts of the corridors. It is often an advantage to shoot seismic profiles using a systematic grid net in the prescribed zones. Here also it is important that the investigated area is not to be restricted.
4. Great emphasis should be put on the interpretative phase having especially in mind the need to confirm or negate special geological conditions of importance for the project. Alternative interpretative models should be evaluated
5. More importance should be given to analysis of the degree of accuracy achieved and uncertainties in the interpretation of geophysical data than is usual today. This should be expressed as minimum/maximum values for depth to bed-rock in the areas investigated.
6. When seismic surveying takes place over great water depth and strong currents prevail measurements of the current velocity should be carried out at different levels. This is to help find the optimal point in time for laying the cable and to ensure it arrives at the planned location.

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The Rennfast Link at the Western coast of Norway with world's longest an deepest sub sea road tunnel.

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ABSTRACT:
The western coast of Norway consists of many long fjords. This article describes how two of these fjords may be crossed by building two sub sea road tunnels, at 5 830 meters and 4 390 respectively. The longer one, the Byfjord Tunnel, will be the world's longest and deepest sub sea road tunnel (223 meters below the sea level). The article describes, among other things, preliminary investigations and geological facts. The Byfjord Tunnel follows, for the most part, phyllite and quartz phyllite rocks, while The Mastrafjord Tunnel follows gnessoid rocks. The tunnels will be constructed for three lanes, with a total breadth of 11 meters. The normal blasting profile is at 72 m^2. The tunnels are designed to operate in drained situation. Leakage water is gathered in a reservoir at the tunnel's low point, from where it is pumped to the ground above through vertical pressure pipes. The tunnels will be constructed from both sides by means of conventional drilling and blasting. The average weekly progress has been at approximately 40 meters per working face. The rock is supported mainly by bolts, shotcrete and concreting. Protection against water and frost is mostly carried out by means of sheet linings/semi linings, insulated and un-insulated, or polyethylene foam, wherever there is a water leakage. The tunnels will be ventilated longitudinally by a combination of symmetrical and asymmetrical ventilators.

1 INTRODUCTION

The Norwegian west coast is split by many long fjords, which are an attraction for the tourists, but a barrier for the onshore communication. Travelling from Stavanger in the south to Trondheim in the north means that you will have to cross 12 fjords by means of ferries. "The regular coastal road" is the term for an idea of a regular road along the coast, leading from

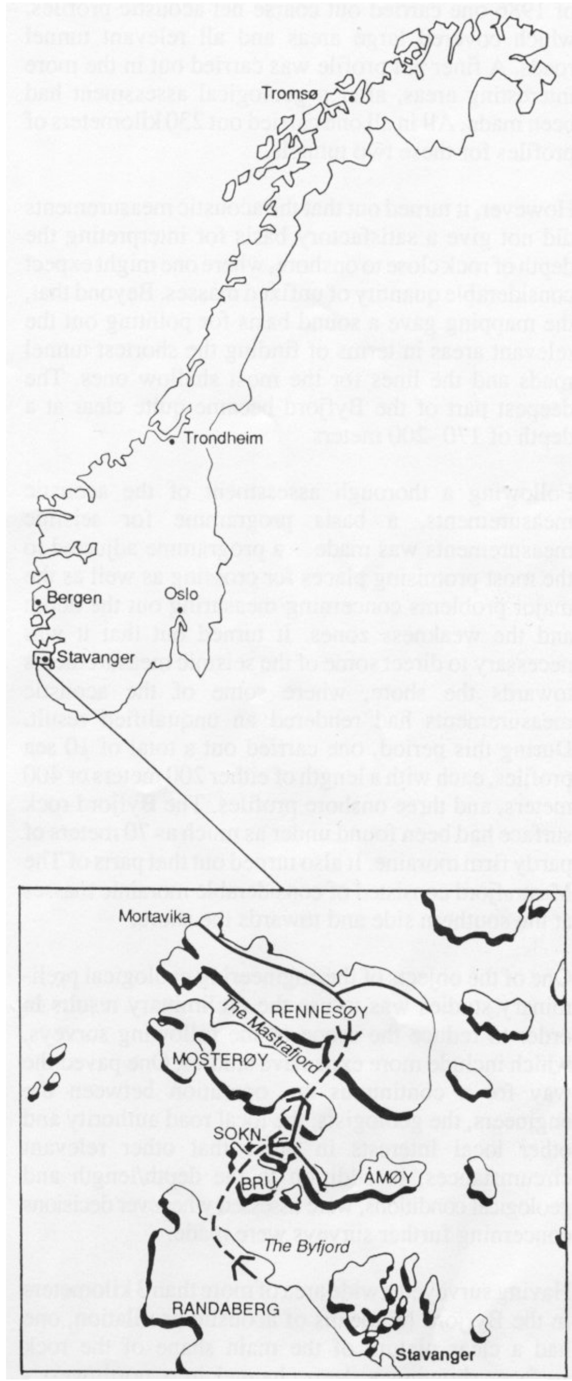

Fig. 1 The Rennfast-project

Stavanger to Trondheim, but with no ferries. Several of the ferries will, within a few years, be replaced by bridges and sub sea tunnels.

However, the crossing of two fjords, The Sognefjord and The Boknafjord, will be extremely complicated and expensive. Due to the fact that one cannot get rid of the ferries crossing these fjords, at least not at this point, it is however possible to reduce the actual crossing. The crossing of The Sognefjord was shortened from 40 minutes to 20 minutes in 1990, while the Rennfast-Project is estimated to shorten the crossing over The Boknafjord from 50 minutes to 20 minutes. In addition, the Rennfast-Project is linking together and providing mainland connection for all the major islands within the Rennesøy County. This county has a population of approximately 3 000 inhabitants, and is situated just north of Stavanger.

Making the project profitable, here speaking of profitability for the community, and the fact that the project can be carried out, is only possible through a combination of the construction of a regular coastal road combined with a replacement/shortening of the local ferries. All in all the project consists of 30 kilometres of new roads, bridges and tunnels, and a total of 22 kilometres belongs to the regular coastal road. As of June the first, 1992, the coastal road will be given the term National Road No 1.

The two sub sea road tunnels crossing The Byfjord and The Mastrafjord, at 5 830 meters and 4 390 meters respectively, make out the two major constructions of the project. The Byfjord Tunnel will, when it is opened, be the world' s longest and deepest sub sea road tunnel.

The project also includes the construction of two conventional bridges, at 170 meters and 280 meters respectively, as well as the construction of a 19,2 kilometre two lane road.

One more special feature of the project is the construction of a new ferry terminal at the northern part of Rennesøy. In order to protect the harbour, one will have to construct a 400 meter long and 8 meter high berm breakwater. The total volume of the breakwater is 400 000 m3, and it consists of blocks ranging from one tonne up to 23 tonnes. All of the rocks being used for the breakwater are taken from the actual terminal area and transplanted out to the breakwater. The construction period lasted for eight months, and the breakwater was finished by December 1991.

The total cost of the project has been estimated to US$ 136 millions (NOK 750 millions, 1991 estimation). The total cost of the two sub sea road tunnels has been estimated to US$ 75 millions (NOK 410 millions) The cost includes all expenses relating to planning, construction management and the actual construction of the tunnels. Thus the unit price per running tunnel meter will be at US$ 7300 (NOK 40 000).

The project is one hundred percent privately financed. A finance company has undertaken the task of financing the project, and in return been given the task of collecting the toll money once the tunnels are opened, until the loans have been paid, but within a maximum number of years, that of 25. It has been estimated that in the year 1993, the year the tunnel will open for traffic, 2 500 cars will use The Byfjord Tunnel within a 24 hour period.

The very first sod of this project was turned in November 1989, while the tunnel construction was fully implemented by August 1990. One had, as a starting point, estimated that the construction period for The Byfjord tunnel would last for 2 years and 7 months, including construction from both sides. The premise was then that one would be able to work through a distance of 60 meters on average per week, 30 meters from each side. However, the actual progress has been higher than expected, close to a distance of 80 meters on average per week, thus resulting in a total construction period of approximately 2 years and 4 months. March 1992 saw The Byfjord Tunnel breakthrough, and the tunnel will be ready for traffic by November 1992.

Even though the project is privately financed, the Government -through The Public Road Administration of Norway -is the one body undertaking the building, and thus the responsible body concerning the planning and the construction of the project. As a consequence of this, The Public Road Administration is the one body that will own and run the construction, once it is opened.

The Public Road administration is also constructing one half of The Mastrafjord Tunnel. The other half of this tunnel, and all of The Byfjord Tunnel, is constructed by a local contractor, a company named A.F. Rogaland Contractors. This company obtained the tunnel contract in stiff competition with other large Norwegian building contractors, which also had the necessary tunnel construction expenses.
In the chapters hereafter each of the two tunnels is described corresponding to the contract parts in this way:
Byfjord I: The southern part, 3020 m length
Byfjord II: The northern part, 2810 m length
Mastrafjord. I: The southern part, 2600 m length
Mastrafjord. II: The northern part, 1790 m length

2 PRELIMINARY INVESTIGATIONS

The preliminary investigations were initiated during the winter 85/86 by a geologist carrying out field surveys. These surveys, as well as economic maps in the scale 1 :5000, aerial photographs with stereoscopic coverage and ordinary geological maps which are available, formed the basis of the first sketches of the corridors for possible tunnel roads. During the summer of 1986 one carried out coarse net acoustic profiles, which covered large areas and all relevant tunnel roads. A finer net profile was carried out in the more interesting areas, after a geological assessment had been made. All in all one carried out 230 kilometres of profiles for these two tunnels.
However, it turned out that the acoustic measurements did not give a satisfactory basis for interpreting the depth of rock close to onshore, where one might expect considerable quantity of unfixed masses. Beyond that, the mapping gave a sound basis for pointing out the relevant areas in terms of finding the shortest tunnel roads and the lines for the most shallow ones. The deepest part of the Byfjord became quite clear at a depth of 170 -200 meters.
Following a thorough assessment of the acoustic measurements, a basis programme for seismic measurements was made -a programme adjusted to the most promising places for crossing as well as the major problems concerning measuring out the depth and the weakness zones. 11 turned out that it was necessary to direct some of the seismic measurements towards the shore, where some of the acoustic measurements had rendered an unqualified result. During this period, one carried out a total of 10 sea profiles, each with a length of either 200 meters or 400 meters, and three onshore profiles. The Byfjord rock surface had been found under as much as 70 meters of partly firm moraine. It also turned out that parts of The Mastrafjord consisted of considerable morainic masses at the southern side and towards the shore.
One of the objects of the engineering geological preliminary studies was to use the preliminary results in order to reduce the scope of the following surveys, which include more expensive studies. One paved the way for a

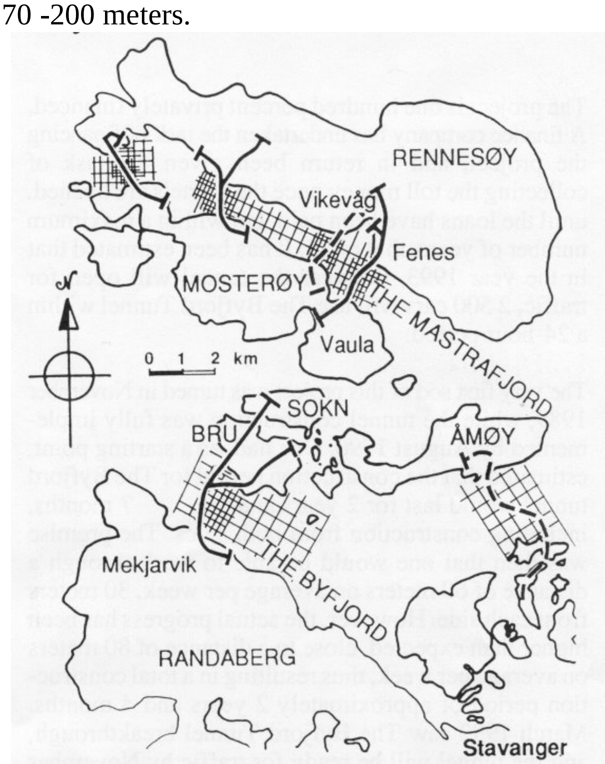

Fig. 2 A general view of the investigated tunnel lines, including a net of acoustic profiles.

continuous co- operation between the engineers, the geologists, the local road authority and other local interests in order that other relevant circumstances, in addition to the depth/length and geological conditions, were assessed whenever decisions concerning further surveys were made.
Having surveyed a wide area of more than 3 kilometres in the Byfjord by means of acoustic profiling, one had a clear picture of the main shape of the rock surface, showing a deep channel in a north/west - south/east direction. However, the interpretations turned out to be somewhat uncertain, due to great depth to rock on the south-eastern side of the Fjord. By analysing the longitudinal profiles, it turned out that the shortest achievable tunnel road was a curve between Mekjarvik and Bru (approximately 5,8 kilometres).
The results of the first seismic profiles of The Byfjord came up with a less safe determination of the rock surface than the acoustic profiles. Some weakness zones appeared (low seismic velocities), but not of such a degree that one became doubtful to the implementation of the tunnel construction.
The preliminary seismic profiles confirmed that the Mekjarvik -Bru line still was the favourable one.
Similar investigations were carried out in The Mastrafjord, where one came up with three relevant places for crossing, and they were made the subject of a technical, economic and social analysis of consequence before the final tunnel road was chosen.
During the 1986- 89 period of planning, and following preceding acoustic profiling, one carried out a total of 15 refraction seismic sea profiles and 4 onshore profiles concerning The Byfjord Tunnel. When it comes to The Mastrafjord Tunnel, one carried out 14 sea profiles and 4 profiles onshore.
There is a continuous lengthwise coverage of the tunnel roads for the involved distances, supplemented by some crosswise profiles. The scope and the placing were, in the main, directed by the relevant approach for each planning step of the project.
One did also carry out control drilling of the rock at some supposedly weak points along the tunnel route onshore. In addition core drilling was carried out for The Mastrafjord Tunnel, mostly to map the course of a large grinding zone. one made an assessment whether to carry out more core drilling both from land and sea, but one reached the conclusion that the costs were higher than the utilitarian value of such surveys here.

VELOCITY (METER/SECOND)	LENGTH OF PROFILE AT GIVEN VELOCITY
6 500 -6 000	1 100 METERS
5 000 -5 500	775 METERS
4 500 -5 000	245 METERS
4 000 -4 500	200 METERS
3 500 -4 000	145 METERS
3 000 -3 500	60 METERS
<3 000	0 METER

Table I: Arrangement of seismic velocity in rock in The Byfjord.

3 LONGITUDINAL SECTION AND CROSS SECTION OF THE TUNNELS.

The total length of The Byfjord is 5 830 meters, with the deepest point situated in the middle of The Fjord, 223 meters below the sea level. A normal interpretation of the refraction seismic measurements rendered, at this point, a rock surface at a depth of 168 meters below the sea level. Including maximum uncertainty of the interpretation and of the accuracy of measurement, one has decided the "measuring out" rock surface to be at 175 meters below the sea level. This renders a minimum rock cover of 40- 41 meters, while the likely rock cover is 45 -46 meters.

1. The roof of the tunnel must have a load-carrying capacity in order to carry the superjacent load, consisting of rock, unfixed masses and water pressure.

2. There must be both space and counter-pressure for an effective high-pressure pre-injection.
3. Concerning weakness zones where rock slides may occur, there must be a safety margin for any development of slides, until counter-acts have become effective.
4. In zones of weakness containing clay, the cover should contribute to keep the inlet water gradient low.

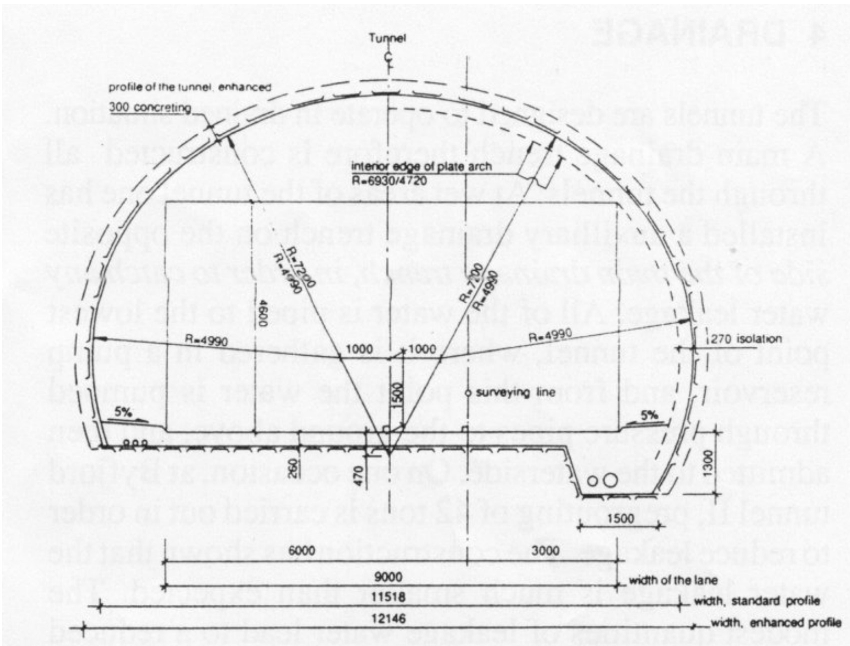

Fig. 3 Standard cross section.

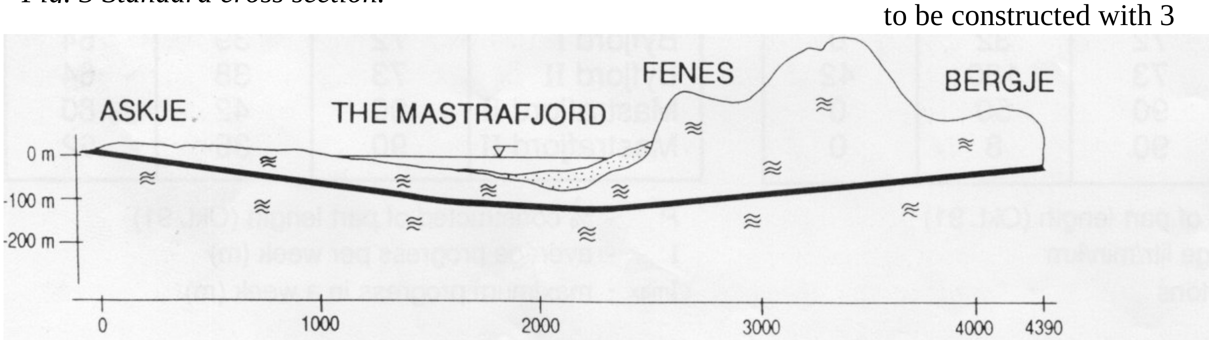

Fig. 4 Longitudal section of The Byfjord Tunnel

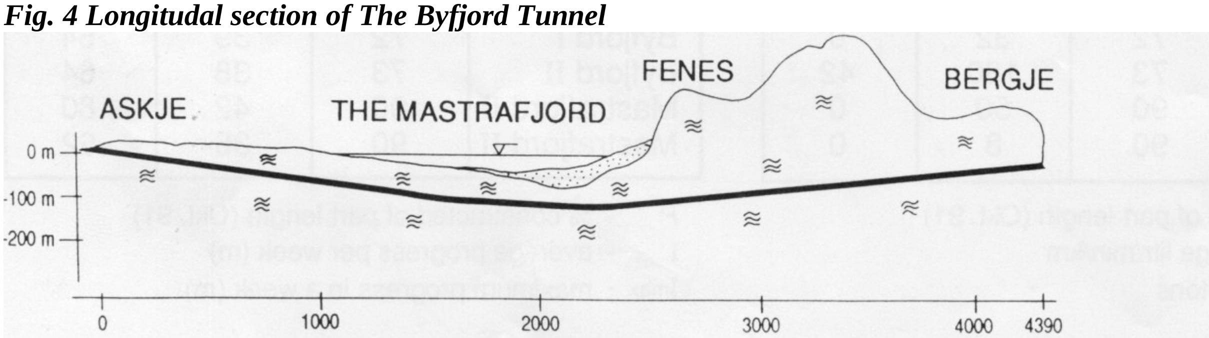

Fig. 5 Longitudal section of The Mastafjord Tunnel

Primarily requirements 3 and 4 have been measuring out, due to the great depth below the sea level.
The total length of The Mastrafjord Tunnel is 4 390 meters, with its deepest point situated 133 meters below the sea level.
Demands stating that the gradient should not exceed 80 per thousand (1:12,5) were put forward due to the traffic and safety requirements. Long and steep gradients brought about that the tunnels had to be constructed with 3

lanes, where the lane in the middle would function as an overtaking lane in the gradients. The total width of the tunnel is II meters, with a total of 9 meters carriageway.
The standard projected blasting profiles for the tunnels are that of 72 square meters, including main drainage trench. However, in zones where there is a danger for a complete concreting, the profile is enhanced to 79 square meters.

4 DRAINAGE

The tunnels are designed to operate in drained situation. A main drainage trench therefore is constructed all through the tunnels. At wet areas of the tunnel one has installed a auxiliary drainage trench on the opposite side of the main drainage trench, in order to catch any water leakage. All of the water is piped to the lowest point of the tunnel, where it is gathered in a pump reservoir, and from this point the water is pumped through pressure pipes to the ground above, and then admitted to the waterside. On one occasion, at Byfjord tunnel II, pregrouting of 42 tons is carried out in order to reduce leakage. The construction has shown that the water leakage is much smaller than expected. The modest quantities of leakage water lead to a reduced volume of the emergency reservoirs and reduced pump units, which, as a starting point, should be measured for a capacity of 300 litres per minute per kilometre. The drain pipe system is all over designed with considerable overcapacity .Plastic pipes with minimum diameter 200 mm are used. The picture of leakage (October 1991) of each tunnel part is shown in the figure.

Tunnelpart	P %	L litres	G tons
Byfjord I	72	32	0
Byfjord II	73	180	42
Mastrafjord II	90	50	0
Mastrafjord II	90	8	0

P - % constructed of part length (Okt. 91)
L - average leakage litr/min/km
G - pregrouting in tons

Table 2 Water leakage

5 GEOLOGICAL CONDITIONS

The dominating rocks along The Byfjord tunnel are phyllite and quartz phyllite, while amphibolitic and thonallitic gneiss are main rocks along The Mastrafjord tunnel. As described above there is a minimum rock cover of 40-41 meters. The actual construction of the tunnels has, so far (October 1991), shown a rather high degree of accordance between the expected rock conditions and the real ones (faults, joints, stratification/ folding). Core drilling, probe zoning arid geo-electrical measurements are carried out w hen constructing, in order to obtain further information concerning the fore going rock bottom, not least concerning any water leakages. The probe holes (2 or 4 holes) are drilled at a length of 24 meters, with a minimum overlap of 8 meters.

6 TUNNEL BLASTING

The tunnels are constructed from both sides of the fjords, by means of conventional drilling and blasting. A modem hydraulic drilling rig, equipped with data- controlled booms and enclosed noise suppressed operator-room, is used on one of the heading faces. Moreover one has made use of open, conventional hydraulic drilling rigs with manual marking of the drill profile and manual control of the drill booms.

Perimeter holes must be placed within 100 mm of the theoretical collaring point, but not inside the tunnel perimeter. Blasting of perimeter holes is carried out cautiously so as to avoid damage to the tunnel crown and walls. Hole spacing on the perimeter is 500 - 700 mm, with the distance to the next outermost row of 700 -900. The pen-meter holes are primed with weaker explosives, such as pipe-charges, that are less effective than those used elsewhere in the cross section. Hereby smoother perimeter surface is achieved and lower costs of support measures and tunnel maintenance.
The average progress per week, as well as maximum progress per week, are shown in table (October 1991).

Tunnelpart	P %	I m	Imax. m
Byfjord I	72	39	64
Byfjord II	73	38	64
Mastrafjord II	90	42	80
Mastrafjord II	90	36	62

P - % constructed of part length (Okt. 91)
I - average progress per week (m)
Imax - maximum progress in a week (m)

Table 3 Construction progress

The average progress includes the weeks with a low progress, for instance due to concreting. One week consists of 100 working hours.
The progress of the rounds is between 4 and 5 meters, depending on the rock conditions.
One has carried out two core drillings in zones regarded as weak. The drill strings, at 51 meters and 55 meters respectively, gave evidence of considerable better rock than expected. One has also carried out geo-electric al measurements in some of the probe holes, in order to determine more accurate low quality rock ahead. The method is based on the electrical conditions of rocks.
From approximately station 2700 the Byfjord I tunnel is passing a fault zone which is estimated to have a width of 75 m and a seismic velocity of 31 00 -3600 m/s from the preinvestigation. The fault zone has partly been excavated by short rounds and concrete lining at tunnel face.
The quality of the rock mass at the time of writing this paper, has been so poor that excavation is carried out by the excavator, without doing any blasting work. Double layer of spiling bolts, length 7,5 m and c/c 0,40 m is installed before excavation. Temporary support before installation of concrete lining is carried out by steel fibre reinforced shotcrete and rock bolts.
Mapping of the real character of the rock surface along the tunnel road as well as a description of permanent support, is made by the engineering geologist.

7 SUPPORT MEASURES
The permanent support has, among other things, as its starting point that the carriageway must be close to one hundred percent free of sprinkling. When it comes to the narrow emergency exit pedestrian lanes on each side, the requirements are somewhat less stringent.
The main support measures are :

-bolts
-shotcrete
-concreting

All the bolts being used are treated with both hot-dip galvanization and powder-lacquering with epoxy. To a large degree one has utilized bolts with a combined function, immediate end anchoring as temporary support and grouting as permanent support. End anchoring is achieved by means of a two-compounded polyester cartridge or by an expansion unit. The lower part (100- 150 mm) of the bolt holes are tightened by means of polyurethane foam. Separate pipes for grouting and air evacuation are put through base plate and foam (the air evacuation pipe up to top of hole). Grouting is carried out from below. Bolts of deformed steel bars are used. The principle is shown in the figure. One also has made use of pipe bolts, with expansion unit, including grouting from above through the pipe. Ordinary grouted bolts (with grout pumped into the bottom of the borehole via a flexible hose in advance) of deformed steel bars are used as permanent support, when put in away from heading face.

The bolt lengths of 3 (in most cases) and 4 meters are most in use, with the diameter 20 mm.
In cases of spiling one has utilized bolts of deformed steel bars which are approximately 8 meters long and have a diameter of 25 mm.
Shotcrete with addition of steel fibre reinforcement is applied almost systematically, not least in the phyllite. The quantity of fibre is 55 kg per cubic meter of concrete. Length of fibre is 25 mm. Total thickness of shotcrete layers is normally 60-80 mm. In poor rock quality one makes use of a thickness of shotcrete up to 150 mm, if necessary combined with perimeter shotcrete beams/ribs having a thickness of 200- 300 mm.
Concreting is carried out in larger unstable zones or in zones of swelling clay. The minimum thickness is 300 mm. All concreting should leave room for a subsequent membrane installation or actual frost protection.

The concrete quality of shotcrete and concreting is to be at least C45 with requirements of environmental class MA, very aggressive, according to NS 3420, 2nd edition.
The table shows quantities of support measures so far (October 1991) including permanent support of each tunnel part.

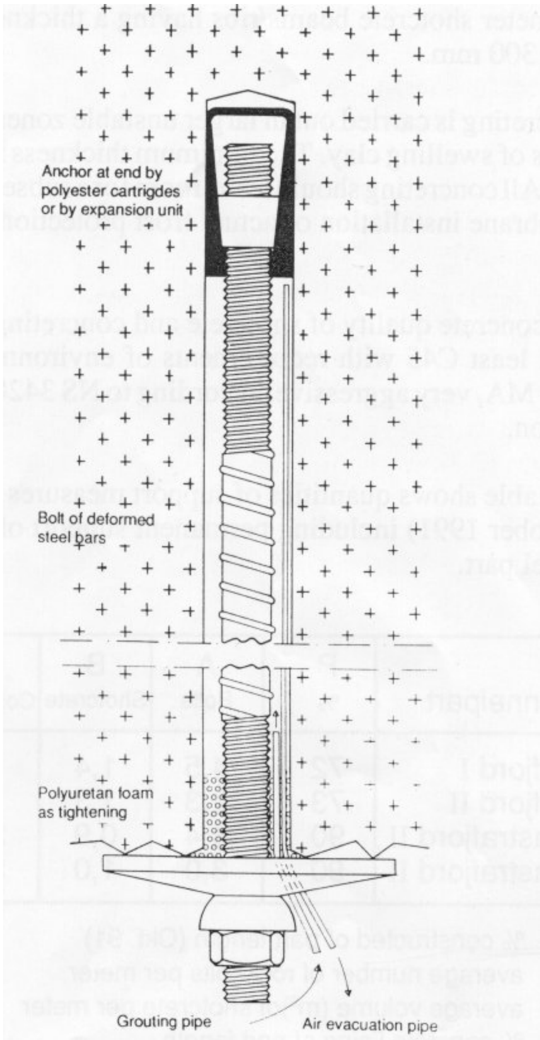

Fig. 6 Combibolt

Tunnelpart	P %	A Bolts	B Shotcrete	C Conreting
Byfjord I	72	4,5	1,4	1,6
Byfjord II	73	2,3	1,3	0
Mastrafjord II	90	4,4	0,9	2,1
Mastrafjord II	90	3,0	1,0	3,1

P - % constructed of part length (Okt. 91)
A - average number of rockbolts per meter
B - average volume (m^3)of shotcrete per meter
C - % concrete lining of part length

Table 4: Support measures

8 WATER AND FROST PROTECTION

The frost quantity where the fjord meets the open sea on the Norwegian south-western part, is very low. However, the tunnel ventilation (longitudinal ventilation) increases the penetration of frost. As a basis in order to measure out the frost protection, the zone of frost has been set at 500 meters entering from each of the tunnel mouths.

Moreover, it is a superior object that the carriageway should be close to a hundred percent dripless. The very first 100 meters entering each of the tunnels and major, coherent areas of leakage somewhere in the frost zone must be protected by means of insulated sheet-metal linings. Mineral wool of 50 mm thickness is used as insulation and sheets of aluminium. Smaller leakage areas within the frost zone will be protected by means of 50 mm thick mats of polyethylene foam (FE foam), fastened to the rock by means of bolts.

Tunnelpart	SI1	SI2	SI3	PE	SP
Byfjord I	6	1	0	10	5
Byfjord II	12	2	23	0	3
Mastrafjord II	18	1	11	29	4
Mastrafjord II	10	5	6	17	6

SI1 - Sheet linings, insulated
SI2 - Sheet linings, uninsulated
SI3 - Sheet semi linings (roof), uninsulated
PE - Polyethylene foam
SP - Thin plastic sheet, uninsulated

SI1-3 - % of part length
PE,SP - % of total rock area

Table 5: Water and frost protection

To water protection outside the frost zone, one will make use of un-insulated sheet metal linings or semi linings (root) of corrugated aluminium. Smaller leakage areas and simple leakages will be protected by means of the PE foam or thin plastic sheets with watertight bolt lead-through.

The aluminium sheet linings, insulated as un-insulated, are connected to beams/arches made of fibre glass. The traffic causes considerable suction/pressure loads, which the construction has been dimensioned to stand. Based on registrations so far (October 1991), the scope and types of water and frost protection are estimated to be approx. as shown for each tunnel part in the table.

9 EQUIPMENT

The tunnels will have longitudinal ventilation by means of a combination of symmetrical and asymmetrical impulse ventilator fans of stainless steel. One will install a total number of about 60 fans in The Byfjord Tunnel, and a total number of about 32 in The Mastrafjord Tunnel.

Lightning, including a scaling down for each zone towards "nightlight" in the middle zone, will be installed in both of the tunnels.

Moreover, one will construct lay-bys for damaged vehicles and turn around niches, and install emergency telephones, CO-meters, wind-meters, equipment for radio communication etc.

Control and surveillance of the tunnels will be executed by a larger security agency being planned in the region where the tunnels are situated.

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Hvaler sub sea rock tunnel, the use of drill ships for investigating fault zone

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ABSTRACT
Experience obtained from using a large ocean-going drill ship for investigating a fault zone under the strait Løperen between the islands of Asmaløy and Kirke øy in the Hvaler archipelago by vertical drilling, is considered. A total of five boreholes were attempted but only two holes provided useful data. Satisfactory information was, however, obtained for making a decision as to the vertical alignment of the tunnel under the fault. From this point of view the operation was successful. Had the rock head not been encountered at a fairly high level, it might have proved difficult, however, to penetrate through sediments down to the proposed tunnel level due to the high rate of wear of drill bits. Also sampling sediments proved difficult with available equipment. The ship itself, however, provided an excellent drilling platform both as regards positioning of the vessel and drilling operations. Based on either the total length or the length of effective boreholes, the drilling costs vary in the range of 17- 23.000 NOK per m.

l. PRELIMINARY INVESTIGATIONS
Plans for providing a road link between the islands of Asmaløy and Kirke Øy in the Hvaler archipelago, county of Østfold were considered for many years after toll collection was terminated on the road from the mainland to Asmaløy. Both bridge and sub sea rock tunnel solutions were studied, but in 1986 the planning process was concentrated on a tunnel solution located in the southern part of Løperen, the strait between the two islands.
Here geological mapping and acoustic sounding indicated a favourable tunnel alignment crossing under the small island of Kvernskjær. The acoustic results showed only thin layers of sediments above bedrock, 5-10 m at the most, and shallow waters apart from the main ship channel near Asmaløy where the sea- bed was located 40 m below the sea level. Here a fault zone was predicted running approximately in a direction north-south.
Further geophysical investigations, using seismic refraction methods, revealed, however, that moraine deposits had masked the acoustic results in some areas. Also further fault zones indicated by geological mapping, were confirmed. The main zones, however, were the one under the ship channel near Asmaløy and another quite close to Kirke Øy, again running in a north-south direction. With the upper part of the fault zones filled with moraine sediments underlying sandy material, and relatively steep side slopes it was not possible to determine an exact width of the zones or determine the level of transition between moraine and bedrock in the zones exactly from the seismic results.
Since determining a safe tunnel level at the se two fault zones was critical for how deep the tunnel had to be located and hence for the tunnel length it was of major importance to obtain more and reliable information on the zones.

2. DRILLING FROM BARGE
For the zone near Asmaløy, vertical drilling using a Rock 601 drill rig and Odex equipment was performed from a barge in the autumn of 1986. With the relatively strong current in Løperen (approx. 3 knots near the surface) it proved difficult to anchor the barge and keep it in position and at the same time pre- vent obstructing the ship traffic. Furthermore, sampling with the Odex system did not prove successful as sandy/silty material filled the casing. Hammer drilling without casing

was, however, successfully performed down to a depth of 51 m below the seabed. Having passed through sediments with varying rock content, the rate of penetration in the lower part of the borehole was fairly constant (70 sec. per m). This rate is too high to indicate normal, not fissured, rock, particularly taking the total length of the drill string of approximately 90 m into account. Based on this information and the indicated width of the fault zone at this location (around 50 m), it was decided to realign the tunnel further north on the Asmaløy side. This would not avoid the fault zone altogether, but supplementary seismic refraction investigations had indicated that the zone was less wide further north (approximately 25 m). Still it was again necessary to obtain more exact data on zone width and depth to "bedrock" in the zone.

3. AVAILABLE OPTIONS FOR INVESTIGATING FAULT ZONE

Further seismic investigations was not considered to have a potential for providing the necessary data. Therefore some sort of drilling activity was visualised as the only way to obtain further information. Sloping core drilling from the shore and through the zone was considered as one alternative. This method however could require repeated drilling depending on the condition of the fault zone material at the level of intersection. This would certainly be the case if zone penetration was prevented due to loose material even after stabilizing the hole by grouting. Secondly even if penetration in fissured rock or sediments was possible locating
possible firm rock further down would require a new hole. Also if firm rock was indicated in the first hole, locating the rock head at a higher level would require a new hole. Repeated drilling would mean increased cost and successful results were not quite guaranteed.
Vertical drilling also has its disadvantages as far as locating the perimeters of the zone is concerned and detecting deviations of the fault plane from the vertical. The accuracy of the information depends on the spacing of the boreholes and hence cost is involved. This time the cost of drilling from a barge was considered as well as hiring a drill ship used in off-shore oil- related geotechnical investigations.
From experience with the difficulties in anchoring a barge in Løperen, and the cost involved in rigging and operating necessary drilling equipment from a barge, it was decided that a drill ship would be preferable even with the higher day rates. This implied that locating the drill ship and performing the drilling-operation would require far less time. Also compared with the uncertainties and possible cost of coring from shore, the project planning group decided that hiring a drill ship would be preferable for investigation the fault zone under the ship channel. The Norwegian Road Research Laboratory was entrusted with the task of hiring a drill ship and supervising the drilling-operations.

4. DRILLING OPERATIONS

In order to reduce costs by utilizing off-season periods for the drill ship industry, a contract was signed with Offshore Support Ser- vices for the use of the drill ship "Bucentaur" (see fig. 1.) in January 1987. The shipping company claimed to have experience in winter drilling and predicted no problems in this connection. However, January 1987 turned out to be severely cold and winter temperatures off-shore and near-shore are not quite the same. With temperatures in the Hvaler archipelago dropping to -20 °C, concern for both ice-floe hazards in Løperen and difficulties with frozen drill mud etc. caused a cancellation of the contract.
A new contract was signed for drilling-operations in May 1987 with increased rates. The drilling programme was based on a minimum of three holes continued down to bed- rock and 10 m into the rock head or a depth of 115 m below sea level if bedrock was not encountered at a higher level. Options for further holes were provided in the contract.
For drilling in Løperen Bucentaur was equipped with a radio positioning system, (transponders hired from A/S Geoteam), API drill string and sampling equipment as well as piggy- back diamond coring equipment (Diamond Boart) and a Seaclam re-entry bottom frame (Fugero). Drill bits and

drill mud was to be reimbursed according to use. Bucentaur operates with a crew that makes drilling possible 24 hours a day in two shifts.

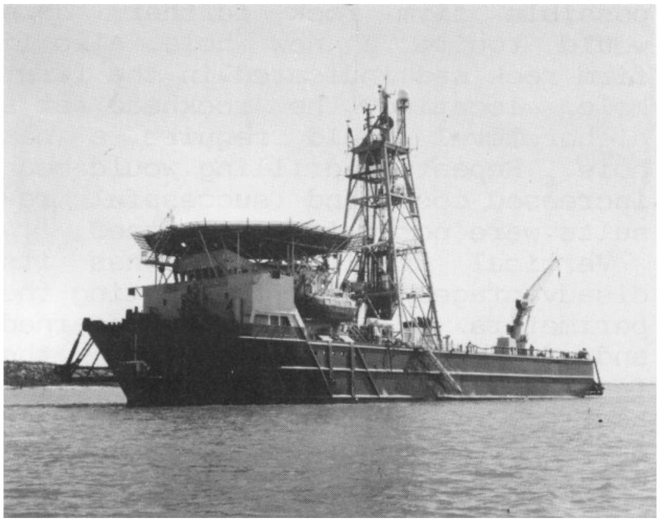

Fig. 1 Bucentaur on location in Løperen

A soil mechanics laboratory was available on board, and this was manned by laboratory personnel from the Road Authorities. In addition two mission leaders (one for each shift) and a drill inspector from NRRL and one representative from the County Roads Office of Østfold participated. Also an adviser was hired from the Continental Shelf and Petroleum Technology Research Institute (IKU), Trondheim together with a geologist from a private consultant (Taugbøl & Øverland). In order to measure possible deviations of the drill string from the vertical, necessary instruments and an operator was hired from the Foundation for Scientific and Industrial Research (SINTEF), Trondheim.

The idea behind the latter was to map the boreholes sufficiently accurate to avoid intersection with the tunnel and prevent uncontrolled water leakage w hen driving the tunnel. An alternative would be to fill the boreholes with cement grout after drilling was completed.

Bucentaur was on location in Løperen at 06.00 hours on May 4th 1987. Due to difficulties with the navigation system, drilling preparations did not start till 10.00. The delay was entirely the drill ship's responsibility also cost wise, but it created some uncertainties as to the exact position of the ship. In order to clarify this point a check was made by teodolite from shore proving that the correct position had been located.

In this position Bucentaur remain- ed stationary without the aid of anchors, but guided by the dynamic positioning system (DP). Any deviation from the selected location was detected by sensors placed on the seabed. The sensors in turn activated the thrust propellers bringing the ship back on location. Movements were recorded continually and plotted and the maximum deviation from the selected point was proved to be within + 2 m. The DP-system also made it easy to shift location by telling the system to execute orders like 2 m aft, 1 m starboard.

With the ship in position the Sea- clam re-entry frame was lowered to the seabed. But the seabed proved to be rather uneven, and since the Seaclam only could operate within deviations less that 3° from the vertical, the re-entry frame was shifted around the bottom to locate a suitable position. This operation took more than 4 hours having tried more than 30 locations. Also for later boreholes this procedure proved difficult and for the two last hales the use of the Seaclam was dropped altogether.

In all 5 holes were drilled. Through the upper sediments the API drill string was used with drill bits in the range of 73/4" -8 1/2". The API string has an inner diameter of 4" making it possible to take samples with hammer samplers (2" or 3") being lowered down to the API drill- bit and hammered into the sediments below or by Christensen sampler locked together with the API-drill- bit during drilling.

When further penetration with the API-string stopped, drilling was continued using the piggy-back system with Diamond Boart HR or NR diamond coring equipment through the API string.

Bucentaur is equipped with heave compensator so that drilling operations will not be disturbed by waves or tides. During the drilling in Løperen the water was very calm and tide variations are relatively small.

Observations made during drilling in the various locations are as follows:

Borehole l:
Drilling was started on Monday 4th May 1987 at 14.30 hours and continued till 23.30 hours, i.e. a total of 9 hours. The water depth was record- ed as 41,05 m. After 5 m penetration of the API-drill string into the sea- bed, sediment samples were taken with the 3" hammer sampler (5,0
5,45 m). The retrieved material was classified as gravelly sand. At a depth of 6 m below the seabed the API-drill string ceased to penetrate, and it was decided to continue with the piggy-back Diamond Boart coring equipment -HR dimension. Rock cores from a 1,5 m thick boulder was retrieved, but further down the drilling resistance varied and only parts and fragments of cores were retrieved, indicating sediments. W hen a depth of 11,5 m below the seabed was reached (- 52,5 m below sea level) further penetration stopped and the equipment was withdrawn exposing a completely worn down drill bit.

Borehole 2:
Drilling started Tuesday 5th May at 00.30 hours and continued till 07.00 hours, i.e. a total of 6 1/2 hours. The water depth was recorded as 42 m. The API-drill string penetrated easily down to 5,5 m below the seabed, and then nearly stopped. Due to the slow penetration it was decided to change drill bit. The new drill bit (Stratapax) had a slightly larger diameter and problems were encountered in passing through the re-entry frame and down into the borehole. It was therefore decided to move to a new location.

Borehole 3:
Drilling started Tuesday 5th May at 11.00 hours and continued till Wednesday 6th May at 23.00 hours, i.e. a total of 36 hours. In addition 3 1/2 hours were spent on locating a suitable position for the Seaclam re-entry frame. The water depth was recorded as 44,4 m. The API-drill string with Christensen sampler penetrated easily down to 5 m below the seabed. Drilling resistance further down varied as boulders were passed. The average rate of penetration was 1,2 m per hour including sampling. The Christensen sampler proved unsuitable for sediments with stones and boulders in a loose matrix as the sampler entrance was blocked by stones while finer particles were washed away with the drill mud. The lower part of the API-drill string was furthermore blocked by sediments w hen the Christensen sampler was withdrawn and hence preventing the sampler from locking into position w hen re-entered into the drillstring. The 3" sampler was used to remove the sediment plug from the API-drill string, and sampling was continued using the hammer sampler. The retrieved samples show sandy and gravelly material apart from the sample taken 5 m below the seabed where uniformly graded sand was encountered.
Penetration with the API-drillstring ceased at a depth of 16,7 m below the seabed. It was therefore decided to continue with the Diamond Boart equipment -NR dimension. Rate of penetration and samples indicated that the drill string passed through sediments with boulders and blocks. W hen reaching a depth of 29,65 m be- low the seabed (- 74,05 m below sea level) it became clear that firm, solid rock was being penetrated. By careful study of the cores, it was concluded that the rock head was at a level of 67,9 m below sea level (23, 5 m below the seabed) .Drilling was therefore terminated.
Since drilling was stopped at a level well above the tunnel level, it was decided to omit deviations measurements in the borehole.

Borehole 4:
Drilling was started on Thursday 7th 1987 at 03.00 hours and continued till 06.30 hours, i.e. a total of 31/2 hours. The water depth was re- corded as 44,8 m. W hen again it proved difficult to locate a suitable position for the Seaclam re-entry frame, it was decided to omit using the frame. W hen the API- drill string had penetrated 4,8 m below the seabed, the drill bit changed direction creating a bend in the drill string, and the drill string was retracted.

Borehole 5:
Drilling was started on Thursday 7th May at 06.50 and continued till Friday 8th May at 10.00. The water depth was recorded as 44,3 m. Omitting the re-entry frame, the API- drill string penetrated easily down to 3,5 m below the seabed. Further down boulders were encountered in the sediments. The average rate of penetration for the first 13 m was 2 m per hour. At a depth of 14,2 m penetration almost stopped and drilling was continued with the Diamond Boart coring equipment. Drilling continued down to a depth of 35,25 m below the seabed (- 79,55 below the sea level) where continuous cores indicated solid rock. By reviewing the core samples it was decided that the rock head was at a level -72,85 below the sea level (28,55 m below the seabed).
Before the drilling equipment was retracted, deviation measurements were performed in the borehole casing. This showed a deviation of only 0,5 m from the vertical at the bottom of the borehole as related to the point of entry at the seabed. The deviation of the part of the drill string between the seabed and the drill deck was of the order of 1 m, correlating well with the deviation measurements recorded by the automatic DP-system on board the ship.

5. RESULTS AND COST
Location of the boreholes and the recorded depths and samples for borehole l, 3, and 5 are shown in fig. 2. As seen from the borehole- log above, only borehole 3 and 5 were carried down to bedrock and into the rock head. The results from these two boreholes are also shown together with the seismic profile in fig. 3. In the original plan it was 78
intended to drill three holes down to and into the rock head or down to the proposed tunnel level if rock was not encountered at a higher level. However, the location of boreholes 3 and 5 and the rock head level observed in these two holes resulted in a conclusion that sufficient data had been obtained, and it was decided to terminate the mission in view of the difficulties experienced in penetrating the overlying sediments with the available equipment.
The total mission costs were NOK 1.485.000 (1987 value) which in addition to the contract with Offshore Support Services (NOK 1.319.237) includes all other hired personnel and services as well as personnel from the Road Authorities. The total length of drilling operations below the seabed was 86,6 m giving an average cost of approximately 17.000 NOK per m. If only boreholes 3 and 5 are counted, the average cost for 65 m is approximately 23.000 NOK per m.

6. EXPERIENCE
Positioning and operating a drill ship of the likes of Bucentaur proved very efficient in Løperen, where neither the current nor the ship traffic caused any difficulties. The drilling operations on the drill deck also functioned well and the drill-crew proved experienced and efficient.
The use of the Seaclam re-entry frame created difficulties due to
the narrow range of permissible deviation from the vertical. Too much time was spent on locating a suit- able position for the frame on an uneven seabed. The frame could possible have been omitted from the start (taking water depths into account) although borehole 4 had to be abandoned at a shallow depth of penetration due to bends in the drill string. Less time was used then on retracting the drill string and moving to a new position than the time used initially to locate a suitable position for the re-entry frame. Fugero has later been looking into various concepts for tilting the frame (e.g.

by hydraulic jacks) into a vertical position after being positioned on the seabed, but the problem is not completely solved.

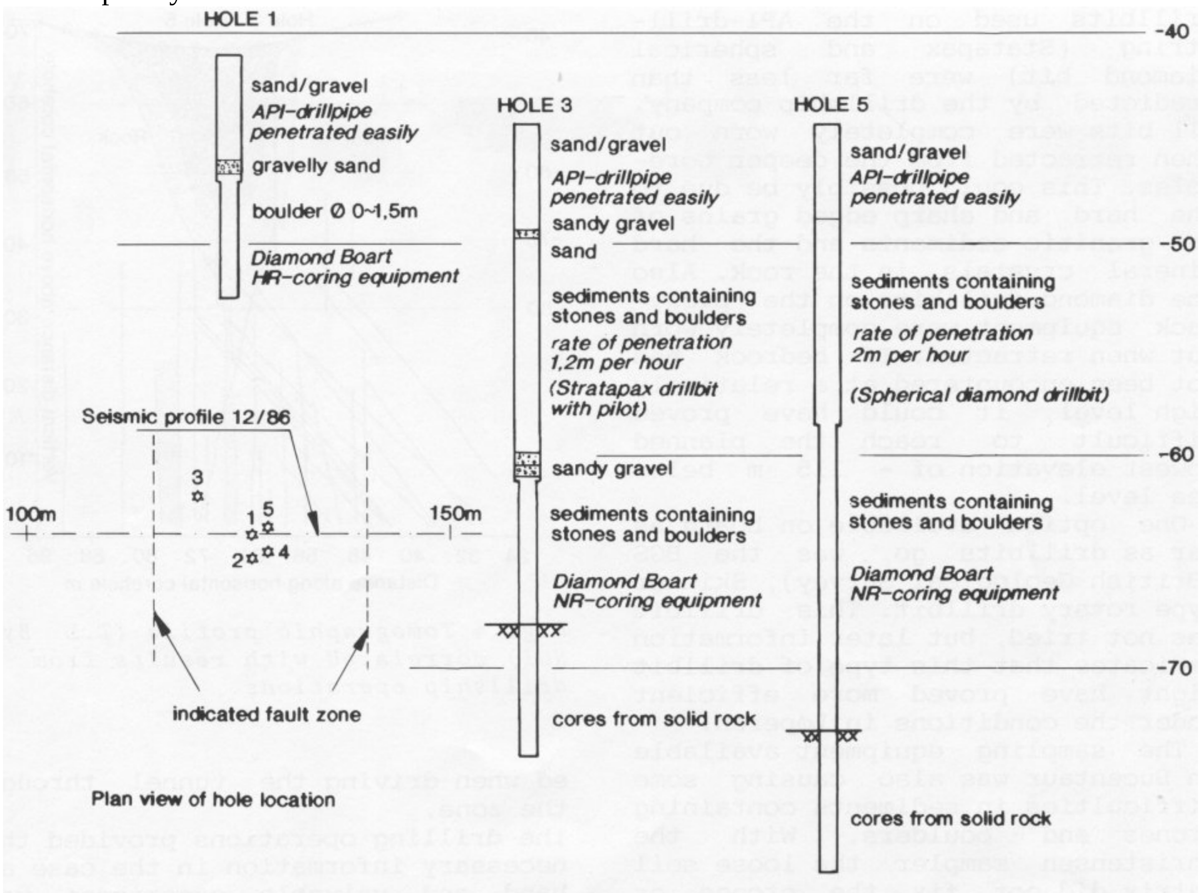

Fig. 2 Borehole positions and results from boreholes 1,3 and 5

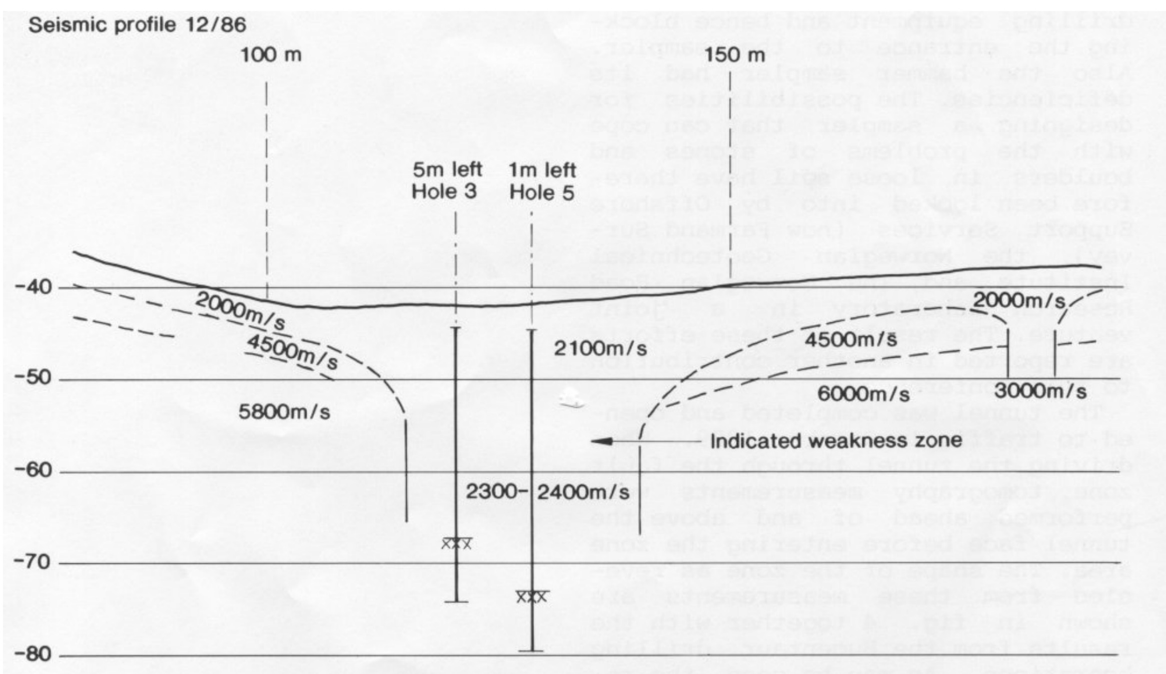

Fig. 3 Part of seismic profile and results from drill ship operations

Penetration abilities of the drill bits used on the API-drill- string (Statapax and spherical diamond bit) were far less than predicted by the drill ship company. All bits were completely worn out when retracted from the deeper bore- holes. This could possibly be due to the hard and sharp edged grains of the granitic sediments and the hard mineral crystals in the rock. Also the diamond drill bits on the piggy- back equipment were completely worn out w hen retracted. If bedrock had not been encountered at a relatively high level it could have proved difficult to reach the planned lowest elevation of -115 m below sea level.
One option available on board as far as drill bits go was the BGS (British Geological Survey) Skinner type rotary drill bit. This drill bit was not tried but later information indicates that this type of drill bit might have proved more efficient under the conditions in Løperen.
The sampling equipment available on Bucentaur was also causing some difficulties in sediments containing stones and boulders. With the Christensen sampler the loose soil matrix did not fix the stones or boulders sufficiently for the sampler to drill through the obstructions. The stones were apparently torn loose and rotated with the drilling equipment and hence blocking the entrance to the sampler. Also the hammer sampler had its deficiencies. The possibilities for designing a sampler that can cope with the problems of stones and boulders in loose soil have there- fore been looked into by Offshore Support Services (now Farmand Survey)f the Norwegian Geotechnical Institute and The Norwegian Road Research Laboratory in a joint venture. The result of these efforts are reported in another contribution to this conference.

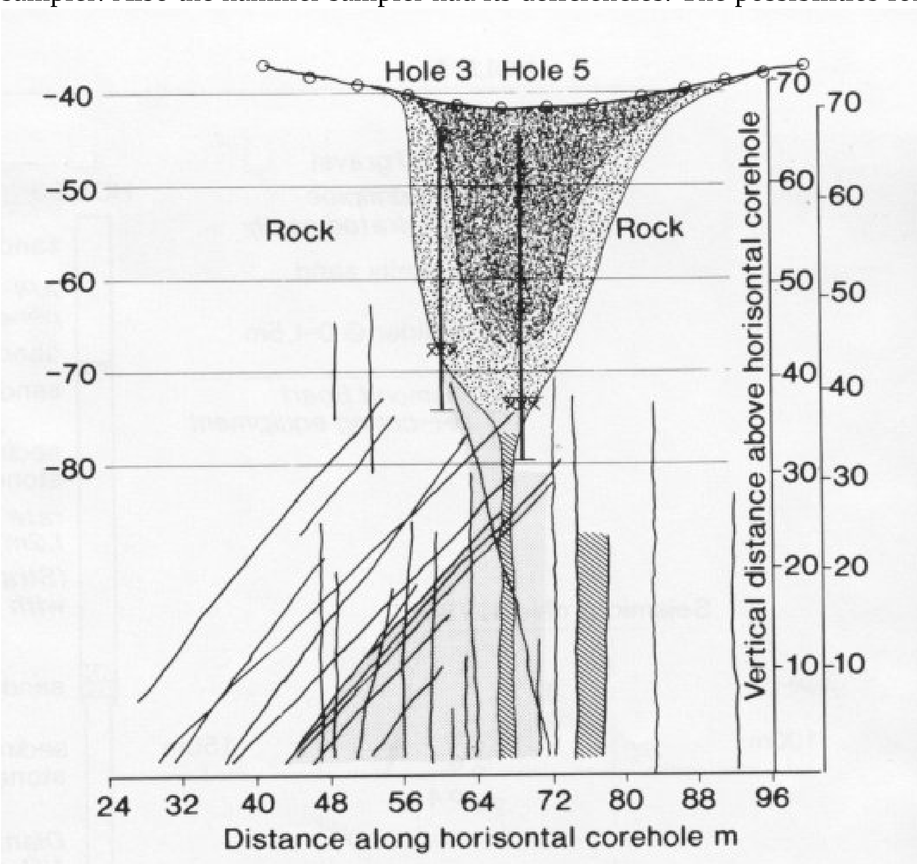

Fig. 4 Tomographic profile (T.L. By, NGI) correlated with results from drill ship operations

The tunnel was completed and opened to traffic in October 1989. When driving the tunnel through the fault zone tomography measurements were performed ahead of and above the tunnel face before entering the zone area. The shape of the zone as revealed from these measurements are shown in fig. 4 together with the results from the Bucentaur drilling operations. As may be seen the results are in fairly good agreement. No adverse surprises were encountered when driving the tunnel through the zone.

the drilling operations provided the necessary information in the case at hand, and valuable experience was achieved related to the use of drill ships and the potentials of available drilling equipment for future strait crossing projects.

Monitoring and control of electric equipment, traffic control and safety installations in the Oslo tunnel

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ABSTRACT:
The Oslo Tunnel was opened 18th January this year. This first stage of the new E18 motorway through the city centre of Oslo is an 1800 m long rock tunnel with three lanes in each direction. The average daily traffic is expected to reach 50 000 vehicles in 1990. Due to length, traffic volume and geometry there are lots of technical installations for ventilation, safety and traffic control in the tunnel.
A Traffic Control Centre with 24 hours operation and a sophisticated control system monitors the installations and the traffic flow through the tunnel. The Traffic Control Centre will in the future monitor all the tunnels in the Oslo region and also sections of the surface main road network.

1 A BRIEF DESCRIPTION OF THE TUNNEL

The Oslo tunnel is part of the E18 Motorway through the City Centre of Oslo. This East-West connection has been on the political debate and planning agenda for over 30 years. Being a rock tunnel, the Oslo tunnel represents a new alternative that was chosen after a brief evaluation process. The master plan was ratified 10th February 1987 and the construction started spring 1987. Inconvenience to road users and the general public due to construction works were minor , and the effects on the environment of the City centre were negligible during the construction period. The tunnel was opened to the public 18th January 1990.
The project is part of the so-called "Oslo- package", a finance package for the development of the main transportation network in the Oslo region towards the year 2000. Investments in the period will be app. 10 000 million NOK. The greater part of the investments relies upon income from a

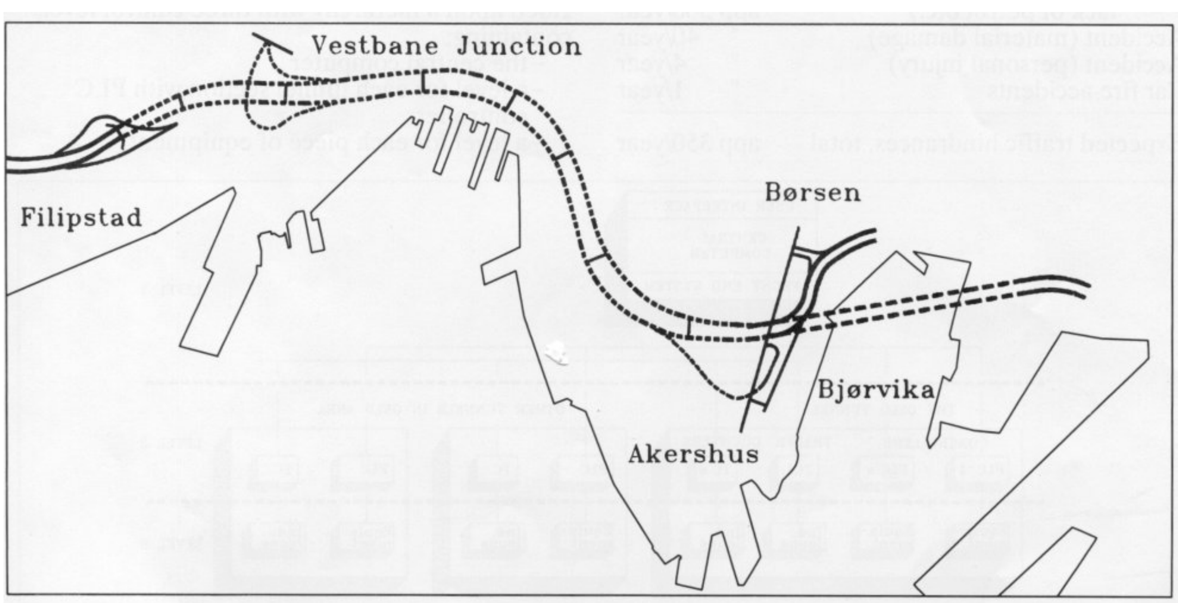

Fig. 1: The Oslo tunnel

toll booth system around the centre of Oslo. The toll-ring system was taken into use 1st February 1990.

The two main benefits from the Oslo tunnel are:

- -Traffic capacity increase through the centre of Oslo
- -Environment improvement in the southern part of the City area

The first stage of the Oslo tunnel consists of two three-lane tubes, each 1800 m long. The second stage consists of the so-called Vestbane Junction with ramps between the tunnel and the road network above ground. The third stage of the development will be a prolongation of the tunnel towards east in a submerged 600 m tunnel through Bjørvika.
In 1990 the tunnel is expected to have an annual daily traffic of app. 55.000 vehicles, while after the second construction stage is completed in 1994, the traffic will increase to 80.000 vehicles a day. The tunnel slopes down to 45 metres under sea level. From the low point there is an upward gradient of 5% in both directions for about 800m. The minimum horizontal radius is 280 m in the main tubes an 60 m on the ramps.

2. THE MAIN OBJECTIVES OF THE CONTROL SYSTEM

A tunnel like the Oslo tunnel needs an extensive monitoring system for two main reasons:

- -The severe consequences of incidents in the tunnel.
- -Large quantities of electronic equipment.

The expected number of incidents in the tunnel during the first year of operation is calculated as shown in the following table:

Stops (motor breakdown/lack of petrol etc.)	approx 300/year
Accident (material damage)	approx 40/year
Accident (personal injury)	approx 4/year
Car fire accidents	approx 1/year
Expected traffic hindrance, total:	approx 350/year

It is assumed that a large part of the incidents will require the use of safety equipment and traffic control.
In the tunnel there are 160 axial fans, 10 shaft ventilators, 3000 light spots, 36 cameras, 55 variable traffic signs, 176 detectors, 327 traffic lane signals, 43 emergency telephones, 13 traffic bars etc.

3. THE CONTROL SYSTEM

The control system is primarily designed for the surveillance of the Oslo tunnel. The system will later be extended to control other traffic tunnels in the Oslo region and road sections on the surface. The system is an integrated one, controlling all types of installations in the tunnel simultaneously. Potential suppliers of system equipment in Nor- way bad in advance shown little interest in developing a common protocol for data and signal transmission. A common protocol was desirable in order to facilitate the use of different equipment in the different tunnels while still having a full communication with a central computer. The problem was solved by the use of extended tenders. The supplier for the equipment in the Oslo Tunnel also submitted a quotation for standard control equipment designed to be used for other tunnels already in planning or under construction.
The principles for the surveillance of the tunnel requires an extensive traffic monitoring. This results in a large number of loop detectors for traffic data and incident detection. The large quantity

of data from the detection system is separated into a self-sustaining but parallel system, connected to the central computer.
Quite early in the planning process it was decided upon a hierarchy with three control levels containing:

- the central computer
- a level for each tunnel section with PLC controllers
- a level for each piece of equipment

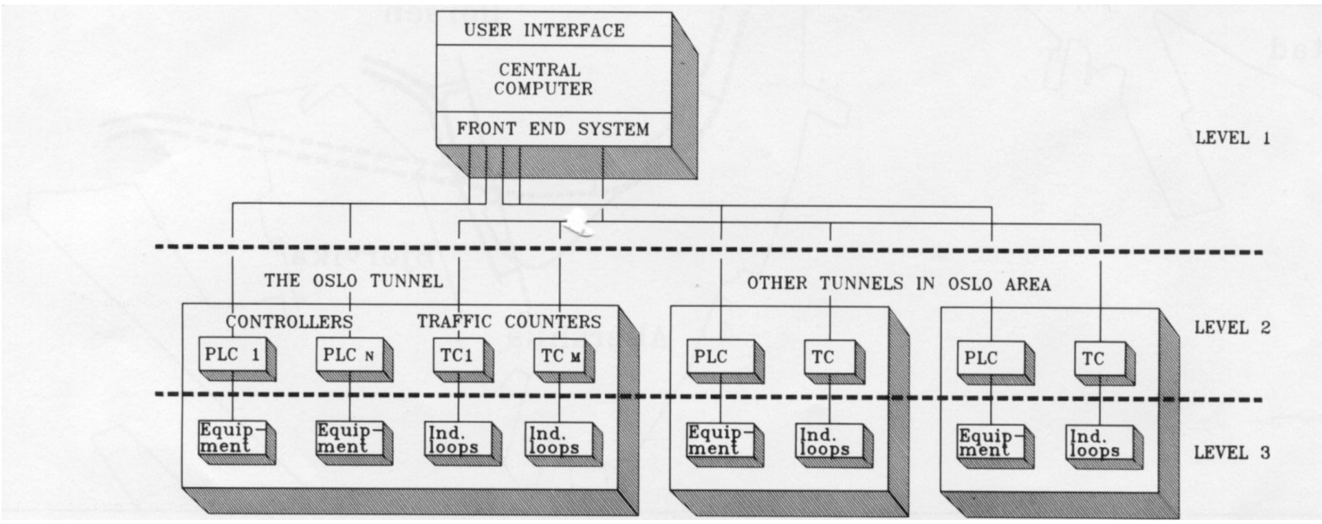

Fig. 2: The control system -control levels

The central computer is a Siemens Sicomp M70. It is a minicomputer with 4 MB memory, 320 MB hard disc, streamer tape for back-up and a separate front end system. The user interface is based upon three intelligent screen terminals of the COROS type.
A separate emergency back-up system guarantees continuous operation of the system.
The emergency system is based upon a separate COROS terminal with a PLC computer communicating with the control equipment through a separate data bus physically separated from the normal communication. In the event of a break- down in the higher levels of the system, signals and equipment will maintain their state of function.
In all there are 12 PLC controllers. Eleven of these are equipped with signal programs for the control of different equipment in the tunnel, such as traffic signals, variable signs, traffic bars, ventilation, lights, pumps etc. They also convey alarm messages from all the technical equipment. Each of these eleven PLC computers covers a section of the tunnel or daylight zone, and most of them are placed in the technical rooms in connection with the tunnel.
The 12th PLC controller is used in the Traffic Control Centre (TCC), controlling the indications on the large map boards, the alarm output channels and the communication with the Urban traffic Control system (UTC) for traffic signals in the city of Oslo. This latter function makes automatic switching to diversion programmes on the surface road network during emergency closing of the tunnel.
The control system in the Oslo tunnel comprises approximatly 1000 control objects. The feedback from the system includes status check of all signals and instruments in addition to ordinary mal-function alarms. The actual state of all objects controlled in the tunnel is displayed. The system has a comprehensive self-surveillance of internal processes and communication.
In addition to automatic control according to pre-programmed signal plans, possibility also exists to control each single object via the terminals.

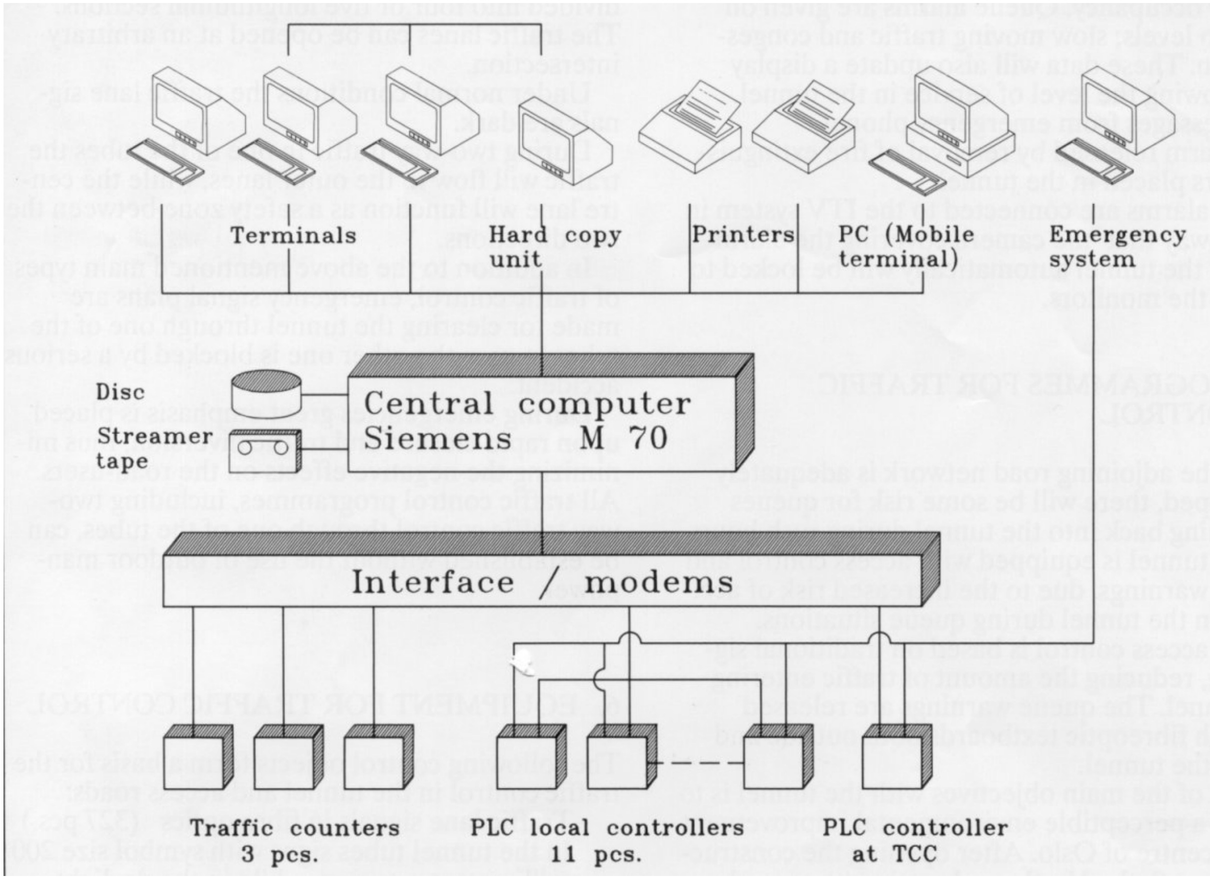

Fig. 3: The control system -technical configuration

This opportunity is especially important for maintenance and repair purposes, and to ensure free access during emergency situations for ambulances and other emergency vehicles by opening a single barrier .

The user interface is simple and easy to understand, and the error ratio is consequently very low. The requirement for short respond times were met, both for display updating and for the execution of control commands. All displays are connected in a menu system and serviced by the operator via soft keys III the displays.

4 TRAFFIC MONITORING AND INCIDENT DETECTION

Double inductive detection loops are placed in the binder course every 200 metres, in each of the traffic lanes throughout the tunnel. There are 3 traffic counters, each with a capacity of 64 loops. Registered data from the detectors are utilized to produce traffic statistics and for the automatic incident detection.

The statistics form an integral part of the control system. Data for traffic volume, traffic speed and vehicle length are transmitted to the central computer every 5 minutes. All data are stored in a data file for 48 hours before they are transmitted to a PC for further detailed analysis.

The central computer produces traffic statistics as required for the every day operation of the system only. Traffic data from a limited number of loops are stored in a separate database. Data from the last two years are available to the operators. Variation patters for traffic volume, speed and vehicle length can be shown graphically or alphanumerically.

Incidents in the tunnel will be reported to the Control Centre in three different ways:

-Automatic incident detection.

The incident detection is based on queue detection, reacting upon traffic volume and speed, distance between vehicles and detector occupancy .Queue alarms are given on two levels; slow moving traffic and congestion. These data will also update a display showing the level of service in the tunnel.

-Messages from emergency phones.

-Alarm released by removal of fire extinguishers placed in the tunnel.

The alarms are connected to the ITV system in such a way that the camera covering the alarm spot in the tunnel automatically will be locked to one of the monitors.

5 PROGRAMMES FOR TRAFFIC CONTROL

Until the adjoining road network is adequately developed, there will be some risk for queues stretching back into the tunnel during rush hours.
The tunnel is equipped with access control and queue warnings, due to the increased risk of accidents in the tunnel during queue situations.
The access control is based on traditional signalling, reducing the amount of traffic entering the tunnel. The queue warnings are released , through fibre optic text boards both outside and inside the tunnel.
One of the main objectives with the tunnel is to obtain a perceptible environmental improvement in the centre of Oslo. After opening the construction stage 2, the Vestbane Junction, the area between the City Hall and the quayside will be free for pedestrians, thus connecting the city closer to the harbour. This area was previously part of the E18.
A tunnel like this one, with a heavy traffic load, will have to undergo extensive maintenance, which at times will require the closing of one of the tunnel tubes. It is emphasized that the surrounding part of the city centre shall not be burdened with diverted traffic. Instead of using the surface road network during such operations, one tube will be used for two-way traffic when the other one is closed for maintenance.
Signal plans for the following main categories of traffic control are pre-programmed:

Type of control	**Frequency**
Access control of the tunnel	Daily
Queue warning	Daily
Closing of lane(s) in tunnel	l00/year
Two-way traffic in one of the tubes	60/year
One tube closed, diversion on local road network	15/year

The closing of one or more lanes will always start outside the tunnel. It is considered essential that a possible conflict and/or queue situation originating from the reduction of the number of lanes shall not arise in the tunnel, but, if so hap- pens, in the daylight areas. The tunnel tubes are divided into four or five longitudinal sections.
The traffic lanes can be opened at an arbitrary intersection.
Under normal conditions the traffic lane signals are dark.
During two-way traffic in one of the tubes the traffic will flow in the outer lanes, while the centre lane will function as a safety zone between the two directions.
In addition to the above mentioned main types of traffic control, emergency signal plans are made for clearing the tunnel through one of the tubes in case the other one is blocked by a serious accident.

During emergencies great emphasis is placed upon rapid closure and traffic diversion, thus minimizing the negative effects on the road-users. All traffic control programmes, including two-way traffic control through one of the tubes, can be established without the use of outdoor man-power .

6. EQUIPMENT FOR TRAFFIC CONTROL

The following control objects form a basis for the traffic control in the tunnel and access roads:
-Traffic lane signals in fibre optics (327 pcs.) In the tunnel tubes signs with symbol size 200 millimetres are used, while in the daylight areas signs with background display and 300 millimetres symbol size are used.

-Tilting barriers (8 pcs.)

The barriers are lowered automatically in response to signals from loop detectors beneath the barriers. The barriers are not equipped with light signals.

-Pivoting barriers (5 pcs.)

In connection with this project a new concept for pivoting barriers has been developed.

The barriers are moved between tilted positions in three steps:

1 Elevation
2 Rotation
3 Lowering

This alternative has several advantages over conventional pivoting barriers.

-The barrier foundation is less space- consuming.
-Lower costs.
-Increased safety during lowering as this function is controlled by the loop detectors, instead of swinging the barrier directly a cross the lane.
The barrier are equipped with "running" yellow light signals.

-Mechanically variable signs (46 pcs.)

The use of mechanically variable signs ensures a layout of the signs corresponding to ordinary traffic signs. The signs have an open solution without front cover and are made in Norway.

-Variable information signboards in fibre optics (9 pcs.)

The signboards are designed with a limited number of fixed texts. They serve various purposes:

-Information to the road-users at the closing points.
-Queue warnings.
-Early warning when traffic is diverted and information to the road-users in emergency situations about radio frequencies for messages.
-Traffic lights

Alternating red, amber blinks, and three light signals for access control.

7 INSTALLATIONS FOR SAFETY AND EMERGENCY

The tunnel is equipped with a number of safety installations:

-Three connections between the parallel tubes accessible to vehicles.

The connections have 7 m wide folding gates which can be manoeuvred locally or from the control centre. Next to the gate there is a door for pedestrians.

-Four foot walks between the tubes.
-Twelve emergency niches.
-43 emergency telephones.

The telephones are placed in closed, sound proofed cubicles. The telephones are connected to an addressing system.

-196 fire extinguishers.

The extinguishers are connected to an addressing system which sets off an alarm in the TCC when an extinguisher is removed from its socket.

Applying to requests from the fire department, automatic or manual fire alarm systems are omitted. All fire alarms are supposed to be given through the emergency telephones or by the alarm system connected to the fire extinguishers. The operators then relay the messages to the police or fire department after verification via the ITV system.

-Beacon lights, placed every 50 metres.

-Radio transmitting.

The aerial cable in the tunnel transmits the two national radio stations Pl and P2, and the cellular phone systems NMT450 and NMT900. operators may interrupt transmission of Pl and P2 to broadcast important messages to the road-users in the tunnel.

The aerial cable also transmits internal communication for the police and fire departments, ambulances and the local Road Department.

-ITV

The tunnel and the access roads are fully surveyed by 34 fixed television cameras. In each of the daylight zones there is an additional dirigible camera.

There is a separate ITV system with four monitor output channels for each traffic direction. In addition there is a system continuously saving pictures from all cameras 24 hours on each cassette.

-Emergency power supply.

The tunnel system is powered by three independent distribution systems.

In case of power failure, the UPS (Uninterrupted Power Supply) immediately starts to function, and a diesel generator will start after app. 20 seconds. The emergency power sustains the central computer and the PLC outstations, the traffic control system, the radio communication network, pumps, night- lights and beacon lights, the surveillance system and the counting loops.

An emergency manual for the installations has been prepared for the operators, which describes the type of action to be taken during various emergency situations.

For serious accidents requiring assistance from the police, fire department or ambulance, access points for the emergency vehicles are pointed out m cooperation with the departments.

Before the tunnel was opened to the public a major emergency exercise was carried out.

8 THE TRAFFIC CONTROL CENTRE

The control centre is located in a building at the eastern tunnel outlet.

The Oslo Municipal Road Department has the operational responsibility for the centre, which is manned through 24 hours. During the period 6 a.m. to 10 p.m. there is an additional qualified operator on duty in the communications room of the Road Department, which is located in the same building. In addition to this, engineers from the Traffic Operations Unit in the Road Department are on-call.

A light patrol vehicle and driver are at disposal 24 hours a day at the centre.

The control centre is constructed and equipped for the future monitoring of the whole Oslo Region main road network.

Two complete operator workstations with colour monitors and communications equipment are installed. A large surveillance board shows the main road network of the Oslo Region. There are

two additional boards showing detailed sections of the tunnels under surveillance. Light diodes show the actual state of traffic control in the tunnel and location of calls from emergency telephones. Ten TV monitors are at the operators' joint disposal.
Visitors may use a gallery which permits a good view of all functions in the control room.

Fig. 4: The traffic control centre of the Oslo tunnel

9. INVESTMENTS AND OPERATIONAL COSTS

The total construction costs are app. 1200 million NOK, financial costs not included.
The technical contracts (including fans, pumps, lights, power supply, traffic control equipment, safety equipment, monitoring equipment etc.) amount to app. 110 million NOK. In addition an electrostatic filter for cleaning the ventilated air has cost app. 7 million NOK.
Operational costs for the installations are estimated to 10 -15 million NOK per year. The most important cost factors are electric power, cleaning and manpower costs. The electric power alone amounts to 4 -6 million NOK per year .