PRACTICAL USE OF ROCK STRESS AND DEFORMATION MEASUREMENTS

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ABSTRACT
This paper describes the development and use of rock stress and deformation measurements by the Rock Mechanics Laboratory, Mining Division, the Norwegian Institute of Technology. A short description of measuring techniques and their accuracy and reliability is given. Case histories from mining and civil engineering are presented.

1 INTRODUCTION

Rock stress and deformation measurements have been carried out regularly by the Mining Division of the Norwegian Institute of Technology (NTH) since 1964.
From the very beginning our engagement in the rock mechanics field has been very practically aimed, and our Rock Mechanics Laboratory has all the time enjoyed very good co-operation with the Norwegian mining industry. At a very early stage the industry realized that proper measurements with proper equipment could be a very valuable tool both in mine planning and mine control during mining.
Since 1964 measurements have been carried out in the great majority of Norwegian mills, including the coal mills on the Arctic Spitzbergen islands. Several mines have engaged the Rock Mechanics Laboratory for control measurements at regular intervals. During the last 8-9 years a large number of measurements have also been carried out in connection with hydroelectric power plants, underground sports halls, road tunnels etc. Quite extensive measuring programmes have in recent years also been carried out in Finland, Sweden and Spain. The measuring results are used both as a general control of stress levels and deformation or subsidence, and as a tool in planning the geometry and orientations of underground chambers.

2 MEASURING METHODS

2.1 Rock stress measurements

During the years several measuring techniques have been used. Today, both two-dimensional gauges ("doorstoppers") and three-dimensional gauges are used. The latter will give the triaxial state of stress in the rock from measurements in a single borehole. Both methods are based upon so called over coring i.e. the stresses of the rock are released by coring and the corresponding strains are recorded by strain gauges. W hen the elastic constants of the rock are known (from laboratory measurements) the stress that acted on the area before the drilling started may be computed from known elastic theory (fig. 1).
During the years the question of accuracy and reliability of rock stress measurements has been thoroughly discussed throughout the world. Thorough laboratory investigations have been carried out at NTH, and these tests show that the measuring techniques as such work excellently under the se controlled conditions. However, at a

very early stage of the development of the measuring equipment it had to be recognised that rock is rock i.e. however good the measurement system as such may be, its accuracy will never be as good in situ rock as in steel. With other words, if rock stress measurement should ever become a practical tool, it had to be accepted that the accuracy always will be more or less limited.
From a mining or civil engineering point of view , it is not too important to know if the stress is 40 or 50 MPa, but it is very important to know if it is 4 or 40 MPa.
To judge the general accuracy and reliability of the measurements it has been necessary to establish a guide based upon experience from field measurements.

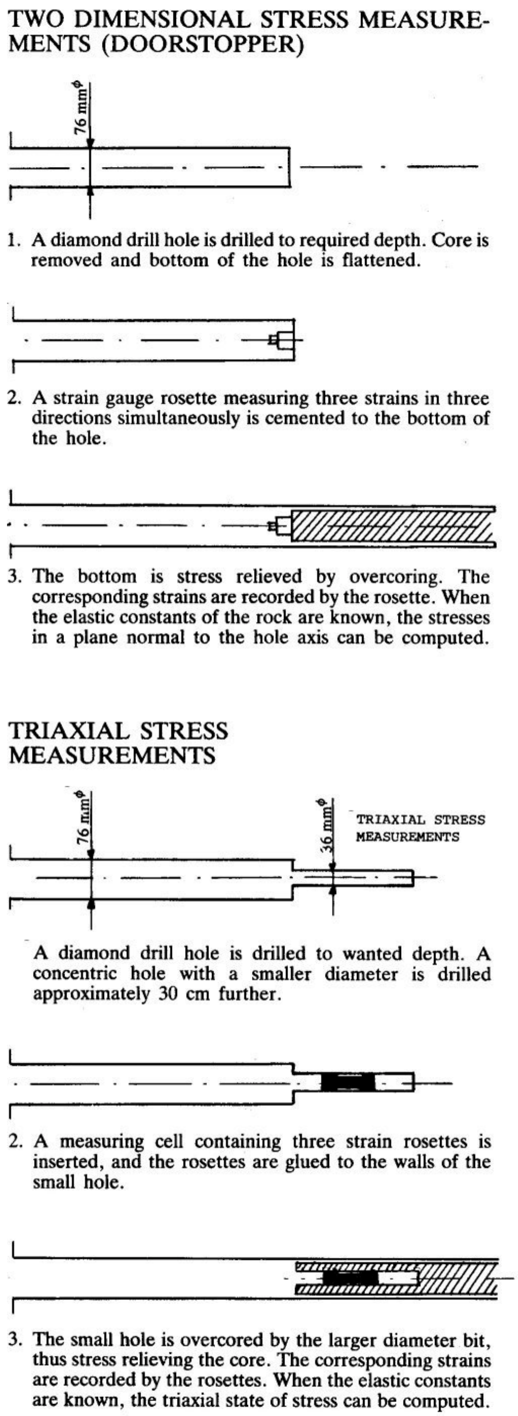

Fig. 1 Rock stress measuring methods.

For the triaxial gauge the following checks have been used:

a) The measured vertical stress component in most cases coincides with the vertical stress computed from overburden and density.
b) The occurrence of rock bursts or "pop- ping)) at different locations around the periphery of a tunnel can be explained from the magnitudes and directions of the measured virgin stresses.
c) In deep fjord or valley slopes the measured directions (and partly also the magnitudes) may be related to the topographic conditions.
d) The measuring technique includes measurements of three different strain components along the measuring hole axis which theoretically shall be equal.

These values may vary from measuring point to measuring point in a borehole, but the average values from 8-10 measurements taken at about 0,5 m intervals in most cases show an almost surprising accordance with theory.
The above mentioned checks have been verified through measurements at more than 80 locations, with the conclusion that the methods have a reasonable accuracy and reliability in the field. An important factor is the quality of the measuring crew. Our experience is that reliable rock stress measurements can hardly be carried out without a highly experienced and skilled crew, who are willing to do the job under (often) severe conditions. In our case the increase in

experience and skill of the crew has resulted in at least 100% increase in the number of measurements per shift, with the result that the cost of a measuring programme is nearly the same today as 10 years ago.Another important factor is that the crew is as "self propelled)) as possible. Our crew brings all necessary equipment, including a compressed air diamond drill machine, in a van, and in many cases it is possible to drive the van directly to the measuring site. The only external assistance needed is to supply the site with compressed air and water for the drilling.
The cost of a typical triaxial stress measurement will be in the order of US $ 6000-7000 plus travelling expenses. This includes approximately 10 single measurements in a 10-15 m borehole, necessary laboratory determinations (Young's modulus, Poisson's ratio, compressive strength, point load strength) and interpretation/reporting.

2.2 Deformation measurements

The deformation behaviour of underground openings and rock slopes may be monitored through deformation measurements. For this purpose our Rock Mechanics Laboratory is using different methods:

2.2.1 Geodetic surveying

On some occasions simple geodetic levelling techniques have been used to measure deformations in rock structures. The levelling telescope is then situated in an assumed stable location, from which movements in unstable areas may be monitored. Fig. 2 shows some examples.

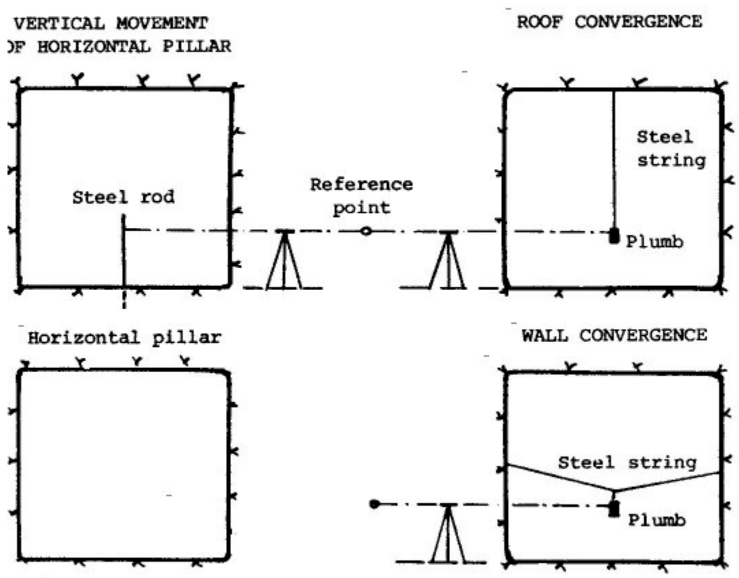

Fig. 2. Deformation measurements by levelling.

The reading accuracy of the levelling technique may be within a few tenths of a millimetre if the distance is not too long.

2.2.2 Telescopic rod extensometers

For measurement of deformations of drifts and tunnels, telescopic invar steel rod extensometers with built in dial gauges are used. The measurement set up is usually as indicated on fig. 3.

The reading accuracy of the telescopic rod extensometer may be in the 11100 mm range, while the total accuracy taking especially the temperature changes into consideration is in the 1/10 mm range.

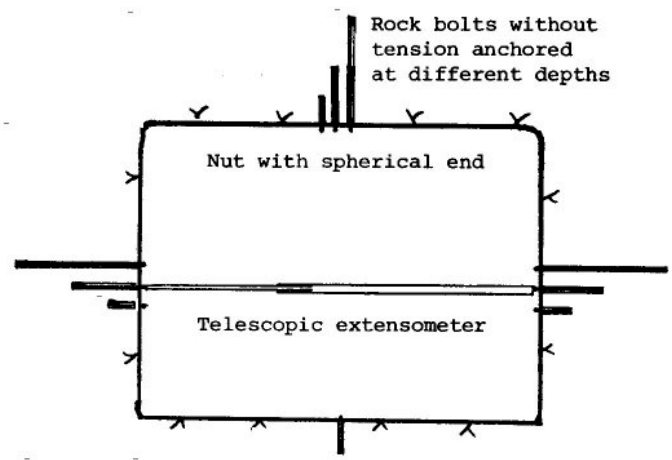

Fig. 3 Set up for rod extensometer measurements in a tunnel.

2.2.3 Borehole extensometers

For measurements of movements and deformations of larger rock masses single position or multiple position borehole extensometers are used. These may be of the string type, where movements between anchors at different depths and the surface are transferred through stainless steel strings, or the rod type where the movements are transferred through stainless steel rods. Even if the latter type is more ex pensive, it has been preferred during the last years due to its better stability and easier installation.

The readings may be made mechanically using dial gauges, or electronically using inductive transducers (LVDT's).

Fig. 4. shows a borehole extensometer installation. Where electronic readings are used, the measuring bridge may be connected to a plotter to give continuous monitoring.

Rod type extensometers have been installed in boreholes as deep as 140 metres. In one case (in Western Norway) a borehole extensometer installed in an unstable highway slope was connected to a traffic signal. W hen the movement exceeded a predetermined limit, a red light was switched on. The system worked excellently and finally the red light went on and stopped the traffic two hours before a major slide blocked the highway completely.

The practical accuracy of a borehole extensometer will depend on several factors, but will probably be in the range 0,1-0,5 mm for rod type and 1,0-2,0 mm for the string type.

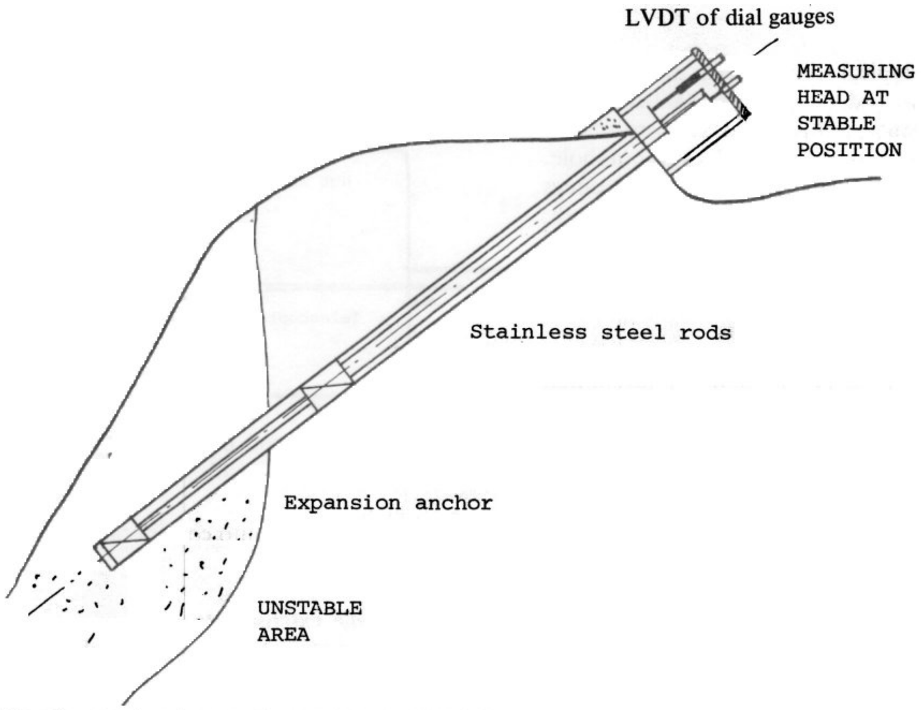

Fig. 4. Rod type multiple position borehole extensometer.

3 CASE HISTORIES

During the last 17 years rock stress measurements have been carried out in more than 80 locations (including different levels and locations in mines and tunnels) .The overall trend is that horizontal stresses in excess of the theoretical horizontal stress due to overburden seem to exist in most cases. This is also the case in young, Tertiary sediments on Spitzbergen. Quite often the major principal stress is horizontal, and relatively high stresses may exist near the surface.

The vertical stress, however, in most cases is equal to the overburden vertical stress.

In the next paragraphs one case history from mining and one from civil engineering will be described.

3.1 The Skorovas Mine

The Skorovas deposits is situated in central Norway and consists simplified of two more or less cigar shaped main ore bodiesof copperpyrite. The surrounding rocks consist mainly of rather massive greenstones, but part of them may be more schistose. Both the ore body and the greenstones are moderatly jointed. The ore body has been mined by so called transverse sublevel stoping, leaving tranverse pillars as indicated on fig. 5.

The roofs of the chambers are completely without support. About 30% of the ore body was originally left as pillars, and the mine management was very much interested in recovering at least parts of the pillars. In 1972 a rock mechanics investigation started including:

1) Three dimensional measurements to determine the virgin stresses.
2) Stress measurements in selected pillars with doorstoppers.
3) Photo elastic model studies to determine the stress distribution around the stopes (based upon the three dimensional stress measurements) .

4) Control of the hanging wall (roof) subsidence by means of borehole extensometer measurements.
5) Classification (ROD) of the hanging wall rock based upon available diamond drill cores.

The pillar measurements showed that while the outermost pillars had a normal stress level according to overburden, the inner pillars had no stress at all i.e. these pillars did not seem to carry any load. The three dimensional stress measurements showed no high stress values, but the photo elastic model studies showed that the horizontal stresses were high enough to create compressive stresses in the roof of the chambers in the inner part of the ore body. This, combined with a natural arching of the hanging wall, results in a stable self-supporting roof. Hence, the pillars are not necessary from a stability point of view. It was therefore decided to remove the central part of a pillar as indicated on fig. 5.

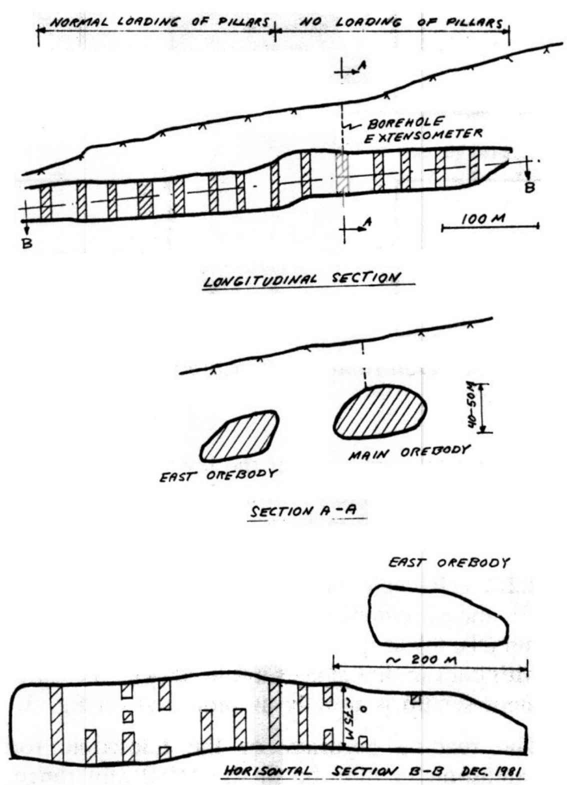

Fig. 5. Simplified lay-out Skorovas Mine.

To record the movement of the hanging wall a string type multiple position borehole extensometer was installed in a 55 m vertical diamond borehole drilled from the surface above the pillar in question.
In the autumn 1973 the pillar was blasted in steps, and readings were taken on the extensometer after each blast until the final opening was established. The only movement recorded was in fact a slight upward movement of the lower anchors. Visual control from available openings showed only minor outfalls from the roof.
During the years after 1973 most of the pillars in the inner part of the mine have been removed, and in addition the so called 'east' ore body has been mined out. In December 1981 the situation was as indicated on fig. 5, i.e. the main chamber has a span up to 75 m, a height up to 45 m and a length of approximately 200 m along the axis. Extensometer readings have been taken regularly during the whole period, showing only minor fluctuations, probably due to temperature changes throughout the year .
The east chamber has about the same span and height and has a length of about 150 m. Thorough visual inspections each year have shown no sign of instability.
This mine will be mined out in 1983, but has during its period of operation had a remarkable total recovery of ore due to the pillar extraction.

3.2 The Oppljos Highway tunnel [1]

This tunnel is part of one of the mountain highways between Eastern and Western Norway. The tunnel is excavated in gneissic rocks, and the tunnel axis, along the major part of its length, forms an acute angle with the foliation. Overburden varies between 200 m and 600 m (fig. 6).

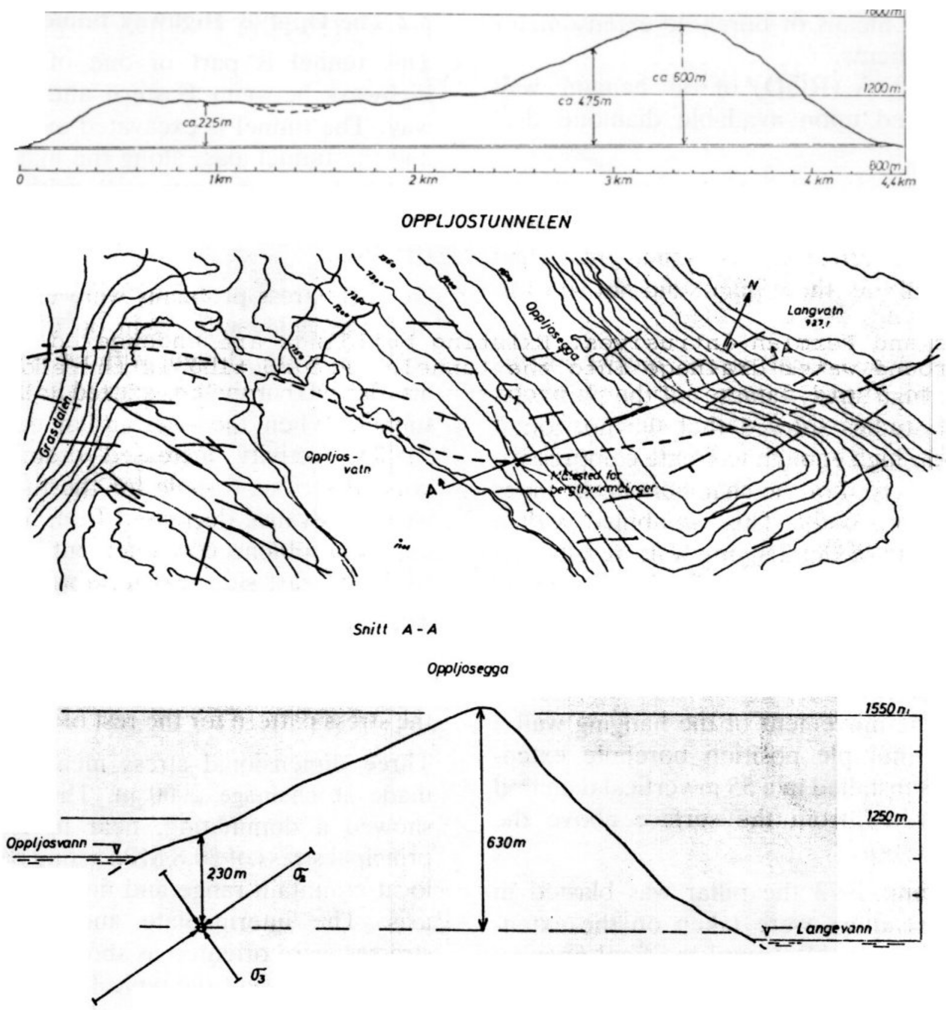

Fig. 6. The Oppljos Highway tunnel.

No rock stress problems were expected except near the valley side (right on fig. 6). The first 1600 m were driven without problems, but around 1600 m spalling started in the roof of the tunnel. W hen the overburden decreased, the spalling activity increased dramatically, and created serious trouble for the tunnelling. The weekly advance decreased from 50 m to 15 m, and the problems continued until the tunnelling from the east side was terminated at 2300 m. The rest of the tunnel was to be excavated from the west side (left on fig. 6). Before starting up, the State Road Authority wanted a prognosis of the stress pattern for the rest of the tunnel path. Three dimensional stress measurements were made at chainage 2000 m. The measurements showed a dominating, near horizontal major principal stress of 20,8 MPa acting parallel to the local mountain range and normal to the tunnel axis. The intermediate and minor

principal stresses were oriented as shown on fig. 6. They seem to represent the typical valley side stresses due to gravity.
A stress pattern as measured win give a maximum tangential stress in the roof of the tunnel of about 60 MPa. The compressive strength of the rock was in the range 47 MPa-127 MPa. This explains the reason for the heavy spanning in the roof.
Theoretically, the tangential stress will decrease with overburden, as the vertical stress will increase and counteract the effect of the horizontal stress. This may explain why spanning did not occur while the overburden was largest.
Based on the measurements and geological data a prognosis was made, stating that similar conditions might be expected except for the last 500 m towards Grasdalen valley (fig. 6).
When the tunnelling started from Grasdalen about 900 m were driven before the problems started in full again, but the rest of the tunnel was driven under difficult stress conditions as predicted in the prognosis. However, the tunnelling crew was now prepared beforehand to meet the problems, and managed a considerably higher advance than on the east side.
Similar measurements taken later in connection with three other highway tunnel projects show the same trend: A dominating, horizontal major principal stress acts parallel to the local mountain range, while the intermediate and minor principal stresses are acting parallel and normal to the mountain or valley-side and represent the typical valley-side stresses due to gravity.
This has created considerable spalling problems in the roof of the tunnels, but with a decreasing intensity with increasing overburden.

4 CONCLUSIONS

Rock stress and deformation measurements have for many years been used widely in the Norwegian mining industry, and are fully accepted as a useful tool in mine control and planning. In the civil engineering field an increasing interest in similar use in connection with hydroelectric power plants, bomb shelters and sports halls, highway tunnels etc. is seen. Within a cost range of US $ 5000-$15000 very important information for planning and control of underground chambers and rock slopes can be obtained.

5 REFERENCES

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TUNNEL EXCAVATION AND PREGROUTING WITH BOUYGUES BORING MACHINE.

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SUMMARY

Two Bouygues TBMs of 3.0 m diameter. are presently boring 10.5 km. of sewer tunnels under the City of Oslo. The rocks are basically shale and limestone sediments and Permian intrusive rocks. Existing buildings are endangered by settlements due to ground water drainage into the tunnels. pregrouting is therefore required to avoid damage. Tunnel progress is done in cycles of tunnel boring and pregrouting at the gallery face. The combination of pregrouting and tunnel boring has been a success with the Bouygues TBMs. These machines have also interesting aspects in excavating through unstable rock. The boring capacity is ranging between 1.2- 2.5 m/machine-hour.

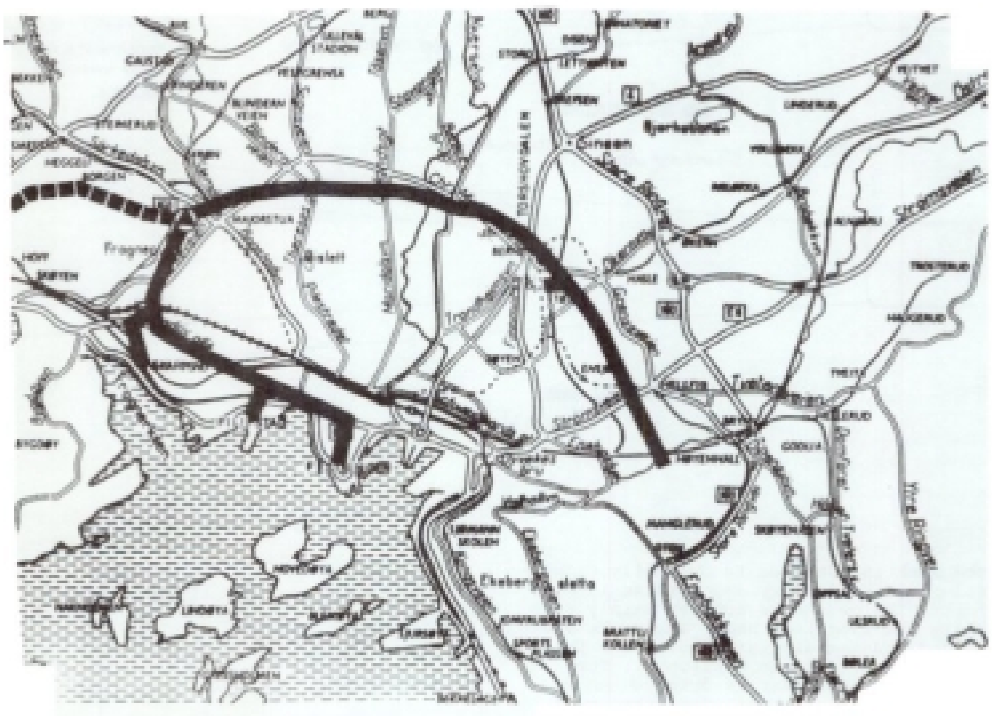


Fig 1.
Tunnel Sewer Oslo Area.
Dark line shows the tunnels under construction, the exiciting tunnels are shown dottet. The triangle signifies the Frognerparken pum station.

INTRODUCTION.

The eastern part of the tunnel sewer in the Oslo Fjord area is located under the City of Oslo. The basic part of the project in this area is about 10.5 km of full profile tunnelling. The tunnels pass under urban areas where ground water drainage can cause harmful settlements of clay deposits on which existing buildings are founded. To avoid the risk of settlement damage and subsequent economical compensation of an unknown extent, the client requires use of extensive pregrouting and, if necessary, post-grouting. The tunnelling is a combination of grouting cycles and subsequent excavation. One has to drill probe holes, measure the water loss and grout cement suspensions or chemical fluids until the water loss in the control holes is below 0.02 Lugeon. This is the criteria for sufficient water tightness that the contractor has to meet before starting the next excavation cycle.

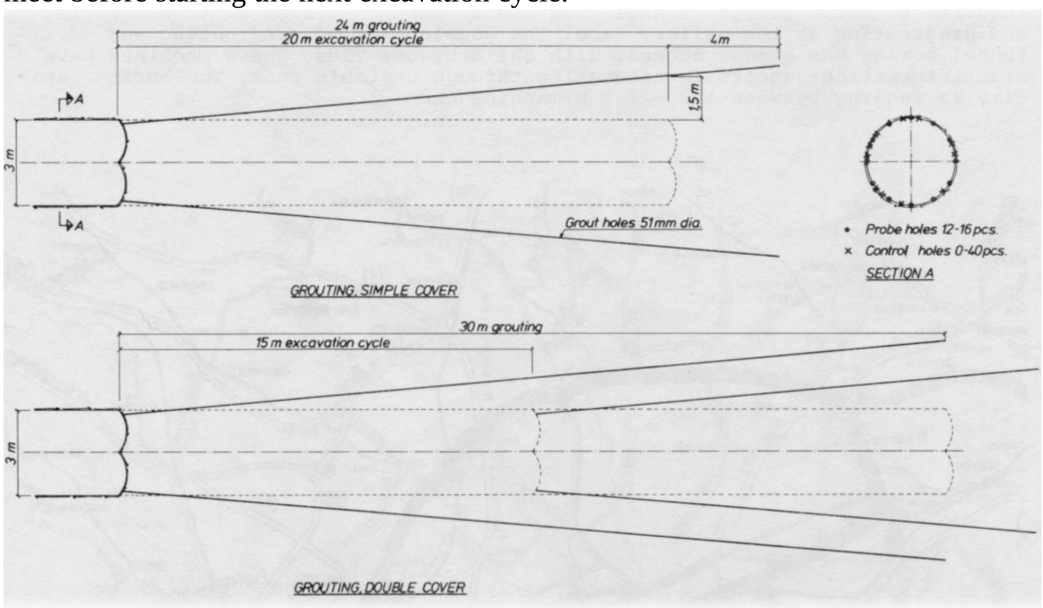


Fig 2. The principle of pregrouting.

GEOLOGY.

The rock in the area is basically Cambro- Silurian sedimentary and Permian igneous rocks. The sediments consist mainly of shale and limestone beds. The rocks are folded and the dip varies with the folding from vertical to horizontal. The igneous rocks appear as dykes of variable width in the sediments. The dykes consist mainly of diabase, syenites and porphyries. This rock, which constitutes between 10- 15% of the tunnel length, is by far a greater challenge to a tunnel boring machine than the sediments.

	SEDIMENTARY ROCKS	IGNEOUS ROCKS
Point load index Is 32 (Mpa) (parallel to bedding plane)	2,7-3,2	10,8-18,6
Uniaxial compressive strength (MPa)	50-80	150-250
Hardness (Cherchar)	10-30	80-200
Abrasivity (Cherchar)	0,1-1,0	2,4-3,9

Table 1.
Key features of Rock Materials

SELECTION OF TBM.

To find a tunnel boring machine which, in the most satisfactory way, could combine the client's strict grouting requirements and the contractor's necessity of working with an economical and reliable full profile machine, the choice was the *BOUYGUES TB 300C.* Two units are running in this tunnel project.
The selection was based on the assumption that the job was, both from an economical

and a time consuming point of view, mainly grouting. It was considered a great advantage to have access to the tunnel face and to allow both men and grouting equipment in front of the boring head. In addition, the job required a TBM which was able to excavate through the hard and abrasive igneous rocks in a proper way, considering that the sedimentary rocks were no problem.

MACHINE CONSTRUCTION.

The boring unit consists of:

a)A rotating boring head on which 3 deflectors and 3 oscillating arms, each carrying one 12" disc cutter, are installed. The arms are synchronised mechanically to a single, oscillating jack arranged in the axis of the boring head.

b)A non-rotating head with a bottom shoe, which slides on the tunnel invert level, kept in position by a top shoe.

Manufacturer	Bouygues machine, Paris
Machine type	Tunnelier Bouygues TB 300 C
Boring diameter	3,0 m
Total power	530 kw (head only: 440 kw)
Number of cutting tools	3 pcs. (12" Robbins)
Frequency of rotation on the boring head	Adjustable from 3 to 42 revs. pr. minute
Hydraulic pressure	25 N/mm^2
Machine thrust	120 tons
Torque on cutter head	37 tm
Electrical power supply	5000 volts
Stroke length (thrust)	48 cm
Weight (boring unit)	35 tons
Min. curve radius	50 m
Max. gradient	15%

Table 2. Machine Key Features.

c)3 grippers and 3 thrust cylinders.

d)A chain conveyor suspended underneath the non-rotating head, oscillating 30º on either side of the vertical position, it scrapes away the excavated material.

WORKING PRINCIPLE.

The working principle of the BOUYGUES machine is quite unique. However, using standard cutters, the cutting mechanism in rock basically is identical to the cutting mechanism of conventional TBMs.

The boring head rotates with a fixed frequency of 40 revs. per minute. Various frequencies have been tried out, but this frequency seems to be most suitable in this type of rock.

By letting the arms oscillate, the disc cutters cover the entire gallery front. Each cutter sweeps within a spherical, concentric ring of half a meter width which, in combination, forms a double domed tunnel face. The machine advances in cycles of a rapid penetration and a subsequent sweeping movement of the arms from one position to the opposite end position.

The penetration and sweeping movements are automatically controlled, but it is possible for the operator to easily adjust both penetration and sweeping speed (spacing) .One cycle lasts for about 20- 30 sec. dependant on the spacing, i.e. the rock properties. In this type of rock the spacing is between 40- 100 mm and the penetration between 5- 15 mm. In this way the adjustment of spacing and penetration makes it possible to match the machine to the rock conditions on the tunnel face continually. Optimal advance, however, is particularly de pendant of the operator's skill and watchfulness.

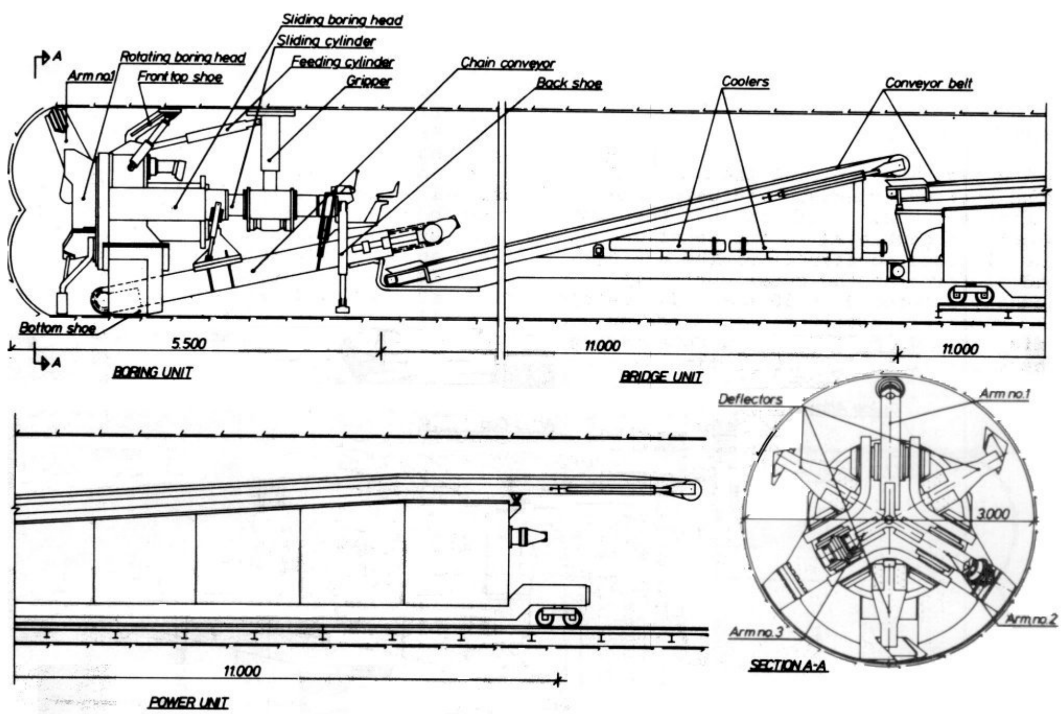


Fig. 3. The Bouygues Tunnel Boring machine.

CUTTING TOOLS.

A 3 m diameter TBM applying the conventional disc cutting technique and using only 3 single row disc cutters, has been considered a gamble as to the constancy of the 12" tool construction. The average radial force on the outer cutter is presumably between 20- 30 tons. In addition, sweeping movement introduces an axial force on the cutters. Considering these conditions, the life-time of the cutter rings in sedimentary rock is satisfactory. Here the cutter on the outer arm cuts up to 150 m^3 of rock or rolls up to 750 km. In igneous rocks, however, the most unfavourable figures respectively are 1,5 m of fractured rock or 16 rolling km per cutter ring. The average lifetime is five cutter rings per bearing/hub construction.

GUIDING.

The tunnels have many curves with a radius from 100 m. To avoid changing the position of the laser emitter constantly, prisms are being installed to refract the be am as a polygon through the curves. This system can easily be installed and is reliable both horizontally and vertically.

BACK-UP SYSTEM.

The back-up system is split into the equipment-, the ventilation- and the loading wagon. The railway is built up by steel sleepers and four rails each of 5 m length, making two tracks of 600 mm gauge approximately 500 mm above the invert level of the tunnel. The double rail system is kept permanent in the whole tunnel and makes it possible to excavate continuously. During pregrouting cycles at the gallery front, this system also allows post-grouting and other works behind to be done without interruption.

GROUT HOLE DRILLING.
While the TBM is excavating, the grout hole drilling machine is attached to the left side of the boring unit. When an excavation cycle is finished, the TBM is reversed 6 m and the boom of the drilling machine is fixed to arm no. 2 on the boring head. To obtain a successful grouting result it is of importance to have a minimum of hole deviation. Thus, by sweeping the arm and rotating the head, the starting position of the holes is being achieved as required. The system is rigid and makes it possible the to drill the grout holes with as small deviations as the rock and machine equipment allow.

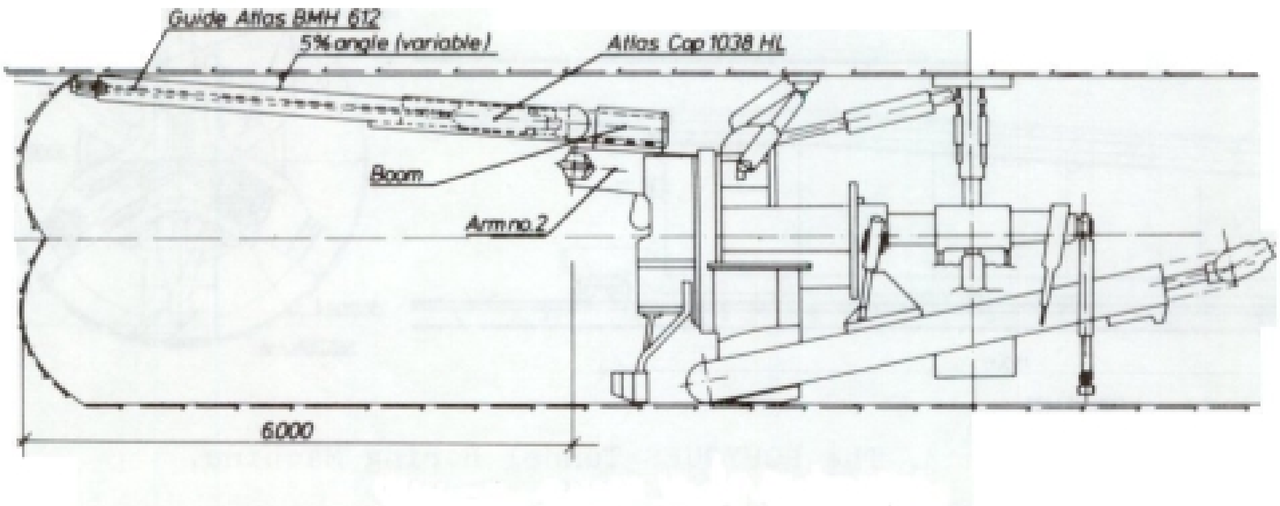
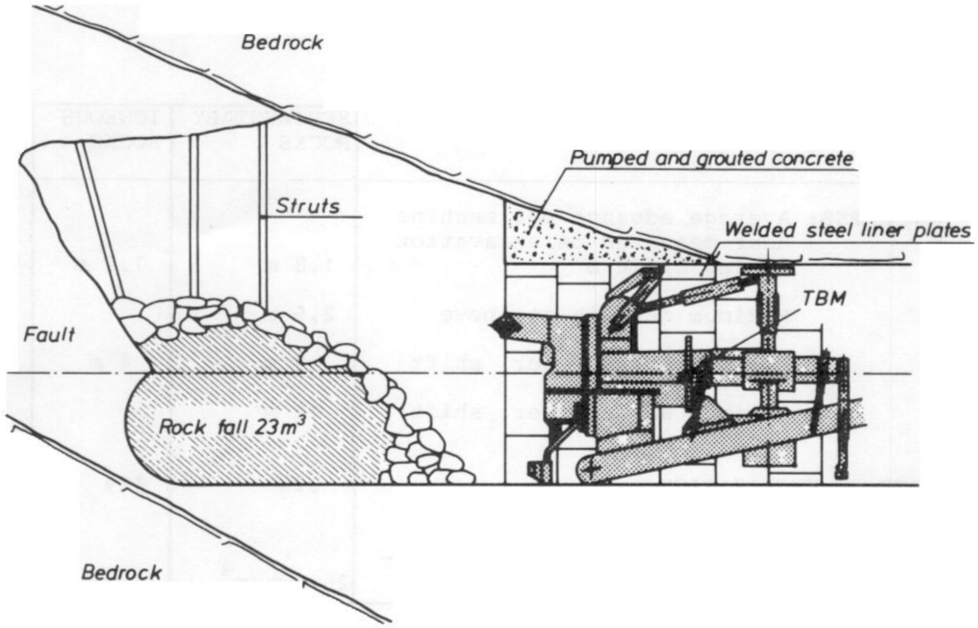


Fig 4. Grout Hole Drilling.

EXCAVATION THROUGH UNSTABLE ROCK.
The rock is in general rather stable, but an unpredicted fault of 25 m thick- ness initiated problems. Boring through the fault was made possible by installing wooden supports, liner plates, pouring concrete or grouting cement in front of the boring head. Finding ourselves in an emergency situation like this, we were lucky to have access to the gallery front. Realizing that tunnel collapses may occur also in full profile tunnelling, the manufacturer of the BOUYGUES machine has developed special tools to be installed on arm. no. 1 to make it possible to install more sophisticated rock support systems at the gallery front. It is possible with these tools to excavate circular cavities and install ring beams outside the tunnel profile. If the rock conditions are very bad, it is possible, in combination with the ring beams, to force sheet piles or wooden planks into the rock, again using arm no. 1. Rock support by the method of spilling is also made possible by using a rock drilling machine fixed to the boring head. Access to the tunnel front brings a wide range of rock support variations which meet most of the support problems in tunnelling. When using the above mentioned methods, tunnel advance may continue safely without damaging the support construction.

Fig. 5 Excavation through unstable Rock.



COSTS.

Table 3 shows in percent the cost figures of the tunnel excavation. All machines involved in the excavation are depreciated through the capital costs. The major share, however, is the TBM. Transportation costs involve the railway and maintenance of locomotives, wagons etc. Energy costs consist of electricity, oil, ventilation, compressed . air and water supply. Costs of labour payment, which also includes social taxes, make the main share of the total costs. Norwegian tunnel workers earn 150.000-200.000 kr., or US $ 27.000- 36.000 per year.

Capital costs	25%
Spare parts	5%
Tool costs	10%
Transportation	7%
Energy	11%
Labour payment	36%
Administration	6%

Table 3. Cost Figures

CONCLUDING REMARKS.

The capacity data are shown in table 4.

The maximum advance is similar to the predicted capacity, but still not competitive to conventional TBMs. The machine utilization is defined as the ratio between the machine hours and the hours at disposal for excavating one cycle including changing shifts and all standstill that may occur. The two Bouygues TBMs have now been operating in this project respectively 3 and 4 years. Altogether 7 km of tunnels have been excavated in rock of various hardness and abrasiveness , and 200 km of grout holes at the gallery face have been drilled.

The machine's good ability in combining tunnel excavation and pregrouting should be emphasized, and, regarding the fact that approximately 2/3 of the job is pregrouting, the machines have been a success in this special tunnel job.

	SEDIMENTARY ROCKS	IGNEOUS ROCKS
PROGRESS: Average advance per machine hour measured on excavation cycles of 20 m	1,8 m	1,2 m
Maximum advance as above	2,5 m	
Average advance per. shift	9 m	4 m
Maximum advance per. shift	12 m	
RATE OF UTILIZATION:	62%	47%
ENERGY: Average consumption of electricity.	25 kwh/m^3	38 kwh/m^3

Table 4. Datas of capacity.

EXCAVATION , SUPPORT AND PRE-GROUTING OF TBM-DRIVEN SEWER TUNNEL

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SUMMARY

For the collection of sewer water from Oslo and suburbs a 38 km TBM-driven tunnel system is being constructed. The project described here is a 14,2 km section, driven from a 900 m adit tunnel located at Holmen (17 km west of Oslo). The tunnel was excavated by two Robbins TBMs with a 3;5-m diameter.
Due to soft clay deposits and populated areas above the tunnel alignment, groundwater lowering had to be prevented. The solution to this problem, was continuous pre-drilling and pregrouting of the rock ahead of face. The equipment for this technology had to be designed specifically for this work, as no standard components could be utilized. Even the TBMs were modified. Weekly average progress on the two headings was 60,5 m and 63,3 m [for five 15-hour days.]. This includes approx. 6,5 boreholes, 85 kg cement and 6 litres of chemical grout per metre of tunnel. The support criteria and methods are described. Noteworthy is the use of premixed dry mortar with steel fibres for the shotcrete works. The contract was finished on schedule and within budget costs.

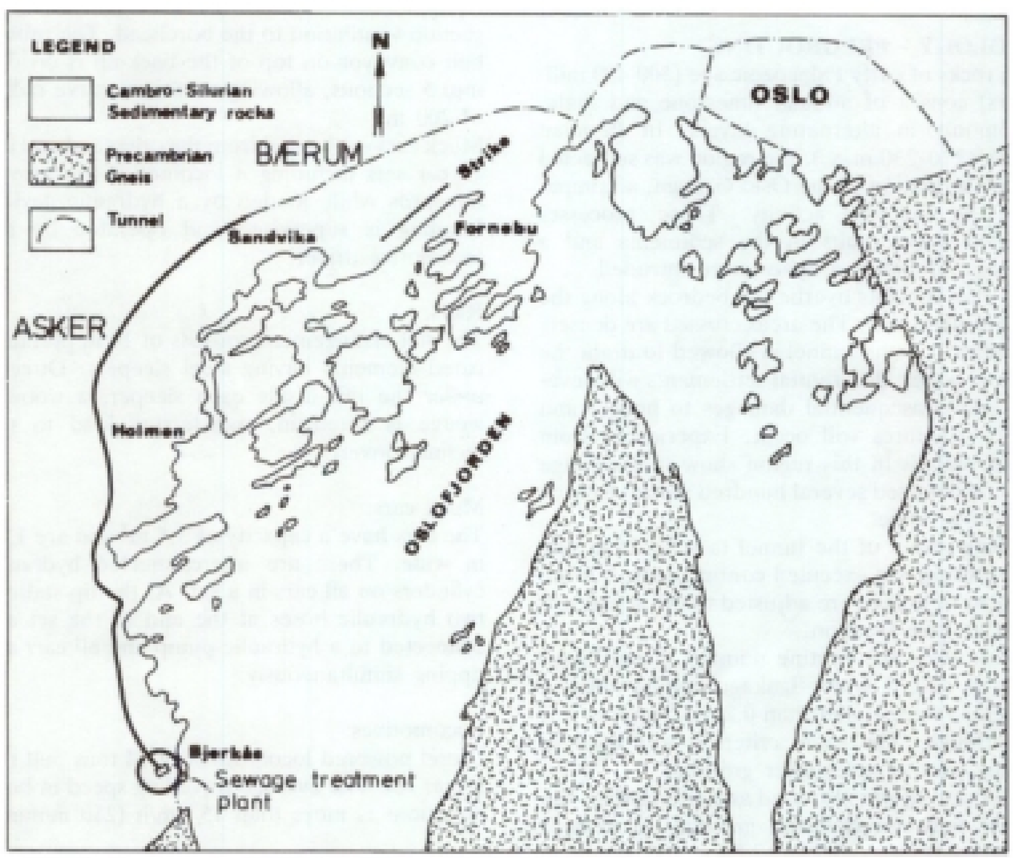


Fig. 1 Project location map and general geology.

INTRODUCTION

The councils of Oslo, Asker and Bærum decided in 1976 to build a regional sewage treatment system. The sewage collection system consists of 42 km of tunnel, out of which 38 km are made by tunnel boring machines. Fig. 1 shows the project location and general geology.

The joint venture of Furuholmen-Prader was awarded the contract of tunnel section Sandvika to Bjerkås to be excavated from a 900 m adit tunnel at Holmen. The contract was signed in May 1977, for the 14,2 km section, amounting to NOK 78 mill. The tunnelling work by TBM was started in October-November 1978 by two machines from the Robbins Company, Model nos. 116-188 and 116-189. Both machines have a 3,5 m diameter. A very special feature of this tunnel system was the need for pregrouting to prevent water leakage into the tunnel. For the first time in the world such a project was accomplished in machine bored tunnelling. The details are described later .

GEOLOGY -PREGROUTING

The rocks of early Palaeozoic age (500-400 mill. years) consist of nodular limestone and shale, commonly in alternating layers. In Permian times (270-230 m. y .), the region was subjected to the formation of the Oslo Graben, accompanied by volcanic activity. These processes created shear faults in the sediments and a number of sills and dykes were intruded. Soft clay deposits overlie the bedrock along the tunnel alignment. The areas crossed are densely populated. If the tunnel is allowed to drain the ground water, differential settlements will develop and consequential damages to houses and other structures will occur. Experiences from other tunnels in this region show that damage may be detected several hundred metres to each side of a tunnel. Probing ahead of the tunnel face by 24 m and pregrouting was executed continuously, but the number of holes were adjusted to the actual risk level at each location.

In risk areas the routine number of test holes were 6 and if water leakage testing showed Lugeon-values higher than 0,2, pregrouting was undertaken. The same criterion was used in testing control holes after grouting.

Water testing was also used as a criterion for the choice between chemical grouting or cement grouting. Between 0,2 and 2,0 Lugeon, chemical grouting was undertaken and above 2,0 Lugeon, grouting was normally done by cement.

EQUIPMENT

TBM -data:

Diameter		*3,5m*
Weight		*90t*
Length		*15m*
Cutters	*14"*	*25*
	12" twin	*2*
RPM		*7,2*
Cutting head motors		*600 HP*
Maximum torque		*60,7 tm*
Maximum thrust		*506 t*
Stroke		*1,2 m*

Back-up equipment:
The back-up equipment was designed and built in Norway by the contractor. The full length of this loading system is 80 m from the rear of the TBM, and it is pulled by the TBM while boring. The system is rolling on rails placed directly behind the TBM. The muck-car set is pushed up a ramp onto rails built into the back-up system. Included in the back-up system are repair tool compartment, flexi-cable drum, TBM power pack, electric equipment and transformers, air scrubber and suction ventilation to the bore head. The rubber belt conveyor on top of the back-up is divided into 5 sections, allowing minimum curve radius of 200 m.
Muck cars are loaded from the other end and the 13 car sets including a locomotive are moved outwards while loaded by a hydraulic device. Loading is supervised and operated by the locomotive driver .

Rails:
The rail-arrangement consists of 10 m prefabricated elements having steel sleepers. Directly under the rail inside each sleeper, a wooden wedge is fitted in, transferring load to the inclined invert.

Muck cars:
The cars have a capacity of 3,5 m^3 and are 1,25 m wide. There are interconnected hydraulic cylinders on all cars in a set. At the tip station, two hydraulic hoses at the end of the set are connected to a hydraulic pump and all cars are tipping simultaneously.

Locomotives:
Diesel powered locomotives of 14 tons pull the 13 car set. The average travelling speed in both directions is more than 15 km/h (250 m/min).

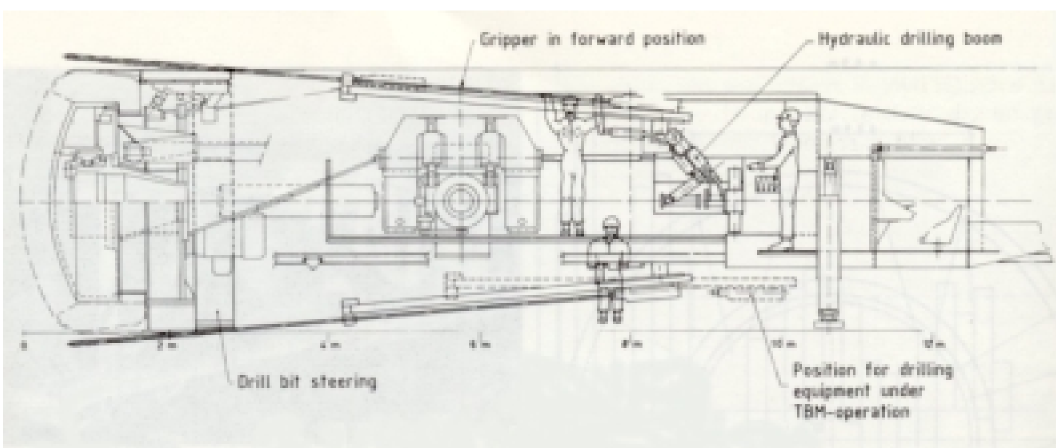


Fig 2. Mounting of drilling equipment on TBM.

California switch:
This is a double track arrangement to allow trains to pass, without blasting of extra room. The California switch is 135 m long inclusive ramps and is wheel-mounted on the tracks. The normal distance between loading point and switch is 300-500 m.
One train of 13 muck cars carries theoretically 45.6 m^3, which is two strokes of the TBM or 2.4 m tunnel. Change of train occurs every second stroke and while regripping the TBM. Pre-boring and pre-grouting:

It was soon realized during planning of this work that combination of TBM-equipment and boring and grouting machinery could not be achieved from any standard components. Every single unit that was to be brought into the tunnel, inclusive the TBM, had to be matched to each other.

The main problem was to do mechanized pre-drilling at the face, where the TBM occupied almost all the space. According to specifications from the contractor and in co-operation with the Robbins Co., the TBMs were modified to allow mounting of two hydraulic booms and feeders for the bore hammers. The design of this equipment was entirely the contractor's responsibility. Standard components were 10 feet feeders and H 70 hydraulic bore hammers from Montabert. (Fig. 2 and Fig. 3)

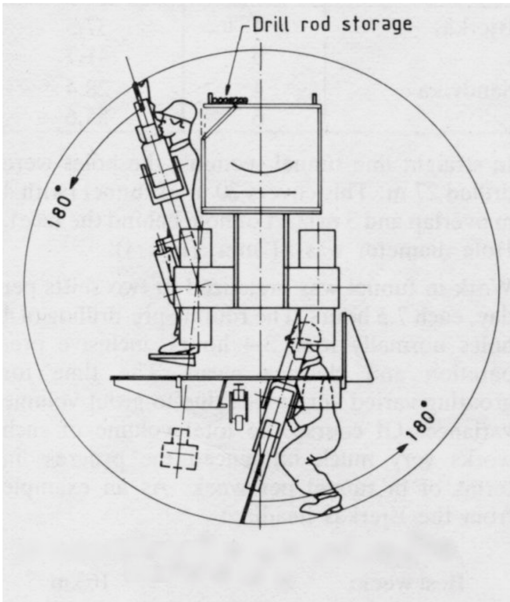


Fig. 3. Movement of booms carrying bore hammer.

There are 17 starting positions for pre-drilling around the periphery at 3 m behind the face. Holes are normally drilled at 4° to the tunnel wall. Within space limits it may be drilled in several directions from one starting point (Figs. 4,5 and 6).

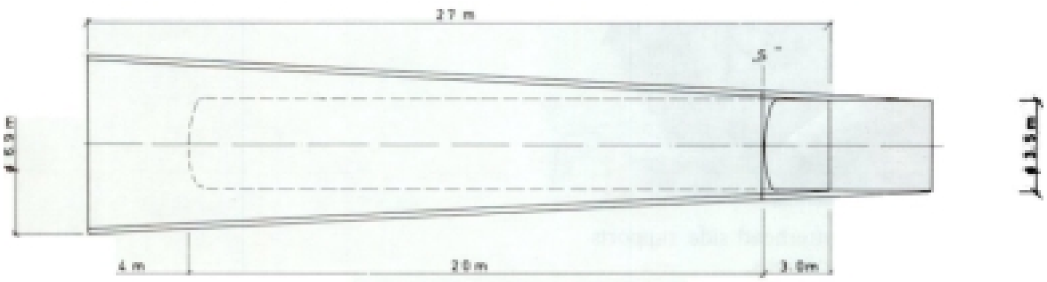
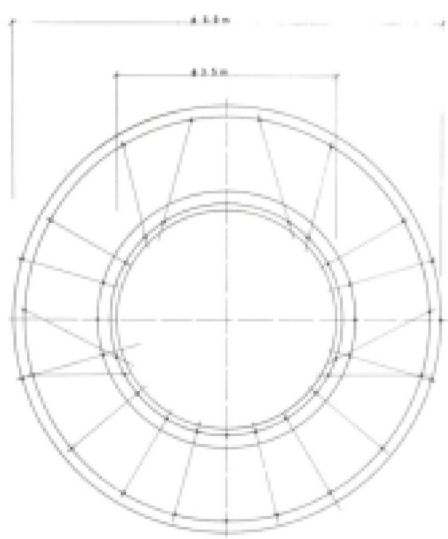


Fig. 4. (above) Predrilling and overlap.

Fig 5. (below) Available start positions for predrilling



The grouting equipment has colloidal mixer , agitator and pump from Montanburo of West Germany. Standard units were modified to fit in the back-up system. The pump maximum output is 100 l/min, and maximum pressure is 50 bar.

These units are permanently mounted 25 m behind the face, and may be used for cement and chemical grouting. See fig. 7.

When necessary post grouting was ordered, a rail mounted mixing- and pumping equipment of the same brand was available for this purpose. At the same time this unit was a reserve for the main equipment, as it could drive on rails all the way to the rear of the TBM.
An one boom rail-mounted drill jumbo for post-grouting and drilling for rock bolts was also available.

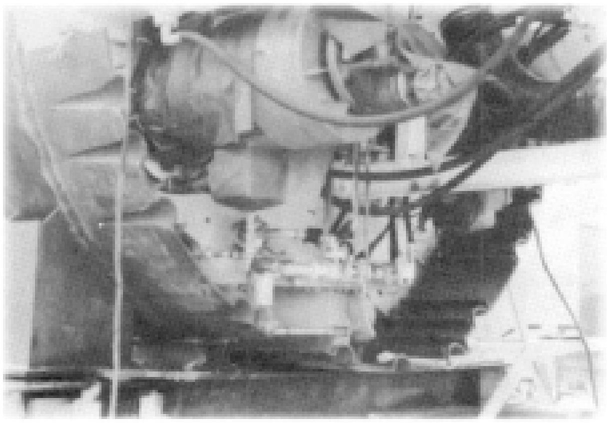


Fig. 6. Modified cutter head side supports with start positions.

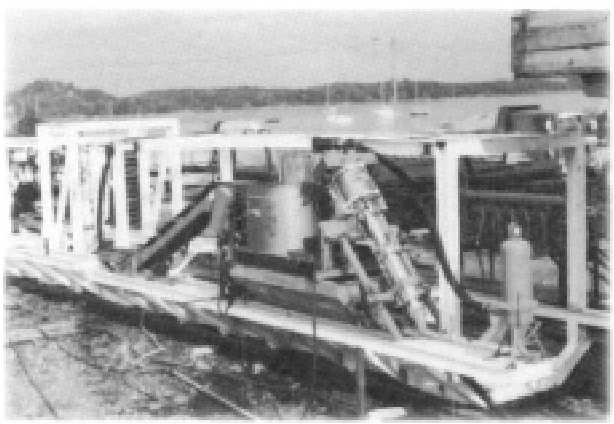
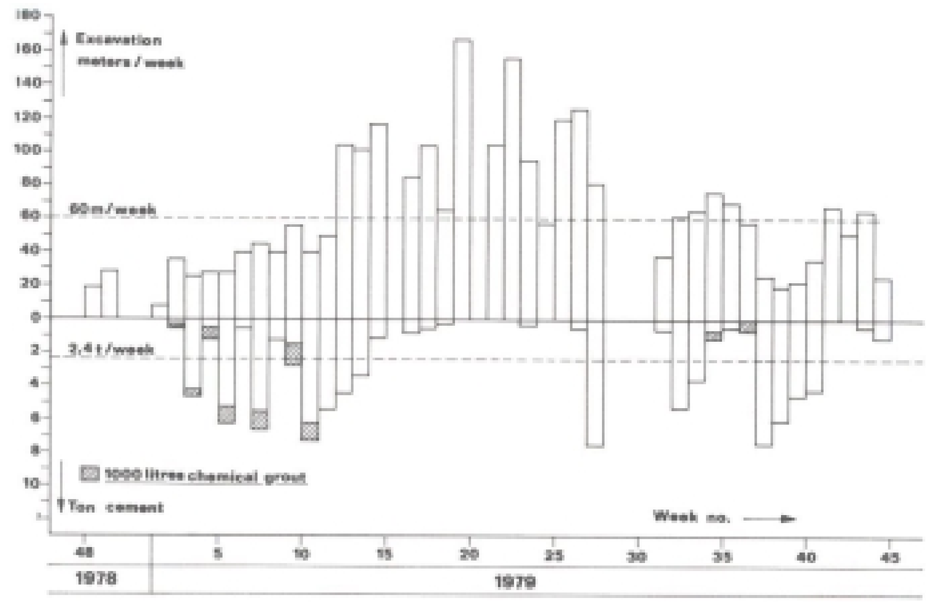


Fig. 7. Grouting station in back-up equipment.

NORMAL PROCEDURE -RESULTS
According to local conditions a number of holes were drilled as a 4° trumpet in front of the face. Generally 4 or 6 holes were the routine:

Heading	Number of holes	Per cent of tunnel length (%)
Bjerkås	4	37,5
	6	41,7
Sandvika	4	28,4
	6	55,6



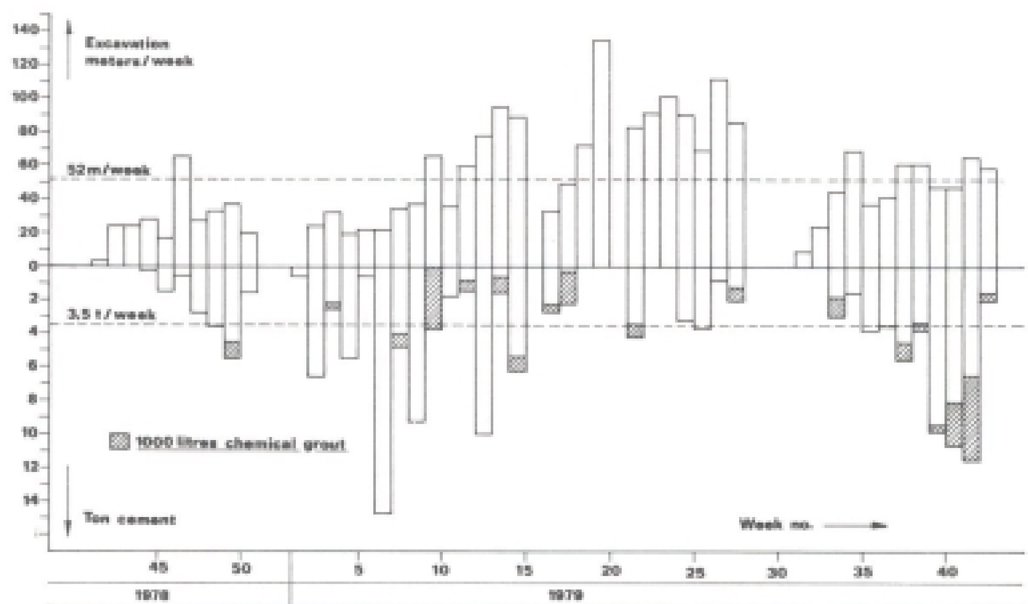


Fig 8 (top) *Tunnel Holmen-Sandvika Progress-pregrouting*
Fig 9 (above) *Tunnel Holmen-Bjerkås Progress-pregrouting*

In straight line tunnel sections, the holes were drilled 27 m. This covers 20 m of tunnel (with 4 m overlap and 3 m start of hole behind the face) . Hole diameter was 51 mm. (Fig. 4)
Work in tunnel was organized in two shifts per day, each 7.5 hours. The routine pre-drilling of 4 holes normally to ok 3-4 hours, inclusive preparation and clearing away. The time for grouting varied very much due to grout volume variance. Of course the

total volume of such works very much influenced the progress in terms of meters of tunnel per week.
As an example from the Bjerkås heading:

Average first 3 km:	50 m/week
Best week:	163 m

	Bid	Heading Bjerkås	Heading Sandvika
Tunnel length in m		7211.2	7066.8
Weekly progress in m	58	60.5	63.3
Net penetration per machine hour in m		3.72	3.59
Number of bore-holes per m. tunnel	6.1	6.95	6.48
Cement per m. tunnel in kg.	11	121.3	54.1
Chemical grouting per m. tunnel in l.	21	6.3	5.9

The close connection between overall progress and grouting may be seen from Fig. 8 and Fig. 9, showing production over a one-year period.

Tab 1.
The average overall results per May/June -81

STABILITY, PERMANENT SUPPORT

The intercalated limestone/shale is generally self supporting and does not need any stability measures to be undertaken. However, there are zones of reduced stability due to shear zones and/or joint systems of unfavourable orientation. The Permian sills and dykes also very often produce poor stability. Altogether there are 150 sections in the 14-km tunnel that are supported. Mostly these sections are short.
The tunnel is mapped by Eng. Geologist F . Løset, using the Q-Method (Barton, Lien, Lunde, 1974). Each zone of poor quality rock may change very much in width and direction and hence there is a considerable variance of Q-value within the same type of support. This should be kept in mind while going through the general relation between Q-value and choice of support method in this tunnel.

Q > 10:
Generally in the case of a self-supporting tunnel. To prevent local, small wedges from falling out, some spot bolting and shotcrete are used.

Q = 5- 10:
In such sections only rockbolts are normally used. A considerable part is considered self supporting.

Q = 1- 5:
The main part of such sections is supported by shotcrete and in a few cases this is combined with rockbolts.

Q = 0,5 -1:
Normally this quality range consists pf separate shear zones or dykes of markedly poorer quality than the surrounding rock. Normal support is a combined use of shotcrete and rockbolts. If the width of a zone is small (some dm), shotcrete only is used.

Q < 0,5:

In such cases the shear zones carry a large percentage of clay material and very often the surrounding rock is also fractured with clay- filled joints. Normal support is reinforced shotcrete and rockbolts. In some wide zones rebar steel baskets are mounted as ribs at 1,0-m centre and shotcreted.

Shotcrete:

As mentioned, the zones of shotcrete support were normally. small and distributed over approx. 14 km of tunnel. The shotcrete works also had to be coordinated with rockbolting, post grouting and other operations. Because of this, the possibility of using factory made concrete delivered by truck mixer in the adit was ruled out. Because of cost efficiency, the net reinforcement was substituted by steel fibres.
A dry-mixed mortar produced by Rescon A/S was chosen for the shotcrete work. this mortar (Confix) contains a very well grad ed sand 0-4 mm, silica dust, plastifier and optionally 18-mm steel fibres (75 kg/m3). At the bottom end of the adit tunnel, Confix was delivered in 500 kg big bags. A continuous mixing station was erected for delivery of water mixed mortar into rail mounted rotating drums. Two such drums transported 3 m^3 each.
In the tunnel a rail mounted shotcrete equipment was pulled by its own locomotive. Shotcrete pump and accelerator pump were driven by a compressed air hydraulic power pack. The rotating drum delivered the mortar directly into the pump. At the nozzle, a compressed air jet .
containing the accelerator threw the shotcrete
on the rock surface.
The shotcrete team was 3 men in the tunnel and 1 man at the mixing station. Inclusive shifting time between shotcrete locations, the production was 10 m^3 per 7,5-hour shift. The rebound was by this system not more than 5%.

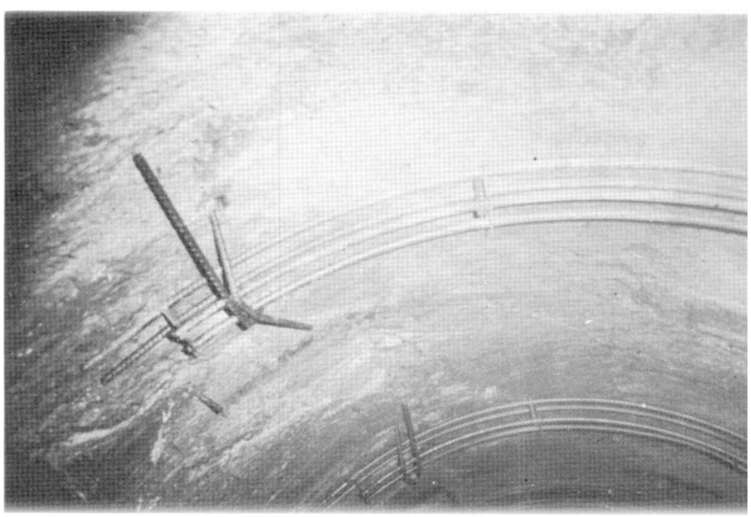


Fig. 10 Tunnel wall with rebar steel basket for shotcreting of ribs

Total volume of shotcrete:

without fibre	340,4 m^3
with fibre	703,2 m^3
Total	1043,6 m^3

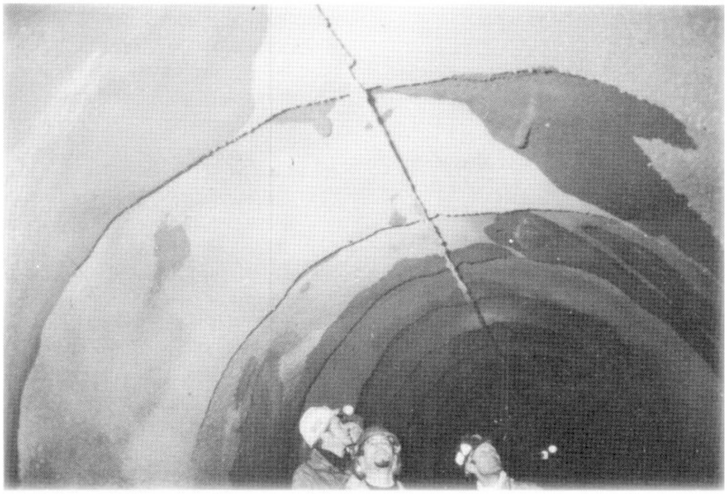


Fig. 11. Finished section.

Concrete testing was carried out by an officially authorized testing laboratory, showing the following results:

	Number of samples	Average strength in MPa C	Average strength in MPa T
Samples from mixer			
With steel fibre	4	77.7	
	1		10.4
No steel fibre	3	69.5	
	3		8.8
Diamond drilled samples from shotcrete surface			
With steel fibre	7	46.1	
	3		6.6
No steel fibre	5	38.3	
	3		6.5

All test after 28 days
Values given in MPa
Compressive strength: C
Bending tension strength: T

The shotcrete adhesion to rock was also tested, but as usual the results varied very much.
One has to realize that the shotcrete is applied on a poor quality rock and the ruptures went partly through rock, concrete/rock contact and polyester glue. The average of nine pull tests having a strength above zero was: 2.1 MPa

Rockbolts:
The normal type of rockbolt was steel of 25 mm diameter fully grouted by cement based mortar. With few exceptions, the length of bolts are 2.4 m. The bolts are not equipped with plates.
The total number of bolts in the TBM-tunnel was 4177 pieces.

Heading Sandvika	2512 bolts
Heading Bjerkås	1665 bolts

COST DATA
W hen 67% of the 14.2 km of tunnel was bored, the cost of the support works at face and the pregrouting works relative to the cost of excavation of the tunnel were:

Support works at face :	2.4%
Pregrouting works :	37.8%

The costs are based on bid prices and actual quantities executed. The permanent supports were adapted from normal procedures to fit the specific conditions of this project. The relatively expensive dry mixed shotcrete mortar gave the necessary flexibility and operational advantages to pay off. The quality test results are excellent and steel fibre reinforcement in the shotcrete was a success. Some of the poorest zones would normally be lined by poured in place concrete. Because of distance between locations, small zones and cost of formwork, rib reinforced steel fibre shotcrete was applied. The ribs were prefabricated from rebar steel and shotcreted (Fig. 10 and Fig. 11).

CONCLUSION
In the tunnelling work described, a new concept of excavation procedure is accomplished. Continuous, mechanized pre-drilling and pre- grouting combined with TBM-excavation was made possible by careful planning and the results of the work are very satisfactory. Disturbance to populated areas above the tunnel was practically nil.

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COMPUTER CONTROL COMES TO HARD ROCK DRILLING

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Ingeniør Thor Furuholmen A / S. Oslo

Electronics and computer technology are rapidly spreading in to the field of tunnel construction. Micro-processor controlled hydraulic drilling jumbos have been successfully used by a Norwegian contractor to drill in excess of half a million metres of hole, and are now ready for series production.

Electronics and computer technology can be expected to influence productivity and economy in the field of construction equipment in the same way that manufacturing and process industries have already been affected.

Looking back a few years, the first really significant change to affect drilling jumbos was the introduction of the hydraulic percussion hammer in the early 1970s. This started to replace the old pneumatic type, producing great improvements in environmental conditions at the face: less noiseless dust and the absence of the unpleasant oil/air mist associated with pneumatic drilling. But even more important, the hydraulic drills had significantly higher penetration rates. up to twice as great as their pneumatic counterparts, and consumed less energy per metre drilled (see Tunnels & Tunnelling. December 1979, p5 and p14).

Hydraulic hammers can save money

As always. doubts were raised about reliability-hydraulic systems are prone to fail if dust gets into the system, and special precautions had to be taken, including the use of dust-free workshops for maintenance-and about whether the equipment was robust and sturdy enough for the rough handling of perhaps inexperienced labour on remote overseas sites. The investment required for hydraulic drills is also much higher and at first their total economy was questioned until it was proved by experience that they really could save money. Now there is little doubt that hydraulic hammers will eventually take over the entire market.

Although the developments mentioned have brought the drilling process to a high level of mechanisation, there are still a number of shortcomings. In practical tunnelling situations the following points may be observed:

1. Although a detailed drilling and blasting plan may as a rule have been worked out, the operators make their own drilling pattern, which they all believe is the best one, although they may be quite different in position and number of holes from gang to gang, and all different from the official one.
2. Contour holes have of ten been placed at random, whereby they, together with overloading, have increased the overbreak unnecessarily. The direct cost of overbreak is limited, but its effect on stability can result in high extra costs. If the tunnel has to be concrete lined, this becomes evident, as the total overbreak volume has to be replaced with concrete.
3. On multiboom jumbos, one of the booms is often idle while the operator attends to some other problem. Also, the time spent moving from one hole to the next depends very much on the operator's skill. This greatly influences the total drilling time.

4. The penetration rate of the drill hammers depends very much on the adjustment of parameters like percussive energy, feed pressure rotation speed, etc.

Optimum working conditions vary along the tunnel, from hole to hole, and some times even within the length of each hole. Therefore the hammer parameters should actually be under constant surveillance and modification, which has been impossible to attain up to now. From the highly variable results observed on the same hammers under assumed equal conditions it can be deduced that they frequently work far from their optimum adjustment.

Development programme for automation

Finally. rather rough treatment of the equipment, for instance very abrupt boom movements. is a common sight in tunnels.

The Norwegian engineering and contracting company Ingeniør Thor Furuholmen A/S had foreseen as far back as 1970 that electronics and computer technology would one day affect its main business of hard rock tunnelling and so ill 1972 started a development programme for automating tunnel drilling rigs.

During the study, an ordinary full scale one-boom rig was mounted in the laboratory equipped with instruments and a computer and used for test runs of mock positioning and drilling. It was concluded that, from a technical point of view, computer controlled tunnel drilling rigs could be developed with the specified performance and reliability, but the equipment cost was high and at the time it was thought that the market was not ready to accept the technology .However, work was started again in 1977 and the first working prototype, a three-boom jumbo, was produced and tested by 1978.

At the present time the company's experience includes drilling more than half a million metres of 45 mm holes using the micro-processor controlled prototype drilling jumbos. The latest versions are equipped and programmed for automatic and precise drilling of complete rounds according to predetermined drilling plans defined by fixed coordinates. The microprocessor-based control systems are now being licensed to manufacturers of equipment.

The computer and general electronics technology applied in Furuholmen's automatic tunnel jumbos have be en developed by The Central Institute for Industrial Research, Oslo, Norway, which has accumulated much experience from other industries such as ship building, paper processing and instrumentation of North Sea oil platforms.

Furuholmen started tunnelling in 1945 and since then has completed about 600 km of tunnel at the rate of up to 50 km per year as well as about 60 major rock caverns for hydroelectric power, oil storage, sewage treatment and other civil and military purposes. At present the company owns a total of about 50 tunnelling jumbos of all types, most of which are of its own design and manufacture, and in addition three full-face machines owned and operated jointly with Prader of Switzerland.

Prototype development

W hen the project was started again in 1977 the market outlook was still unclear , but several conditions had changed in favour of this technology. The competition for skilled operators had increased. The cost of labour had increased, more than the cost of equipment. New government rules for safety and working conditions for operators, such as sound insulated operator cabins with clean air, had been introduced. Huge improvements in micro-electronic technology that facilitated cheaper and more reliable design of computer control systems for this purpose had by this time become

available. Experience from the use of micro-processors had already been obtained from other applications.
It was concluded that these factors, together with Furuholmen 's own need for new, modem tunnel drilling rigs, justified further development leading to working prototypes.

Fig. 1. Computer controlled jumbo designed by Furuholmen.

Design of the first prototype
For the first working prototype the aims were limited to having the physical equipment necessary for automatic control installed. The jumbo had however to be operated manually and run with only the measurement and display part of the programs active to test out reliability and suitability of the different components.
The main specifications were set as follows: the first prototype was design ed with three booms to drill on face areas from 35 to 90 m^2 in one set-up. The boom reach was 8.5m high by 12.5m wide. The boom geometry and chassis together with the hydraulic system were all of new design. Total weight of the jumbo was 42 tons.
In designing the booms, much emphasis was placed on getting a simple but rigid construction to facilitate ruggedness and precision. It is one of the benefits of using advanced control systems that the mechanical part of the system can be optimised for stability and mechanical simplicity without being limited to geometries that are easy to operate manually.
The booms each have seven independent axes of movements in addition to the motion of the drilling hammer on the drill guide. The sensors, measuring the motion of each axis with great precision, were designed as an integral part of the boom so that they are well protected against damage and humidity.
The hydraulic power packs were designed as modules, one for each boom with the same motor and pump unit for powering the hammer work as well as boom movements and with value manifolds comprising all the electro hydraulic control valves as an integral part.

The electronic operator panel, including a CRT graphical display and electronics cabinet containing the micro- processors and interface circuits, were installed unprotected on the rig, without any operator's cabin or other shelter .
The prototype was first used on a large sewage treatment plant at Slemmestad in the Oslo area, where a system of 12 large caverns for process equipment was drilled and blasted. With so many drilling faces available in a concentrated area, the rig was operated almost continuously on a two shift basis. During this period of about a year the rig drilled 380000m of holes for blasting out approximately 270000 m^3 of solid rock (limestone and shale). This demonstrated a high mean availability of the rig. Maintenance problems in general were very small, for the electronic equipment and sensors as well as for hydraulic and mechanical parts.
When this job was finished, a thorough inspection and checkout of the whole rig was performed. It turned out that the wear and tear on the booms also was quite negligible compared with the general experience from other rigs. This is probably due to a great extent to the soft and controlled manoeuvring of the booms obtained through the electronic control system.

First prototype at work
The first prototype is now drilling on the mainland side face of a road tunnel under an arm of the Barents' Sea out to the city and island of Vardø. Until now it has drilled roughly 100 000m in very fractured sandstone.
Extremely difficult geological conditions have caused great problems for the drilling and blasting. Concreting has been necessary to a great extent, and it has therefore not been possible to utilise the rigs' capacity in the same manner as on the first job. However, the cold and humid climate in this area, with a seawater infiltration through the tunnel roof, represents a severe environmental test of reliability for the electronics. So far all parts of the jumbo, including the electronic part, have withstood the test.

Design of the second prototype
The second prototype was ready for testing in September 1979. Mechanically, this rig is identical to the first one with the same size of chassis, booms, hydraulic power units, etc, but several additional features were included. These were a sound insulated and heated operator's cabin, and the computer was programmed for automatic positioning and drilling of complete rounds in curved tunnels, as described later. After a testing and debugging period of two months, the rig was set in production on its first job. The first]ob for this second prototype was two road tunnels in the Arendal/Grimstad area in southern Norway. Approximately 120000m of hole was drilled in hard basalt stone. The cross-sectional area was $70m^2$, and both of the tunnels were curved in the horizontal as well as the vertical plane, with variable cross sections.
As this job was a combined training and test-out of computer control under almost full production, the system was first operated in the manual computer controlled mode. Gradually automatic controlled positioning and drilling, according to the programmed drilling plan, was applied.

Satisfactory operation
Technically the equipment operated satisfactorily with few maintenance problems. After some adjustment of the procedures for automatic collaring and bending compensation, the automatic control mode was operated regularly with satisfactory results. The procedures for drilling curved tunnels according to the programmed plan

worked perfectly with few problems in drilling the contour holes when successive rounds were drilled by program control.
When a round for some reason had been drilled manually, minor geometrical transition problems had a tendency to occur. Systematic statistics have not been collected so far but there is no doubt that the contour was generally smoother and more regular when programmed drilling was applied.
After the job was finished the whole system was carefully checked and the same conclusion could be drawn as with the first rig concerning wear and tear and maintenance cost. The next job for this jumbo is expected to be a hydro tunnel in Greece now under negotiation and planning.
A third prototype with the same equipment and specifications as number two was ready in January 1981, but has not yet been assigned to any job.
Two prototypes of small size jumbos have been built and are now on their first job on hydro tunnels at the Aurland power plant in the western part of Norway. These jumbos both have two booms designed for face areas from 10 to 20m^2. The booms have in principle the same type of geometry as the three big prototypes, but dimensions and weights have been reduced. The booms have been designed and prepared for computer control. They are however, at this first stage operated manually from an electronic control panel.

Description of equipment and methods

The control system consists of:

- Electronic control unit with printed circuit boards based on microprocessor technology
- Operator's panel with easy-to-learn joy-stick controllers
- Visual display unit for graphic display of drilling plan; drilling directions relative to selected axis; drilling penetration rates; alarms and other operator and maintenance aids.
- Sensor units for measuring positions, angles, pressures, etc
- Hydraulic power pack with electro hydraulic control valves for controlling boom movements and drilling.

The control system is based on a modular design and can therefore easily be adapted to rigs with a different number of booms. different geometry of the booms, different drill hammers, etc. The electronics, cabinets and sensors have been specially designed to survive the rough environmental conditions found in tunnels.

Method of operation

The jumbo can be operated in three different modes:

1. Automatic positioning and drilling according to a pre-programmed drilling plan
2. Manual operation by using the manipulator mode of the control panel
3. Direct control, which is a standby option to enable it to be operated even if a failure should occur in the control system.

The mode of operation can be selected individually for the different booms by means of push-buttons on the operator's panel. The design of the control system makes it easy to choose between the different modes and to change from one to the other.

Manual override in the automatic mode can therefore easily be performed, in case a hole cannot be drilled in the planned position for instance. After the manual override is performed, the system is set back to the automatic mode again by a push button.

Automatic positioning and drilling

In the automatic mode the control system moves the boom to the programmed position for drilling a hole according to a programmed drilling plan. Then the feed rail is moved towards the rock and drilling is started, entering the drill bit smoothly, so that no side-sliding occurs. After the drilling is controlled to programmed depth, the drill is returned and the boom is automatically moved to the next hole position and the procedure is repeated.

The programmed drilling plan is displayed on the Visual Display Unit (VDU) so that the operator can follow the progress of the drilling at all times. Separate symbols are used to mark the holes that have been drilled, the hole being drill ed and the next hole to be drilled, so that the operator can override if any obstacles should make this necessary.

If, for some reason, the hole cannot be drill ed in the planned position, the operator may override and move the drill bit to a better position by means of the manipulator position joy-stick. The direction of the drill rod will automatically be parallel to the programmed one.

If the operator presses the button ORIENTATION, the drill rod will change direction so that although it points toward the bottom of the hole, which will remain as planned, the start of the hole will have been manually moved to a new position.

Precise drilling without precise positioning of the rig or marking on the tunnel face may be carried out as follows: the different blast holes are programmed in a local coordinate system in relation to the tunnel axis at the face. The description of all the holes relative to this coordinate system constitutes the *drilling plan*. All the holes can have individual positions, directions and depths.

The local coordinate system is described relative to the fixed coordinate system as a function of tunnel length by the *tunnel description*. The fixed coordinate system is defined by the direction of a laser beam and the direction of gravity. The *tunnel description* and the *drilling plan* are stored in the control system 's memory.

Several alternative drilling plans can be stored simultaneously and selected by a button. New *drilling plans* and *tunnel description'* can be applied by means of a magnetic tape cassette in the field.

Position of blast holes computed

After the jumbo has been moved to its working position in front of the face, the position of its local coordinate system relative to the fixed system is measured. This is performed by moving one of the drill guides by means of the manipulator joy-sticks until it coincides with the reference laser beam. When the *navigatio'* button is pressed the computer will then read the different boom-angles together with the rig's tilt-angle and compute the rig position. This information is stored and used to compute the position of the blast holes relative to the rig's coordinate system.

The necessary angles and boom- extensions to position the drill rods for drilling of a certain hole in the drilling plan are computed by means of a mathematical model of the geometry of the rig and the booms. This mathematical model is operated dynamically in real time, so that the motion of the boom from where it actually is to the programmed drilling position is smooth and well controlled.

Manipulator mode
Precise positioning and directioning of the drilling tool is a difficult task on an ordinary jumbo, because each individual motion influences the position as well as the direction of the rod. On a rota-boom this is especially complicated. In the MANIPULATOR mode precise manual control of the drill guide is a very easy operation and the boom movements are taken care of automatically through the computer. The control is performed by means of two three-axis joy-sticks. One joy-stick is for change of position and the other for change of direction of the drill rod- The coordinates of the joy-sticks are relative to the position and direction of the drill rod, so that the operator does not need to think of the complicated geometry of the boom. The direction of the drill rod is shown graphically on the VDU relative to the predetermined reference axis. Initially the reference axis is parallel to the axis of the rig. By setting one of the drill rods in a new direction and pressing the navigation button the direction of this drill rod will be the new reference axis.
When pressing the orientation button on one of the other booms, this boom will automatically change its direction to be parallel to the reference axis, without changing the drill bit position. The manipulator mode is convenient to use when overriding manually in the automatic mode, positioning one of the feeds parallel to the reference laser beam for establishing the rig's position; or for every drilling task where a pre-programmed plan is not used.

Advantages
Based on experience so far , Furuholmen is confident that the equipment and methods work reliably in the rough environmental conditions encountered in tunnelling. Improvements in operational performance and drilling precision have been demonstrated. Maintenance of the booms and associated parts has been greatly reduced due to the smoother action of the computer controlled movements.
The computer controlled drilling rig is also a tool with the inherent possibility of working systematically towards optimum results for hole patterns, hammer work, etc. This comes from the fact that the whole work procedure is recorded by the computer so that results can be measured against choice of parameters. This means that the tunnel engineer takes a well equipped laboratory with him into the field.

Future developments
Although the systems described are technically advanced, they represent only the beginning of the application of sophisticated techniques in tunnelling. It is an inherent feature in the design of the systems that further developments may be added later .
Further developments envisaged in drilling include automatic drilling without supervision, optimisation of hammer- work, and automatic monitoring and indication of equipment failure.
Among other applications in tunnelling, scaling and shotcreting could be mentioned as interesting areas.
The methods and equipment developed are now being prepared for series production, and will be marketed world wide. Licence agreements with rig manufacturers are being negotiated for application to their products for mining and construction. Furuholmen, as a contractor, will offer demonstrations of equipment and methods during the performance of actual tunnelling jobs.

The development and manufacturing of the equipment and methods for computer controlled tunnel drilling are performed by the following group of companies and research institutes:

Ingeniør Thor Furuholmen A/S
A Norwegian civil engineering contracting company. In tunnelling work they are one of the biggest operators in northern Europe with ! approximately 40 jumbos and three full-face machines in operation.

Andersens Mekaniske Verksted A/S
A Norwegian machine shop specialising in design and manufacture of equipment for mining and tunnelling.

Central Institute of Industrial Research
A Norwegian industrial research institute, which has a wide experience in the development of industrial automation systems for any purpose (including rough environments), bas ed on advanced technology.

Norwegian Electronics A / S
A company specialising in development and manufacture of professional electronics for industrial purposes.

CRITERIA FOR THE SELECTION OF FULL FACE TUNNEL BORING OR CONVENTIONAL TUNNELLING *

* Reprint from Proc. 4th Congress. Int. Soc. Rock Mech., Montreux 1979.

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SUMMARY:
The main elements in an empirical model for predictions of net penetration rate and cutter costs for tunnel boring with disc cutters are presented. Systematic cost calculations based on this model allow comparisons between tunnel boring and conventional tunnelling. This is an important tool for feasibility studies for tunnels and in design of larger hydro power projects. Even if tunnel boring under Norwegian conditions still usually are more costly per meter tunnel than conventional tunnelling, tunnel boring may allow large savings in construction costs due to different design and shorter construction period.

PREDICTION MODEL FOR TUNNEL BORING.
The first tunnel boring projects in Norway have mostly been in populated areas where the wish to avoid blasting vibrations has been decisive in the selection of construction method. The rocks have usually been rather weak. The possibilities of tunnel boring in harder rocks are improving, mostly due to the development of single disc cutters with high thrust capacity. In order to develop a tool for the planner of tunnel projects a prediction model for tunnel boring has been developed in a cooperation project between the departments of geology and construction engineering at NTH, The University of Trondheim. A short resume of the main elements in the model will be given here. The background for the model is empirical and it is presently under revision and updating since more experience with tunnel boring has been gathered since the introduction of the model in 1976. The model is based on three factors describing the bore-ability of the rock mass:
A drilling rate index, a bit wear index and the intensity of weakness planes in the rock mass. Figure 1 shows the expected net penetration rate for a tunnel boring machine (TBM) with 3,5 m diameter quipped with single disc cutters, as a function

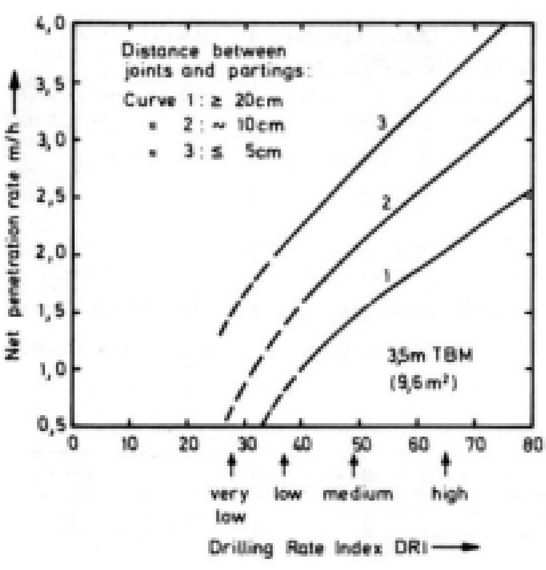

Fig 1. Expected net penetration rate for a 3,5 m TBM as a function of the Drilling Rate Index and the intensity of weakness planes in the rock mass.

of the Drilling Rate Index DRI and the distance between joints and partings.
The cutter costs per solid m^3 rock as a function of the Bit Wear Index and the intensity of weakness planes are given in figure 2.
The two indexes have primarily been developed for percussive drilling and are detailed described by SELMER-OLSEN & BLINDHEIM [l]. The indexes have been extensively in use in preliminary investigations for tunnels over the years in Norway and have proven to be a valuable tool for drillability classifications for percussive drilling both for light and heavy drilling equipment. Rough correlations have also been found between these indexes and tunnel boring results and this have justified the use of this indirect method also for bore- ability classifications for tunnel boring. The verbal labelling "low", "medium", "high" etc. for these indexes (see figure l and 2) are based on experience in our range of rock types from soft, but usually quite competent sedimentary rocks, to metamorphic extremely hard rock among the hardest ever found.

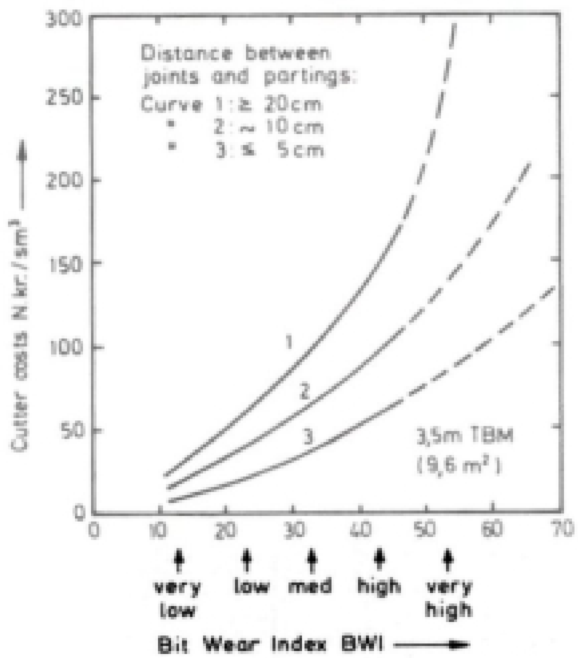

Fig. 2 Expected cutter cost for a 3,5 m TBM (with singel disc cutters) as a function of the Bit Wear Index and the intensity of weakness planes

Together with the average distance between joints and partings in the rock mass these indexes include the three important factors in bore-ability, rock strength, abrasiveness and the weakness planes. It must be remarked that as weakness planes not only clear intersecting joints are counted, but also weakness planes such as plane continuous partings of mica or other laminated minerals. These have a tensile strength perpendicular to the plane which is equal or practical equal to zero and will make an increase in net penetration possible compared to solid rock. From the diagrams it is clear that this effect is considerably large especially w hen the distance between the weakness planes is down to the same size order as the distance between the cutter grooves at the tunnel face.
The model also consists of correction factors for increasing tunnel diameter, considering that the cutter head rpm is reduced with increasing diameter and on the other hand that larger diameter allows room for cutters with large-capacity bearings. Since the method is empirical the diagrams will only be valid a short period of time as the tunnel boring technique is under constant development. The correlation with practical results has, however, been as good as could be expected from an indirect method like this. Based on the experience gathered in the two years the method has been in use it will be re- fined and updated during springtime 1979. It is hoped that the orientation of weakness planes and the thrust level can be included in the model in a more specified way. WANNER [2] has shown that the net penetration can increase as much as up to 100% with increasing angle between tunnel axis and foliation planes. OZDEMIR & MILLER & WANG [3] have also clearly demonstrated how

important the thrust level is for penetration rate. This is verified by experience and the prediction model must presently be limited to boring with single disc cutters with diameter 30 and 35 cm, a good utilization of the thrust capacity, relatively large cutter spacing (50- 75 mm) and boring approximately parallel to the dominating system of weakness planes.
Of course the penetration rate and cutter costs may vary much due to the handling of the equipment. The predictions should be valid for good working practice and cannot be considered as a guarantee.

TUNNELLING COSTS.
Based on detailed systematic cost analysis of tunnel boring, on the background of the shown model, and on similar studies for conventional tunnelling, a comparison of the unit costs for the two tunnelling methods is possible. Such detailed studies are reported in the project report [4] and the main conclusions will be given briefly here.

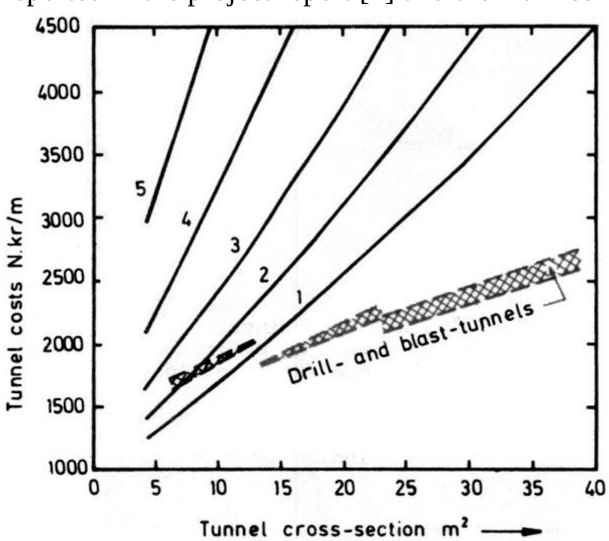

Fig.3. Unit cost for machine bored and conventional tunnels. Tunnel lenght 3 km.

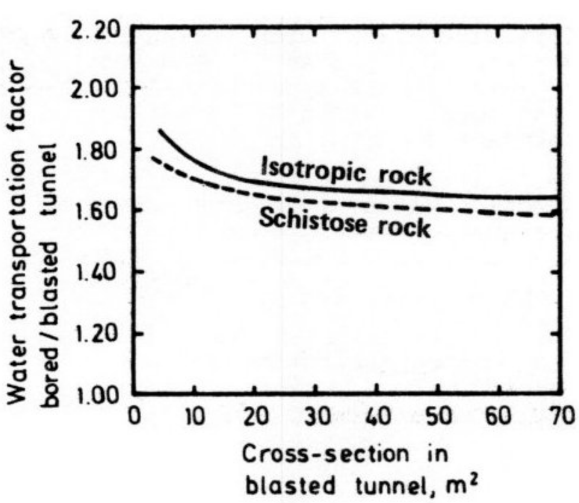

Fig 4. Relative water-transportation capacity of bored tunnels compared to blasted as a function of tunnel size.

Figure 3 shows the tunnel costs in unit costs per m tunnel as a function of the tunnel size. The cost estimates are based on a tunnel length of 3 km and are not including supporting or rock reinforcement. As can be seen the costs for tunnel boring are lower than for conventional only under the most favourable rock conditions. The cost level is based on Norwegian working conditions and 1976-prices.
Important assumptions are a total utilization of the TBM of 40% and a good stand-up time of the rock mass. Delay due to serious stability problems, such as crushed zones and heavy rock spalling are not included.
A typical medium-strength gneiss would usually follow curve 3 or 4 and in such rock tunnel boring cannot presently compete with conventional tunnelling under our conditions in direct unit costs.
Bored tunnels will have a better water-transporting capacity as figure 4 illustrates based on both theoretical analysis and direct measurements.
Relative water-transportation capacity of bored tunnels compared to blasted as a function of tunnel size. If this is taken into consideration the picture is more

favourable for tunnel boring.
Figure 5 shows the unit costs as a function of tunnel, size with equivalent water-transportation capacity. More rock can then be bored economically especially if the possible reduction in supporting costs is included. Under our conditions this cost reduction is mainly due to the fact that schistose and jointed rock of ten can be left unsupported in a bored tunnel, where as it might need a lot of systematic scaling and bolting in a blasted tunnel. Supporting work and tunnel lining which can wait until it can be done behind the tunnel boring machine, can also be done in a more rational way. On the other hand, problems with intense rock spalling and in loose-consolidated crushed zones with swelling clay may be more costly to handle close to the tunnel face during tunnel boring.

Table 1. Curve exploration for machine bored tunnels.

Curve No.	DRI	BWI	Distance between joints and partings
1	High	Low	< 5 cm
2	Med. High	Med. Low	< 5 cm ∿ 10 cm
3	Low Med. High	High Med. Low	< 5 cm ∿ 10 cm > 20 cm
4	Low Med.	High Med.	∿ 10 cm > 20 cm
5	Low	High	> 20 cm

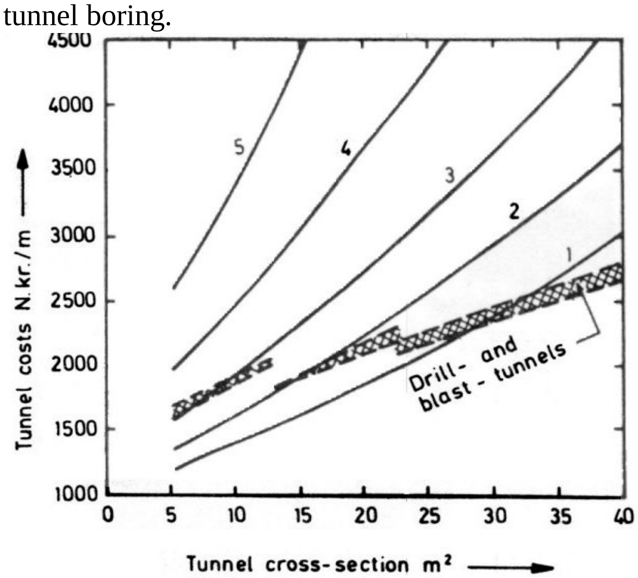

Fig. 5. Costs for machine bored tunnels (curve 1 to 5) compared with equivalent water-transportating capacity.
Curve explanation as in figure 3.

CALCULATION EXAMPLES .
Besides the possible savings in tunnelling costs under favourable geological conditions, important cost reductions can be obtained by the project design. Stor-Glomfjord power project in northern Norway is a good example of this.
As figure 6 illustrates the project will be constructed different with tunnel boring than with conventional tunnelling. With tunnel boring one construction site can be spared. This means saved road building and all the problems with winter construction in areas with extreme weather conditions.
Reduction of construction interference in the virgin nature shown in figure 7 is also important even if this is difficult to express in hard numbers. The geology is relatively favourable.
There are few large crushed zones in the area. Some problems with rock spalling are expected, hence the tunnel boring machines will have to utilize rock bolting equipment immediately behind the cutter-head. The rock types are mica schists and

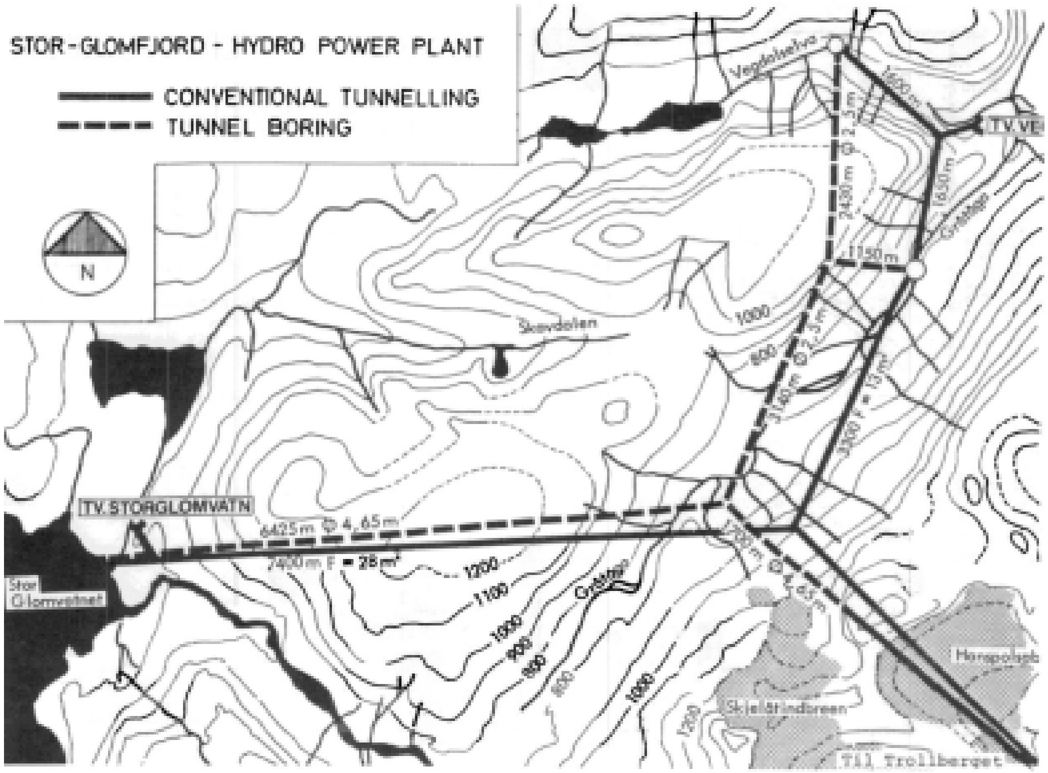

Fig. 6. Stor-Glomfjord hydro power project. With tunnel boring one adit including road building can be saved (in the north-eastern part of the project).

crystalline limestone with a jointing which is favourable for the penetration, but not expected to give problems with stability in the bored tunnels. The total construction time with tunnel boring with one adit is expected to be the same as for conventional tunnelling with two adits. The bored tunnels are also placed to encounter as little as possible of rock types with low boreability.

The total savings by tunnel boring in Stor-Glomfjord Hydro power project are given below

Construction sites, operation of construction sites, adits (cross-cuts) , road building:	7, 400 000 ,- NOK
Different construction of tunnels:	7, 100 000,- NOK
Interest during the construction period:	7, 700 000,- NOK
General costs:	6, 300 000,- NOK
Total possible saving:	**28, 500 000,- NOK**

The main savings in this case are not due to differences in unit costs for the tunnelling itself, but connected to the total design of the project.

In the areas in the neighbourhood of Svartisen glacier a total of about 200 km of hydro power tunnels are planned. The topographic and weather conditions make it desirable to use long tunnels from each construction site or adit.

The rock types are mica schists, limestone and granite. The mi ca schists are found over large distances with a varying mineral content and schistosity. Often the foliation is developed to an intense schistosity with continuous layers of mica. At the surface where the rock is exposed to frost weathering the outcrops often look like the picture in figure 8. At the depths of the tunnels all these partings will not really be present as

Fig 7. Stor-Glomfjord area by Svartisen glacier in northern Norway

clear discontinuities in the rock mass, but the mica partings will still work as weakness planes influencing the bore-ability. Boring results are expected to be quite favourable even if the rock itself between the partings can be quite hard and abrasive. This is already confirmed by experience from tunnels in phyllites and quarts schists in western Norway when the rock is not intensely folded on the scale of the distance between the cutter grooves.

The main factors considered in the study of this large hydro power project have been:

1. Rock mass properties, bore-ability and stability.
2. Tunnel cross-section.
3. Tunnel length from each adit.
4. Design changes (tunnel layout, number of adits) .

An increased tunnel length from each adit is possible due to less need for ventilation and possible higher overall tunnelling speed.
The first preliminary evaluation of this project based on the general geological investigations in the area gave the results as indicated in table 2.

Fig. 8. Frost-weathered outcrops of mica schist with plane mica particles.

For 12 tunnels the engineering geological preliminary investigations were done by field mapping of rock types, weakness planes and sampling for drillability/drillability tests in the laboratory.
After these studies 16 tunnels with a total length of more than 100 km are expected to give savings by tunnel boring of the size order of 125 million NOK.
The change-over to tunnel boring demands considerable investments. Table 3 shows, however, that the need for investments and the rest value of the tunnel boring machines after the construction period are considerably smaller than the possible total savings.

	Number	Tunnel length, km
Planned tunnels	48	200
Expected to be bored with savings	19	120
Geological preinvestigations	12	80
Savings not expected after geol. preinvest. (out of the 12)	3	18

Table 2. Overview of the number of tunnels and tunnel lengths considered for tunnel boring at Svartisen glacier area hydro projects.

TBM diam. m	Machine costs 10^6 N.kr.	Rest value x 10^6 N.kr.	Number of tunnel faces	Bored length, km
2,5	6,5	0	5	31,4
2,5	6,5	0	5	30,6
3,1 - 3,5	9,0	3,6	2	8,8
4,3	12,5	1,1	2	16,4
4,3	12,5	5,6	1	7,8
4,65	14,0	6,6	1	7,9
Sum	61,0	16,0	16	102,9

Table 3. Need for investments and rest value of tunnel boring machines for construction of Svartisen hydro power project.

By tunnel boring the penetration and costs are more dependant on the rock mass properties than by conventional tunnelling. The limit costs for an increase in tunnel size is larger for tunnel boring, as can be seen from the steeper curves in figures l and 3, than for conventional tunnelling. As a result of this the equivalent optimal cross-section for bored tunnels may be less than for drill and blast tunnels. The optimal tunnel cross-section is then defined as the Cross-section for which the limit costs for construction and capitalized production value are the same.
An optimisation of the cross-section for bored tunnels may result in smaller tunnels and hence from a wider economical point of view in an unfortunate reduction of the utilization of the water power resources and therefore lost energy production. To a certain extent this will be self regulating as unfavourable geological conditions for tunnel boring for a certain tunnel will result in a higher tunnelling cost which makes it less actual to bore that tunnel. Comparisons of the optimal cross-sections for the different methods may therefore also have an influence of the choice of tunnelling method.

CONCLUSION.
Full face tunnel boring is al ready an advantageous method for driving water-transporting tunnels. The method has today a greater potential of development than conventional tunnelling. The change to the two-shift working day will further favour tunnel boring. Since tunnel boring still is more sensitive to geological conditions than conventional tunnelling, it is important that the selection of tunnelling method takes place after thorough analysis of all the factors involved including bore-ability, stability and supporting together with design economy.

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Ground freezing techniques used for tunnelling in Oslo city centre.*

* Reprint from Proc. The 2nd International Symposium on Ground Freezing, Trondheim 1980.

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Introduction

The new 3,6 km long railway tunnel under Oslo, which has recently been opened for traffic, is connecting an eastern and a western railway system. The tunnel passes through the centre of the city , and the varying ground conditions, a number of old and vulnerable buildings, and Streets with heavy traffic imposed a series of problems for the designers and the contractors. One major problem was crossing under the street Drammensveien, with heavy traffic and situated in the most representative part of the city , close to The Royal Garden. At this location there is a depression in the rock surface, going down to the roof of the tunnel, and making normal tunnel blasting impossible.

The tunnel, which is normally double tracked, is here divided into two single track tunnels going into the nearby National Theatre Station. The station is blasted in rock. just above the railway tunnel, and same metres aside of it, there is an old underground tunnel for turning the trains on the city's western underground system.

Ground conditions

Underneath Drammensveien the bedrock surface is shaped like a canyon, crossing the tunnel axis at an angle of approx. 55°. According to the site investigations, the bottom of the canyon is located about 0,5 m over the blasting profile for the tunnel. On the north side of the tunnel the canyon is very steep and narrow, and widening to the south. Over the critical part of the tunnel the width of the canyon is approx. 20 m at the top and 5 m at the bottom. The bedrock is shale with some limestone. Experience has shown that the bedrock in the bottom of such canyons is fissured and of bad quality for tunnelling.

The canyon is filled with sandy gravel and stone. Under the street pavement there is a 3 m thick layer of stone fill, and from this layer to the bottom there is natural well-graded sandy gravel. Fig. 2-3.

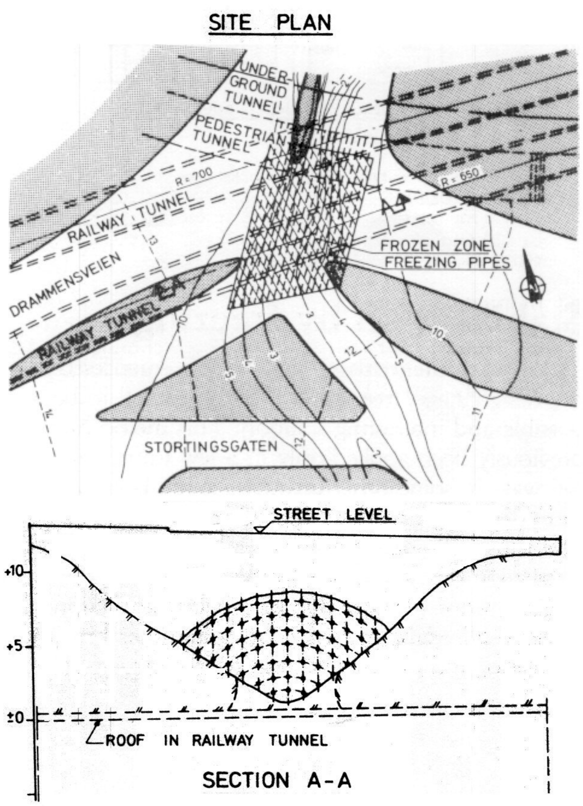

Fig. 1. Site plan and section.

The natural water content of the gravel varied from 12% to 18%. The ground water level was registered at a depth of approx. 5 m under street level.
It was obvious that under these conditions we had to find a special way to construct the tunnel. Discussions with the Road Authorities and the
Traffic Police also made it clear that an ordinary cut-and-cover method at this spat would give almost impossible traffic problems, and we were strongly requested to find a solution without an open cutting.

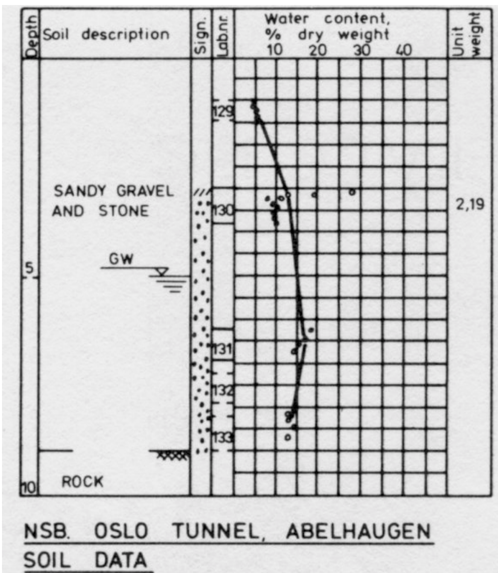

Fig. 2. NSB Oslo Tunnel, Abelhaugen. Soil data.

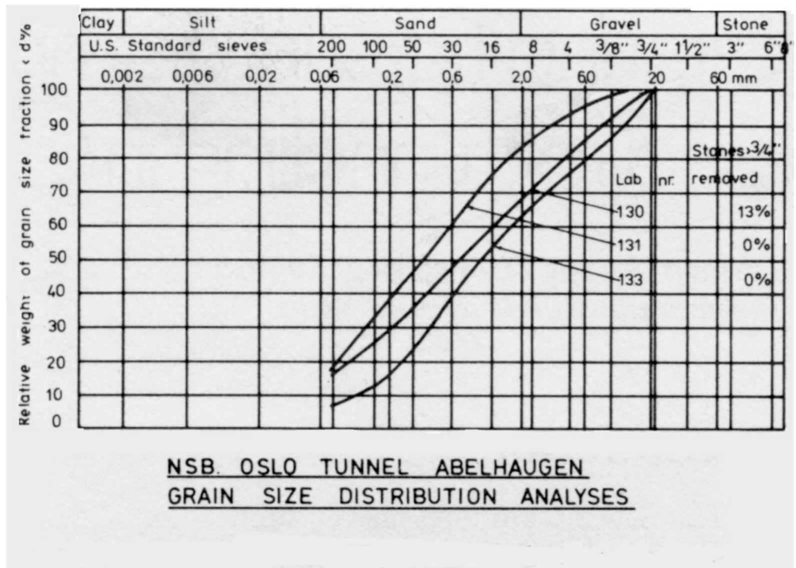

Fig. 3. NSB Oslo Tunnel Abelhaugen. Grain size distribution analyses.

Stabilization by freezing.

Among the methods which were considered at the design stage, freezing of the soil seemed to be a possible and interesting solution. "This method had previously been applied only in a few tunnels in Norway. In some other countries it has been a rather common method applied to similar problems, although not always too successfully. Expansion of the soil due to the freezing, and concentration of water due to capillary attraction Under the freezing process have sometimes brought problems, and even damage to nearby structures. In this case the conditions for a stabilization of the sail by freezing seemed to be rather favourable. The only structure which might be exposed to any damage was a pedestrian tunnel to the underground station. A section of the tunnel would be situated directly over the frozen zone, and there would probably be same cracks due to frost heave. This was not considered to be a serious problem, because it was assumed that repair of such cracks would be modest. Frost heave in the street would possibly make it necessary to adjust the tram rails from time to time, but for the other traffic it would hardly be of any harm.

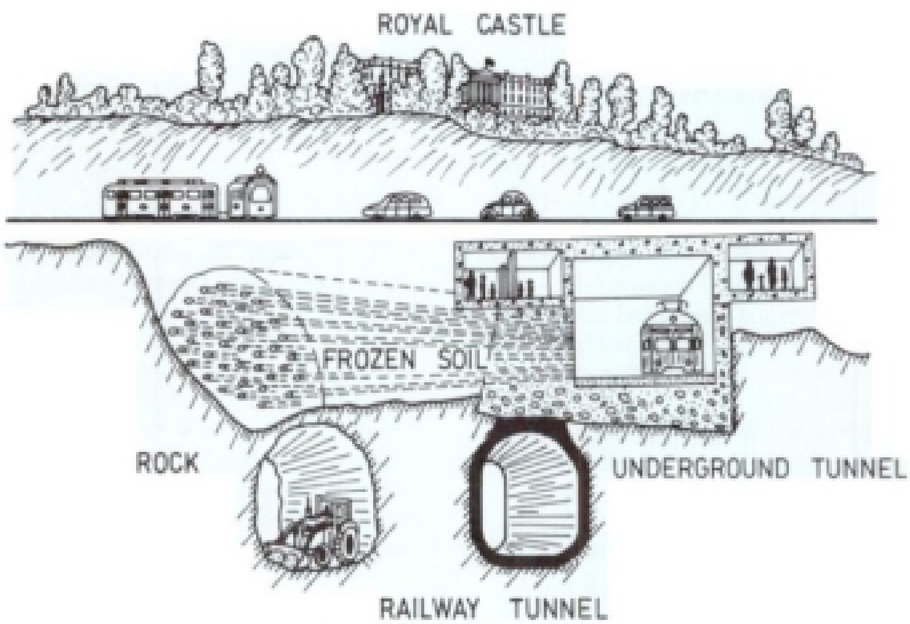

Fig. 4. Cross section of tunnel system.

Strength of the frozen soil

To apply the freezing technique it is necessary to have a sufficiently saturated soil. Due to the capillary action it was estimated that this sandy gravel would be 100% saturated up to about one metre above the ground water level.
Earlier research on frozen soil has shown that the strength is greatly dependent upon the following factors:

1. The soil, its grain size distribution, water content, porosity , and salinity of the water.
2. The type and rate of loading.
3. The temperature of the frozen soil.

For this project no laboratory tests on frozen soil were performed. Karlsrud (1974) tried however to evaluate the strength of the soil on the basis of comprehensive tests carried out by Heiner (1972) on a very similar sandy moraine, as described in the following.

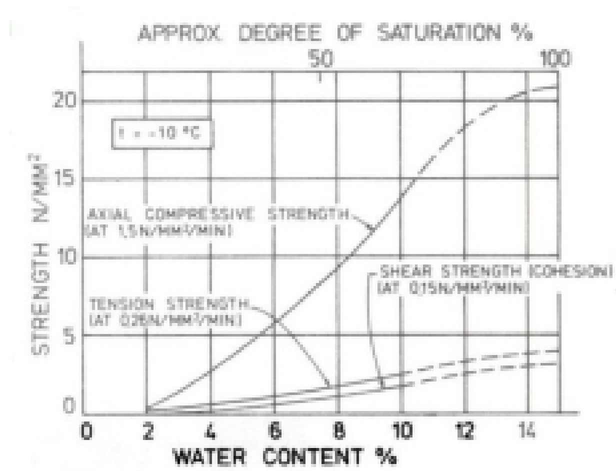

Fig. 5. Strenght versus water content curves (Heiner’s test).

In Heinert's tests the soil samples were compacted with a standard equipment with water content varying from 2-10%. This gave

a porosity of approx. 0,44, which means a water content of approx. 15% in a 100% saturated test sample. This is close to the values at the site in question.
Heiner made three types of shear strength measurements:
a) Simple axial compression tests (unconfined).
b) Tension test (indirect Brazilian method with radial loading of a cylindrical test sample between two plates).
c) Direct shear tests.
All the tests were performed with a water content in the samples ranging from 2-10% and temperatures -+-0,5, -+- 2, -+- 5 and -+- 10°C.
Typical test results are shown in Figs. 5 and 6. The influence of the rate of loading on the axial compression strength is shown in Fig. 7.

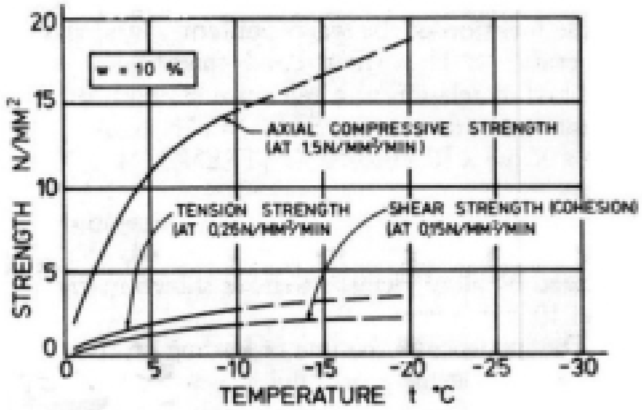

Fig. 6. Strength versus temperatur curves (Heiner’s test).

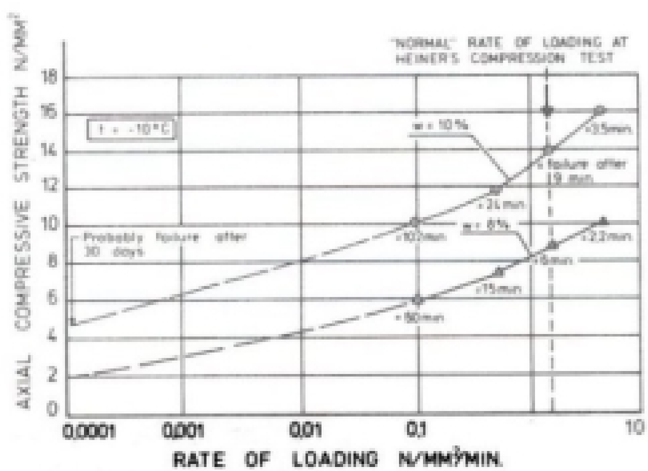

Fig. 7. Influence of rate of loading.

As may be seen, there is a great difference in strength values determined on one side by axial compression tests and on the other side by tension tests and direct shear test. This may be explained using the Mohr-Coulomb equation.

(1) $\tau_f = c + \delta_n \cdot tg\ \Phi$

where

f = shear strength

c = cohesion

n = normal stress

Φ = apparent angle of friction

At t= + 10°C, w= 10% and a rate of loading of 1,5 N/mm/min, the results of Heiner's tests were:

Single axial compression strength, $\delta_c = 14$ n/mm^2

Tension strength, $\delta_t = 2,5$ N/mm^2

Cohesion (direct shear strength at $\delta_n = O$), c- 2,5 N/mm^2.

Fig. 8 shows that there is a rather good agreement between these values, w hen one assumes Φ= -45°.
All the tests performed show very similar dependency on temperature and water content. Thus one may assume that the apparent friction

is constant Φ = 45°, and that the cohesion c is a unique function of the water content w and the temperature t. Thus the strength might be described in relation to a reference strength or cohesion, for instance by STRENGTH (t, w) = K (t) x K (w) x REFERENCE STRENGTH.

This is confirmed by the unique temperature factor K (t) and water content factor K (w), indicated by all of Heiner's tests, as shown in Figs. 9 and 10.

The influence of the rate of loading on the cohesion is a bit more doubtful. It was assumed, however, that it would be comparable to the influence on compression strength, as shown in Fig. 7.

When assuming a reasonable time for the tunnel work, it was estimated a correction factor 0,25, which gave a reference cohesion

c (t= + 10°C, W= 15 %)= 0,9 N/mm^2.

For other values of temperature and water content, corrections could be done according to the curves in Figs. 9 and 10.

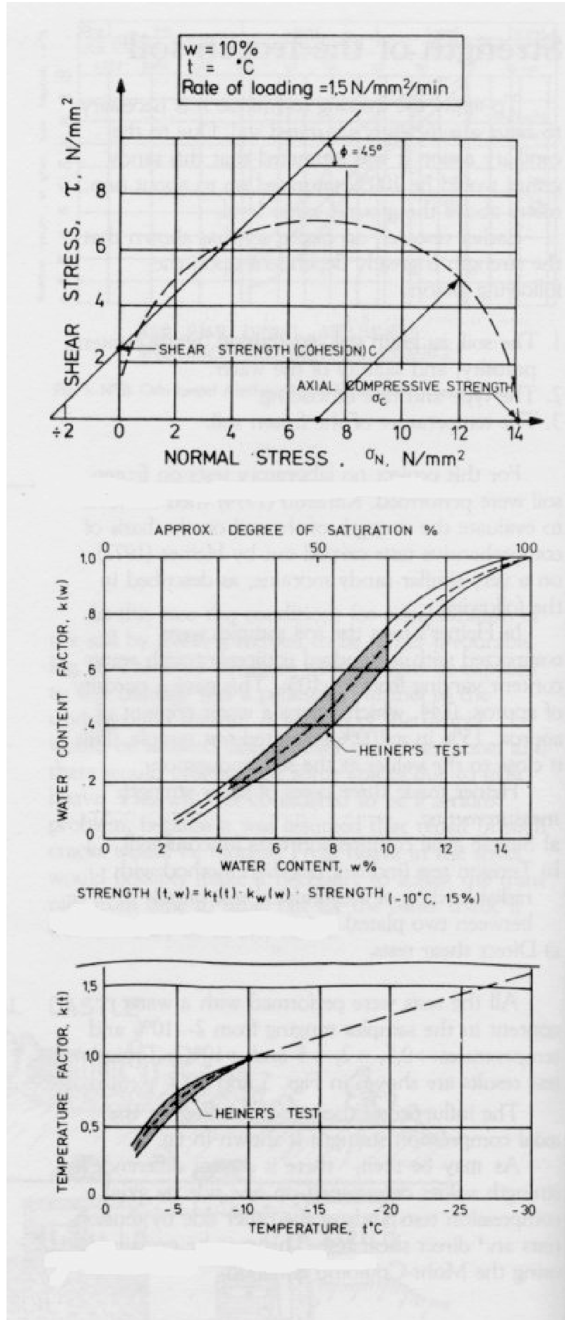

Fig. 8. (top) Representation of stress values by the Mohr circle.

Fig. 9. (middle)Water content factor (Heiner's test).

Fig 10. (bottom) Temperature factor (Heiner's test).

By calculating the bearing capacity of the frozen soil, a safety factor of at least 3 was applied in the actual design.

Planning and execution of the work.

To establish a frozen zone in the ground, pipes for the freezing liquid have to be drilled and mounted. There were three possible alternatives for drilling of the freezing pipes:

1. Vertical drilling from street level through the gravel to the rock.
2. Drilling from the tunnel ah('ad of the tunnel face, at different angles with the tunnel axis to cover a sufficient volume outside the tunnel profile.

3. Drilling from the underground tunnel near to the National Theatre Station. This tunnel is running parallel to the railway tunnel, same metres to the side and just above the tunnel profile.

Alternative 1, with drilling operations and trenches in the street for the freezing pipes, was rejected due to traffic reasons.
Alternative 2 would result in a stop in the tunnelling work during the time for drilling the pipes, establishing the freezing system and freezing of the ground. This time was calculated to approx. 8 weeks.
Altogether the alternative 3 seemed to be most favourable. As can be seen by the drawing, the underground tunnel is situated in a perfect position relative to the freezing zone. Fig. 4.
There were however two serious disadvantages. It was necessary to drill through a solid concrete wall with a thickness up to approx. 7 m and the running of the train in the underground tunnel could not be interrupted. This meant that all the work in the underground tunnel had to be done during the few hours in the night w hen the underground traffic stopped and the electric current could be turned off. Available working time was 5-6 hours per night. During these hours the drilling equipment had to be transported into the tunnel, rigged up and removed again because of the limited space. The effective working time was thus reduced to 3-4 hours.
Collecting pipes for the freezing liquid were mounted on the tunnel walls and led to a pedestrian tunnel near to the platform in the underground station, where the freezing aggregates were also installed. Cooling water was led to the aggregates from water pipes in the tunnel.
The most significant advantage of this system was that the whole freezing operation could be done independent of the tunnel work, hence the frozen soil could be established before the drift of the tunnel under the canyon.
The freezing pipes were placed in a position such that the frozen soil would have the shape of an arch spanning over the rock canyon. The theoretical distance between the pipes was 1,0 m horizontally and 1,2 m vertically, with some variations due to the different angles of the pipes. The pipes were placed in concentric circular arches parallel to the theoretical surface of the frozen arch. Total number of pipes was 56.
The frozen arch had to be designed for the soil overburden and for traffic load in the street. The necessary thickness was calculated to be approx. 5 m.
Two alternative freezing systems were considered. One was an evaporizing plant with freon gas directly in the freezing pipes. The other one was a two-step plant with freon in a primary cycle and CaClz as cooling liquid in the pipes.
The choice was left to the contractor, who
preferred the two-step system, mainly for safety reasons. Necessary capacity for the freezing plant was calculated to approx. 100.000 kcal/hour. The temperature in the frozen soil was assumed to be at least + 15°C after a continuous freezing time of approx. 8 weeks, which was in good correspondence with the temperatures measured at the spat.
The temperature of the frozen soil was recorded by a system of 60 electrical resistance feelers placed in vertically drilled pipes. Of particular importance was the temperature check at the rock bearings of the frozen arch. The length of the freezing pipes was up to 26 m and due to inaccuracy by the drilling it was assumed to be some variation in distance between the pipes.
Pipes for the temperature feelers had to be drilled vertically from street level. Connecting lines were placed in grooves in the pavement from the temperature feelers

to the freezing central room. After the freezing of the soil the blasting for the tunnel could continue like normal in a rock tunnel. Cast-in-place concrete lining was carried out as soon as possible and also as near to the blasting face as possible. A particular problem was concreting towards the frozen blasted rock surface which had very low temperatures. Evidently there could be freezing of the concrete nearest to the rock surface, and accordingly a destruction of the outer part of the concrete. Therefore the thickness of the concrete lining was increased to maintain the safety factor. A particular advantage by tunnelling in frozen rock was the total absence of water leakage in the working period.

From a technical and safety point of view The State Railways had no objections to the method. It was, however, obvious that the work would be rather expensive. The main reason for this was the location of the drilling, which made it necessary to do most of the work during a few hours in the night, including transporting, rigging up and removing all the equipment in the underground tunnel every night, thus reducing the efficiency of the work considerably. The drilling for the freezing pipes through the thick concrete gravity wall, which in addition turned out to have a steel sheet pile wall at the backside, also gave an appreciable addition to the costs. The comparatively small content of volume to be frozen gave a high cost per m^3 frozen soil for the rigging and operation of the equipment. The total price for the freezing operation was NOK. 3,6 mill. (720.000 US$), which, with a frozen volume of approx. 1100 m^3, gave NOK 3.300,- per m^3 (660 US$).

For the choice of method the safety considerations was of vital importance. The freezing method also satisfied the traffic authorities by not disturbing the traffic flow in the street. In addition attention was paid to the environmental conditions, a reduction of the number of open cuts in the city centre for the tunnel construction obviously was valuable for the city. Therefore, in spite of the high cost, The State Railways accepted the method, even though it was a "first-time" experiment for the Railway administration, the consultant and the contractor.

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UNDERGROUND EXCAVATION AND SUPPORT OF A MAJOR SEWAGE TREATMENT PLANT FOR THE OSLO AREA

K. Garshol
Ingeniør Thor Furuholmen A/S, Oslo

SUMMARY

The sewage treatment pl ant is located 30 km SW of Oslo in sedimentary rocks of Palaeozoic age. The excavation amounts to 350,000 m^3 and rock support works to 14,000 m^3 of shotcrete and 25,000 rockbolts. The time schedule allowed one year for excavation and support of I1 caverns of 16 m span, 100 m length and cross sectional area of 150 m^2. Rock classification according to the Q-method of Dr . Bartoll, gives classes "very poor" to "fair" in areas not intersected by shear zones. To achieve the necessary rate of production, the rock support work was highly mechanized to give overcapacity. In 7,5 hours shifts, rockbolts were placed in numbers around 70-80 pieces and shotcrete applied from 25-51 m^3, by 10% rebound. Excavation procedure, equipment and labour force and wet-mix shotcrete test results are also described.

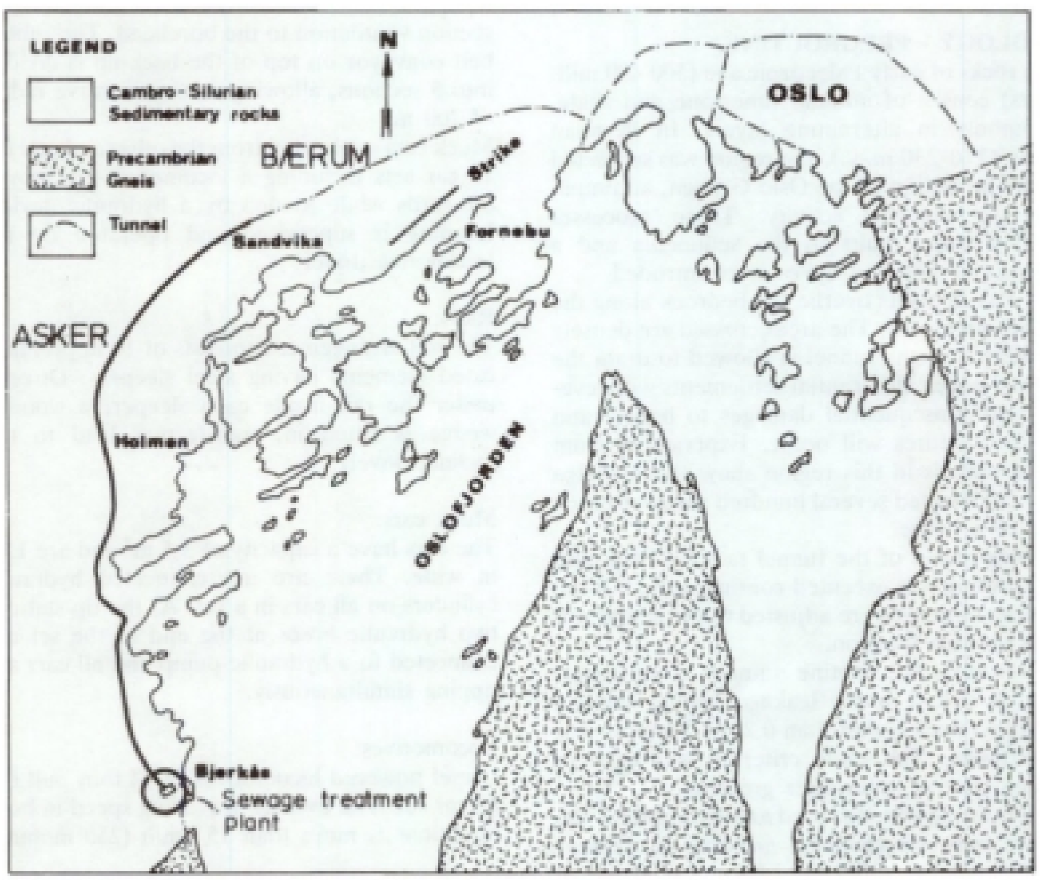

Fig. 1. Project location map and general geology.

INTRODUCTION

The councils of Oslo, Asker and Bærum decided in 1976 to build a large regional sewage treatment plant underground at Bjerkås in Asker. This location is about 30 km SW of Oslo and is the end point of approximately 40 km of TBM made tunnels for the transport of sewerage. Fig. 1.

For further information on background, planning and plant details, see reference (1).

The underground volume of the plant, now excavated, is 350,000 m^3. Concrete works amount to 25,000 m^3. Rock support works executed was approx. 14,000 m^3 of shotcrete and more than 25,000 rockbolts.

The bid sum in august 1978 was NOK 120 mill.

(approx. US$ 20 mill.), which may be subdivided in 30% excavation, 30% rock support, 30% concrete and installations and 10% works above ground. The time schedule was extremely tight, allowing only one year for rock excavation and support.

The pl ant lay-out constitutes 11 parallel caverns having 16 m span and 12 m pillars. Fig. 2. The main caverns are 100 m long and the cross- sectional area is 150-160 m^2. There are also some connecting tunnels, traffic tunnels and a 925 m long outlet tunnel, beneath the fjord sea bottom. The outlet tunnel ends in a vertical shaft.

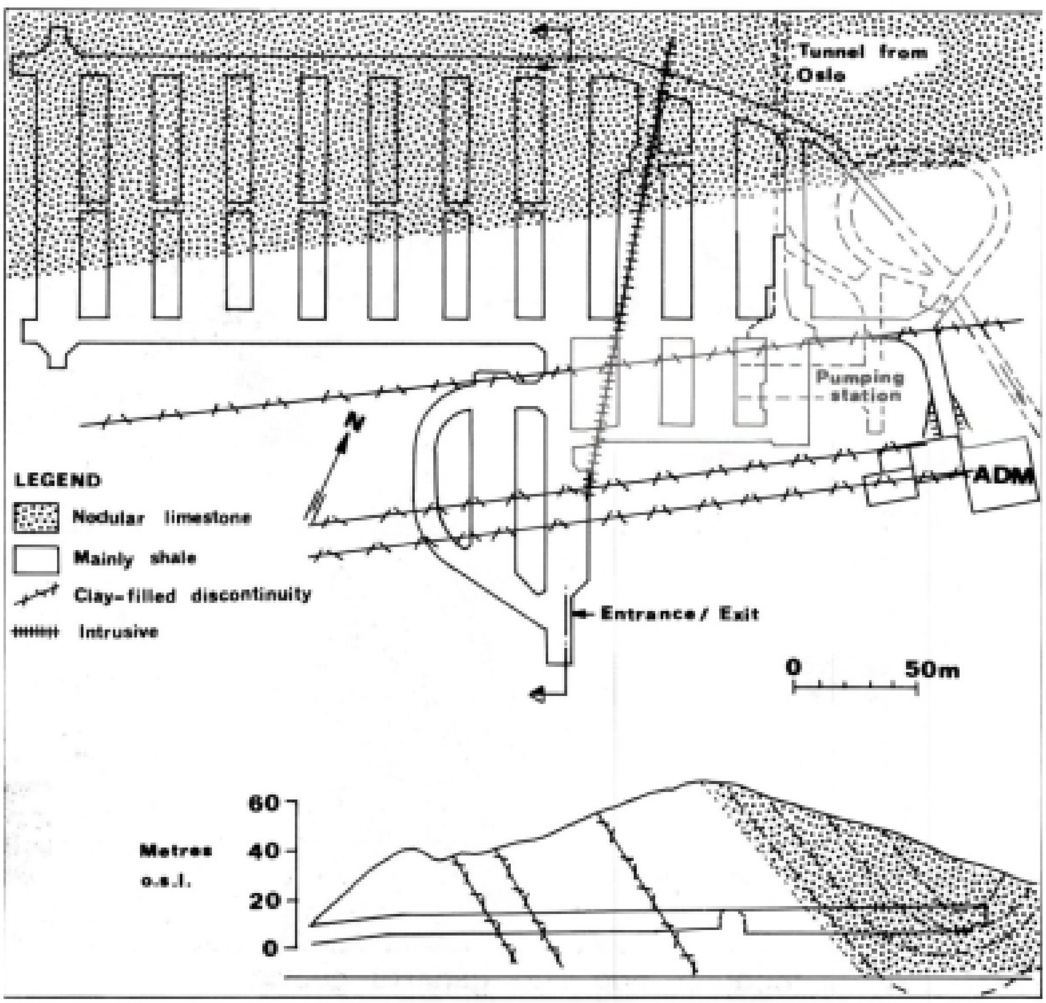

Fig. 2. General plan lay-out and geology.

GEOLOGY

The sedimentary rocks in the Bjerkås hill are of early Palaeozoic age, ranging from 400 to 500 mill. years. In the Caledonian orogeny, the rocks were slightly folded along an axis ENE/ WSW, which is reflected in the present strike direction. In Permian times (250 mill. years ago), the region was subjected to the formation of the Oslo Graben, with volcanic activity. These processes created shear faults in the sediments and a number of sills and dykes were intruded.

The northern part of the plant is located in nodular limestone, and the rest in a shale. Fig. 2. Seismic refraction velocities in nodular limestone was about 5000 m/s, while the shale showed 2500-4000 m/s. Point load strengths derived from cores were 2-4 MPa in limestone and down to 0.1-0.2 MPa in the shale.

At the stage of site investigation, analysis of the rock quality was made by Dr. Barton according to the Q-method (2). The correlation between prognosis and actual conditions was very good. The types and volumes of rock support works executed, also fitted well.

Q-analysis underground shows:

Rock	**Q-value**	**Class**
Nodular limestone	*5-10*	*Fair*
Shale	*0.5-10*	*Very poor to fair*
Shear Zones	*0.05-0.5*	*Extremely poor to very poor*

Fig. 3. Modified backhoe excavator for mechanical scaling.

Fig. 4. Integrated wet-mix shotcrete robot system.

EXCAVATION AND ROCK SUPPORT

The main caverns were blasted full cross-section in a maximum 8 m high top heading. Drilling was done by an electro-hydraulic jumbo with a net round length of 5.0 m. The jumbo was a prototype computerized rig, allowing complete control of drill hole directions, giving very smooth periphery blasting (3).

After blasting, the round was loaded on to 70 t. Komatsu dumpers for transportation. The next step was scaling, done mechanically by an adapted backhoe excavator (Fig. 3). The shotcrete team moved in, covering roof and walls with approx. 10 cm of shotcrete in one operation. The shotcrete nozzle was carried by a remotely controlled hydraulically operated arm. The necessary amount of shotcrete for one round, approx. 10 m^3, was placed in less than 3 hours.

Following shotcreting, the holes for rock bolts were drilled, done by one man operating a drill jumbo. Depending on actual conditions the holes were spaced 1,5 x 1,5 m to 1,0 x 1,0 m. The last step before new blast hole drilling, was placement of rock anchors. The holes were filled from the inner end by a- specially designed cement-based grout. Then a rebar steel of 3,5 m length and diameter 25 mm was driven into the grout.

Step 1.	Drill and blast
Step 2.	Load and haul
Step 3.	Mechanical scaling
Step 4.	Shotcreting
Step 5 .	Drilling of anchor holes
Step 6.	Grouting and placement of rebar steel

To operate a system like this there is a need of at least 6 available headings and an extremely accurate co-ordination of successive steps. It is also a prerequisite that there is a good capacity balance between operations and an excellent reliability of equipment.

This working procedure normally was followed. The only adoption made concerns number of rounds blasted before application of shotcrete and bolts.

In the nodular limestone, rock conditions were such, that two to three rounds could be blasted and then followed by reinforcement.

Mostly the shotcrete carried out at the heading was completed by net reinforcement connected to the rebar bolts and covered by 5-10 cm of shotcrete, carried out behind heading works.

EQUIPMENT AND LABOUR FORCE

Drilling for rock bolts was a one man job, operating a drill jumbo equipped with 3 booms. The feeders were specially mounted to do bolt holes in different directions. To place grout and rebar bolts, a specially equipped truck was operated by two men. The truck carried a mixing and pumping device remotely controlled from the hydraulic work platform. Several holes were grouted from bottom out and rebars then driven in. Shotcreting was executed according to the wet-mix process. This means premixed concrete from a standard mixing plant, transported in a rotating drum. The concrete is poured directly into the pump. The pump is either of piston type or a mono-type. Concrete is pressure-conveyed to the nozzle where it is accelerated by compressed air to be shot against rock surface.

In the compressed air stream, the liquid stiffening accelerator is added by a proportioning pump.
Concrete pump, accelerator pump and tank and the hydraulic robot for nozzle operation, were all together carried by a wheel-loader. Arriving at a new heading the rig was ready in 5-10 minutes. (Fig. 4). The job site had two shotcrete-rigs of this type. A supplementary equipment consisting of a small tractor-mounted robot and a separate shotcrete pump was also available, mainly as a reserve.
The labour force under ground exclusive fore- men and engineers, consisted of 8 men in drill and blast, load and haul. Scaling, shotcrete and rockbolting was executed by 7 men at the heading. All figures are per shift and there were two shifts per day, each 7.5 hours.
The integrated wet-mix robot shotcreting sys- tem is designed and manufactured by Ingeniør Thor Furuholmen A/S, based on 18 years of shotcrete experience as a contractor. Presently rigs of this design are operating in Greece, France, W. Germany, Sweden and several in Norway.

CAPACITIES
The work at site started in August 1978 and at the end of 1979 the caverns, excavation and rock support were finished according to schedule. A rough estimate shows that each working shift (7.5 h), totalling 20 men exclusive foremen and administration, produced on average: Excavated rock volume 500 m3 Rockbolts (L=3.5 m) 42 bolts Shotcrete 23 m^3
A team of 3 workers would normally produce 70-80 rockbolts pr. 7.5 h shift.
The shotcrete crew, consisting of two men plus
truck driver, produced between 25 and 35 m^3 each shift, out of which 90% stays on the rock surface. At top capacity and w hen enough rock surface was available, the shift production was up to 51 m^3.
The capacity and reliability of rock support works were so balanced that excavation itself was the limiting factor.

SHOTCRETE TEST RESULTS
The normal shotcrete mix was composed of natural sand aggregate, with grain size from zero to 10 mm, standard Portland Cement, silica dust, plastifier and water. The accelerator added in the nozzle was sodium silicate (Waterglass).

Mix design per m^3:

Sand 1600 kg
Cement 350 kg
Silica dust 60 kg
Plastifier 3 kg
Water added to slump 15 cm

The waterglass amount used varied with conditions, but normally ranged from 20 to 35 litres per m^3 of concrete. This gives 7 to 13% of the cement weight. The waterglass has a negative effect on compressive strength, but in no way destroys the final quality. There are also control results showing a stronger strength increase between 28 and 90 days than for a normal concrete.
The shotcrete control executed at the job site by the Norwegian Institute for Rock blasting Technology, was carried out by two methods.

Test panels were shotcreted on to wooden boxes, cured and cut in cubes by diamond sawing. Cubes 1000 cm^3 were loaded vertical to and parallel to shotcreting direction. Core drilling was also made in the cavern roof. Diameter of core 60 mm. Drilling was continued into the rock and the concrete core was pull ed to measure bond strength against rock. The cores pulled were brought to laboratory, cut to 60 mm length and loaded to measure compressive strength.
Compressive strength results range was: 30 -44 MPa
Bond strength was not measured at this job site
because rock fragments were pulled together with the cores. Experience from other conditions shows that the normal bond strength range on sound rock is:
1.0 -2.0 MPa
Ten years ago Furuholmen made only dry mix shotcrete. W hen comparing test results of compressive strength and bond strength, between dry-mix and wet-mix method, the following tendencies are found:

1. Dry-mix shotcrete has a slightly higher compressive strength.
2. Bond strength is close to equal.
3. The spread of test results within specific projects are lower for the wet-mix process, which means a more uniform quality.

The owner agreed to pay the shotcrete per m^3 by
deducting 10% rebound. This complies with
measurement of rebound carried out at other projects of wet-mix application.

COST DISTRIBUTION

The final cost distribution was not far away from the original estimates, as shown below:

Concrete 29,4 %
Rock support works 20,3 %
Drill and blast excavations 27,2 %
Grouting 0,6 %
Mobilization 17,6 %
Pier, quay, roads etc. 4,3 %

The time schedule bad a final construction date of October 1st 1981.
Trial-operation of the cleaning plant bas been run since autumn 1981 and the first sewage water was treated March 8th 1982. The complete plant was ready according to schedule and was officially opened in June 1982.

CONCLUDING REMARKS

In a very tight time schedule and in rock conditions that are poor, especially compared to normal conditions in Norway, the rock reinforcement system chosen has been a success. In an integrated system consisting of 6 different steps exclusive works behind heading, every single step m u s t perform reliably ..
Robot shotcreting by wet-mix process with pump capacity of 200 litres per minute and a simple and effective rock bolt procedure, has given full stability control while working and permanently.
In this project, the normal situation for most underground job sites, the stability works being the project bottleneck, is totally turned around. This is the key to the excellent overall performance recorded at the Bjerkås job site.

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THE USE OF TUNNEL BORING MACHINES (TBM)

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A/S Høyer-EIlefsen is one of the oldest and largest construction and engineering companies in Norway. Annual turnover in 1981 was about 800 mill. Norwegian crowns (US $135 mill.), and a total of 1700 persons are now employed.
During the 90 years of construction, over 500 km of tunnels and shafts have been excavated, and from these 14 km with TBM. All excavation with TBM has be en done in co-operation with the Swiss company Bauunternehmung Murer AG of Erstfeld. This company has since 1976 been involved in a substantial amount of bored tunnels and shafts, mainly in Central Europe.
So far 4 different projects are completed in Norway, 3 horizontal tunnels and one 45° shaft. See table 1.

TABLE 1.

Plant	Tunnel length in m	Rock type	Rock properities σ_p (MPa)	DRI	Joint spacing	Average net. advance in m/h	Utilization in % excl. grout and support	TBM Type/Diam.
Kjøpsvik	340	limestone with zones of amphibolite and mica shist	100		wide	2,0		WIRTH TB II/3,32
							43	
	810	»	100			1,8		TB II/3,32
Sandvika - Lysaker	7600	limestone and shale	80-90	50	medium	2,3	55	WIRTH TB II/3,35
		10% diabase dykes	170-180	37	wide			TB II/3,35
Floskefonn	2800	phyllite quartz shist	80		medium	2,8) 1,2)	48	WIRTH TB I/2,53
		granitie gneiss	180		medium			
Sildvik Shaft 45° incl.	800	biotite quartsite	150	50	wide	1,1	52	WIRTH TB I/2,53

σ_p = Compressive strength in MPa
DRI = Drilling rate index
(Geological Institute, Technical University of Norway)

Tunnel Kjøpsvik
As the first tunnel boring job, A/S Høyer-Ellefsen was awarded the contract consisting of 1150 m transport tunnel for the Norwegian cement manufacturer NORCEM at their Kjøpsvik plant.
First an access tunnel of 340 m was bored. Then the machine was turned 90° and the remaining 810 m was executed in less than 11 weeks. Diameter was 3,32 m.
The rock consisted mainly of limestone and zones of amphibolite and micashist.
Compressive strength of the limestone was about 100 Mpa, and the strike of the bedding was at 90° to the access tunnel line, and parallel to the main tunnel.
As TBM a German WIRTH TB II H was used.

This is a machine with electric/hydraulic drive and variable speed of the cutter head. 19 double disc-cutters and two carbide insert centre cutters were used. This gave a spacing of 45 mm and a thrust per cutter of 12 -14 tons. Normal speed of the cutter head was 10 RPM.
For the mucking 1-2 Hågghaulers HT 1290 with rubber tyres, each of 9 m^3 capacity, was used. They had the advantage of not needing rails and a very simple back-up could be used. Weekly advance rates of 83 m and 74 m were achieved for the access and main tunnels respectively. (96 hour working week). First assembly took 2 weeks, and the turning of the machine was carried out in 4,5 weeks including the blasting of a new starting chamber .
The use of dump trucks with rubber tyres for the mucking caused some downtime due to the fact that the capacity of the dumper was only half a stroke of 0,8 m.
In addition there were some problems with the tyres of the dumper in the circular profile. The tunnel was finished without any form of permanent lining.

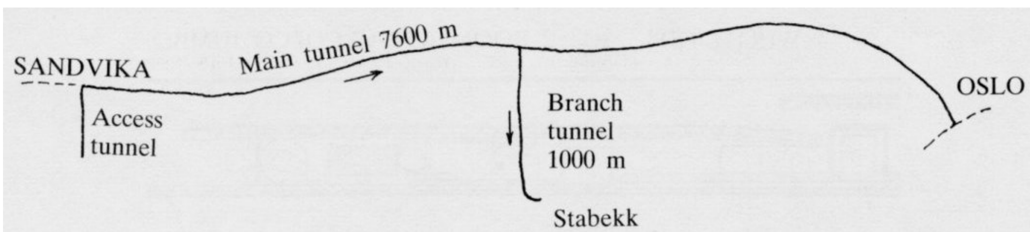

Fig. 1. Tunnel system within the contract. Arrows indicate boring direction.

Tunnel Sandvika -Lysaker
This tunnel is a part of the new sewer system for the city of Oslo. The contract consisted of a 7600 m long main tunnel and a branch tunnel of 1000 m.
The rock in this tunnel system was mainly limestone and shale. In the sedimentary rock formation igneous rock such as diabase appears as dykes and constitutes approx. 10% of the tunnel length.
The following typical figures show the compressive strength of the rock to be:

-For the sedimentary rock 80-90 Mpa
-For the igneous rock 170-180 Mpa

Also for this project a WIRTH TB II H was used. Diameter of 3,35 m and electric/hydraulic drive of the cutter head. During the first part of the boring period double disc-cutters were used together with carbide insert centre cutters.
This configuration was later changed to a combination of single and double disc-cutters, something that gave about 35% increase in net penetration rate.
The machine was equipped with a single back-up for the loading of 5 Mühlhauser 5,5 m^3 muck cars. This made it possible to load two strokes of 0,8 m in one set of cars.
To prevent ground water lowering in the area, the whole tunnel length was pre-grouted with cement and chemicals. For the drilling of grout holes, a two boom Atlas Copco jumbo was placed between the TBM and the back-up, to allow for 24 m long holes to be bored in front of the tunnel face.
The amount of grouting was much larger than expected. A total of 1,500,000 kg of cement and 350,000 l of chemicals was pumped into about 80,000 m of grout holes. This operation was quite time consuming, and took over 50% of available working time.

The TBM was assembled in the starting chamber just before Christmas 1977, and the breakthrough was celebrated at the end of August 1981.
Average net penetration rate of 2,3 m was achieved together with a utilization of 55%, grouting and support work excluded.
Rock bolts, shotcrete and liner plates had to be used occasionally in sections of bad rock stability. During crossing of a major fault zone, the TBM was nearly buried in loose material. To advance this zone both fibercrete and liner plates had to be used, and it to ok one month to pass this zone of 8 m.
As permanent lining 550 m^3 fibercrete and 3000 grouted rock bolts were used. This means that most of the tunnel was left unlined.

Tunnel Floskefonn
As part of the Eidfjord Hydroelectric Power plant a 4,6 km tunnel should be excavated high up in the mountains at the Floskefonn lake. A bored cross-section of 5 m^2 was needed and A/S Høyer-EIlefsen got this contract at the end of 1978.
Rock was mainly phyllite, but in the first 1,8 km and the last 0,8 km hard and massive granite gneiss was encountered. For this reason the 1 ,8 km was excavated conventionally and the TBM could start boring directly in phyllite.

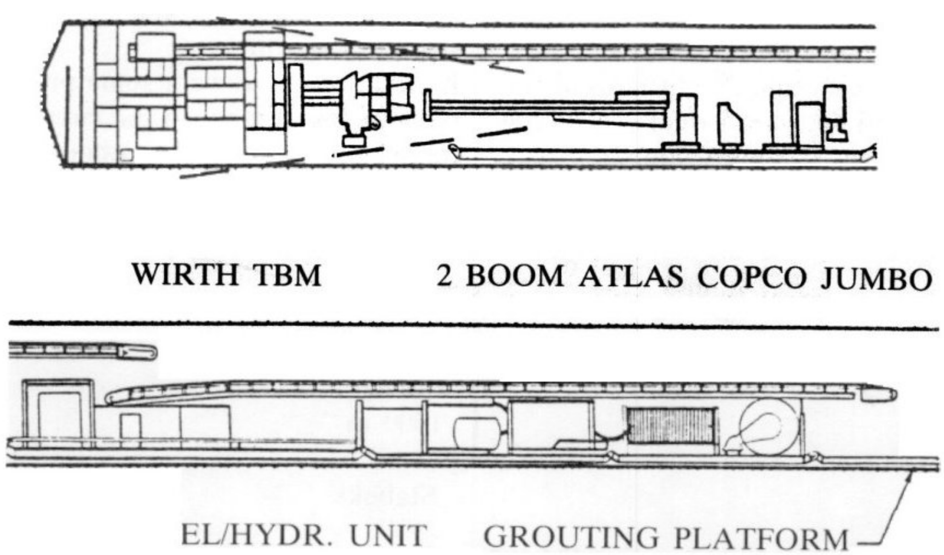

Fig. 2. TBM with grouting equipment incorporated.

A WIRTH TB I H was used with diameter 2,53 m together with a single track back up, so that 5 m^3 Mühlhauser muck cars could be loaded be hind the machine.
After 2 weeks of assembling, the TBM could start boring with an average rate of 127 m per 106 working hour week in the phyllite. Best week was 148 m.
Due to some mechanical problems with the TBM in a fault zone between the phyllite and the gneiss, and also to the hardness of the rock at the end, average weekly penetration over the whole 2,8 km was 98 m with a utilization of 48% . Also here the need for permanent lining was very limited. Only this fault zone had to be concreted.

Shaft Sildvik
The latest boring job was the 800 m long penstock at the Sildvik Hydroelectric Power plant in the north of Norway. This was the first shaft to be bored with TBM in Norway. inclination of the shaft was 45°.

The same machine as earlier described for Floskefonn was used together with an extra anti-slip device. No back-up was needed due to the fact that the muck runs by itself down this steep gradient.
For transport of people, spares and rock support material a small trolley pulled by a winch at the lower end of the shaft was used.
The rock consisted of massive biotite quartzite of medium to low bore-ability. In the access to the power house high rock pressure was experienced. This continued into the shaft area and a large amount of support work had to be carried out immediately behind the TBM. For this reason a specially developed rock bolting machine was built in behind the anti-slip device. In addition to rock bolts, also wire-mesh was used to a large extent to catch popping stones from the side of the shaft. By means of this bolting machine the support work was done without interrupting the. boring operation.
Average penetration for an 80 hour working week was 45 m, and the best week was 75 m. Due to the massive rock formation, high cutter load was necessary and therefore carbide insert disc-cutters was used.

Concluding remarks
In general, experience with the use of TBMs in A/S Høyer-Ellefsen is good. These 4 projects have been completed successfully both for the clients and the contractor . Therefore, the outlook for future boring jobs is regarded as positive.

TUNNEL BREAKTHROUGH BENEATH 85 m WATER AT RINGEDALSVATN

Sivilingeniør Øystein H. Stormo
Ingeniør Chr. F. Grøner A/S.

INTRODUCTION

To increase the usable storage volume of a water reservoir one can raise the water level making use of the existing topography and through construction of dams. Another approach to have more storage volume available is to pierce the bottom of an existing lake (or artificial reservoir) which is often an in ex pensive but technically a reasonably demanding solution.
In Norwegian hydroelectric developments such submerged inlets for water tunnels are often used and lake bottoms are pierced at water-depths up to 100 m. This solution was also chosen for the headrace tunnel of the high head Oksla Hydro Power plant, Tyssedal.

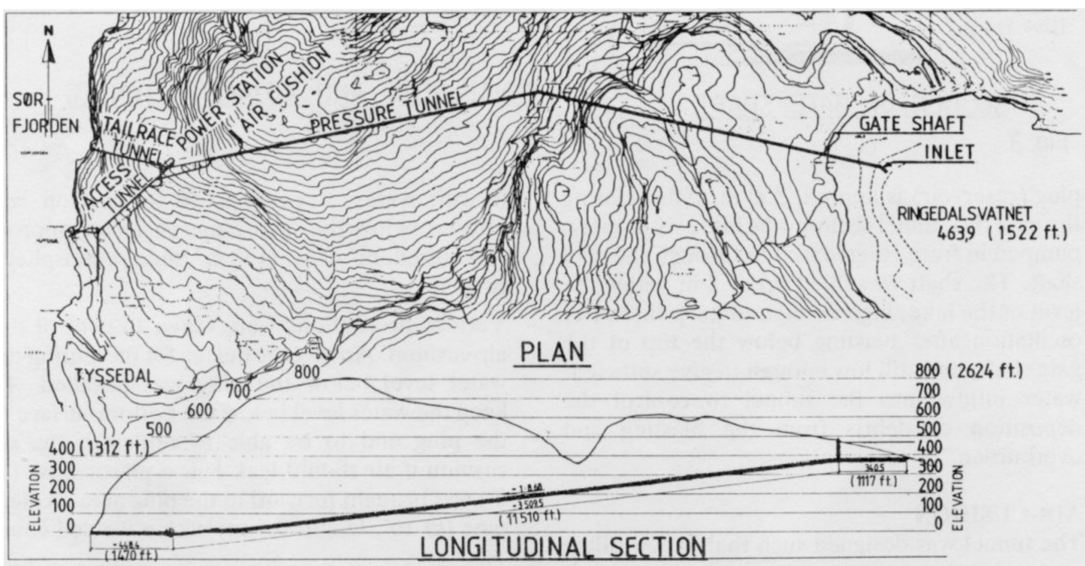

Fig. 1

OKSLA HYDRO POWER PLANT

The Oksla hydroelectric plant is owned by NVE-Statskraftverkene (Norwegian State Electricity Board) and was also built by NVE acting as its own contractor. The Oksla pl ant makes use of an existing reservoir, Ringedalsvatn. Ringedalsvatn was originally natural lake whose water level was raised by 35 m through construction of a concrete-masonry dam in the years 1912-18 as a part of the Tysso I hydroelectric plant.
As shown in Fig. 1 the Oksla development consists of a 3.9 km long inclined (approx. 1:9) and unlined headrace tunnel (35m2 cross section) connecting Ringedalsvatn with an underground power station, from which an outlet tunnel just below the sea level leads to Sørfjord. Instead. of a conventional surge shaft the air cushion solution is used. The total head is approx. 465 m. In order not to lose any production from the existing plants NVE decided to maintain the water level in Ringedalsvatn during

construction of the Oksla plant. This led to the somewhat extraordinary water depth of 85 m during the piercing operation.

"OPEN PIERCING"

In breaking through the last section of rock (or "plug") under the lake bottom, different methods can be applied. In this case the so-called "open piercing" was chosen by NVE- Statskraftverkene in co-operation with Vassdrags- & Havnelaboratoriet -V HL (Norwegian Technical University, River and Harbour Laboratory) and ingeniør Chr. P. Grøner A/S. In open piercing the main inlet gate (Figs. 2 and 3) is closed, but the gate shaft is open and both gate shaft and the tunnel section between gate and the plug are filled with water .

The length of the tunnel from the gate to the plug (reservoir) is approx. 350 m. After closing the gate this tunnel section was filled with water , pumped in from Ringedalsvatn through the gate shaft. The shaft was filled up to 8 m below the level of the lake, high enough to keep the water oscillation after blasting below the top of the gate shaft, and still low enough to give sufficient water inflow into the tunnel to control the deposition of debris from the blasting and overburden.

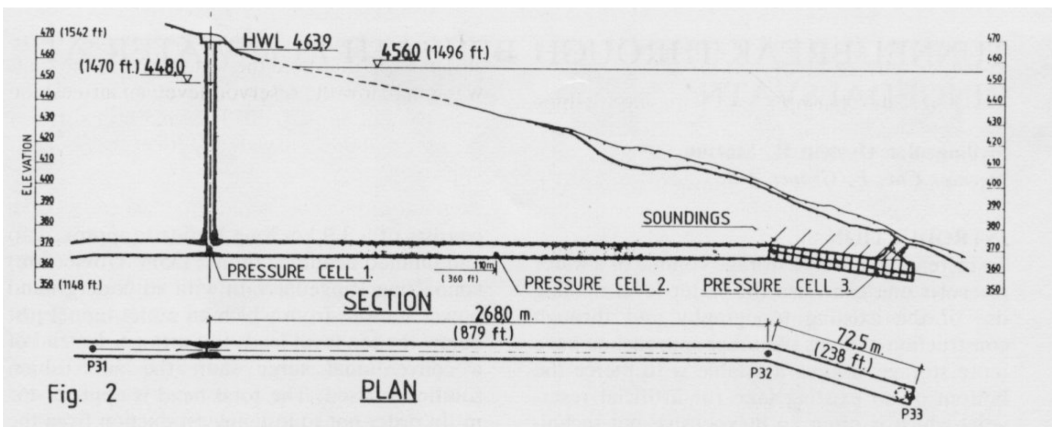

Fig. 2

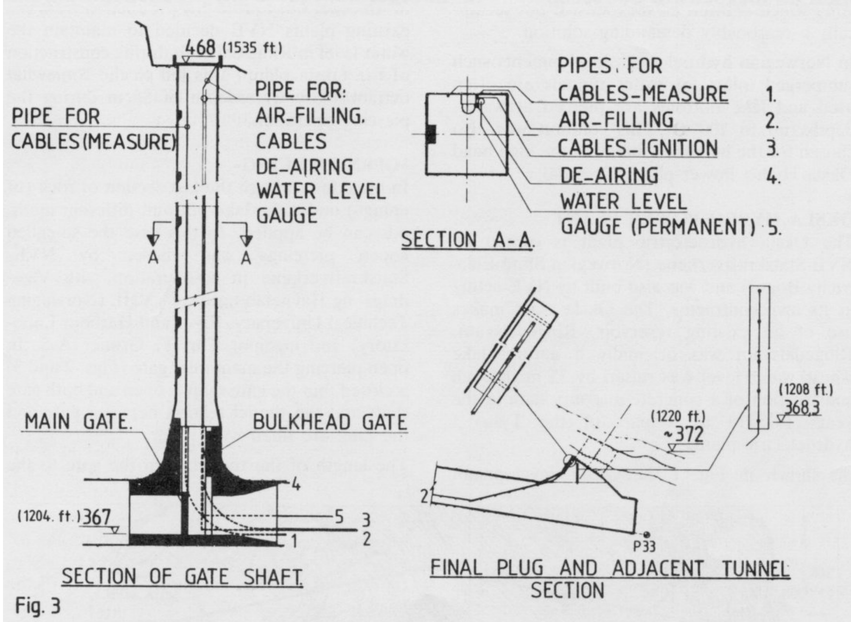

Fig. 3

AIR-CUSHION

The tunnel was designed such that, when filling it with water, a compressed air-cushion was created below the final plug, enclosing approx. 150 normal-m^3 of air (150 m^3 under atmospheric pressure).
Various precautions were taken to control this air-cushion. An electric device for measuring the water level below the plug was installed. To keep the water level below the bottom surface of the plug and to be able to replenish the air cushion if air should leak out, a plastic hose (Ø 2") was brought forward to the plug area. A steel pipe (Ø 10", length 0.5 m) with a welded cover was mounted at the end of the hose to give a smoother current of outflowing air .
To avoid the harmful and "uncontrollable)) pressure shock from the break through blast being transmitted through water against the main gate, it is of vital importance to create such an air-cushion and to make sure that it is maintained until the blast occurs. The air- cushion interrupts the water column and the pressure shock is thus dampened. Also, it has been theoretically established, that the pressure rise will be smaller, the larger the air-cushion.
As part of a research programme (of the River and Harbour Laboratory) measuring pressure transmission in water tunnels, three pressure gauges were installed. One at the gate, one close to the plug and one approx. half-way between the plug and the gate.

DRILLING AND LOADING OF THE PLUG

The drilling and loading plan for breaking through the plug was prepared by Dyno Konsulent A/S. Drilling was done such that the last (lakeward) 0,5 m of the plug was left undrilled, which led to drill hole lengths of approx. 3,5 m in centre and approx. 4,3 m in the periphery of the plug.
There were a total of 151 loaded holes (Ø 45 mm) in the plug and a total charge of 716 kg Extra Dynamite. The plug consisted of approx. 150 m^3 of rock, and the specific charge was thus 4,8 kg/m^3. The blasting caps were millisecond. German Delay caps, 2 caps in each hole, specially made to resist 100 m of water pressure for 72 hours.

WATER AND AIR FILLING OF TUNNEL AND GATE SHAFT

To fill the tunnel and gate shaft with the required approx. 15,000 m^3 of water, two pumps (each with 350 l/sec. capacity at a lift height of 20 m) were used. The pumps, hanging from a mobile crane, pumped the water from the lake and approx. 20 m down into the gate shaft through a Ø 10" ARMTEX pipe. The section over the gate shaft edge consisted of a 90° steel bend (Ø 10").
After the tunnel was filled up and water began to rise in the gate shaft, filling with water and air proceeded alternatingly to maintain the desired water level in the air-cushion under the plug.
After 15 hours of filling the pumps were stopped. At that time the water level in the shaft was 8 m below the reservoir level; an air-cushion consisting of approx. 1080 normal-m^3 existed at the plug and the lake water depth at the plug was approx. 85 m. Preparations had be en made to blast earlier than intended, if the air by any chance should seep from the air-cushion. In order to accommodate an unforeseen water rise if an early blast should become necessary, there were openings of 3 m2 total area in the concrete floor at the top of the gate shaft. Although eventually not needed for this purpose these openings were very useful for installations and for the blast preparation.

SUCCESSFUL PIERCING

The blasting of the plug took place successfully on 26th January, 1982. The pressure in the air-cushion before blasting was approx. 9 bars. During the blast a pressure increase of 39% (to 12,5 bars) was measured near the plug. At the main gate a pressure rise of 17% from 7,8 bars to approx. 9,1 bars was observed.
The water in the gate shaft oscillated to a level 5,8 m above the water level in the reservoir which corresponds to a pressure rise at the main gate of the same magnitude as that caused by the blast (as expected). The water level rise in the gate shaft took 26 sec. , which agrees well with the predicted value of 29 sec.
The 5,8 m by which the gate shaft water level exceeded the reservoir level is 72,5% of the 8 m water level difference between shaft and reservoir that existed before the blast. This result corresponds to previous experience with this type of piercing and to the computations. One can assume with reasonable certainty that the water in the gate shaft will reach a level above the reservoir corresponding to 70 -90% of the level difference before the blast, depending on losses due to friction and turbulence.
In conclusion, the piercing operation can be considered a great success; also, all measurements and controls were performed to the satisfaction of all parties.

ACKNOWLEDGEMENT

The author expresses his gratitude to Prof. H. E Einstein of M.I.T. for reviewing the paper .

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EXPERIENCES WITH PREGROUTING IN SEWAGE TUNNELS IN THE OSLO AREA

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SUMMARY
For the municipalities of Oslo, Bærum and Asker a co mm on sewerage system has been constructed. The system consists of 42 km of tunnels of which 38 km are constructed by tunnel boring machines. The main part of the system was put into operation in May -82.
The deposits in the Oslo area consist mainly of soft clays. A lowering of the ground water table will introduce settlements in the clays with possible damage to buildings as the result. Consequently, to prevent such damage, tunnels have to be performed almost completely water tight. For the described tunnel system a method based on pregrouting was chosen.
This paper describes the pregrouting method and is summing up the main experiences and results from the grouting works.

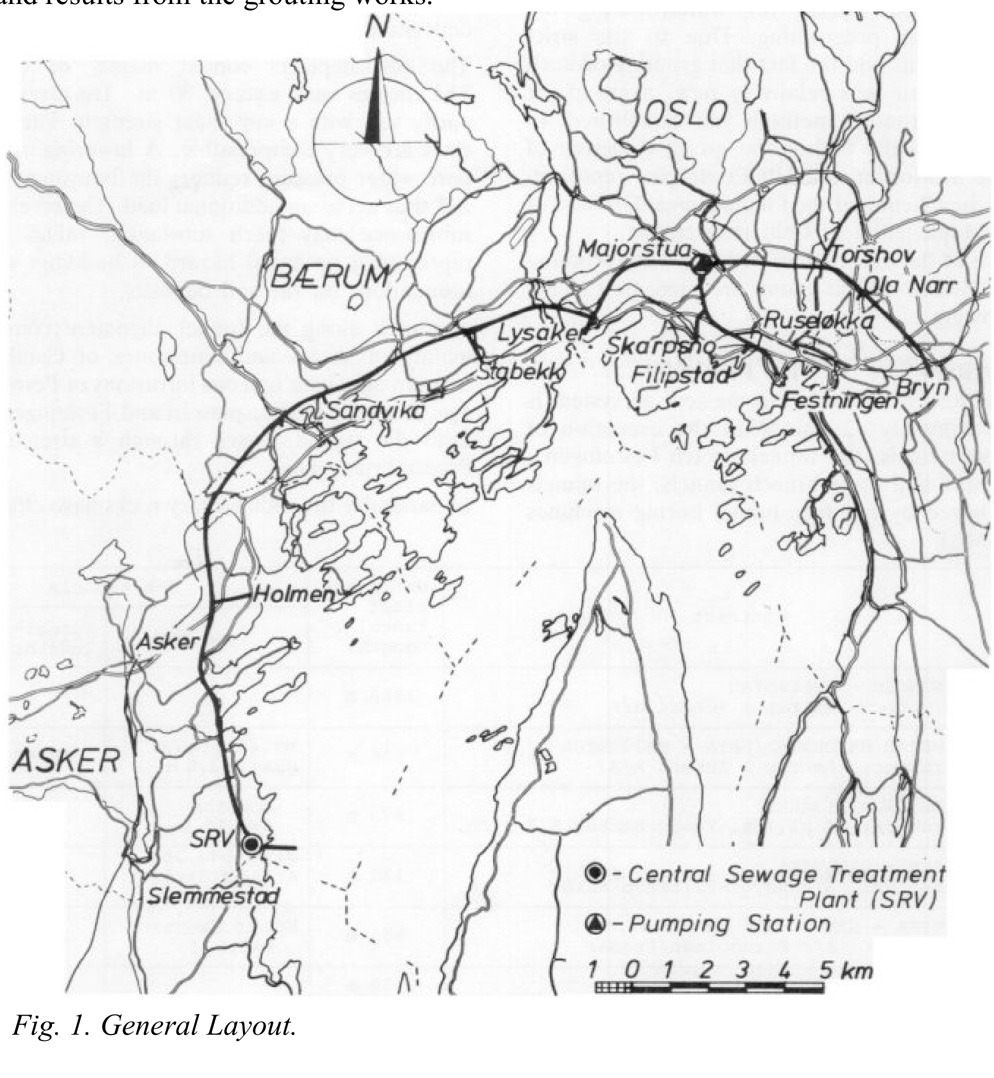

Fig. 1. General Layout.

INTRODUCTION

For the municipalities of Oslo, Bærum and Asker, a common tunnel system for sewage is being constructed. The system will collect sewage and conduct it to a common Sewage Treatment Plant at Slemmestad (Bjerkås). The overall layout is shown in Fig. 1.

A well known problem with tunnelling in the Oslo area is the lowering of pore water pressure in the marine soft clay deposits following water leakages into the tunnel. The lowering of pore water pressure leads to settlement of the clay which in turn may cause severe damage to buildings. For the actual tunnel system requirements for waterproofing had to be made. The required degree of watertightness varied along the tunnels. In Central Oslo with heavily built up areas the requirements were very strict. The method chosen for waterproofing the tunnels was pregrouting. Due to the strict requirements and the fact that grouting on such a large scale was relatively new, a group for study of grouting methods was established. In addition to the author the group consisted of Arne Storjordet and Ulf Fredriksen representing the Client and the Geotechnical Division of the Municipality of Oslo respectively.

Some of the experiences with the waterproofing work learnt by this group are presented in the following.

Contract	Drill and blast tunnels. Lengths	TBM tunnels	
		Type TBM	Tunnel-lengths
1 FESTNINGEN - FILIPSTAD Contractor: Astrup & Aubert A/S	1360 m		
2 FILIPSTAD MAJORSTUA/BRYN - MAJORSTUA Contractor: Astrup & Aubert A/S	510 m	No. of 2 Bouygues D=3,0 m	10770 m
3 MAJORSTUA - LYSAKER Contractor: Dipl.ing. Kaare Backer A/S & Co.	670 m	Robbins D=3,15 m	4180 m
4 LYSAKER - SANDVIKA Contractor: A/F Høyer-Ellefsen/Murer	230 m	Wirth D=3,35 m Atlas Midifaser 2,1x3,14 m	8510 m 1000 m
5 SANDVIKA - SRV Contractor: A/F Furuholmen/Prader	800 m	No. of 2 Robbins D=3,5 m	14200 m
Total length of tunnels	3570 m		37660 m

Fig. 2. Tunneling Contracts

DESCRIPTION OF THE PROJECT

Total length of tunnels in the sewage system is approximately 42 km. With the exception of access tunnels, the tunnel stretch Festningen - Filipstad and some branch tunnels, the tunnels are bored by full face tunnel boring machines (TBMs).

The tunnel system is divided into five main contracts. The first contract, Majorstua - Lysaker, was completed in 1976. The rest of the tunnel length from Torshov to Slemmestad was put into operation in May 1982, at the completion of the Central Sewage Treatment Plant (SRV). The remaining lengths of tunnel in Oslo will be made operational in steps with the final completion expected in 1984.

The table of Fig. 2 shows the tunnel lengths attached to each contract.

The experiences described in this article are mainly concerned with the last four contracts to be completed, i.e. contracts 1,2,4 and 5.

GEOLOGY

To define the required degree of watertightness it is necessary to know the rock quality and the thickness, composition and quality of the soil deposits along the tunnel alignment.

The soil deposits consist mainly of clays. The thickness may exceed 30 m. The clays are partly soft with a low shear strength. The soft clays are very compressible. A lowering of the pore water pressure reduces the buoyancy and will thus act as an additional load. The resulting subsidence may reach substantial values and represent a potential hazard to buildings with foundations on the soil deposits.

The rock along, the tunnel alignment consists mainly of shales and limestones of Cambro- Silurian age, with igneous intrusions of Permian age. Between Rådhusplassen and Festningen in Oslo the tunnel passes through a stretch of Precambrian gneiss.

In particular the sedimentary rocks have closely spaced jointing that make the grouting work demanding.

THE GROUTING PROCEDURE

W hen tendering for the first contract, Majorstua -Lysaker, in 1973, the tunnel was supposed to be waterproofed by pregrouting. A main basis for this was the good results achieved with the Holtekil tunnel in the same area (1971). The TBM used in this first contract did not, however , have sufficiently good access to the tunnel face for the achievement of pregrouting. An extensive post grouting was carried out, but it could not prevent damages to buildings from subsidence.

W hen tendering for the three remaining con- tracts in the autumn 1976 and spring 1977, pregrouting was still chosen as the principle for waterproofing of the tunnels. The requirements for fitting the TBMs for pregrouting equipment were strongly underlined.

The Tenders received this time were based on equipment much more suitable to a pregrouting procedure. Further adjustments of equipment were also made by the Contractors after the Contracts were let.

A brief description of the procedures for
pregrouting chosen by the different Contractors is lined up below:

> **Astrup & Aubert A/S:**
> TBM: Bouygues. The machine is withdrawn approx. 6 m. A hydraulic drilling machine is mounted on the arms of the TBM. The access to the tunnel face is relatively easy.
>
> **A/F Høyer-EIlefsen/Murer:**
> TBM: Wirth. A rig with two hydraulic drilling machines is placed immediately be hind the TBM. The drilling cut is made behind the back grippers, approx. 6 m behind the tunnel face.
> TBM: Atlas Midi. The machine is withdrawn and a drilling machine is mounted on the cutter head.
>
> **A/F Furuholmen/Prader:**
> TBM: Robbins. One hydraulic drilling machine is mounted on each side of the TBM. Drilling cut is made immediately behind the TBM head, approx. 3 m from the tunnel face.

The procedure for pregrouting may be described as follows:

- A predetermined number of probe holes are drilled ahead of the tunnel face.
- A water loss test is carried out by water pressurizing of the holes.
- The water losses are expressed in terms. of Lugeon (L) .The measurements give an indication of the watertightness of the rock and are taken into account for determination on the type of grout material that shall be used.
- The probe holes are grouted until a predetermined counter pressure is achieved.
- If leakages as measured in the probe holes are greater than acceptable, a new round of control holes between the first ones is drilled. Water losses are measured and grouting is carried out according to the same criteria as for the probe holes.
- The procedure is repeated until the criteria for watertightness are fulfilled.

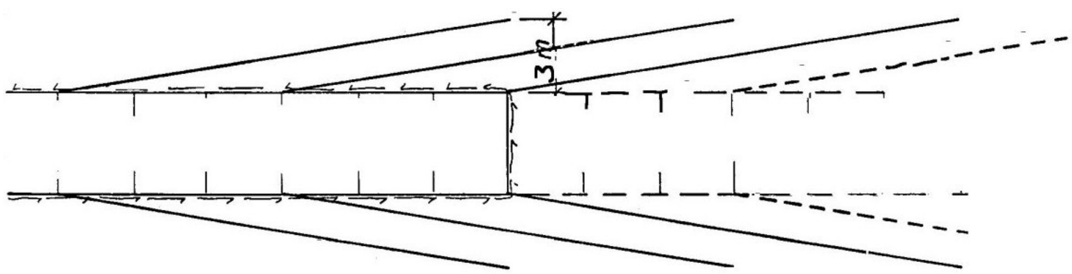

Fig. 3. Drillholes for Pregrouting. Drill and Blast Tunnels.

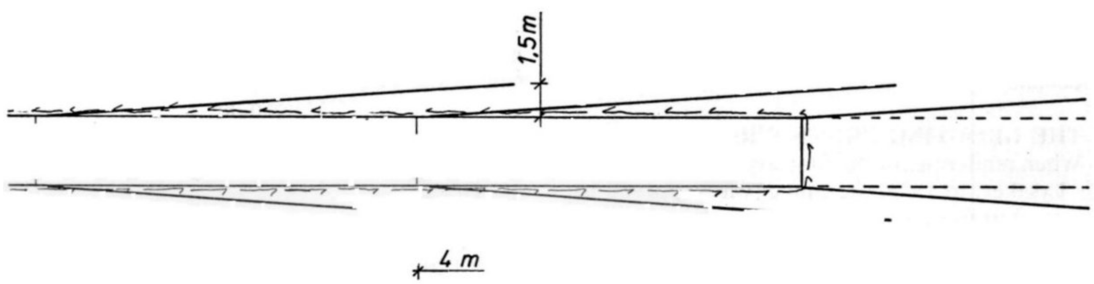

Fig. 4. Drillholes for Pregrouting. TBM Tunnels.

The Client decides on the number of boreholes and the number of grouting rounds. Compensation is made for the quantities spent. The construction time is adjusted according to special rules related to the quantities spent.
The principle for setting out the holes for pregrouting is shown in Figs. 3 and 4.
As may be seen from the figures, the holes are placed closer to the tunnel wall with the full face boring than with conventional drill and blast method. The cracks induced by blasting make it necessary to place the grout curtain further away from the surface of the drill and blast tunnel. The main problem with the TBMs has be en placing the drill holes in sufficient numbers and in the right direction. Due to the narrow spaces left unoccupied by the TBM parts, it was not possible to cover the desired area at the tunnel face with the Robbins and Wirth machines. By inclining the holes, however, this was solved in an acceptable manner. With the drilling cuts made on the tunnel surface behind the cutter head it was very difficult, or say impossible, to drill with the small angle to the tunnel axis as described. The extra length of the holes due to the setting out some distance behind the face also contributed to an inaccuracy.

The Mining Institute of the Norwegian Technical University has carried out single controls with the inaccuracy of drilling at all main tunnel faces. The controls show that the deviation from the described direction in most cases are too big. In addition to the deviation of the direction of setting out, the holes in many cases have a change of direction along their length. The borehole deviation registered in the drill and blast tunnels was generally less significant, but even here the deviation some places was in excess. From the Client's point of view the accuracy of the drillings was not satisfactory.
Normally the full face bored tunnels are driven with a single grout curtain. The usual interval length between each round of pregrouting is approximately 20 m. In some stretches with difficult conditions along the tunnels in Central Oslo a double curtain is, however, carried out. The boring stage between each round of grouting will thus be come shorter. Tunnelling with double grout curtain accomplishes improved watertightness, but driving speed and costs of tunnelling will naturally be seriously influenced.

REQUIREMENTS FOR WATERTIGHTNESS

Requirements for watertightness vary along the tunnel alignment due to both variation in soil deposits and type of buildings.
There has be en no basis for attaching the requirement for tightness to the amount of leakage water allowed in the tunnel because it would involve comprehensive calculation of the ground water balance in each ground water basin. Such calculations would in any case be attached with great uncertainties. Later w hen more data of experience is available, the criteria for tightness based on such calculations may be favourable.
In the case of the sewage tunnels requirements for watertightness are attached to the maximum allowable water loss in the probe and control holes. These requirements are in turn related to the behaviour or pore pressure measurements which are made in drill holes from the ground surface. The variation in ground water level indicates also to w hat extent post-grouting will have to be carried out. Careful surveillance of the pore pressure level is thus regarded to be an important activity throughout the construction period.

GROUT MATERIALS

The grout materials may be divided into two main groups:

-Suspensions

-Chemical solutions

The suspensions contain particles and will therefore have poorer ability of intrusion compared with the chemical solutions. The two groups of materials are complementary to one another. With large water losses a cement suspension is used while the chemical solutions are used with smaller losses.
The criteria for changing from cement to chemicals have be en varying from contract to contract, but on average the limit has been in the area of 1-3 Lugeon.
As the number and the size of the joints varies, water loss measurement of the entire length of the probe hole does not give sufficient know- ledge of the leakage distribution along the hole to choose ideally the right grout material. Specifications for several packer positions along the probe holes to analyse the leakage distribution were made in the contract. In practice, however, this procedure has proved too time consuming and thus too costly. The inconvenience of measuring the water losses for the entire hole length only is therefore accepted.

Suspensions
Cement represents the major part of the suspensions used. Mainly Rapid Portland is used. In the area at Festningen where the rock encountered consists of some alum shale, sulphate resistant cement was also made use of. As an admixture to the cement 2% Bentonite was specified.
In the contract Sandvika -Lysaker considerable amounts of the product Cernsil were used in connection with the cement pregrouting. This material is added in the end phase of the grouting procedure and consists of cement, silica, bentonite and rapid hardening admixtures. The purpose of using the se materials is to reduce the hardening time and at the same time make the grout more stable by reducing the amount of water.
Experiments with the use of specially fine ground cement delivered by Norcem were also carried out in the tunnel Sandvika -Lysaker . Maximum grain size of the cement (D90) was approximately only half that of the Rapid Cement. Later Norcem has been considering wind sieving as a more suitable method of obtaining a fine grained cement. The plans are, however, shelved for the time being.

Chemical solutions
The resin grout Geoseal MO 4/8 was used for the contract Sandvika -Slemmestad. For a short period the product Lignin (Ligno- Sulphonate) was used in the contracts Sandvika - Lysaker and central Oslo. Another product, AM 9 (Amino-methyl acryl) was also used to some extent. AM 9 and Lignin, however, were later considered by the Medical Authorities to entail a health hazard. The necessary protection measures against the risks would be so extensive, that the use of the se materials in fact proved prohibitive to the Contractors. As a substitute different solutions of silicates were used in the se contracts. The solutions under the trade names of *Stabgel FR* and *Stabilodur F* are those mainly made use of. One drawback with the silicates is the expelling of water during formation of gels. The phenomenon is called syneresis. One has, however, achieved a mixing ratio giving only minor syneresis, i.e. 2-3% .

Tunnelling Method	Total length of tunnels (m)	Excavated distance (m) per. 15.6.82	Total quantities carried out for pre-grouting per 15.6.82 Boreholes (m)	Cement (kg)	Chemicals (l)
Drill and blast	3.010	3.010	103.850	939.300	554.400
Full face bored	33.000	30.330	387.370	4.434.400	1.880.300
Total*	41.400	33.340	491.220	5.373.700	2.434.700

Contract 3, Majorstua - Lysaker, excluded.

Fig. 5.Total Quantities of Tunnel Lenght and Amounts of Pregrouting.

POST GROUTING
The required tightness is not always achieved by the pregrouting of the tunnels. It is therefore necessary to carry out some post grouting. Similar with the experience of other tunnel projects it is rather difficult to achieve good results with this method. With great efforts and, unfortunately, at relatively high costs some good results are obtained. Both chemicals and cement have been used for post-grouting.
It should also be mentioned that some 350 m of continuous concrete lining has been placed in tunnel Majorstua -Torshov. It was here expected that the requirements for watertightness could hardly be met with post grouting alone.

GROUTING EQUIPMENT

During the past few years the re has been a development in the field of equipment. All contractors in this connection have applied equipment for grouting of several holes simultaneously. Such equipment gives less possibilities for control with the grouting of each hole. Partly this may be dealt with by using measuring devices for each hole. Though the contracts were formulated with single hole grouting in mind, it was found reasonable to apply equipment for simultaneous grouting of several holes due to the enormous amounts of grout involved with this project.

Tunnels	Excavated length per 15.6.82 (m)	Pregrouting. Average quantities per m length of tunnel		
		Boreholes (m)	Cement (kg)	Chemicals (l)
Oslo (Contr. 1&2)				
- TBM Tunnels	7.500	28,7	215	210
- Drill & Blast Tunnels	1.860	51,0	476	287
Lysaker - Sandvika (Contr. 4)				
- TBM Tunnels	8.530	8,8	183	25
Sandvika - SRV (Contr. 5)				
- TBM Tunnels	14.280	6,7	88	6

Fig. 6. Average Quantities per metre Lenght of Tunnels.

QUANTITIES OF GROUT USED

To illustrate the amount of grouting work undertaken the tables of Figs. 5 and 6 give some data on the quantities of pregrouting performed.

COSTS

To give an idea of the costs involved for pregrouting these are shown in comparison with the cost of tunnel boring and rock support, see Fig. 7. Percentage cost of support and reinforcement and pregrouting compared to the tunnel boring are shown in parenthesis. All costs are in NOK in contract prices, not adjusted for escalation. Costs of rigging, administration etc. are not included.

RESULTS

To register the amounts of water leakage into the tunnels, small dams for measurements are constructed. In areas with strict requirements the final results for TBM tunnels will be in the range of 3-5 litres per min. per 100 m. The best results from drill and blast tunnels are 1-2 litres per min. per 100 m.

As mentioned before, leakage measurements are not applied directly to decide whether sufficient watertightness is achieved. Measurements taken at different times show the development in water leakage. Due to difficult conditions at the measuring stations the readings are, however, not always 100 per cent reliable.

Pore pressure measurements along the tunnel alignments show mainly satisfactory ground water levels, indicating acceptable conditions of leakage into the tunnels. At some limited locations the pore water pressure has decreased below an acceptable level. In such cases it has be en necessary to put into operation artificial groundwater re charge wells to compensate the leakages. The post grouting or in one case lining of the tunnel has so far made the use of permanent recharge wells redundant.

The extensive programme of pore pressure measurements has in some cases revealed that decrease in the pore water pressure may be caused by activities with no connection to the sewage tunnel system.

Contract	Price level	Tunnelling method	Tunnel lenghts (m)	Average tunnelling costs (NOK)	Average costs compared with tunnelling costs	
					Rock support	Pregrouting
2 Oslo City	Autumn 76	TBM	7.500	3.155	135 (4%)	4.500 (143%)
4 Lysaker - Sandvika	Autumn 76	"	7.530	3.486	125 (4%)	1.615 (46%)
5 Sandvika - SRV	Spring 77	"	14.205	3.144	58 (2%)	845 (35%)
1 Festningen - Filipstad	Autumn 78	D&B	1.050	3.675	1.685 (45%)	12.144 (330%)

Fig. 7. Cost of Pregrouting and Rock Support Work Compared with Excavation. Rigging Costs excluded.

CONCLUDING REMARKS

The procedure of the works has so far shown that the grouting mainly has been in compliance with the requirements for water tightness of tunnels and caverns. Up to now there has been no need for permanent pore water recharge wells. The amount of grouting in the contracts Slemmestad- Sandvika and Sandvika -Lysaker has been approximately as expected or somewhat higher. In Oslo the quantities of grouting material are substantially higher than specified in the contract. The extensive grouting should, however, be considered in view of the extremely strict requirements that have to be specified in densely built up areas.

When tunnelling through areas sensitive to subsidence and requiring a high degree of watertightness it is important to be sufficiently careful and thorough in the grouting work. An unsuccessful grouting may result in lowering the ground water level and the high costs involved are then more or less wasted.

Pregrouting may vary a lot and be different for each new tunnel face. Considerations with respect to number of boreholes, choice of material etc. will have to be made continuously. Grouting work demands far more supervision than usually applies to other kinds of work, a fact that both Contractor and Client must bear in mind. A crucial factor is the motivation of all parties to recognize and perform the important task of making a waterproof tunnel.

The work with the sewage tunnels confirms the experience made with other tunnels, that pregrouting is far more efficient than post grouting. Post grouting and other types of post tightening of tunnels will anyhow have the drawback of ground water lowering with the attached subsidence damages being experienced before the water proofing is put into effect.

In the above, experiences with water tightening of sewage tunnels are reported. Similar grouting procedures are also carried out in connection with Majorstua Sewage Pumping Station and caverns for the purpose of Contractor's rig areas. The pregrouting of the larger cross sectional areas has also been carried out satisfactorily.

TUNNELLING WITH HYDRAULIC DRILLS AT ÅBJØRA HYDRO ELECTRIC PLANT IN NORWAY *

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SUMMARY
Tunnelling with hydraulic drills has been a success in Norway. In the fall of 1976 excavation of tunnels for the development of the Åbjøra scheme was started. Two and a half years later a total of 25 kilometers of tunnels were completed. During the most busy period 5 rigs with hydraulic drills were in operation.

GENERAL INFORMATION .
Åbjøra Hydro Electric Plant is located in the central part of Norway (Fig. l) .
The scheme covers a mountainous area between the Tosen Fiord and the valley of Namdalen. Water from the area of precipitation is transferred through 5 tunnels to the main reservoir. From here the water is conducted via a 10 km long, unlined supply tunnel down to the Kolsvik power station. The power station is situated underground with a short tailrace tunnel out in the fjord. Average head of water is 507 m.
The Åbjøra plant is jointly owned by Helgeland Kraftlag A/S, 8650 Mosjøen and Nord-Trøndelag Elektrisitetsverk, 7700 Steinkjer. The Civil Engineering Division of the latter company has acted as Consulting Engineers for the scheme.
General contractors for the whole scheme has been Ing. F. Selmer A/S.

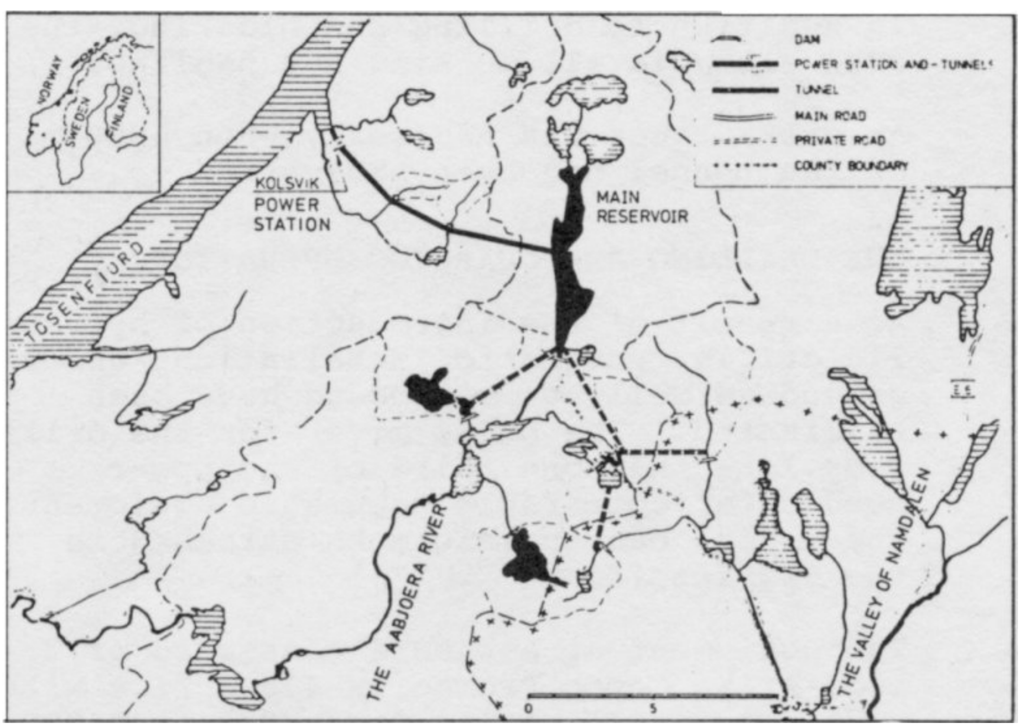


Fig. 1. Survey of the Aabjoera Hydro Electric Plant.

GEOLOGY- STABILITY PROBLEMS.

The rocks of the region are of Cambro-Silurian age and may conveniently be divided into two main groups: In the western part one finds highly metamorphic rocks such as various gneisses and amphibolites. The eastern part is dominated by coarse-grained igneous rocks like granites, diorites and monzonites. Point load strength indexes for the rocks vary from 3 to 9 MPa measured on water saturated cores of 32 mm. They may thus be described as being of very high strength[l]. Drilling as well as blasting properties of the rocks along the tunnels have been normal according to Norwegian standards [2 and 3]. Only in limited parts of the total 25 km of tunnels have stability problems been experienced. Approximately 5000 rock bolts have been installed, mainly to pre- vent spalling in highly stressed parts of the tunnels. In parts with weak rocks a total of 450 m of the tunnels has been lined with shotcrete. The most severe problems have been experienced where the tunnels have met faults and crushing zones containing swelling clay. Such problems have been solved by the use of approximately 200 m non-reinforced concrete lining. As usual in Norway the linings have been cast in place and have a minimum thickness of 30 cm.

ORGANIZATION OF THE WORK.

Norwegian tunnelling is traditionally autonomous. Payment for the workers is according to the premium system. Initially, there was no experience available regarding the use of hydraulic drill rigs. The initial stage thus presented a great challenge to the engineering staff as well as to the workers.

From January l. 1978, according to a new act of law night work underground is no longer allowed in Norway. Consequently, it was necessary to reduce from 15 shifts per week to 10 shifts. (All shifts are 7,5 hours).

The work is organized along the same lines in all tunnels. Below is used the western part of the supply tunnel as an example, totalling a length of 6510 m from point of attack to break through. The area is 23,3 m^2. After a period of 4 months the crew had gained sufficient experience to master the equipment, and two blasting rounds per shift were easily obtained.

The number of workers at two shifts were:

Face crew		2 x6 men=	12 men
Road work	day shift	1x1 man =	1 man
Electrician	day shift	1x1 man =	1 man
Foreman	day shift	1x1 man =	1 man
Total			15 men

In addition to drilling and blasting, the crew also did all mucking and hauling. An extra truck was necessary when 5000 m of the tunnel had be en excavated.

THE DRILLING AND CHARGING OPERATION.

As a result of the introduction of hydraulic drills, pneumatic installations under-ground as well as over ground have been eliminated. The prime mover for the drill uses less than one third of the power needed for comparable pneumatic equipment. The motors can therefore be attached to the rig itself.

The equipment at Åbjøra consisted of 3- boom Atlas Copco Promec TH-470-3 rigs with hydraulic drills Atlas Copco COP 1038 HO and service platform (Fig. 2) . When an operation cycles starts, the rig leaves a niche 300- 500 m behind the face.

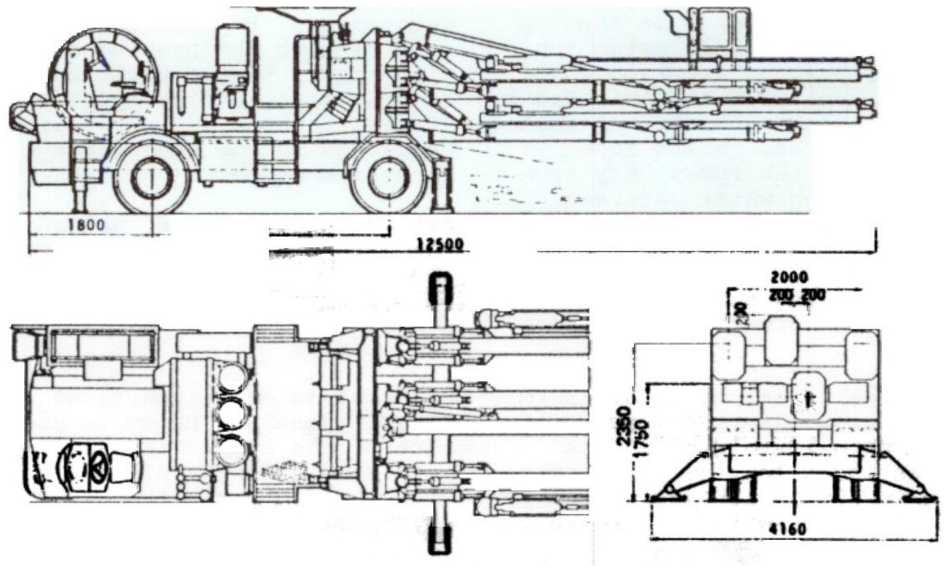
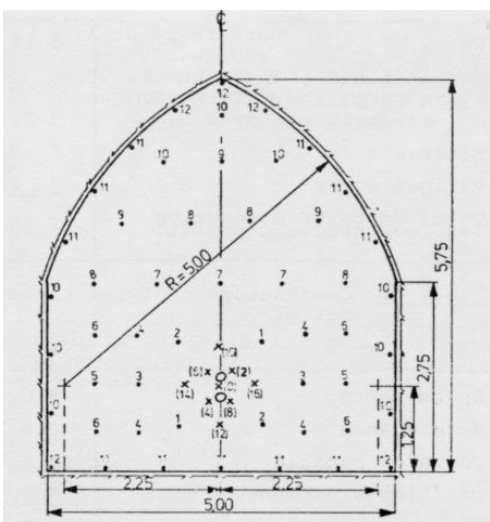


Fig. 2. Atlas Copco Promec TH-470-3

A drum mounted on the rig, contains 125 m cable of 500 V, 3 x 75 Cu. The cable is attached to the voltage terminal when the rig passes by. Simultaneously, 2 men mark the tunnel face to prepare for a new round. After experimenting with varying length of drill rods, 14 ft rods with 45 mm cross bits were chosen. During the drilling operation, the shot firer and the maintenance man are drilling. They are assisted by one man who changes drill bits and does the charging.



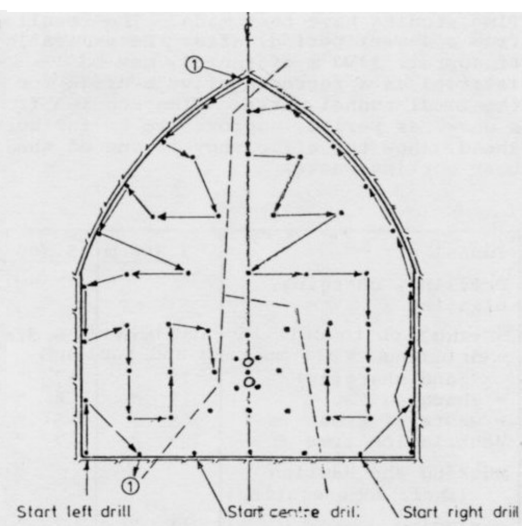


Fig. 3. Drilling pattern.

o	uncharged cut hole
x	charged cut hole
•	other charged hole
( )	short delay blasting cap

Fig 4. Sequence of drilling.

Fig. 3 shows the pattern of drilling with parallel-hole cut and two large holes of 4 in. diameter. The sequence of drilling is shown in Fig. 4. Average depth drilled is 3,80 m, and the advance per round is 3,60 m.

For one round 295 kg of explosives is used, including 200 kg of ANFO(ammonium nitrate fuel oil). Specific drilling, including overbreak, is about 2,5 drilled m per m^3 rock, and specific charging, including overbreak, is about 3,5 kg per cu m. The figures are 10% to 20% higher than stipulated for normal resistance to blasting [3]. However, good fragmentation of the rock and easy mucking are obtained.
The fixed installations in the tunnel consist of electric high voltage cables for 11 kV (3 x 150 Al) installed by bolts and wires along the tunnel wall. Dry-type transformers, II kV/500 V and 11 kV/200 V, are placed in niches with a maximum distance to the tunnel face of 750 m. Ventilation is based on Korfmann GAL 14 fans and 1,40 m flexible ducting. Centrifugal pumps, .type Vogel 42 DGE supply the rig with water. All works in the tunnel concerning ventilation, water and electricity are carried out by the three men from the face crew who are not occupied at the face during the drilling operation.

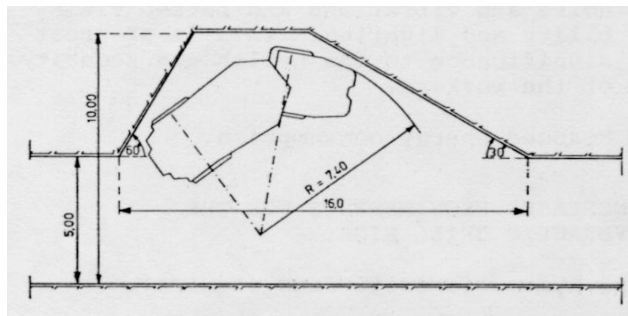


Fig. 5. Plan, loading recess.

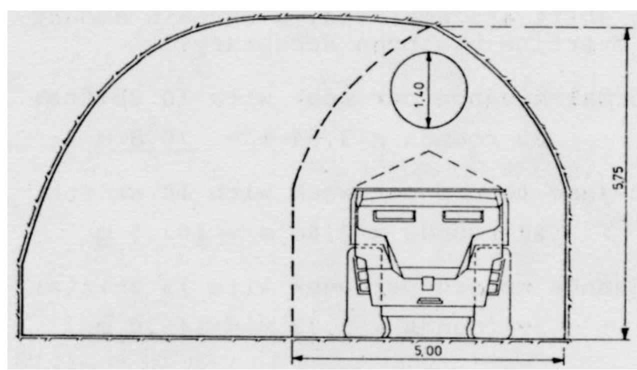


Fig.6. Section, loading recess.

THE MUCKING AND HAULING OPERATION.

Ventilation period after blasting is combined with meal breaks. A special barrack for this purpose is placed in one of the niches and is supplied with fresh air from the ventilation ducting. The equipment for mucking and hauling is as follows: Two CAT 980 B, 26 tons, one in operation at the face and one as reserve. Five Kiruna trucks K-250, 22,9 tons, with a hauling capacity of 35 tons. Road grader CAT 14 E for road maintenance.
During the mucking and hauling period when four trucks are in action, the face crew of 6 men is organized as follows: The maintenance man is tied up with the drilling rig, four men do the hauling and scaling. When the fist truck is loaded, it is parked in a recess or niche, and the driver starts scaling of the roof from the muck pile. Three men drive two truckloads each before the mucking and hauling operation is completed. The remaining scaling is preformed by two men from the mucking machine. The mucking is done by the load-and carry method, with recesses at intervals of 110-130m (Fig 5, Fig 6)

CAPACITY.

Time studies have been made. The results from a 4-week period, after the excavation of approx. 3300 m of tunnel, may be regarded as a representative average for the total tunnel works. Time studies from a one-week period, approx. two km further ahead, show the efficiency in one of the best working weeks.

Tunnel at:	3.300 m	5.300 m
Drilling, charging, blasting		
- rigging	11 min.	9 min.
- drilling (and charging)	75 "	67 "
- charging	15 "	8 "
- waste of time	9 "	5 "
Ventilation time	27 "	17 "
Mucking and hauling (incl. some scaling)		
- rigging	11 "	7 "
- mucking and hauling	51 "	45 "
- waste of time	7 "	7 "
Scaling	24 "	15 "
	230 min.	175 min.

Table 1.
Time studies. Results in a week with normal advance (tunnel at 3 300m), and results in a week with advance record (tunnel at 5 300 m).

By combining the meal breaks with the ventilation time 8-hours-shifts are easily obtained. A normal shift will give two rounds. In the cases when three rounds per shift are achieved, a certain amount of overtime has been necessary.

Normal advance per week with 10 shifts: 20 rounds x 3,54 m = 70,8 m
Advance record per week with 10 shifts: 28 rounds x 3,66 m = 102,5 m
Advance record per week with 15 shifts: 39 rounds x ~,72 m = 145,0 m

Average advance in 1978 was 70 m per week (10 shifts) including blasting of recesses and the installing or rock bolts. The blasting of the recesses corresponds to 5 m of tunnel per week. The result per shift is about 70% higher than scheduled. The tunnels were completed earlier than originally planned, even though the number of shifts available was reduced by one third from January l. 1978.
Drilling, blasting, mucking, hauling, scaling and work with the fixed installations were carried out with a total of 7,7 man-hours per tunnel metre. Payment to the tunnel workers has been approx. $ 13 per hour.

Table 2.
Contractors cost pr tunnel metre.

Wages	$ 143,00
Costs of machine, incl. of spare parts and maintenance, ex. electric energy	" 217,00
Materials	" 171,00
Various costs	" 25,00
Total costs of production (ex. overhead and profit)	$ 556,00

Table 3.
Cost of rig as calculated pr drilled metre.

Spare parts	$ 0,60
Maintenance	" 0,20
Costs of capital	" 0,70
Drilling equipment	" 0,50
Total costs	$ 2,00

ADVANTAGES OF HYDRAULIC DRILL RIGS.

The advantages of the hydraulic drill rigs may be summarized as follows:

- Increased drilling rate of up to 100% ..compared with pneumatic drills.
- Increased reliability. The rigs proved to function very well, and loss of rounds rarely happened.
- Improved working conditions with less noise and vibrations and better visibility and lighting. This is of great significance to the health and security of the workers.
- Reduced energy consumption.

INCREASED REQUIREMENTS FOR THE HYDRAULIC DRILL RIGS.

The hydraulic drill rigs require:

-Higher investments than for pneumatic drill rigs, spare parts included. In our case the cost figures for spare parts are probably too high as they include various improvements of the rig.

-Increased demands for the skill of the maintenance man.

-More severe security regulations. The rigs must be equipped with sensitive earth- leakage breakers; only HU (heavy) detonators may be used and special isolation of the ends of detonator leads are needed.

-Increased requirements to electric power-system stability. The automation of the rigs is sensitive to fluctuations of voltage.

CONCLUSION.

The experiences from the Åbjøra Hydro Electric Plant point clearly in favour of the hydraulic drill rigs. This is strengthened by the fact that up to now one dealer alone during a few years has delivered approx. 60 hydraulic drill rigs to Norwegian sites. The advance records at Åbjøra are so far the best ones obtained in Norway. And as far as the authors know among the best results in hard rock tunnelling.

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EXAMPLES OF THE BEHAVIOUR OF SHOTCRETE LININGS UNDERGROUND*

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Introduction.

The following examples of the use of shotcrete are taken from a number of different tunnels and permanent underground openings in Nor- way. Roughly calculated only 1,5 % of our total tunnel length is lined with shotcrete. This 1,5 %, however, is equivalent to more than 15 miles of tunnel length. Shotcrete linings of varying lengths are to be found in railway, highway, water supply and sewage tunnels, but primarily in hydro power tunnels. Shotcrete is also used in a number of different underground rooms for lining and supporting. Compared with (his, concrete linings cast in place make about 3 % of the total tunnel length.

Without exception all these tunnels and underground openings are situated in bedrock of Palaeozoic or Precambrian age. Sandstones, limestones, shales, mica schists, marbles, green- stones, different intrusive rocks and gneisses, granites, amphibolites and quartzites are the most common rocks types encountered.

Shotcrete linings are of ten regarded as an alternative to concrete linings cast in place. Hence the se two types of support have to be carefully compared in cost, quality and flexibility. This will often set a limitation on the use of shotcrete.

For instance, in Norway a 20 cm thick reinforced shotcrete is more expensive than a concrete lining of 30 cm minimum thickness, not reinforced and cast in place, and the time it takes to produce it is longer. W hen supporting close to the face, the shotcrete, however, involves, in general, less delay in the advance of the tunnel if thin linings are used. It also allows a greater flexibility w hen the unstable area has irregular boundaries. This makes shotcrete favourable as a temporary support after each or after every second round. When determining the permanent support of a tunnel one has not only to consider the cost and the type of stability problem, but also different circumstances concerning the prospective use of the tunnel. For instance the degree of safety against outfall of small chips with time and accessibility for later supplementary works. The low total per cent of linings in tunnels in Norway is due to such assessments and to the fact that more than 90 % of our tunnels are made for hydro-electric power schemes.

Furthermore, the physical and mechanical conditions for a tunnel often change after it has been taken into use. For instance it could be exposed to freezing and thawing or it could be used for conveying water, perhaps under high pressure. The choice of type of linings depends also on behaviour fixed by the utilization of the tunnel. Not all stability problems that arise during use express themselves equally clearly during the period of excavation.

Experiences of different methods of tunnel linings in relation to the type of stability problems and the use of the tunnel have be en valuable for the design of tunnel support in Norway. Some of these experiences concerning the use of shotcrete linings will be described in this paper.

Underground openings with dry conditions and temperature above 0º.

The first shotcrete linings used in tunnels and underground openings in Norway date back to 1952. In the early years their use was restricted to rock masses of good or very good quality. Shotcrete was used in underground petrol storage excavations to prevent sparks caused by falling rocks, or to avoid repeated scaling control every second or third year in tunnels where people worked daily. Before shotcreting the rock was scaled down and on stable areas were supported by rock bolts. Complicated stability problems were solved by concrete linings cast in place.

What was called 6 cm shotcrete was generally used, only in a few cases 10 cm. We have no information about the dosage of accelerators used in these old shotcrete linings and w hat strength they intended to gain. But we know that admixtures, if used, were treated with great caution at that time in Norway.

An inspection a month ago of some of the 10- 25 year old shotcrete linings confirmed that they had served their purpose perfectly. In general their strength and adhesion to the rock seemed to be very good. A few small are as with no adhesion were of course observed, but such occur also in newer linings, and are the results of unsatisfactory cleaning of the rock. This was probably partly due to oil coatings from the drilling equipment and contaminants from the explosives, and partly a result of the problem of keeping clean upward-facing rock surfaces which easily collected dust. A careful cleaning of the rock surface is essential for the strength of the lining.

We also experienced early on that adhesion was not obtained on surfaces formed by joint planes coated with clay. Further, that the adhesion to surface areas parallel to the foliation in mica schist, phyllite or the like, and bedding planes in certain types of shale, was often very low and considerably less than the minimum tensile strength of the rock. This is likely due to a finely splintered rock surface having microscopic cracks parallel to the surface which result from the splitting of the rock. The minerals on schistosity planes that we regard as the most dangerous are mica, chlorite, talc, graphite, hematite and clay minerals.

Rock bolts and reinforcement nets were gradually taken into use at places where such minerals occurred, in particular at protruding parts of the tunnel surface. This strengthened the shotcrete, increased the interaction and helped to build bridges to are as which gave good adhesion.

At protruding corners or edges the extending plane is often a joint or a schistosity plane with the mineral coating already mentioned. In such a case, the shear strength along the very thin shotcrete cover at the jutting edge and the very low adhesion to the joint plane reduces the strength of the construction. The use of a reinforcement net and rock bolts at the joint plane allows the possibility of increasing the thickness along the edges and the total strength of the lining.

Another thing that was tried in the early sixties was to use shotcrete linings with reinforcement net and rock bolts as permanent support in openings with high anisotropic stresses and rock spalling phenomena. Briefly described the method used was expansion rock bolts in the roof, if needed close up to the face after each round. Behind the drilling rig, supplementary bolts were installed and in some cases also a temporary net. Later on, loosened slabs were carefully taken down, the rock bolts replaced one by one with grouted dowels and the temporary net replaced by a reinforcement net. Finally 10 -20 cm shotcrete was applied. This supporting method has proved to be a success and is now frequently used.

Also w hen benching 20 m down in an underground opening in rock of good quality, but with high stresses, a shotcrete lining seems to take up the small, unavoidable

deformations of the crown in a very satisfactory way. A single tension crack in the reinforced shotcrete is, in this case, generally of no consequence for the stability.
As mentioned, however, the changes of physical conditions in a tunnel after the time of completion have given serious failures of shotcrete linings. I would like to comment on some of these types of situations that have led to great disappointment in Norway.

Underground openings exposed to freezing and thawing

In Norway frost may in some cases be active in railway and highway tunnels over stretches of up to 1,5 km. It of ten takes, however, some years before shotcrete used in such tunnels loosens, cracks and falls out due to the freezing process. The destruction process starts in humid places and places with low adhesion to the rock.
The are as with low adhesion are, as earlier mentioned, schistosity planes, bedding planes and joint planes with mica, chlorite, talc, graphite, hematite and clay minerals, as well as areas coated with oil, dust and contaminants from explosives. Some highway tunnels, railway tunnels and metro tunnels have had great maintenance costs. Figure 1 shows a rather small failure due to a joint plane and frost action in a railway tunnel near Oslo. A temporary reinforcement with bolts was carried out.
Due to shotcreting, the seepage water will be held back in joints, pores and the large number of microscopic cracks in the splintered rock surface and cause frost burst phenomena.
Draining by means of drill holes has only a limited effect. A much more effective drainage or an insulation against frost is needed to prevent failure. Rock bolts also have a limited effect. A reinforcement net, however, often prevents larger loosened slabs of shotcrete from falling out.
The shotcrete seems to resist the frost action better on dry fracture planes in crystalline rocks that do not follow the foliation or joints. But in tunnels one seldom gets such idealized conditions on all parts of the surface containing swelling clay.
It should be unnecessary to mention that also in open cuttings shotcrete exposed to frost action has, in general, been a failure. Figure 2 shows such an open cutting in shales coated with shotcrete. The picture is from Oslo.
It is obvious that instead of solving a problem, shotcrete lining on surfaces exposed to frost action very of ten leads to new stability problems of considerable extent.

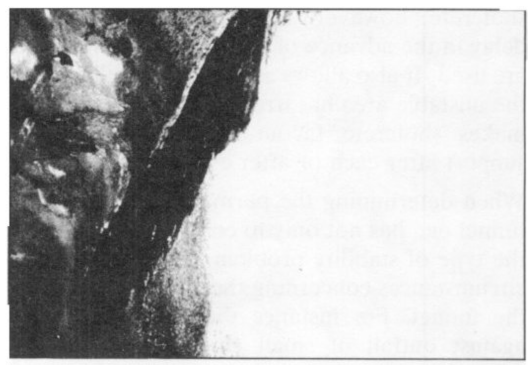

Fig. 1. Failure due to joint plane and frost action in a tunnel.

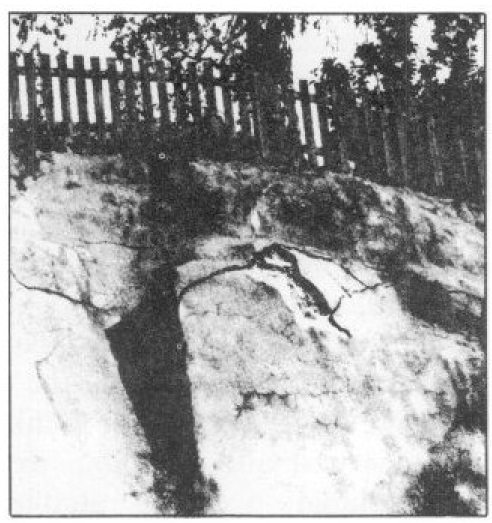

Fig. 2. Shotcrete failure in an open cutting due to frost burst.

Water tunnels with gouges containing swelling clay

Gouges containing swelling clay occur frequently in Norway. Often they are heavily consolidated and individually several meters, in some cases more than ten meters, wide. They have developed along joint planes or fault planes. Often the feldspar in the wall rock is altered to montmorillonite. In other cases a secondary schistosity, defined by thousands of microscopic fillings of montmorillonite, characterises the wall rocks of the clay gouges.

Often the only thing that could be observed in the tunnel surface is a change in colour, and the softer nature of the rock. In such cases the water percolation through the rock is often less than the evaporation so that the swelling process does not start and the real stability problem does not become apparent. Almost nothing may happen to the gouge during the time of construction. Only in the ditch and partly in the invert the water has softened the clay material by soaking and made it swell. The situation, however, still seems harmless to the miner .

If the tunnel is taken into use as a water tunnel, the altered rock swells and breaks down part by part and fins up the tunnel. Several thousand m^3 could fan out during a month or two if the water is flowing. If the water is not flowing, more or less watertight plugs may be built up. Shotcrete linings often seem in the se cases to be of no help even if the shotcrete is 25 cm thick, reinforced and bolted. It makes no difference whether shotcrete is inserted by the wet or the dry method.

Figures 3 and 4 show situations in a pressure tunnel for water supply from the south of Norway a few months after the water was introduced in the tunnel. Shotcrete linings of different design and up to 25 cm thick and reinforced with net were cracked or broken down in about 30 places and the gouge material had completely plugged the tunnel in three places.

Fig. 3. Failure due to swelling clay in water supply tunnel.

The rock mass was intersected by gouges up to 5 m wide containing a very active swelling clay in the form of altered amphibolites and gneisses. Only a few of the gouges caused difficulties during the advance of the tunnel due to seepages and a relative low degree of consolidation. They had be en temporarily supported with shotcrete.

The permanent support was done by shotcrete adapted to the size of the zones. The strength was tested during the construction by spraying in pans. A uni-axial compressive strength of more than 30 MPa was claimed.

In the approximately fifteen years' older water supply tunnel running parallel to the one in question, the linings were cast in place and no failure had occurred. Linings cast in place have also be en the usual means of supporting the very many tunnel areas with gouges containing swelling clay. This is, however, not the first time that shotcrete linings have failed in water conveying tunnels. Since 1960 we have had four different tunnels with more or less the same case history as the one mentioned.

From experience it is known that all damage to shotcrete linings of thickness between 10 and 25 cm, occurring before or after the tunnel is taken into use, have happened w hen the swellability is greater than 0,25 MPa after Brekke's swelling test. Furthermore, we know that the clay content in average for the total affected area of larger zones has been more than 5 %, but chiefly concentrated in one or more gouges. In smaller single gouges with thickness down to 20 cm, it seems that both the concentration and the swellability have always been somewhat higher in cases of failure. An interesting and favourable experience with shotcrete is the sealing method used to support small clay gouges where the wall rock is of good quality. It has been used with success on clay zones of widths up to one meter w hen supporting inactive clays. The arching effect that can be obtained with the good, clean and bolted side rock as abutments, and the eventual swelling or squeezing effects that may occur, should be carefully evaluated in each case. The method has also be en successfully used on single zones with active swelling clay w hen the width of the zone is less than 20 cm. The sealing method is, however, a typical piece of craftsmanship, both the blasting of the wedge along the gouge as well as the bolting and spraying of the zone. On the other hand concrete is saved and the profile of the tunnel is not reduced. In some cases reinforced shotcrete linings have been used in combination with rock bolts where small veins of swelling clay occur in different directions. To attain a satisfactory result, however, a sufficient number of big blocks are needed between the veins to facilitate effective bolting. Furthermore, the total content of clay in the whole rock mass should be less than one per cent, no larger concentrations should occur, and the direction of the veins should not be too unfavourable in relation to the tunnel axis. In certain types of rock the method should not be used, for instance: dunite, serpentine and soapstone.

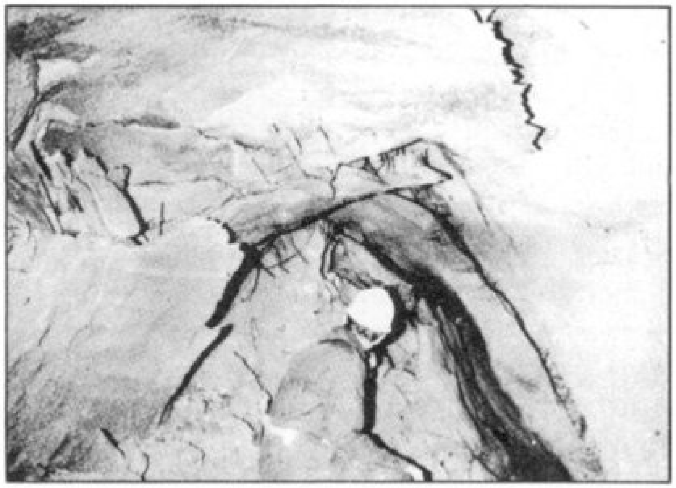

Fig. 4. A total break down of a shotcrete lining in a water supply tunnel due to a gouge with swelling

It is obvious from the examples that shotcrete linings in general must be able to withstand higher stresses from the swelling clay than do cast linings. If the rock pressure is not so high in relation to the strength of the consolidated clay, such that a squeezing effect arises, the clay in a gouge will build up a silo effect or arching effect after a very limited and tolerable expansion. This is the explanation of the relatively small expansion which is needed to reduce the mobilized stresses on a lining to an acceptable level. This also explains the different behaviour of the two types of linings. The required expansion seems to be a function of the level of consolidation, the swellability, the amount of clay and the width of the zone. This expansion varies from case to case; usually it is less than one inch, but under certain conditions it can be

considerably more. Because of the cracks formed by the blasting and the de stressing of the periphery , the exact consolidation is in general of minor interest. However, a cast lining in place close to the face gives a larger possibility for expansion of the clay than a shotcrete lining, due to: less scaling and cleaning, less concrete penetration in cracks, more time for swelling, lower adhesion to the surrounding rock, larger shrinking (which depends on the total geometry of the concrete construction) and, in addition to this, the incomplete filling against the crown.
In extreme situations where very active clay in wide gouges has had little possibility to expand due to dry conditions we have also observed cracks and deformations in the walls of cast linings. However, a total collapse caused solely by the swelling process has never occurred in Norway. In some serious cases we have successfully allowed the clay possibility for expansion behind the walls by special techniques.
In case where very active swelling clay in larger gouges has swelled sufficiently without falling out before the shotcrete is inserted, the result in tunnels carrying water has been somewhat more encouraging. However, to insert the shotcrete on the very fissured and soft clay material is difficult, and the penetration of the shotcrete in the fissures reduces the free volume needed for the swelling process. Also the low adhesion and the irregular shape of the lining do not provide reassuring conditions for the support. A critical attitude to the use of shotcrete is well founded in these cases, too.

Water tunnels with squeezing phenomenon in crushed zones

Very fine crushed dikes of altered diabase rich in chlorite have turned in to squeezing rock after having been saturated with the water that penetrated the shotcrete in a tunnel carrying water under pressure at 500 m depth. Fall-out of several thousand m3 from the crushed zone into the tunnel was the result. Also in this case of squeezing induced in fine crushed and dry chloritic material by saturation in few metres wide zones, a lining cast in place served the purpose perfectly.
Similar observations have been made in the case of crushed soapstone, serpentine and dunite, as well as in schists containing talc in addition to a very high content of chlorite or mica. Whatever else the cases may have in common, they all show a number of slickensided fissures. The water saturation gives the rock mass a very low internal friction. A small expansion seems needed also in the se cases to reduce the stresses on the lining as much as in case of swelling clay.

Conclusion

As a conclusion, I will draw attention to the fact that a proper use of shotcrete is a demanding art both for the nozzelman and for the geologist. It is an excellent method for permanent support and lining w hen used in the right way and in the right place. In areas exposed to freezing and thawing and against active swelling clay deposits in water tunnels, however, shotcrete linings are often unsatisfactory. This is also the case in instances of squeezing in crushed zones. Furthermore shotcrete is often effective as a temporary support to increase the stand up time in cases of swelling clay, too. But used close to the face during the advance of the tunnel it often covers the problems without solving them, and one has to make decisions about the permanent support blindly if such decisions were not made before shotcreting.

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THE Q-METHOD AND ITS APPLICATION -A METHOD FOR DESCRIBING ROCK MASS ST ABILITY IN TUNNELS

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SUMMARY
The Q-method is a numerical description of the rock mass quality with respect to tunnel stability. The Q-value is defined by a function consisting of six parameters which may be estimated either from geological mapping or from in situ measurements and from drill cores. The Q-method is used internationally for general description of rock mass quality, and as a guide for estimating tunnel support requirement.

DEFINITION OF Q
The Q-value is a numerical description of rock mass quality in regard to tunnel stability. The Q-value depends on six parameters.

$$Q = RQD/Jn \;\; Jr/Ja \;\; Jw/SRF$$

where
RQD = rock quality designation
Jn = joint Bet number
Jr = joint roughness number
Ja = joint alteration number
Jw = joint water reduction factor
SRF = stress reduction factor

The number of Q ranges from 0.001 (for exception- ally poor rocks) up to 1000 (for exceptionally good rocks) (Barton, Lien, Lunde 1974),

EVALUATION OF Q
One estimates the Q-value by assigning numerical values to the six parameters. This can be done by geological mapping on the surface, by core- logging, or later on by mapping in the excavation. The six parameters are given values ac- cording to well-defined descriptions.

RQD
Deere (1963) defined rock quality designation from the jointing of bore cores. The RQD number is the sum of the lengths of all core pieces longer than 10 cm (between natural joints) in per cent of the total core length. W hen bore cores are unavailable, RQD may be related fairly simply to the number of joints per volume of rock (Palmstrøm 1974).
Deere used the RQD-value as a description of rock quality. In the Q-function the RQD-value is only us ed as a measure of the joint spacing. RQD has values from 0 to 100. The Q-function specifies that 10 is the lowest RQD-value used.

Jn
Joint set number takes values from 0.5, for massive rocks with no or few joints, to 20 for crushed, earth like rocks.

Jr
The joint roughness number varies from 0.5, for slickensided, planar joints , to 4 for discontinuous joints. Usually the value for the weakest significant joint set is used in the Q-function.

Ja
The joint alteration number varies from 0.75 for unaltered joint walls to 20 for rock with thick, continuous zones of swelling clay. In the Q- function the weakest or most unfavourable joint set is generally used.

Jw
The joint water reduction factor has values from 1.0 for dry excavations to 0.05 for excavations with exceptionally high inflow.

SRF
The stress reduction factor takes values from 1.0 for medium rock pressure to 20 for heavy rock pressure. The values are taken relative to the rock strength.
Barton, Lien and Lunde, 1974, in their detailed description of the Q-method present tables for the different parameters used in the Q-method.
The Q-function may be considered as the product of three quotients. The first quotient RQD/Jn is a measure for the relative block size. The second quotient Jr/Ja is a fair approximation to the actual interblock shear strength. The third quotient Jw/SRF describes the active stress. It is generally agreed that these three quotients represent three major parameters for tunnel stability.

ESTIMATING REQUIRED SUPPORT BY Q-METHOD
The Q-value describes the rock mass stability condition. Each Q-value will therefore provide an indication of the support required. The analyses of 200 tunnel case records led to relationship between Q-value and the type of permanent support used. The design of the support constructions depends on the dimension of the excavation. Figure 1 presents the relationship among Q, excavation dimension and support category. The excavation dimension has been normalized with respect to ESR, the excavation support ratio which varies from 0.8 to 5 ac- cording to construction practice for different types of excavation with different safety requirements (see Barton, Lien, Lunde, 1974, for definition).
Figure 1 considers 38 support categories. Barton, Lien and Lunde, 1974, present a description of each category. The figure also shows the values of Q and excavation dimensions where no support have been required.

APPLICATION OF THE Q-METHOD
The Q-method has been used at NGI for 7 or 8 years as a general description of rock masses in tunnel and rock chambers, and as a guide for estimating the need for support. During pre-investigations, the Q-method provides an indication of .the rock stability and required support constructions. Through mapping of the tunnel after excavation, it is possible to compare the pre- investigation Q-value with the actual Q-

value in the tunnel. The comparison usually shows rather good agreement, especially when core drilling has been carried out during the pre-investigations.
For estimating necessary support, the tunnel or rock chamber is mapped systematically according to the Q-value. This mapping should divide the tunnel in sections where the variation of the Q-value should not exceed a factor of 10 i.e. Q = 0,1-1,4-40 etc. The Q-value in each section is estimated by assigning values to the six Q-parameters. Used in this manner, the Q-method has proven a good guide for estimating support. However, the method should be used critically and according to current experiences.
The Q-value may vary according to the way it was obtained. The Q-value from surface mapping may be lower than the actual Q-value beneath the surface. The difference is due to the surface weathering and the usual tendency to more jointed rock near the surface. In similar rock formations, the Q-value estimated in a full face bored tunnel will usually be higher than the Q-value in a blasted tunnel. In a full face bored tunnel, the joints are usually closed. Special care has therefore to be taken not to overlook any significant detail.

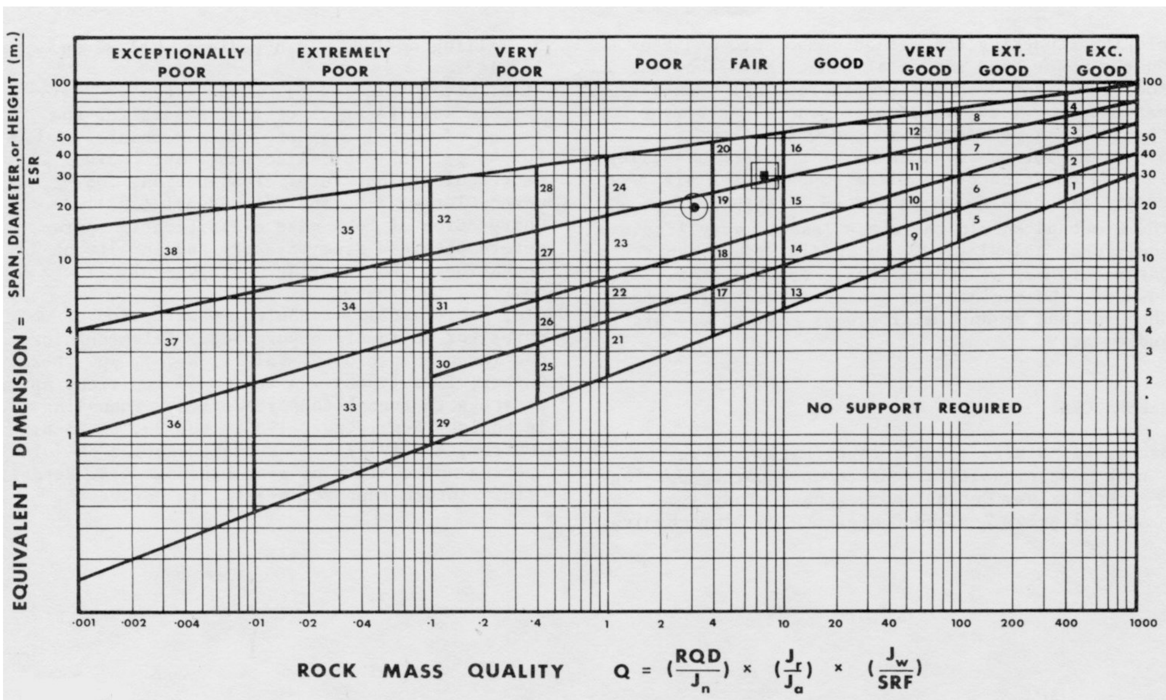

Fig. 1. Relationship among Q, excavation dimention and 38 support categories.

COMMENTS ON THE USE OF THE Q-METHOD

During the last years the Q-method has been one of the most used classification systems for rock masses. The method has been repeatedly mentioned and appraised in the literature.

Baczynski (1980) comparing several classification systems, found that the Q-method appears to be the most promising for classification and determination of stable spans within thedolomitic shales at Mount Isa Mine, Australia.

McCusker (1980) in his review of different classification systems, had some critical remarks on the method. In particular he thought that the method would not be applicable to support types other than rock bolts and shotcrete in rock of the same origin as those on which the study was based.

Cameron-Clarke and Budavari (1980) made a comparison between Q-values obtained from bore cores and in situ observations. They concluded that there was approximately an 80% probability of bore core and in situ Q-values falling within one rock class of each other.

A general criticism of the Q-method is its complexity rendering the method cumbersome and slow. On the other hand some claim that the method is too simple and does not incorporate all necessary parameters. There is however general agreement that the six parameters in the Q-function are important for the evaluation of the rock stability. In a calculation of required support, the six parameters have to be estimated in any case.

The Q-method should therefore lead to very little additional calculations. In special rocks other parameters may be of importance for the stability. However, the engineering geologist, whether using the Q-method or not, will always need to use his judgement.

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