1 THE HISTORY OF TBM TUNNELLING IN NORWAY

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ABSTRACT: Full face boring of tunnels and raises in Norway started early in the 1970's. However, the method for full face boring of tunnels has been known since 1850, but more than 100 years passed until the first tunnel boring machines (TBMs) for harder rock were developed by James S. Robbins, USA.

1 INTRODUCTION

It is almost unknown to the majority in the construction industry that a Norwegian designed tunnel boring machine was built and tested in concrete at Sørumsand workshop in 1922.

However, the tunnel boring machine was never put into operation on a real tunnel project. The design engineer, Mr. I. Bøhn, died shortly after the initial testing of boring into concrete, and further development of his TBM was stopped. The cutter head was equipped with drag bits which were not suitable for the rock types in Norway. The development of the disc cutter started with James S. Robbins in the beginning of 1950, based on ideas from 1850 by the American engineer Charles Wilson. The evolution of the disc cutter accelerated the development of tunnel boring machines.

The rock in Norway and in Scandinavia, was reckoned as very hard, and for this reason fullface boring was considered as an exotic method which served very little purpose in this country.

However, the Mining and Construction industry followed with interest the developments which took place abroad. In 1967 the very first fullface boring in Norway took place, executed by NVE (The Norwegian Hydropower Board) by boring (pilot drilling and reaming) of a 73 m long, 1.0 m diameter raise at Tokke Hydro Electric Project.

Though the rate of penetration was low, costs were high and difficulties were many, tunnellers saw the possibilities which the method could add to the mining and construction industry through future improvements of the equipment.

In 1970/71 Fangel A/S (Mofjellet Gruber) and A/S Sulitjelma Gruber both acquired equipment for fullface boring of raises with diameters up to 1.8 m. The raise boring of the shafts (up to 250 m long) was successful and the experience from this application was very important for the introduction of the tunnel boring technology in Norway.

Sulitjelma Gruber was the first in the world to use disc-cutters in hard rock in connection with raise boring and also the first in the world who was willing to try constant cross section cutter-rings, which brought the fullface technology another step forward.

In 1972 A/S Jernbeton and the City of Trondheim entered into the first contract on fullface boring of a tunnel. A new era in Norwegian tunnelling had started. The contractor A/S Jernbeton leased a second hand Demag TBM Ø 2.3 m and operators/mechanics from a German contractor for boring of the 4.3 km long sewer tunnel between Sluppen and Høvringen.

After 600-700 m of boring and several attempts with different cutter types, it was concluded that the thrust of the machine had to be increased and the cutterhead had to be modified in order to be able to cope with the massive greenstone in the tunnel. The modification took two months. The thrust per cutter-ring was increased by 40-50%. This resulted in a 100% increase in the net rate of penetration. The project was completed in 1974 and both the client and the contractor were reasonably satisfied after the project.

The first part of the main sewer system for Oslo city, the 4.5 km section Lysaker - Majorstua called for tenders in the autumn of 1973. The contract was awarded to Dipl. Ing. Kaare Backer A/S with Sulitjelma Gruber as subcontractor for the tunnel boring. Sulitjelma Gruber, encouraged by the results achieved by fullface boring of raises, bought a new Robbins TBM with diameter 3.15 m for the project. Thus Sulitjelma Gruber became the very first owner of a TBM in Scandinavia.

Comprehensive probe drilling and pre-grouting as well as post grouting were required in order to avoid lowering of the water table and prevent damage to buildings along the tunnel alignment. The drilling of probe holes and holes for grouting were carried out by hand held equipment.

The tunnel boring started in November 1974 and was successfully completed in June 1976. There were few technical problems with the equipment, the advance rate was good and the cutter costs were low in the Cambro-Silurian formation.

The utilisation of the TBM at the project was low because of the comprehensive probe drilling and grouting which was necessary to meet with the contract requirements.

It was recognised that on future similar TBM projects it would be necessary to incorporate special equipment for probe drilling and drilling of groutholes on the TBM.

The experience gained from this project was utilised on the remaining 35 km of the Western Oslofjord Sewer Project.

Encouraged by the promising results from the two first TBM tunnels in this country, the largest contractors decided to promote tunnel boring as an alternative to Drill & Blast. In 1982, 10 years after the initial fullface boring in Norway, a total of 22 tunnel projects were underway or had been completed.

After the sewer project in Oslo, Sulitjelma Gruber's machine was used in Fosdalen Bergverk for the boring of a main haulage tunnel at an 800 m deep level in the mine.

Initially the boring went well, but on the way the tunnel alignment was changed to bore into very tough and massive quartz keratophyre, which resulted in low rate of progress and advance rates and very high cutter costs.

The boring was terminated after 670 m and the equipment moved after rebuilding to the Eidfjord Project in 1977. The diameter was extended from 3.15 m to 3.25 m and the size of the cutters changed from 12" to 14", in order to cope with the granitic gneiss formations at the Floskefonn tunnel.

Of a total length of approximately 4.6 km, the first 1.8 km of the Floskefonn transfer tunnel at Eidfjord HEP consisted of a granitic gneiss with UCS up to 270 MPa. The rest of the tunnel consisted more or less of "forgiving" rock, phyllite with some quartz lenses.

350 m into the tunnel and after three Main Bearing failures, Sulitjelma Gruber decided to pull out of the tunnel and the contract with NVE. At that time there was a business down turn in the mining industry generally with low copper prices. Due to this and together with the financial setbacks at the Oslo Sewer Project (the Main contractor went bankrupt), Sulitjelma mines decided to conclude their contracting activities and to concentrate on mining. The machine was sold to an Austrian contractor for a project in Africa, and has successfully completed many tunnels since then.

Contractor Høyer-Ellefsen took over Floskefonn tunnel project. The remaining part of the tunnel section with granitic gneiss formations was excavated by Drill & Blast and a second hand Wirth TBM Ø 2.53 m was then used to bore the balance of the tunnel consisting of phyllite.

2 THE ROLE OF THE NORWEGIAN UNIVERSITY OF SCIENCE AND TECHNOLOGY (NTH) IN THE DEVELOPMENT OF FULLFACE TUNNEL BORING

It has to be mentioned that NTH (now NTNU), represented by the Department of Building and Construction Engineering, has been a prime force for the method and for the understanding and development of tunnel boring machines in hard rock. At an early stage NTH realised the possibilities and advantages fullface boring would give to the contractors and to the clients.

In cooperation with contractors, machine suppliers and tunnel owners, NTH has used the tunnels as full scale laboratories from the time the method was introduced in Norway, and made a comprehensive collection and systematizising of boring information by using engineering students.

The development of NTH's prognosis model from 1/76 until today's 1/94 has brought about an understanding for TBM, geology and rock parameters which are of vital importance for estimating advance rates and costs.

NTH's development of the model has formed the basis for better understanding and planning of fullface boring projects and has given the contractors a good tool for detail calculations and scheduling for TBM projects, or when comparing TBM and conventional Drill & Blast. The model is being used for planning and bid purposes on several projects abroad.

The first prediction model was developed and published in 1976 in cooperation between the Department of Construction Engineering, and the Department of Geology at NTH (the Norwegian University of Science and Technology).

3 TUNNEL BORING AT AURLAND HYDRO ELECTRIC PROJECT

In 1977-78, the contractor Ing. Thor Furuholmen A/S with technical support from the Swiss contractor Prader AG bored a 6.2 km long transfer tunnel at the remote Aurland HEP with a Robbins TBM 3.5 m diameter. The rock was mostly phyllite with quartz lenses and sections consisting of massive, abrasive sandstone.

The boring operation was very successful. No rock support was required during boring and the need for permanent rock support was minimal. The unit cost per meter tunnel excavated was somewhat higher than calculated for Drill & Blast, but the reduced construction time from one adit only made the TBM tunnel project come out as a great winner.

The Aurland project became an important corner stone for the further development of NTH's prediction model. Here, for the very first time, NTH also recorded the effects of angle between foliation and tunnel axis on the net rate of penetration.

4 THE DEVELOPMENT OF THE WESTERN OSLOFJORD REGIONAL SEWAGE PROJECT

The City of Oslo, together with the neighbouring municipalities of Bærum and Asker, established a jointly owned sewage treatment plant in the period 1974-1981. The scheme comprises connecting tunnels with diameters ranging from 3.0 m to 3.5 m, nearly 40 km in total length.

In 1970 when the Prestudy Report for the project was presented, it was assumed that all tunnels were to be excavated by Drill & Blast methods.

However, the 1973 Feasibility Study Report concluded that an alternative method based on TBM should be requested in addition to tenders based on Drill & Blast.

In 1976, after the successful completion of the first section of the main sewage tunnel system (Lysaker - Majorstua), the tender documents for the remaining contracts specified that all major tunnels were to be excavated by TBMs. Drill & Blast method would not be accepted. In a time span of six years the approach of the owner and his consultants changed from 100% conventional to 100% TBM boring.

The tunnels were bored with seven TBMs from four different manufacturers with quite different concepts. At one time, a total of six machines were in operation at the project - one Wirth, two Bouygues, two Robbins and one Atlas Copco.

The experience gained from the first tunnel drive on the project was used as input for the tender document specifications for later contracts for the same project in 1976 and 1977. The owner made strict requirements for probing and pregrouting in order to avoid or minimise damage due to settlement, caused by lowering of the ground water table. The contractors had to provide and demonstrate mechanised equipment and methods for efficient probing and pregrouting. This became the most extensive probing and grouting program ever executed in connection with TBM operations anywhere in the world.

5 HARDER AND MORE MASSIVE ROCK IS BEING BORED

Norway is generally considered to provide some of the toughest hard rock challenges in the world. With few exceptions, the first TBM projects in Norway started out in the relatively easy greenschists, greenstone, shale, limestone, phyllites and micaschists. Later, tunnels in Precambrian rocks, granites and gneiss have been bored.

The breakthrough for hard rock tunnel boring came in the period 1981-1984 with the accomplishment of the 8 km long, 3.5 m diameter transfer tunnel in Glommedal at Ulla Førre Hydro Electric Project. Encouraged by the results from the tunnel boring in Kleådalen at the Aurland project, NVE decided in 1980 to purchase a new 3.5 m dia TBM to bore the Glommedal transfer tunnel.

The area contained massive granite and gneiss formations with up to 210 MPa unconfined compressive strength. In fact, the rock on this project was so massive, that the NTH (the Norwegian University of Science and Technology, now NTNU) predictor model was revised to include the fracture class 0 (zero). The Robbins TBM 117-220 worked for 2.5 years to cut through the massive rock on the 8,022 m diversion tunnel.

The same TBM 117-220 later drove a 9.2 km diversion tunnel through granitic gneiss at Kobbelv Hydro Electric Project, followed by 9.3 km at Storjord site and 6.2 km at Trollberget site, Svartisen Project.

In November 1991 the TBM set outstanding Norwegian tunnelling performance records at Vegdalen Diversion tunnel, Svartisen project: best shift - 44.2 m, best day - 81.8 m, best week - 421.2 m and best month - 1,176.5 m, averaging more than 200 m per week in the micaschist and micagneiss formations.

Owned and operated by Statkraft (formerly NVE) the TBM has bored a total of 32.7 km of tunnel on four projects, all in hard rock formations. This "oldtimer" is the TBM that has bored the most tunnel metres in Norway and Scandinavia so far, and is also included in the Robbins TBM Honor Roll.

6 TBM INCLINE SHAFT BORING

Boring inclines with TBM + ABS (anti-backslip-system) was first used in Norway in 1980 by contractor Høyer-Ellefsen in cooperation with the Swiss contractor Murer AG, at Sildvik Hydro Electric Project near City of Narvik. A 45 degrees, 760 m long x 2.53 m diameter pressure shaft was completed with a Wirth

TBM in six months, inclusive assembly and disassembly of the equipment. Average advance rate of 45 m per week was achieved.

The performance of the incline borer met with the expectations and encouraged the contractor Astrup-Høyer AS in 1983 to purchase an Atlas Copco Jarva 3.2 m diameter TBM to bore a 1,250 m long, 41 degree pressure shaft at Tjodan Hydro Project.

In 1985 the same contractor used this TBM to bore the 1,370 m long penstock with 100% inclination at Nyset-Steggje HEP. The shaft was completed in seven months, including downtime due to a main bearing failure. The main bearing of the TBM was changed in the shaft, approximately 450 m from the start portal.

The rock bored at Tjodan and Nyset-Steggje consisted of massive granite and granitic gneiss. Shaft boring of longer pressure shafts with TBMs proved to be a very good alternative to conventional Drill & Blast Alimak raising, especially when the pressure shaft is on the critical path.

7 ROCK BURST AND SPALLING CHALLENGES AT KOBBELV HYDRO ELECTRIC PROJECT

During TBM boring in granitic gneiss at Kobbelv Hydro Electric Project in 1984, extreme rock burst and spalling problems occurred, caused by high horizontal in-situ stresses.

Approximately 1,700 m into the 6.25 m diameter Tverrelvdal headrace tunnel intensive bursting and spalling occurred. Extensive and systematic rock support were required to safeguard men and equipment. Rock bolts with plates were installed immediately behind the TBM front supports from the TBM working platforms. After about 200 m the rock burst and the spalling decreased gradually as the rock overburden increased, thus counteracting the effect of the horizontal stresses. The average daily production rates over the 200 m section dropped to approximately 6 m, and the rock stability problems increased the construction schedule by about two months.

Intensive spalling occurred as well during boring of the first 1,000 m section of the 9.2 km long 3.5 m diameter nearby Reinoksvatn transfer tunnel. High tangential stress concentrations in the roof and in the invert, caused by the high horizontal stresses in the area, required immediate, extensive and systematic rock support. Rock bolts, straps and netting, and concreting of the invert were used to stabilise the track. As for the Tverrelvdal tunnel, the spalling decreased as the rock overburden increased. Average weekly production over the first 1,000 m slowed to 52 m per week as opposed to 120 m per week in the remaining part of the tunnel, and the problems added eight weeks to the construction schedule.

The contractor/owner NVE (The Norwegian State Power Board) concluded that a better advance preinvestigation and understanding of rock mechanics and stress conditions at Kobbelv would have resulted in better forecasting, and would have eased the challenges and probably reduced time loss to the half. NVE and renowned rock mechanic experts were also of the opinion that Drill & Blast tunnelling would have created even worse spalling problems and damage to the tunnel surface over longer stretches than for TBM boring, due to the impact from the blasting.

8 TBM FOR ROAD TUNNELLING

For the construction of the Svartisen Hydro Project NVE, the Norwegian Water Resources and Electricity Board (now Statkraft) needed road access to the Western part of the project. In 1983 it was decided to build a 7.6 km single lane road tunnel with meeting niches at every 300 m, and a 6.25 m diameter leased, second hand TBM was commissioned to drive a 4.3 km section of the tunnel between Kilvik and Glomfjord.

After the project was started, government authorities involved upgraded the road standard from a single-lane to a two-lane highway as part of the highway system along the coast of Norway. The TBM driven section therefore was later slashed out from Ø 6.25 (30.7 m^2) to 52.5 m^2 to meet with the T8-standard requirements of the Road Department.

In the period 1984-1986 The Norwegian Public Road Administration built two-lane dual road tunnels, 3.2 + 3.7 km long, through Fløyfjellet, City of Bergen, as part of a bypass motorway system by using a 7.8 m diameter Robbins TBM with backup for trackless muck haulage by regular dump trucks.

The TBM was named “Madam Felle” after a locally well-known lady running a beer pub in the “good old days” in Sandviken, an old part of Bergen. Besides tunnelling advance rates, emphasis was also put on the environmental aspects of TBM boring vs. Drill & Blast because of the dense population along the tunnel alignment, including hospitals near the tunnel portals and exits. The tunnel boring was a success in the hard granitic gneiss formations. However, slashing out the two lower corners by Drill & Blast to meet with the motorway standard was time consuming.

Later, in 1987, the TBM was enlarged from 7.8 m to 8.5 m diameter to reduce slashing and drove a 850 m long road tunnel which was parallel and close to an existing road tunnel in Eidsvåg, a suburb of Bergen city. Boring of the tunnel was almost a must in order to maintain the traffic in the construction period. At 8.5 m diameter the lower corners still had to be blasted.

In order to avoid the time consuming and costly slashing out of corners for the most of the road tunnels to be built in Norway, a TBM diameter of 9.3 - 9.5 m is needed.

In 1996 Statkraft Anlegg AS, who is playing an active role in the Norwegian TBM tunnelling effort domestically as well as internationally, purchased from the Swedish contractor Kraftbyggarna two of the most powerful large diameter TBMs ever built; two Atlas Copco MK 27 TBMs with diameters Ø 6.5 and 9.1 m. The diameter for both TBMs can be extended to 10-12 m depending on the rock formations to be bored.

Statkraft Anlegg in cooperation with the Norwegian Public Road Administration and Department of Building and Construction Engineering at the Norwegian University of Science and Technology (NTNU) started in 1996 a development program for building of TBM bored road tunnels in Norway in such a way that the road bed and tunnel installations behind the TBM is finished at the breakthrough.

9 UPGRADING OF AN EXISTING HYDRO POWER PLANT

In 1986 the owner of Nedre Vinstra Hydro Plant called for bids for extension of the existing power plant and connecting tunnel system in order to double the power production.

The plan included enlargement from 32 to 52 m^2 cross section of the existing headrace tunnel by slashing out the invert over the whole length of 17 km. To minimise lost revenue during plant shut down, the work had to be completed within one or two summer seasons, using large crews to drive simultaneously from five existing access tunnels.

During bidding for the job, the VSF Group, a joint venture of AS Veidekke and Selmer Furuholmen AS, suggested an attractive bid alternative. The VSF Group proposed to drive a nearly parallel headrace tunnel using two second-hand TBMs in the phyllite and sandstone formations. This scheme would enable the power station to remain in operation during the construction of the tunnel. The solution would cut power station downtime from a minimum of six months to one month, and nearly eliminate the need for rock support. New TBMs were not considered because of the 4-5 months required tunnel start up. Two existing Robbins machines (Models 148-212 and 148-213) successfully used by the contractor A/S Jernbeton on the Brattset and Ulset HEPs, were applicable and readily available. The machines were upsized to 4.75 m diameter.

The two TBMs were started from a single adit approximately halfway between the reservoir and the pressure tunnel, and the machines bored in opposite directions, with a common tip station for the muck train.

The project proved to be very successful, and was completed six months ahead of schedule.

At the bidding stage, the TBM alternative for the headrace tunnel was calculated to cost about 15% more than the Drill & Blast slashing out of the invert. However, the income from electricity generation during the construction of the TBM headrace tunnel more than offset the extra cost of construction.

10 FIVE TBMs ON SVARTISEN HYDRO ELECTRIC PROJECT

In cooperation with Statkraft, Robbins introduced in 1988 the High Performance TBMs, using 483 mm (19”) cutters. The three first HP TBMs thus ever built, TBM 1410-251 (Ø 4.3/5 m), TBM 1410-252 (Ø 4.3 m) and TBM 1215-257 (Ø 3.5 m) bored 13.8 km, 11.8 km and 8.2 km tunnels respectively at Trollberget job site, Svartisen Project.

By this development, the cutters, TBM and the TBM performance were taken to a new level, and gave the TBM industry an improved tool for boring hard to very hard rock formations.

On the west side of the Svartisen glacier, Statkraft was using the 8.5 m diameter Robbins machine, “Madam Felle”, the veteran of the twin highway tunnels in Bergen and Eidsvaag, for driving a 7.3 km long incline pressure tunnel (1:13.5) with trackless haulage of muck. A Statkraft designed turntable was used for turning the trucks right behind the backup, instead of providing turning niches.

The TBM from Ulla Førre and Reinoksvatn (Kobbelv Hydro Project), Robbins model 117-220 (Ø 3.53 m) first drove a 9.3 km near-the-surface gutter tunnel on the West side of the glacier and was then moved to Trollberget where it bored another 6.2 km tunnel.

The Main Bearings for the HP TBMs supplied to Svartisen are of the Tri-axial type. The change in the late eighties from Tapered Roller Bearings to Tri-axial Main Bearings in the TBM industry has improved the utilisation of the machines due to improved load characteristics and bearing life.

Another big improvement is the new style wedge lock cutter housings that were introduced by Robbins at Trollberget job site, first time ever. This cutter housing design is superior to any other cutter housing and became instantly the industry standard, leading to improved cutter life and less cutter changes.

The development and introduction of the Robbins 19” (483 mm) cutters rated at 312 kN/cutter was another significant step forward in hard rock boring and enabled the HP-machines to operate at 40-50% higher thrust per cutter compared to the standard TBMs fitted with 17” cutters.

Statkraft tested different type cutters and cutter-rings from other manufacturers (Wirth, Sandvik) as well at the Svartisen project, including a new concept, the “Norway-cutter” designed by Stein Narvestad. The split cutter-ring design allowed cutter-rings to be changed at the face. The conclusion from the fairly limited testing at selected cutter positions was that this concept worked and was an interesting alternative for cutters in abrasive rock formations. However, further modifications were needed, and there was still room for improvements in the design of the retainer device, holding the cutter-ring in place.

11 RECORD PERFORMANCE AT MERAAKER HYDRO ELECTRIC PROJECT

In August 1992 Merkraft, a joint venture of Eeg-Henriksen Anlegg AS and AS Veidekke, completed the boring of a 10 km transfer tunnel at Meraaker Hydro Electric Project with a 3.5 m diameter Robbins High-Performance TBM in less than 11 months. The tunnel boring was finished six months ahead of schedule. In the first full month of operation the TBM achieved the fastest start-up of any Robbins TBM on record by boring 1,028.9 m.

Merkraft set outstanding national performance records along the way with the HP TBM working in geology ranging from hard, massive metagabbro, with UCS of 300 MPa and graywacke and sandstone appearing as mixed face conditions to relatively soft phyllite.

- Best shift (10 hrs.) 69.1 m
- Best day (two 10 hr. shifts) 100.3 m
- Best week (100 shift hours) 426.8 m
- Best month (430 shift hours) 1358.0 m
- Average weekly advance rate 253.0 m

12 NORWEGIAN CONTRACTORS WITH TBM EXPERIENCE

Projects in Norway:

Statkraft	102 km
Høyer-Ellefsen * 18,643 m	
Astrup-Aubert * 12,050 m	
Astrup-Høyer * 1,370 m	
Jernbeton * 34,510 m	
Aker Entreprenør * 6,350 m	73 km
(*: now Veidekke)	
Furuholmen (now Selmer)	35 km
VSF (Veidekke/Selmer-Furuholmen)	17 km
Hordaland Vegkontor/State Road Dept.	8 km
Kruse Smith	8 km
Sulitjelma Gruber	5 km
Merkraft (J.V. of Veidekke and Eeg-Henriksen)	10 km
Completed (1972-1992)	258 km

Projects abroad:

Statkraft	16 km
NOCON (Veidekke-Selmer)	19 km
NOCON/Eeg-Henriksen/Statkraft	9 km
Jernbeton (Astrup-Høyer)	5 km
Completed, under construction or under start up:	49 km
(1994 -)	

As of 1997, the total length of TBM bored tunnels in Scandinavia, completed or under construction is 321 km of which 258 km have been bored in Norway.

13 MANUFACTURERS INVOLVED IN TBM TUNNELLING IN NORWAY

Name of manufacturer	Number of TBMs supplied	Number of tunnel drives	Tunnel length bored
Atlas Copco	7	8	27,6 km
Robbins (now Atlas Copco Robbins)	14	30	184,9 km
Wirth	6	8	30,4 km
Bouygues	2	2	10,8 km
Demag	1	1	4,3 km
	30	49	258,0 km

14 NORWEGIAN TBM CONTRACTORS ABROAD

Jernbeton AS was the first Norwegian contractor operating a TBM abroad. As sub-contractor to Skanska, in the period 1985-86 a 4.5 km section of the Kymmen Hydro tunnel in Sweden was bored. Jernbeton used its 4.53 m diameter Robbins TBM 148-212 in gneiss and granitic rocks.

In August 1992, after the Meraaker Hydro Project was completed, a 20 year period of tunnel boring in Norway had come to a temporary stop, due to lack of projects. Contractors with TBM expertise had to look for projects abroad.

In 1993 Merkraft sold the Robbins HP TBM 1215-265 to a foreign contractor for a project in the Middle East. The machine was upsized from 3.5 m to 4.23 m diameter. However, Nocon (50/50 owned by Selmer and Veidekke) in cooperation with Eeg-Henriksen and Statkraft Anlegg operated this TBM as sub-contractor to the main contractor. The boring of the 9 km section of the tunnel was successfully completed in the beginning of 1997.

Statkraft Anlegg AS entered into a cooperation agreement with the Joint Venture CNO/WBH in 1994, for boring of 6.5 km of the 6.7 km long 3.5 m diameter Midmar Tunnel in South Africa. The Robbins High Performance TBM Ø 3.5 m (with 19” cutters) from Svartisen Project was used. The drive, which began in beginning of February 1995, is part of the first phase of the Umgeni Water project to provide water to the Pietermaritzburg area, west of Durban. Statkraft Anlegg provided the TBM and technical support for the Midmar drive, including the TBM operating crew, cutter shop man, supervisor and manager.

The Midmar tunnel encountered some of the hardest rock ever bored with a TBM. One dolerite sample tested at 420 MPa. The tough dolerite comprised half of the tunnel length. Cutter changes consumed almost half of the available time, and caused the drive to fall 10 weeks behind schedule. Statkraft Anlegg worked closely with Atlas Copco Robbins in developing a new extra heavy duty cutter-ring to sustain the cutter loads required and the impact from open joints for this particular drive. The new cutter-rings reduced cutter consumption by 50% and put Midmar back on track. The breakthrough came on schedule end of February 1996.

The Midmar Tunnel used an extensible conveyor mucking system for the entire TBM bored tunnel length of 6.5 km and Statkraft Anlegg became the first Norwegian contractor to use a Continuous Conveyor System.

In August 1997 Statkraft Anlegg entered into a lease agreement with Leighton Kumagai Joint Venture for the use of the Robbins HP TBM 1215-257 for boring of a 4.5 km section of a 12 km long water supply tunnel in Hong Kong. The contract includes technical services from Statkraft Anlegg for the duration of the tunnel boring. The diameter of the TBM will be extended from 3.5 m to 3.84 m. The boring start up was scheduled for January 1998.

Statkraft Anlegg, in joint venture with Jaiprakash Industries Limited India, is currently operating a 8.3 m diameter Open Robbins Hard Rock TBM at Dul Hasti Project in Kashmir, India. Approximately 1,600 m of the 6.5 km section of the headrace tunnel was completed by a French Joint Venture before it pulled out of the project in 1993 for security reasons. Statkraft has a crew of 15 people on the project.

In August 1997, Statkraft Anlegg successfully completed a 110 m long, 1.8 m diameter raise at Dul Hasti with an old Robbins 61R raise drill that was taken over from the J.V. through the client, and that Sulitjelma Gruber purchased in 1971 new from Robbins.

Statkraft Anlegg has become one of the most aggressive hard rock TBM contractors world-wide, and they are currently bidding on several tunnel projects abroad.

The Norwegian Construction Group (NOCON) signed a contract in January 1996 with the main contractor Astaldi/SAE Joint Venture for boring of a 4 km headrace tunnel and an approximately 15 km long transfer tunnel as subcontractor at Pont Ventoux Project in northern Italy, close to the French border.

On this project NOCON is currently operating two refurbished “old timer” Robbins TBMs, open hard rock type, Ø 4.75 m leased from two different owners. Continuous Conveyor Systems are used for muck transportation for both TBMs. After boring approximately 2,400 m of the transfer tunnel from F2 site, a total water inflow in the range of 20 m^3 per minute over some 900 m bored, seems to be the most difficult task to handle on this tunnel project. Railbound muck haulage under such circumstances would not have been feasible.

15 THE NORWEGIAN ROCK BLASTING MUSEUM

To honour the Norwegian Tunnelling Society for its contribution to the success of hard rock tunnel boring, Atlas Copco Robbins in November 1997 donated the veteran Robbins TBM 81-118 to the Norwegian Rock Blasting Museum. The 2.59 m dia TBM, still in good operating condition, was manufactured by Robbins in 1965, and was the 18th machine built.

TBM PROJECTS IN NORWAY

PROJECT LOCATION IN NORWAY	CONTRACTOR	MACHINE MANUFACTURER	SERIAL NUMBER	DIAMETER	PROJECT NAME	BORING PERIOD	TUNNEL LENGTH BORED
Trondheim	Jernbeton	Demag	TVM 20-23H	2.3 m	Trondheim Sewer	1972-1974	4.300 m
Oslo	Sulitjelma Gruber	Robbins	105-165	3.15 m	Oslo Sewer	1974-1976	4.300 m
Malm	Sulitjelma Gruber	Robbins	105-165	3.15 m	Fosdalen Mine	1977	670 m
Floskefonn	Sulitjelma Gruber	Robbins	105-165-1	3.25 m	Eidfjord Hydro	1978	350 m
Trondheim	Trondheim/Atlas Copco	AC Mini Fullfacer	1524	1.5 x 2.4	Grillstadbekken	1977	120 m
Kjøpsvik	Høyer-Ellefsen/Murer	Wirth	TBII-330H	3.32 m	Norcem Cement	1977-1978	1.120 m
Oslo	Astrup & Aubert	Bouygues	TBM 300C	3.0 m	Western Oslofjord Regional Sewer	1977-1982	5.400 m
Oslo	Astrup & Aubert	Bouygues	TBM 300C	3.0 m	Western Oslofjord Regional Sewer	1979-1982	5.400 m
Oslo	Høyer-Ellefsen/Murer	Wirth	TBII-300H	3.35 m	Western Oslofjord Regional Sewer	1977-1981	7.580 m
Oslo	Høyer-Ellefsen/Murer	AC Midi Fullfacer	FF-2132	2.1 x 3.2	Western Oslofjord Regional Sewer	1979-1980	1.000 m
Oslo	Furuholmen/Prader	Robbins	116-189	3.5 m	Western Oslofjord Regional Sewer	1978-1981	7.200 m
Oslo	Furuholmen/Prader	Robbins	116-188	3.5 m	Western Oslofjord Regional Sewer	1978-1981	7.060 m
Aurland	Furuholmen	Robbins	116-181	3.5 m	Aurland Hydro Project	1977-1978	6.200 m
Lier	Furuholmen	Robbins	116-181	3.5 m	Lier Water Supply Tunnel	1979	1.200 m
Floskefonn	Høyer-Ellefsen/Murer	Wirth	TB I-253H	2.53 m	Eidfjord Hydro	1979-1980	2.807 m
Sildvika	Høyer-Ellefsen/Murer	Wirth	TB I-253H	2.53 m	Sildvika Hydro 45 deg. incline	1980	760 m
Sørfjord	Furuholmen	Robbins	116-181	3.5 m	Sørfjord Hydro Project	1980-1982	5.840 m
Skamfer	Jernbeton	Robbins	148-212	4.5 m	Brattset Hydro Project	1980-1982	8.150 m
Naeverdal	Jernbeton	Robbins	148-213	4.5 m	Brattset Hydro Project	1980-1982	7.000 m
Ulla Førre	NVE	Robbins	117-220	3.5 m	Ulla Førre Hydro Project	1981-1984	8.022 m
Mosvik	Høyer-Ellefsen	Atlas Copco Jarva	MK 12 T	3.5 m	Mosvik Hydro Project	1982-1983	5.390 m
Yset	Jernbeton A/S	Robbins	148-213-1	4.5 m	Ulset Hydro Project	1982-1984	7.300 m
Ulset	Jernbeton A/S	Robbins	148-212-1	4.5 m	Ulset Hydro Project	1983-1984	4.960 m
Lysefjord	Astrup & Aubert	Atlas Copco Jarva	MK 12	3.2 m	Tjodan Hydro 41 deg. incline	1983-1984	1.250 m
Lysefjord	Furuholmen	Atlas Copco Jarva	MK 12	3.5 m	Tjodan Hydro Project	1983-1984	4.865 m
Sørfjord	Furuholmen	Robbins	116-181	3.5 m	Sørfjord Extension	1982-1983	3.010 m
Kobbelv	NVE	Robbins	204-215-1	6.25 m	Kobbelv Hydro Right Branch	1983-1984	2.492 m
Kobbelv	NVE	Robbins	204-216-1	6.25 m	Kobbelv Hydro Right Branch	1985-1986	2.659 m

TBM PROJECTS IN NORWAY cont. (2)

PROJECT LOCATION IN NORWAY	*CONTRACTOR*	*MACHINE MANUFACTURER*	*SERIAL NUMBER*	*DIAMETER*	*PROJECT NAME*	*BORING PERIOD*	*TUNNEL LENGTH BORED*
Glomfjord	NVE	Robbins	204-216-1	6.25 m	Glomfjord Road Tunnel	1984-1985	4.333 m
Kobbelv	NVE	Robbins	204-215-1	6.25 m	Kobbelv Hydro Left Branch	1985-1986	4.181 m
Kobbelv	NVE	Robbins	117-220-1	3.5 m	Kobbelv Hydro Project	1984-1987	9.206 m
Bergen	State Road Dept.	Robbins	252-226	7.8 m	Fløyfjell Twin Road Tunnel	1984-1986	6.850 m
Årdal	Astrup-Høyer A/S	Atlas Copco Jarva	MK 12	3.2 m	Nyset-Steggje HEP 45 deg. incline	1985	1.370 m
Trondheim	Jernbeton A/S	Wirth	TB I-270H	2.7 m	Heimdal Sewer	1985-1986	2.800 m
Gaupne	Statkraft	Wirth	TBIII-450E	4.5 m	Breheimen Hydro Project	1986-1989	9.001 m
Jostedal	Statkraft	Atlas Copco	FORO 1500	4.5 m	Breheimen Hydro Project	1986-1989	5.550 m
Vinstra	VSF-Group	Robbins	148-212-3	4.75 m	Nedre Vinstra Hydro Project	1987-1988	9.789 m
Vinstra	VSF-Group	Robbins	148-213-2	4.75 m	Nedre Vinstra Hydro Project	1987-1988	6.773 m
Bergen	State Road Dept.	Robbins	252-226-1	8.5 m	Eidsvåg Road Tunnel	1987	850 m
Holandsfjord	Statkraft	Robbins	252-226-1	8.5 m	Svartisen Hydro Project	1988-1990	7.308 m
Holandsfjord	Statkraft	Robbins	117-220-1	3.5 m	Svartisen Hydro Project	1989-1990	9.277 m
Trollberget	Statkraft	Robbins	117-220-1	3.5 m	Svartisen Hydro Project	1991-1992	6.161 m
Trollberget	Statkraft	Robbins	1410-251	4.3/5.0 m	Svartisen Hydro Project	1989-1992	13.836 m
Trollberget	Statkraft	Robbins	1410-252	4.3 m	Svartisen Hydro Project	1989-1991	11.861 m
Trollberget	Statkraft	Robbins	1215-257	3.5 m	Svartisen Hydro Project	1990-1991	8.210 m
Stavanger	Aker Entreprenør	Wirth	TBII 325H	3.25 m	Stavanger Sewer	1988-1989	3.850 m
Stavanger	Kruse Smith	Atlas Copco	Jarva MK12	3.5 m	I.V.A.R. / Stavanger Sewer	1989-1990	8.070 m
Haugesund	Aker Entreprenør	Wirth	TBII 325H	3.25 m	Haugesund Sewer	1990	2.500 m
Meråker	AF Merkraft (J.V.)	Robbins	1215-265	3.5 m	Meråker Hydro Project	1991-1992	9.647 m

2 BOREABILITY TESTING

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ABSTRACT: Tunnel boring: The rock is excavated by rolling cutters being thrust against the rock with a high force while the cutterhead is rotated. The rock material directly under the cutter edges is crushed and concentric grooves are formed. Between the grooves the rock chips away in larger pieces. The relevant material properties are therefore hardness, as resistance against indentation, and resistance against impact, as well as the abrasivity and occurrence of weakness planes and discontinuities. The paper discusses different test methods in use with emphasis on the Norwegian boreability testing practice. Advice is given regarding approach and procedures for sampling.

1 INTRODUCTION

The performance of tunnel boring machines is dependent on the ground conditions. This includes all aspects from excavation of the rock material, steering and support of the machine, need for rock support due to rock stresses, jointing, weakness zones etc. and problems with water and gas.

Excavation rates can vary from less than 1 mm per revolution of the cutterhead to 10-15 mm, limited by the capacity of the muck removal system. This can result in weekly progress ranging from less than 30 m to more than 300 m, making a large impact on time schedules and costs. Cutter costs may vary by an even higher factor. Hence reliable prediction methods are necessary to establish realistic cost estimates, progress schedules or performance guarantees.

A variety of testing methods in connection with planning, contracting and follow up of tunnel boring are utilised. These range from general rock material classification tests to specialised boreability tests, intended to simulate the effects of or on TBM cutters. The tests are performed on small pieces of cores or hand samples, and some on large blocks of rock for full scale simulation.

The tests are carried out by TBM manufacturers, by consultants, commercial testing laboratories, and technical institutions, to develop more efficient TBMs and cutters, predictions for contracts, and for research on methods in general.

Efforts have been spent on identifying the most useful tests for measuring decisive factors for the boring operation. This has not yet resulted in any internationally accepted standards. Practice therefore varies a lot, both between and within different countries, with respect to which tests are performed and how the results are used in tunnelling contracts.

The intention of this paper is to identify the rock material properties which are important for tunnel boring, and to discuss the usefulness of different test parameters. The test methods that form the basis for the Norwegian prediction model are described in more detail, as they have been extensively used for predictions, contracts and follow-up of results for many tunnel boring projects in Norway.

2 FAILURE MECHANISMS

2.1 Crushing and chipping

Tunnel boring, in the most common form, is performed by thrusting the cutterhead of the TBM towards the tunnel face, at the same time as it is rotated. By this the cutters roll on the rock surface. If the thrust force is large enough the cutter edges penetrate into the rock.

As the cutterhead rotates, the force on the individual cutter varies all the time. The peak load of individual cutters can be several times the average load. Thus, the action of the cutter includes a percussive effect.

As the cutter rolls, three main failure mechanisms occur:

- Cracks are formed, penetrating radially into the rock from the cutter edge.
- Material under the cutter is crushed to a fine powder, of which some is compacted and left in the groove.
- Chipping of pieces takes place between the grooves.

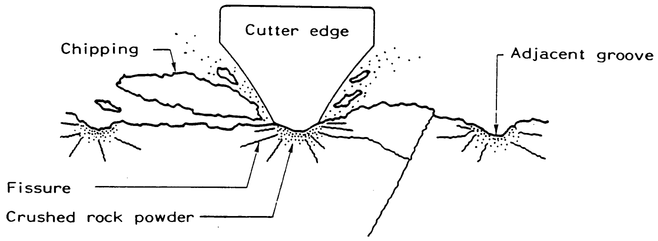

Fig. 1. Failure mechanisms under the cutter edge.

The chipping between the grooves does not necessarily take place for each pass of the cutter. Depending on the rock, the thrust per cutter and the cutter spacing, it may take several cutterhead revolutions before the ridge between the grooves chips away. Local weaknesses in the rock mass will influence the cutting, by allowing easier cracking or chipping, or by increasing the size of the pieces that break loose.

High speed films, performed by Colorado School of Mines of full scale linear disc cutting, show the repeated chipping in front and to the side of the cutter edge, and that part of the rock material is being crushed under the cutter edge and "blasted" out to the side of the edge. The chipping of larger pieces is obviously a more efficient failure mechanism than the crushing that takes place under the edge. Both are inherently part of the process.

It has been discussed whether the chipping takes place as a tensile or shear failure. The crack formation takes place radially from the cutter edge, into the rock or to the next groove, parallel to the high compressive stresses from the cutter edge. The high speed films show that the chips "pop up" from the rock surface before they are pushed sideways. There are obvious similarities to the formation of radial cracks from blast holes or cracking along high compressive stress trajectories in general. On chips of fine grained rocks it is sometimes possible to observe the characteristic "feather" pattern on the surface clearly indicating a tensile type of failure. The highly dynamic effect must be remembered, the cutter is actually exerting a series of impacts to the rock surface.

2.2 Observations at the tunnel face

When a cutterhead is retracted for inspection and change of cutters, the rock at the tunnel face can be observed. The concentric grooves with rock powder are easily recognised, and the ridges between the grooves appear with clean surfaces, protruding or almost flat depending on the efficiency of the cutting.

Except for the size and geometry, the similarity to the bottom of a percussive hole is striking. Both have grooves after the edge of the bit or cutter and chipped surfaces in between.

The effect of weakness planes or discontinuities on the cutting can also be observed, as pieces of rock may have been broken out ahead of the general face surface.

3 IMPORTANT PARAMETERS

3.1 Strength

Ideally, an oriented dynamic tensile strength test should express the properties decisive for crack formation and chipping. As a substitute, the normal classification tests such as Uniaxial Compressive Strength (UCS) or Point Load Strength Index (IS) are sometimes used. UCS may show a correlation to net penetration rate for some rock types or within a rock type. However, experience has shown that UCS for many rock types provides no or poor correlation to net penetration rates. Caution should therefore be taken not to rely on this parameter alone for boreability estimates.

The Point Load Strength Index IS, which expresses the strength against indirect tensile failure, shows reasonable good correlation to net penetration rates for some rock types. The correlation will however vary from one rock type to another.

In conclusion, these parameters should not be used as boreability parameters alone. They can however be useful as supporting parameters for the purpose of general classification and to allow tentative comparisons.

3.2 Toughness

Toughness, or ability to sustain deformation, should in principle be a more directly useful parameter. Several tests are available and are in different ways expressing the resistance against impact.

One is the Atlas Copco Robbins' toughness test, performed by a Charpy swing hammer on a piece of core. Another is the Protodyakonov's fall hammer test which is performed on a sample of crushed rock aggregate. This test has been shown to have a correlation to the Specific Energy for rock breaking.

A similar impact test on crushed aggregate is included in the Norwegian set of tests. It was originally developed as a Swedish test for road aggregate. It is described in more detail in the next section. Also this test must be expected to reasonably express the Specific Energy for rock breaking by impact.

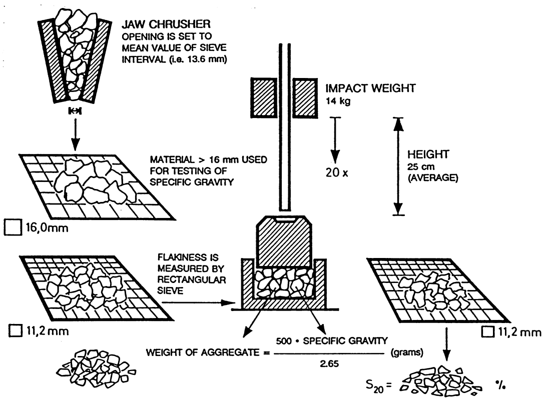

Fig. 2 The impact test to determine the brittleness value.

3.3 Hardness

Hardness has several definitions, but in this context the ability to resist cutting, indentation and/or abrasion may apply. Several tests express the different aspects of hardness. Surface indenting tests such as Brinell and Rockwell are not practical for use on rock samples. Vickers' tests have been used to indicate a combined rock material hardness based on the hardness of each mineral.

Sievers’ miniature drill test was originally introduced as a drillability test for rotary drilling. It is used as the second parameter in the Norwegian set of tests.

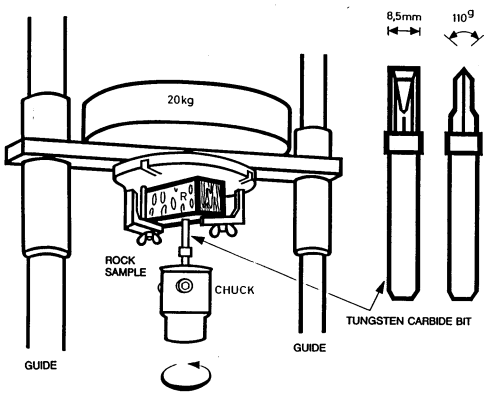

Fig. 3 The Sievers' miniature drill test.

Rebound tests are also considered as hardness tests, e.g. Shore scleroscope or Schmidt hammer. The latter is frequently used as a follow-up tool and is also used for predictions. Caution should be exercised by not relying on this parameter alone. It may have a correlation to compressive strength for some rocks, but the test does not actually break the rock. Combined with strength tests it may serve as a supporting parameter.

3.4 Abrasivity

The cutter edges are exposed to abrasive wear from the intact rock surface, from fragments of rock and from the crushed rock powder.

The Moh’s hardness scale represents a rating of hardness of minerals, expressed as the ability to scratch. The Moh’s hardness of the minerals in a rock will give an indication of whether the rock will be able to abrade cutter steel or tungsten carbide inserts.

For practical predictions, the different simulating tests are more useful to express abrasivity or ability to abrade.

One such test is the Cerchar test, for which the Cerchar abrasivity number is measured as the tip loss of a sharpened steel needle after scratching on the rock surface in a standardised manner. The tip loss expressed as wear flat in tenths of mm, after the needle has moved 10 mm along the surface under an axial weight of 7 kg, gives the Cerchar abrasivity number. The test and the characterisation of the different levels of abrasivity is described in an ISRM suggested standard. The classification is valid for steel. To make predictions of cutter wear, established correlations must be used.

Another test is the Atlas Copco Robbins’ milling test. A steel ball and a piece of cutter steel is milled with rock aggregate for a specified time, and the weight loss is measured. The Los Angeles test, normally used for testing of road aggregate, could be used in a similar manner.

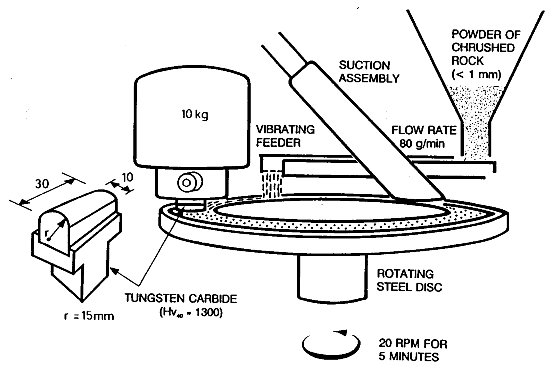

Fig. 4 The Abrasion Value test.

In the Abrasion Value test, which is part of the Norwegian set of tests, crushed rock finer than 1 mm is fed under a test piece on a rotating disk. The test piece consists of tungsten carbide or cutter steel, and the weight loss is measured after a specified time.

3.5 Weakness planes

The effect on tunnel boring of these features together with discontinuities is discussed in a separate paper. The following should however be noted in connection with testing for boreability, as the presence of weakness planes in a rock mass may influence the result of the different tests described above in a different manner for each test.

For example, weaknesses along bedding planes or foliation planes may influence UCS values and Is values differently, depending on the spacing of the planes, size of test specimen and test specimen orientation. Irregularly distributed cracks or flaws in the rock material may also influence various tests differently. The influence may be less for the tests utilising crushed rock than for UCS, depending on the fraction size. The aggregate tests are not orientation dependent. These effects can cause poor correlation between different tests, or between test results and TBM performance. Care needs to be taken considering the possible effects during preparation of test specimens, sampling, testing and comparison of results.

4 NORWEGIAN BOREABILITY TESTING

4.1 Background

The Norwegian set of boreability tests is based on test methods originally developed for prediction of drillability for percussive drilling. This development started in 1955, and the tests have been applied also for tunnel boring since 1972.

In this period, data from tunnel boring projects in a variety of rock types and rock conditions have been collected in Norway and abroad. Results from testing of rock samples and mapping of discontinuities have been analysed and compared with actual TBM performance. Usable correlations have been established allowing predictions of TBM performance, based on geological data and TBM machine parameters, such as thrust, rpm, etc.

A series of updated prediction models have been published by the Department of Building and Construction Engineering at The Norwegian University of Science and Technology, and examples of use discussed in several papers, see literature list.

4.2 Tests and indices

The tests include the following parameters:

- resistance to impact: determined by a drop test on prepared rock aggregate which yields the “brittleness value” S20.
- surface hardness or resistance to indentation or grinding: determined by the Sievers' miniature drill test, which yields the Sievers' J-value SJ.
- abrasiveness of the rock on the bit or cutter material simulating the wear caused by the rock fines on the cutter edge, performed on tungsten carbide giving the Abrasion Value (AV) or on disc cutter steel giving the Abrasion Value Steel (AVS).

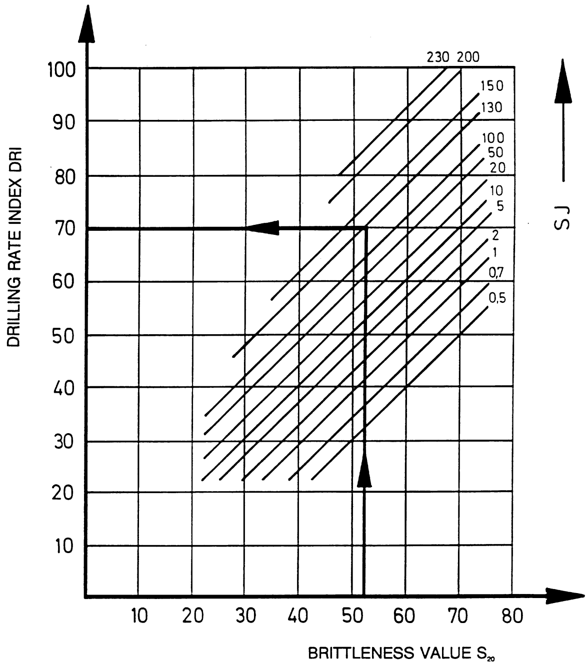

Fig. 5 Diagram for calculation of DRI

The results from the first two tests determine the Drilling Rate Index (DRI) and the results from the last two the Cutter Life Index, as

CLI = 13.84 x (SJ/AVS)*0.3847.

These two parameters are used for TBM predictions.

The Bit Wear Index (BWI), determined from DRI and AV, indicates bit wear for percussive drilling and are used as a supplement. The quartz content is usually determined by differential thermal analysis, and used as a supporting parameter.

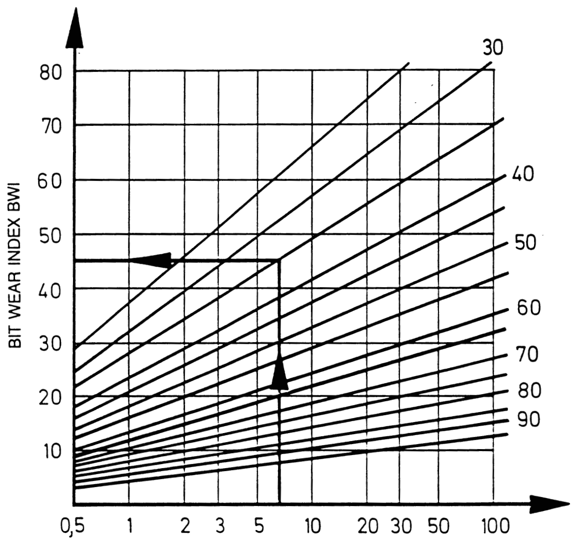

Fig. 6 Diagram for calculation of BWI

The impact test (S20) distinguishes “tough” from “brittle” rock, as both strength and elastic properties influence the result. This is considered as the main reason for the established useful correlations.

For some rocks, a rough correlation can be found between UCS and DRI, but this cannot be expected in general. For example, two rocks with the same UCS values may have different toughness due to different elastic properties, thus having different DRI values and also different TBM performance.

A modified version of the impact test has been developed, in order to make it possible to perform simplified tests on smaller samples.

A rough correlation can be found between Cerchar values and the Cutter Life Index (CLI), but this must only be used for comparison purpose, not as a replacement for direct testing.

4.3 Drillability catalogue

A catalogue is available for the drillability tests performed at the Department of Geology and Mineral Resources Engineering, Norwegian University of Science and Technology, listing results for practically all the tests performed since the start of such testing. The last revision from 1990 contains results from approximately 1,300 rock samples.

The results are listed according to rock types, determined mostly by petrographic visual inspection, normally supported by differential thermal analysis for determination of quartz content, and sometimes by X-ray diffraction for other minerals.

The catalogue contains descriptions of the test methods and diagrams for calculation of the different indices. A characterisation of the DRI and BWI values, from "extremely low" to "extremely high", considering Norwegian rock types, and distribution histograms are included. Relations between some of the parameters are also shown.

The catalogue allows comparisons between previously experienced results, and illustrates the often large variations in mechanical properties that may occur within one rock type. It is emphasised that the catalogue must be used with care when drillability or boreability of specific rocks is evaluated. The catalogue cannot substitute laboratory testing of samples from the actual project.

4.4 Sampling

Sampling of rock for boreability testing, should always be based on geological mapping, supported as necessary by core drilling. The number of samples must be determined by consideration of the occurring number of rock types and the variation in properties for each type.

In order to achieve representative samples, care should be taken to:

- consider petrographic variations, as some darker rocks contain minerals making them tougher.
- consider grain size, as fine grained rocks frequently are stronger than coarse grained rock of the same type.
- avoid weathered samples, unless there is assurance that weathered rock is present at tunnel level, and therefore should be included. Always consider the possibility of the effect of weathering on test results even if samples appear "fresh".
- include not only "typical" samples, thought to be representative for average conditions, but also samples of "extreme" varieties, as they may have a significant impact on the TBM performance, thus determining the feasibility of boring with a specific TBM.

In case the rock structure or texture shows large variations over short distances, samples can be taken from each variety or composed by adding together hand or core pieces in a representative mixture.

Each sample should consist of blocks, or pieces of cores, to give a total of minimum 10 kg, normally 20 kg. Core samples must be larger than 32 mm, but preferably larger than 50 mm since crushed aggregate produced from cores will contain some pieces with partly smooth core surface. This influences the test results, and introduces the need for corrections.

The sample should be accompanied by information about project name, location and orientation of sample, date and purpose of testing.

5 OTHER TEST METHODS

5.1 Indentation tests

Indentation tests in a larger scale than the surface hardness tests have been developed and used in many variations. These range from indentation of buttons to segments of discs. The purpose is to simulate the effect of the tungsten carbide inserts utilised on percussive and rotary drill bits, conical roller discs or disc cutters, including variations in tool edge shape. Atlas Copco uses a "Stamp Test" by indentation of a spherical button for rock drillability classifications.

For testing with segments of disc cutters, the loading velocity is usually so low that there is no impact effect, thus only the static strength has an effect. The test can simulate both the crushing under the tool edge and the chipping to the side. More realistic values are achieved if the indentation is repeated with a spacing, thus including the chipping between the grooves.

Normally the force/penetration relationship is recorded and the force level at first penetration and chipping is considered as indicative of critical thrust.

It is important that the sample is confined by high strength casting in a conical steel cylinder or box. Otherwise the samples may split for a low force. Depending on the size of the tool, the test can be demanding with respect to preparation work and testing equipment. It is therefore more useful for comparative testing, rather than for routine boreability testing.

5.2 Linear and rotational tests

The Excavation Engineering and Earth Mechanic Institute at Colorado School of Mines (CSM), developed the full scale linear cutting tests as a tool for establishing boreability prediction formulas. It has also been used for predictions of TBM performance, by testing with different types of cutters and thrust levels.

Typical block size needed for testing is 1 m x 1 m x 0.5 m, although samples down to 0.6 m x 0.6 m x 0.3 m can be used. Tests have been performed on cores of 150 mm diameter, cut to prisms and cast side by side to provide a sample of sufficient size.

A 1.8 m diameter rotation cutting rig, allowing simulation of cutting in circular grooves has also been developed by CSM. This test needs four rock blocks of approximately 1 m x 1 m x 0.8 m.

Such tests give results useful for predictions of net penetration in massive rock. Due to the need of large rock samples the use will be limited to special cases.

Other research institutions, and manufacturers, have also utilised linear or rotational cutting for research and development objectives or for specific project prediction testing.

6 CONCLUSION

6.1 Use of boreability tests

Tunnel boring machines have become more powerful over the years, both with respect to thrust and torque capacity. Still the performance is strongly dependent on the rock boreability.

Therefore boreability testing has a useful purpose on the planning stage for comparisons between Drill & Blast tunnelling and tunnel boring, on the tendering stage and during the follow-up for contractual purpose or experience analysis.

When boreability test results and indices are included in tender documents, the purpose has to be clearly stated, and the representativeness should be addressed, especially if the number of samples is low.

Sometimes boreability test results and indices have been included in the tender documents, and used as a specified basis for quantified boreability classes. The extent of these classes are then logged during construction and payment adjusted accordingly. This approach has been useful in cases where risk sharing is an integral part of a purchase agreement or a construction contract.

It must however be realised that the performance of a TBM is also very dependent on the amount of weakness planes and discontinuities in the rock mass, and upon how it is operated. Too elaborate contractual regulations according to rock boreability alone can therefore give unforeseeable effects, so caution must be observed.

Today, contractors experienced in tunnel boring will be able to assess the risk of performance in most rock types, provided that relevant results from thorough investigations and testing are presented for their evaluation. For difficult conditions, a boreability evaluation report can be a useful supplement to the test results.

6.2 Development

Because of the large variation in boreability between and within different rock types, as well as the dominant effect of weakness planes and discontinuities on TBM performance, it is important that boreability tests are simple and inexpensive, to allow the tests to be performed in large numbers.

Research is ongoing for the Norwegian set of tests, in order to make it possible to perform impact tests on smaller samples. Such tests will give a first impression, before a test programme on full size samples can be performed. The small sample testing will also ease sampling for follow up purposes.

It must be expected that there will be an increased need to compare results between different test methods. Such comparisons will be useful for the exchange of experience, but one has to be realistic about the general validity of any correlation established only for a few rock types.

In total, boreability testing will continue to form an inherent part of the preparation for tunnel boring and the follow up of TBM performance, of special importance for hard and abrasive rocks.

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3 PREDICTION MODEL FOR PERFORMANCE AND COSTS

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ABSTRACT : Prediction models for excavation rates and costs of tunnelling are used for several purposes, e.g. time planning, cost estimates, tendering, budgeting and cost control, and of course choice of excavation method. This paper treats the pre-diction model for hard rock tunnel boring developed by the Norwegian University of Science and Technology. The model estimates penetration rate, cutter consumption and excavation costs based on rock mass and machine parameters.

1 INTRODUCTION

The complete prediction model is published in /1/, which is the latest revision of a series of prediction models on TBM tunnelling published by the Norwegian University of Science and Technology since 1976. The current model is based on data from 230 km of tunnels, partly in very demanding rock conditions. The engineering geology of the tunnels has been carefully mapped and production and cost data have been analysed.

The step by step model estimates

- Net penetration rate (m/h)
- Cutter life (h/cutter, sm^3/cutter)
- Machine utilisation (%)
- Weekly advance rate (m/week)
- Excavation costs (NOK/m)

The prediction model also makes it possible to analyse the effect of variation in one or more factors on penetration rate, machine utilisation and excavation costs.

2 ROCK MASS PARAMETERS

Through the input parameters, the following rock mass properties are considered:

- Degree of fracturing by Fracture Class and the an gle between the tunnel axis and the planes of weakness.
- Drillability by the Drilling Rate Index DRI, see also Chapter 2.
- Abrasiveness by the Cutter Life Index CLI and the quartz content, see also Chapter 2.
- For some rock types, porosity is also included.

Of the above mentioned parameters, the rock mass fracturing is by far the most important. The estimated penetration rate (m/h) is increased by a factor of five from homogenous to well fractured rock mass. For homogenous rock mass, estimated penetration rate will increase by a factor of two from extremely low to extremely high DRI values.

Since the net penetration rate is an important factor for the weekly advance rate, the cutter life and the excavation costs, it is obvious that great efforts should be made to establish a best possible model of the rock mass fracturing for a tunnel project.

3 MACHINE PARAMETERS

The estimation model uses the following machine parameters:

- Average cutter thrust
- Average cutter spacing
- Cutter diameter
- Cutterhead RPM
- Installed cutterhead power

For boring in hard rock, the average cutter thrust (kN/cutter) is the most important machine parameter. Hence, the development has concentrated on larger cutters to be able to sustain the required thrust. In hard and homogenous rock masses, a High Power TBM (483 mm cutters) will typically have a penetration rate (m/h) that is 40-50 % higher than that of a standard TBM (432 mm cutters)

At the time being, the limiting factor for boring in hard rock is the material quality of the ring steel of the cutters. Further improvement of penetration rate and cutter life in hard rock conditions should be focused on ring material quality and cutterhead design.

4 NET PENETRATION RATE

The basic philosophy of the prediction model is to combine the decisive rock mass parameters into one rock mass boreability parameter, the equivalent fracturing factor kekv, and the relevant machine parameters into one machine or cutterhead parameter, the equivalent thrust, Mekv. The reason for this approach is that the breaking of rock is an interaction between the rock face and the cutterhead.

The equivalent fracturing factor is found by combining

- Rock mass type of systematic fracturing
- Rock mass degree of systematic fracturing
- Angle between planes of weakness and the tunnel axis
- Rock strength (drillability) expressed by DRI

The systematic rock mass fracturing is divided into fissures or joints. Joints are continuous, i.e. they may be followed all around the tunnel contour. One example is bedding joints in granite. Fissures are non-continuous, i.e. they can only be followed partly around the tunnel contour. A typical example is bedding planes in mica schist. The degree of fracturing is divided into classes for practical use when mapping.

Fracture Class (Sp=Joints/St=Fissures)	Distance between Planes of Weakness [mm]
0	-
0-I	1600
I-	800
I	400
II	200
III	100
IV	50

Table 1 Fracture classes for systematic fractured rock mass.

The nature is of course not as simple as the classification above can express. Sometimes, continuous joints predominate, at other times bedding plane fissures or foliation planes are more dominant. In schistose rock, it may be difficult to distinguish between the schistosity of the rock and fissures along the schistosity planes. Hence, one has to simplify and use sound judgement to avoid making too complicated models.

The angle between the tunnel axis and the planes of weakness is calculated as follows:

$$\alpha = \arcsin(\sin\alpha_f \cdot \sin(\alpha_t - \alpha_s))$$

α_s = strike angle of weakness planes
α_f = dip angle of weakness planes
α_s = tunnel of axis direction

If more than one set of weakness planes are included in the model (maximum 3 recommended), the total fracturing factor is

$$k_{s-tot} = \sum_{i=1}^{n} k_{si} - (n-1)\cdot 0.36$$

k_{si} = fracturing factor for set no. i
n = number of fracturing sets

The equivalent fracturing factor is

$$k_{ekv} = k_{s-tot} \cdot k_{DRI}$$

k_{DRI} = correction factor for DRI ≠ 49

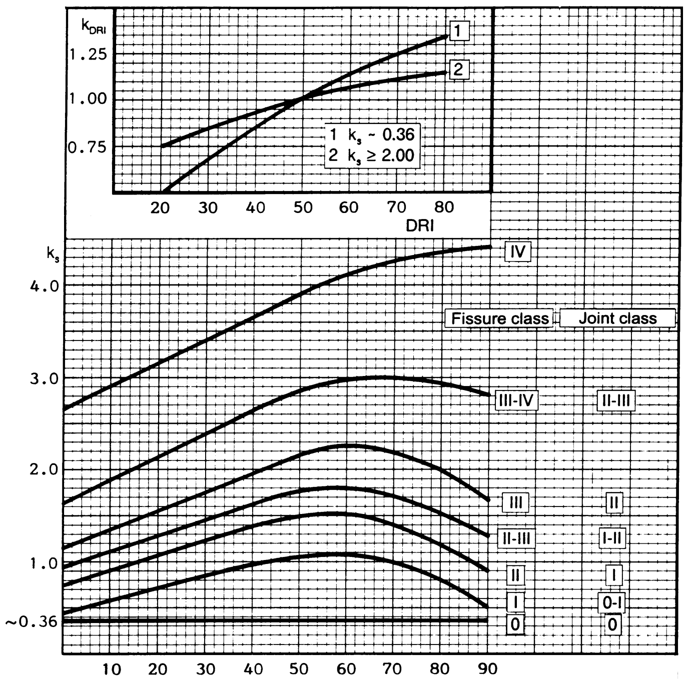

Figure 1 Fracturing factor.

The basic penetration rate is expressed in mm per revolution of the cutterhead. Hence, the decisive machine characteristics are independent of the TBM diameter. The equivalent thrust is expressed as

$$M_{ekv} = M_B \cdot k_d \cdot k_a$$

M_B = gross average thrust per cutter, i.e. not the available thrust capacity of the machine, but the actual thrust (to be) used.

k_d = correction factor for cutter diameter; an indirect expression of the contact area under the cutter.

k_a = correction factor for average cutter spacing.

Average cutter spacing is found by dividing the cutterhead radius by the total number of cutters on the cutterhead.

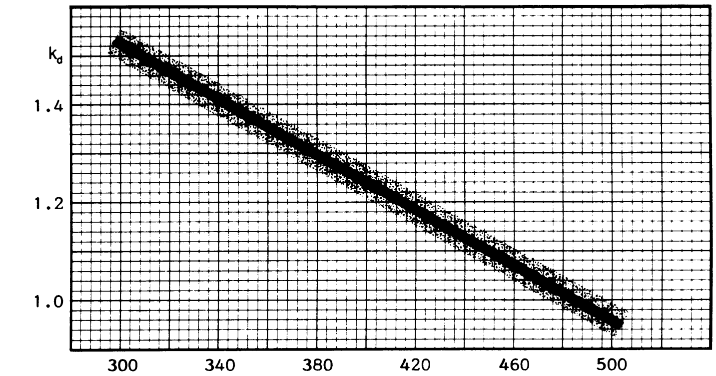

Figure 2 Correction factor for cutter diameter.

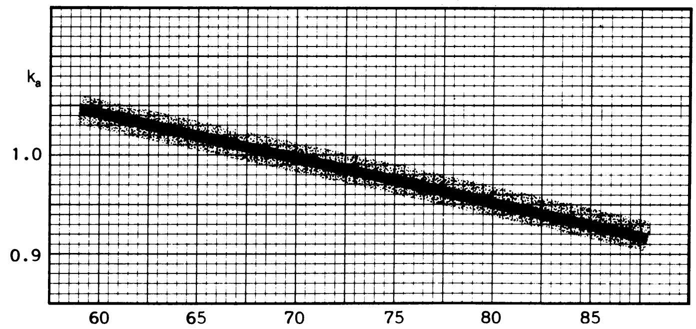

Figure 3 Correction factor for average cutter spacing.

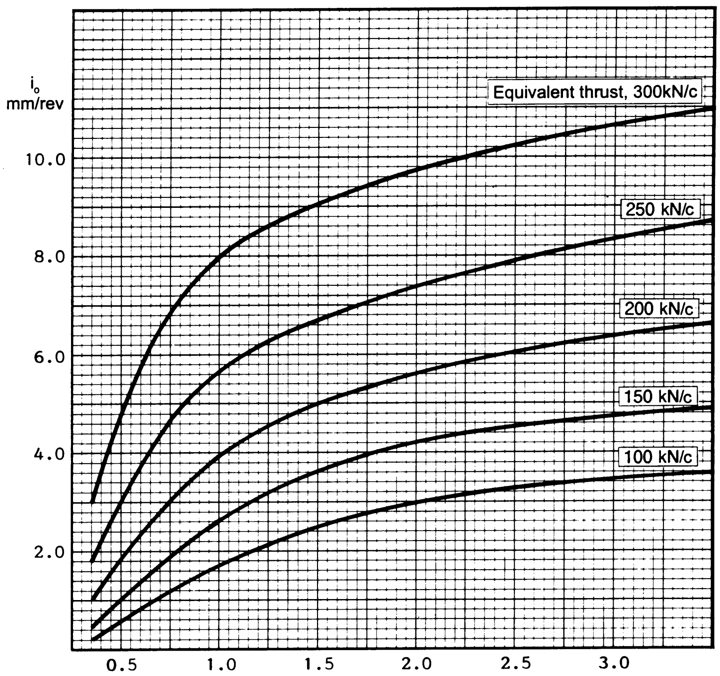

Figure 4 Basic penetration rate.

The estimated basic penetration rate should be checked with regard to the torque capacity of the cutterhead drive system. A detailed model for that purpose is presented in /1/. Net penetration rate in m/h is found by

$$I = i_0 \cdot RPM \cdot \frac{60}{1000}$$

RPM = cutterhead revolutions per minute

5 ADVANCE RATE

The gross advance rate is given as bored metres per week as an average for a longer period, and depends on net penetration rate, machine utilisation and the number of working hours during one week. The machine utilisation is net boring time in per cent of total tunnelling time. The total tunnelling time includes

- boring, T_b
- regripping, T_t
- cutter change and inspection, T_c
- maintenance and service of the TBM, Ttbm
- maintenance and service of the back-up equipment, T_{bak}
- miscellaneous activities, T_a
  - waiting for transport
  - laying and maintenance of tracks or roadway
  - water, ventilation, electric cable(s)
  - surveying
  - cleaning
  - normal rock support in good rock conditions
  - other (travel time, change of crew, etc.)

In addition to the listed items, miscellaneous includes time loss connected to the tunnelling method and organisation, and unforeseen time consumption.

The time consumption is estimated in h/km, and the machine utilisation is given by

$$u = \frac{100 \cdot T_b}{T_b + T_t + T_c + T_{tbm} + T_{bak} + T_a}$$

$$T_b = \frac{1000}{I}$$

$$T_t = \frac{1000 \cdot t_{tak}}{60 \cdot l_s}$$

l_s = stroke length, typically 1.5 – 2 m

t_{tak} = time per regrip, typically 4 – 5 minutes

$$T_c = \frac{1000 \cdot t_c}{60 \cdot H_h \cdot I}$$

t_c = time per changed cutter, typically 45 - 60 minutes

H_h = average cutter ring life

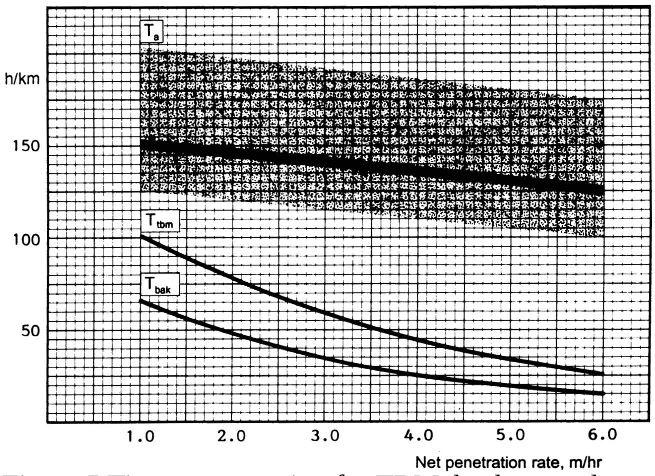

Figure 5 Time consumption for TBM, back-up and miscellaneous.

The time consumption in Figure 5 is based on 101 h/week. If the working hours per week is considerably larger (or smaller) than that, one should evaluate the estimated machine utilisation.

For the total tunnelling time, extra time must be added for

- assembly and disassembly of TBM and back-up
- excavation of tip stations, niches, branchings, etc.
- rock support in zones of poor quality
- time for unexpected rock mass conditions
- complementary rock support and lining
- major TBM breakdowns (e.g. main bearing failure)
- dismantling of tracks, ventilation, cables, etc.
- invert clean-up.

6 CUTTER CONSUMPTION

Cutter ring life is mainly dependent on the following:

Rock Mass Properties	Machine Parameters
Cutter Life Index, CLI	Cutter diameter
Rock content of abrasive minerals, represented by the quartz content	Cutter type and quality
	Cutterhead size and shape
	Cutterhead rpm
	Number of cutters

Table 2 Cutter life parameters.

The average life of cutter rings is given by

$$H_h = \frac{H_0 \cdot k_D \cdot k_Q \cdot k_{RPM} \cdot k_N}{N_{tbm}} \text{ (h/cutter)}$$

H_0 = basic cutter ring life

k_D = correction factor for TBM diameter

k_Q = correction factor for rock quartz content

k_{RPM} = correction factor for cutterhead rpm

k_N = correction factor for number of cutters

N_{tbm} = number of cutters on the cutterhead

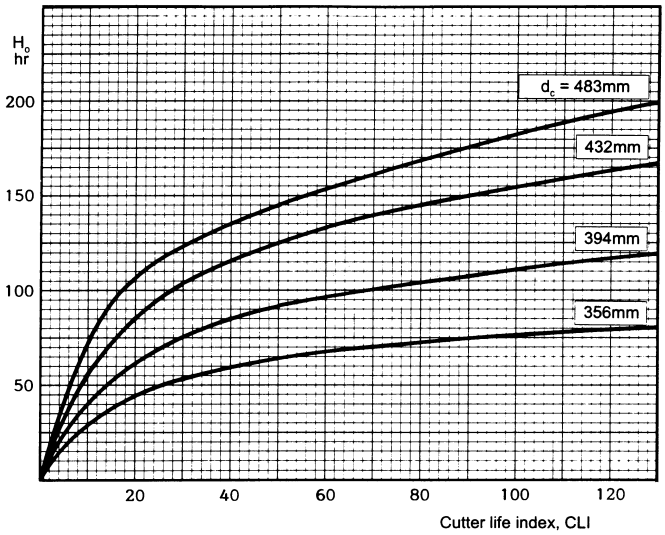

Figure 6 Basic cutter ring life.

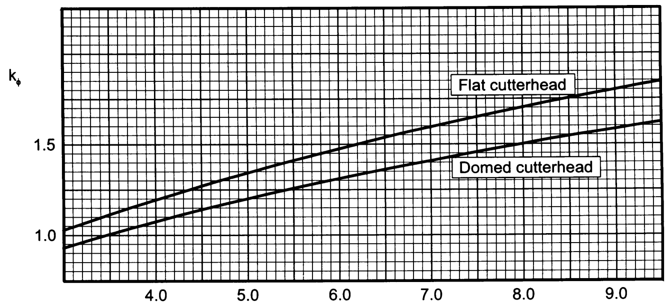

Figure 7 Correction factor for TBM diameter and cutterhead shape.

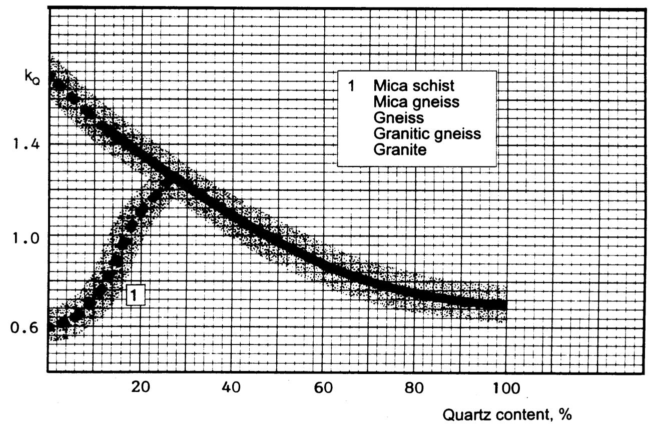

Figure 8 Correction factor for quartz content.

$$k_{RPM} = \frac{50 / d_{tbm}}{RPM}$$

$$k_N = \frac{N_{tbm}}{N_0}$$

d_{tbm} = TBM diameter

N_0 = normal number of cutters on the TBM

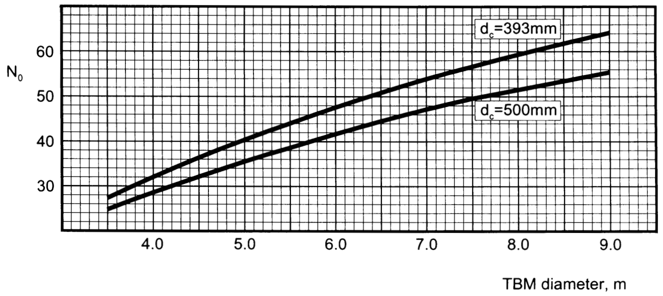

Figure 9 Normal number of cutters on a TBM.

7 NORMALISED EXCAVATION COSTS

The normalised costs are an evened and normalised summary of detailed component cost estimates. The excavation costs comprise

- costs for assembly and disassembly of the TBM and back-up equipment
- capital costs, maintenance, etc. of the TBM and the back-up equipment
- cutter costs
- muck transport, ventilation, electrical installations, water supply
- labour costs
- additional costs related to declined adit (if existent)

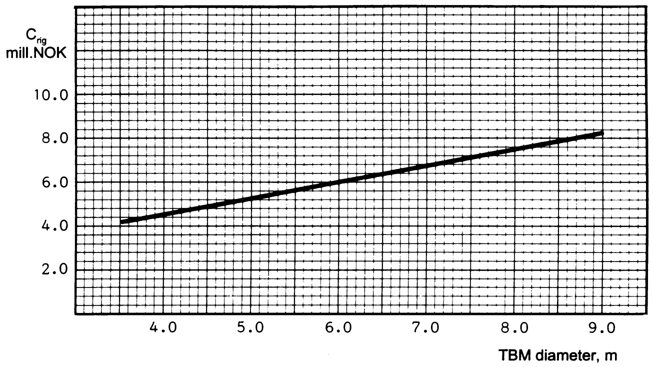

Figure 10 Costs for assembly and disassembly of the TBM and back-up equipment

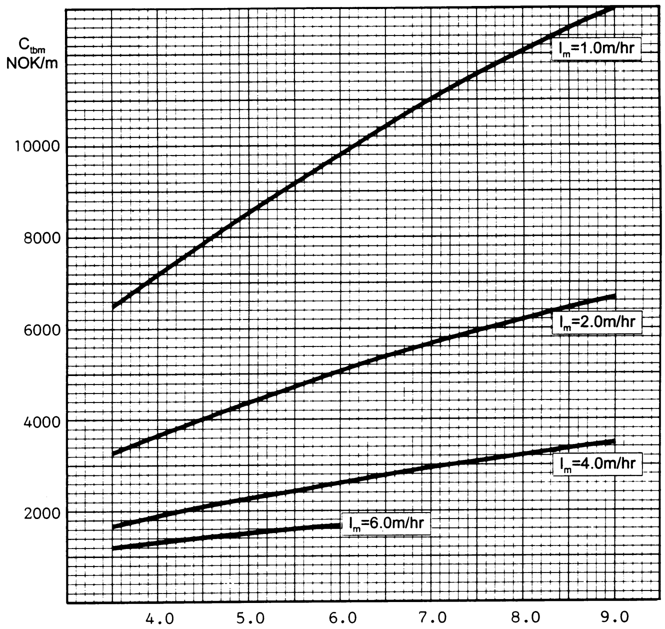

Figure 11 TBM costs.

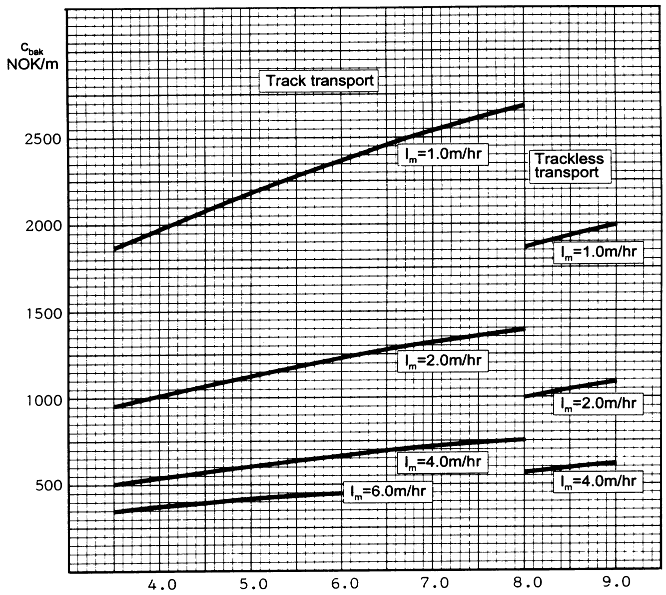

Figure 12 Costs for back-up equipment.

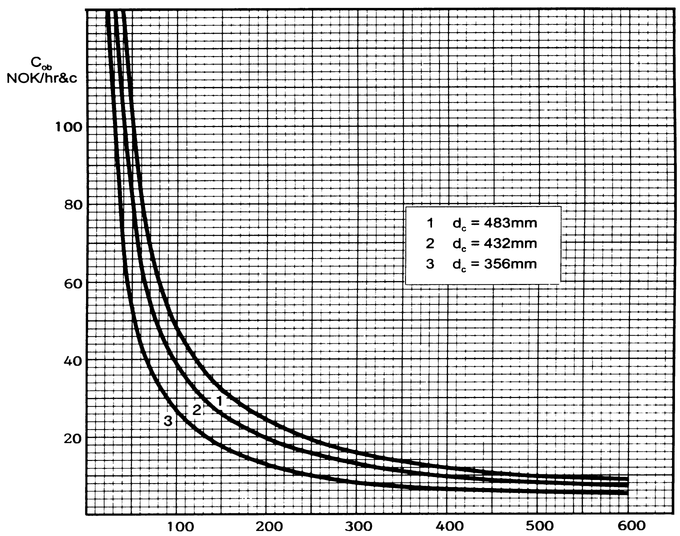

Figure 13 Basic cutter costs.

When using Figure 13, the cutter life on the x-axis is

$H_t = H_h \cdot N_{tbm.}$

$C_b = C_{ob} \cdot \frac{N_{tbm}}{I}$

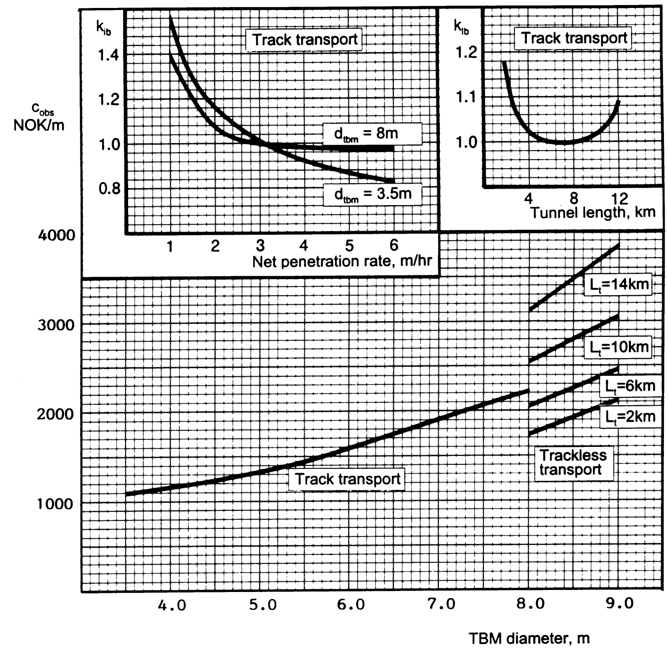

Figure 14 Basic costs for work behind the face, correction factors for net penetration rate and tunnel length.

$$c_{bs} = c_{0bs} \cdot k_{ib} \cdot k_{lb}$$

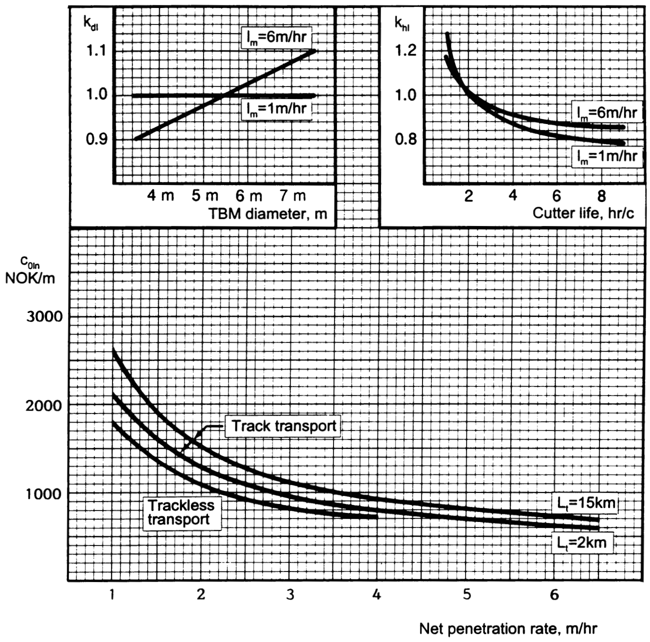

Figure 15 Basic labour costs, correction factors for TBM diameter and cutter life H_h.

$$c_{ln} = c_{0ln} \cdot k_{dl} \cdot k_{hl}$$

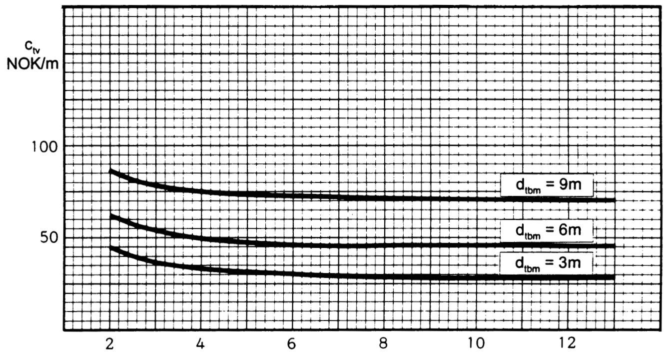

Figure 16 Additional costs for declined adit.

$$c_{sum} = c_{rig} + c_{tbm} + c_{bak} + c_b + c_{bs} + c_{ln} + c_{tv}$$

Unforeseen costs are not included in C_{sum}. At least 10% should be added to cover such costs.

REFERENCES

/1/ The University of Trondheim, NTH-Anleggsdrift: Project Report 1-94 HARD ROCK TUNNEL BORING, Trondheim 1994, 164 pp.

4 DEVELOPMENT OF TBM TECHNOLOGY FOR HARD ROCK CONDITIONS

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ABSTRACT: This paper gives a brief history and development of hard rock Tunnel Boring Machines (TBMs). Since the early 1970's, some sixty tunnel projects have been completed by open, hard rock TBMs in Scandinavia, and all but ten of these projects have taken place in Norway. Atlas Copco Robbins TBMs have played a major role in this development and have been used on approximately 85 percent of the 300 km driven. This paper provides basic description and operating principles of the modern open, hard rock Robbins Main Beam and Main Kelly type machines, and reviews the current state of the art main bearing arrangement and cutters. A listing of important geological, tunnel design and machine factors to be reviewed during a TBM project study is also included.

1 BRIEF HISTORY OF TUNNEL BORING MACHINES

In 1851 an American Engineer Charles Wilson developed a tunnelling machine which is generally considered as the first successful continuous borer for rock. However, problems with cutter technology combined with other mechanical difficulties made it not competitive with the rapidly developing techniques of Drill & Blast tunnelling. Though Mr. Wilson's concept of using disc cutters turned out to be correct, it took almost a century before his ideas were developed and put into use.

Other famous undertakings include the compressed air driven TBM developed by Colonel Beaumont in 1881 for an exploratory tunnel under the English Channel. This machine was fitted with a massive head with bits designed to carve concentric rings in the chalk. This 2.1 m diameter machine may be considered as rather primitive today, but it bored more than 1.8 km with daily advance rate of 24.5 m! In 1882 all work on the English side was cancelled due to political pressure.

Practically no serious attempts were made until 1952 when James S. Robbins designed a TBM to be used for four tunnels at Oahe Dam in South Dakota. This unit was 7.85 m in diameter and with a cutter head consisting to two counter rotating heads - an inner and an outer section. The cutterhead was fitted with fixed carbide drag bits radially arranged, and parallel rows of freely rolling disc cutters, which were protruding slightly less than the carbide drag bits. It is of interest to note that this machine was powered by 2-150 kW motors and had a total weight of approximately 114 t. While the machine was not designed for use in hard rock, it successfully bored through soft shale advancing up to a world record of 45 m/day. 356 drag bits and four disc cutters were reportedly replaced during the four tunnel drives.

The first successful hard rock TBM was built in 1956 for the Foundation Company of Canada for use on the 4.5 km long Humber Sewer Project in Toronto, Canada. This 3.28 m dia. Robbins TBM (Model 131-107) was contracted for boring through sandstone, shale and crystalline limestone - UCS reported from 5-186 MPa. The TBM was designed and equipped with drag bits as well as cutters to cut the rock. During the initial boring period, it was decided to try removing the high wearing drag bits, leaving only disc cutters on the single rotational head. This experiment turned out to be a success, and it became the accepted concept for using disc cutters exclusively in hard rock conditions.

Since the mid 1950's the development of TBMs has progressed in two directions:

- Enable machines to bore tunnels in massive, hard and abrasive rock.
- Enable machines to bore tunnels in stable, competent rock as well as in ground so unstable that the tunnel has to be lined concurrently with the excavation.

This paper deals with the hard rock open type TBMs.

Detail follow-up of more than 300 km hard rock TBM tunnels by Norwegian contractors, consultants and universities have played a major role in the understanding and prediction of the rock cutting process and the developments in the TBM technology.

2 HARD ROCK - DEFINITION

A definition of "hard rock" can easily lead to controversy. Common definitions include:

- Unconfined compressive Strength (UCS) exceeding approximately 50-100 MPa.
- A mineral matter that cannot economically be excavated by a road header.
- Something hard, consolidated and/or load bearing, which, where necessary, has to be removed by blasting.
- A rock sample that requires more than one blow by a geology hammer to split.
- Metamorphic and igneous rocks (i.e. not sedimentary rock).

For the purpose of this paper hard (strong) rock is defined according to ISRM (1980) with UCS exceeding 50-100 MPa.

2.1 Important factors for hard rock tunnelling systems
A. Geological factors:

- Rock mass strength and elastic properties
- Rock types and rock abrasiveness
- Degree and type of jointing and/or fissuring of the rock mass
- Rock porosity and friability
- Dip, strike and direction of drive
- Over burden and in situ stress
- Faults
- Stand up time
- Water inflow
- Possibility of gas
- Ambient rock temperatures

B. Tunnel Design factors and requirements

- Alignment
- Gradients in the direction of drive
- Rock support requirements - bolts, partial or full ring beams, welded wire mesh, steel straps, shotcrete, in situ concrete, concrete segments or steel liner plates
- Tolerance requirements
- Ground pressure limitations
- Probing and pre-excavation injection requirements
- Assembly and start-up schemes
- Contract schedule including excavation period
- Possibility of more than one heading
- Power supply
- Time schedule
- Environmental issues

C Machine design factors - based upon above items A and B

- Cutter capacity, diameter, tip width, spacing, protrusion and cutterhead profile.
- Cutter attachment (cutter housing) type
- Cutter changing (front loading and/or backloading features)
- Cutterhead stiffness and stability
- Cutterhead rpm
- Mucking capacity and systems
- Main bearing and seal assembly
- Cutterhead drive system.
- Power and torque requirements for balanced design
- Hydraulic and lube system requirements
- Electrical system requirements
- Structural integrity
- Thrust and torque reaction systems
- Rock support installations
- Simplicity and ease of operation
- Safety of personnel
- Interphase with back-up system and ancillary tunnel equipment
- Capital cost, estimated production rates, cutters and spare parts costs to fit budget and production schedules.

  Some of the important machine design factors are specifically addressed below:

3 CUTTERS

The hard rock tunnel boring is a form of crushing and chipping of rock with disc cutters applied against the rock face with brute force. The cutters will penetrate a certain depth into the rock face for each revolution depending upon the machine and cutter characteristics and the geological factors. TBM rate of penetration (ROP) is typically defined in mm per revolution of the cutterhead.

In the contact area between the cutter tip and the rock, the rock is crushed to powder. From this zone, cracks will propagate towards the neighbouring groove (kerf) and rock face will spall in a combination of chips and fines. The actual cutter indentation determines the depth of the crack formation and it is a function of the force applied to the cutter and the cutter foot print. For each rock type, a minimum thrust will typically be required to indent the rock effectively. If the spacing between the grooves are too large, the stress pattern created may not reach the influence zone caused by adjacent cutters. This will cause inefficient boring and low rate of penetration. On the other hand if the spacing is too close or the cutter tip is too blunt, only small chips will form, generally meaning inefficient boring and low overall rate of penetration.

The thrust force, cutter type and cutter spacing are therefore among the most important factors in the TBM evaluation. These factors will in turn affect the design and price of a TBM and play a major role in the overall project schedule and cost.

The thrust capacity of the individual cutters may be limited by one or more factors including bearing life, quality of the cutter ring steel and the attachment to the cutterhead. Some thirty years ago the state of the art was to use 12" (305mm) cutters with maximum recommended average cutter loads of approximately 89kN/cutter. Feedback from actual projects, Robbins research and development, as well as competition from steady improvements of the Drill & Blast tunnelling methods, have moved the cutter technology through a dramatic evolution from 11", 12", 14", 15½", 16½", 17", 19" and up to 20" (500 mm) cutter assemblies with average individual rated cutter loads increasing steadily from 80 through 133, 178, 222, 245, 267 and 311 kN respectively. Individual cutter impact loads of three to five times the average rated load on hardrock TBMs have been recorded by use of cutter instrumentation on special Robbins - customer research projects in Norway and Australia.

The development and introduction of the 17" (432 mm) cutter in the early 1980's was an especially important step forward. It enabled machines to operate successfully and economically at 222 kN/cutter rated loading over long drives in massive, gneiss and granitic formations. Today, ACR offers TBMs fitted with 17" (432 mm) and 19" (483 mm) cutters rated at 267 kN and 311 kN/cutter respectively. The machines with 19" cutters - High Performance (HP) TBMs - are considered state of the art for hard and massive rock conditions, with proven experience and results, first from hydro project tunnels in Norway, then followed by projects in Hong Kong, Saudi Arabia, South Africa and USA. It should be noted, however, that TBMs with 17" cutters may still be preferred and recommended solutions where backloading cutters are required and, when tunnel cuts through mixed rock mass conditions and geology. A lower cutter loading combined with a closer spacing may result in a higher ROP and lower cutter cost.

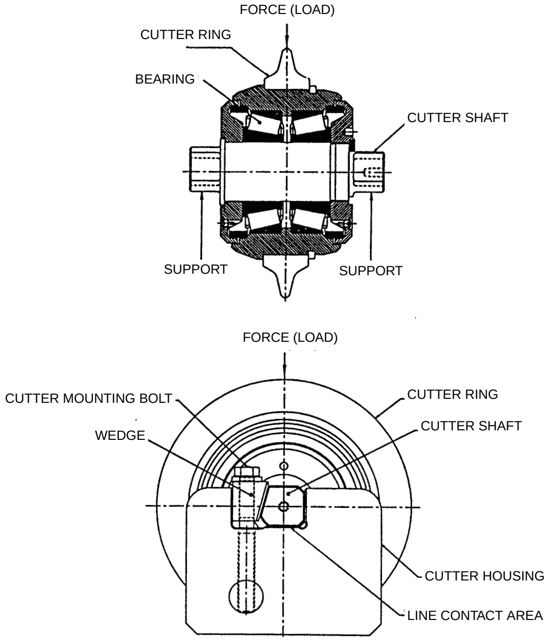


Fig. 1 Modern Wedge Lock Cutter Assembly and Housing

4 MAIN BEARING ARRANGEMENTS

In order to design a suitable main bearing for a TBM it is important to define the various loads that will act on the cutter head This includes using loading conditions determined by geological factors as well as cutter head geometry. The L10 main bearing life is determined for specific application by applying formulae established by the TBM as well as bearing manufacturers using ISO 281. L10 is expressed as the number of revolutions at which the bearing has a 10% statistical probability of failure. The required bearing life for a specific application therefore becomes a function of the cutter head thrust, ROP, the rpm and the tunnel length.

The desired L10 life converted to machine hours should normally exceed the anticipated total TBM operating hours with a factor of 2 or more, provided that physical restraints will allow. Examples of physical restraints include weight and size limitations to transport the main bearing assembly and other large components from the TBM factory to the start-up station at the site.

Several types of main bearing are commonly used in the modern hard rock TBMs of today:

1. Low angle taper roller bearing
2. Steep angle taper roller bearing
3. Asymmetrical taper roller bearing
4. Three axis roller bearing
5. Spherical bearings with front and rear bearings.

The first four bearing types are all normally located in the cutterhead cavity immediately behind the cutterhead, and they may be used in all types of TBMs - open or shielded. The spherical type consisting of a radial bearing on the front and thrust bearings at the rear of the TBM may only be seen used in the larger diameter Robbins Kelly drive TBMs (MK27).

The capacity and cost of the roller bearings generally follows the list above i.e. the highest capacity and the highest cost pertaining to the 3 axis roller bearing. All current Robbins (MB) High Performance TBMs have been fitted with this type main bearing. These main bearings have large diameters up to approximately 70% of the actual tunnel diameter, in order to withstand the high loads and overturning moments.

The spherical bearing type could exceed the L10 life of a 3-axis bearing with a factor of two or more. The longer bearing life (L10) combined with smaller bearing diameters and reduced transport size requirements may thus become important factors in project feasibility studies and machine recommendations on long tunnel drives. All the newer Robbins MK27 TBMs with diameters from 6.5 m to approximately 12.5 m diameter have been equipped with this arrangement.

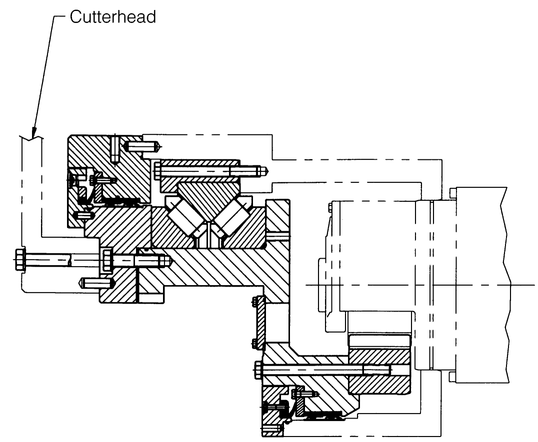


Fig. 2 Typical steep angle taper roller bearing

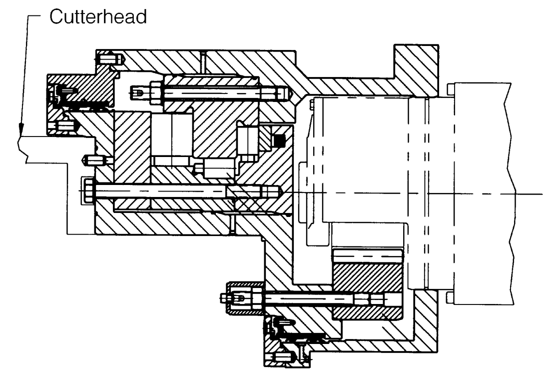


Fig. 3 Typical three axis roller bearing

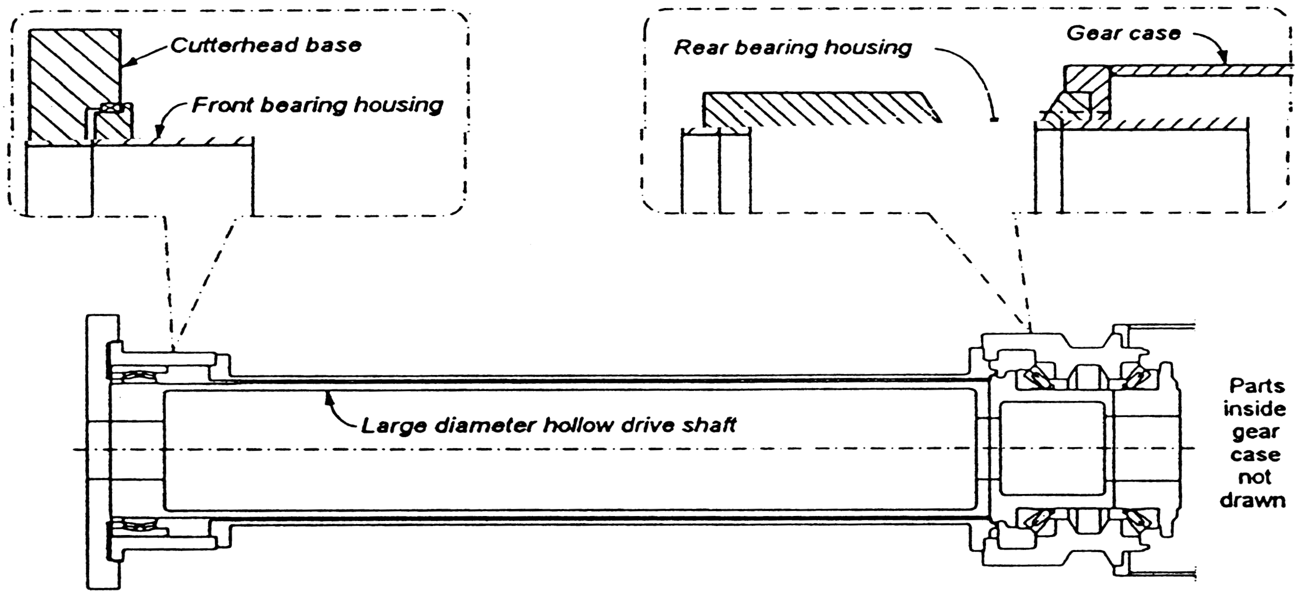


Fig. 4 Typical spherical roller type bearings (MK27)

TBM	Projekt/ Country/ Year	Dia. (m)	Rocktype/ Approx. UCS (MPa)	Max. Recom. Operating Thrust (kN)	Cutterhead Drive (kW)	Cutter No. & Type	Rated Load kN/Cutter
910-101	Oahe Dam/ USA/1952	8.0	Shale/ 1,4-2,8	445	298	(dragbits)	N/A
231-152	Paris Subway/ France/1973	7.0	Limestone/ 150-200	7,050	670	13 - 12" 6 - 11"	134
212-173	Seabrook Tunnel/ USA/1979	6.7	Diorite, Gneiss/ 59-248	10,422	895	48 - 15½" 4 - 12"	200
252-226	Fløyfjell Tunnel/ Norway/1984	7.8	Gneiss, Granitic Gneiss/ 138-241	12,983	1,640	57 - 17"	222
235-282	Queens Tunnel/ USA/1996	7.06	Granite, Gneiss, Rhyolite/ 193-275	15,588	3,150	50 - 19"	311

Table 1 Brief overview of ROBBINS MB TBMs 6.7 to 8.0 m diameter 1952 - 1996

5 TBM OPERATION

The following basic modes of operation are common for today's hard rock. The descriptions include the Robbins main beam (MB) and Kelly (Mk) style TBMs.

5.1 MB Series - Open TBMs of Robbins Origin
The ground work for this design was established in 1952 and it provided the base for all TBMs. The MB TBM is applicable and has successfully been proven in broken ground and medium stand-up time to applications in hard, massive ground. Some 140 units of MB Robbins have bored more than 1,900 km of tunnels as of 1996. TBMs of this design hold a number or world records for tunnel lengths and performance.

5.1.1 Anchoring Section
The anchoring section towards the rear end is equipped with a single pair of horizontal grippers. These carry the weight of the machine's rear end and serve to react thrust and torque during the boring operations.

5.1.2 Working Section
During boring, the working section is propelled forward by the thrust cylinders. The drive units are mounted on the main gear case which is integrated with the cutterhead support which also contains the main bearing which carries the cutterhead. The cutterhead support rests on the invert and is centered and stabilized by the side and roof supports.

5.1.3 Drivepower and Torque
Rotational power is normally generated by electric motors. The motors drive the common ring gear which in turn transmits the torque to the cutterhead. To start the head, the drive motors are brought up to speed running free and then the clutches are engaged.

Thrust is generated by extending hydraulic cylinders acting between the gripper pads and the main beam and is transmitted to the cutterhead via the cutterhead support structure.

The support shoe acts as a fulcrum around which the rear end of the machine can be moved in all directions. The machine may therefore be steered during boring to provide smooth curves and to enhance cutter life. Steering is done by individual control of the grippers and of auxiliary cylinders acting vertically between the anchoring section and the main beam.

5.2 Typical Boring Cycle for a 5 m diameter MB TBM
A normal boring cycle can be described as consisting of the following steps:

5.2.1 Step Description

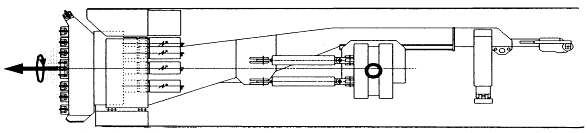
1. At the beginning of the cycle, the anchoring section has been brought forward in relation to the working section, and grippers are extended against the side walls. The TBM has been properly aligned and the rear support legs have been pulled up from the in vert. With the head rotating, the thrust cylinders are energised and advance the working section over the full 1.8m stroke.

2. At the end of the forward stroke the cutterhead rotation is stopped, and the hydraulic operated rear support legs are set against the invert to carry the weight of the rear end of machine.

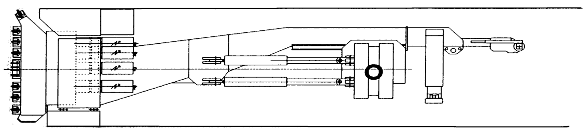
3. With the TBM supported the grippers are retracted, the anchoring section becomes free for resetting which is done by retracting the thrust cylinders.

4. When these are fully retracted, the grippers are pressurized against the rock again. The rear support legs are pulled up from the invert and the machine is aligned for line and grade if necessary.

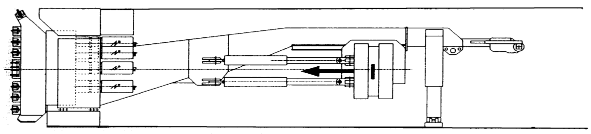
1. Then cutterhead rotation is started once more and the TBM is ready for the next boring cycle



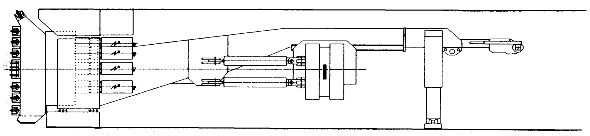
1 Start boring stroke



2 End boring stroke



3 Start reset stroke



4 End reset stroke

O = Gripper out **I** = Gripper in

Fig. 5 A complete boring cycle - step by step

5.3 MK Series - Open TBMs of Jarva Origin

5.3.1 MK as in Kelly

The basic design goes back to 1965. In the Robbins product range, the Jarva design has been designated as the MK Series. This TBM is applicable and has successfully been proven in soft, broken ground conditions with medium stand-up time to hard, massive rock. 53 units have been delivered and have bored more than 515 km of tunnels as of 1996.

5.3.2 Anchoring Section

The anchoring section in the centre of the machine is equipped with two pairs of horizontal grippers. When the grippers are energized against the tunnel walls, they provide the full support of the machine, and also serve to take the thrust and torque reactions.

5.3.3 Working Section

During boring, the working section is moved forward by the thrust cylinders and transmits the torque reaction from the main motors at the rear end of the machine - via the square torque (reaction) tube - to the main body and by the grippers to the tunnel walls.

5.3.4 Torque

Rotational power is normally generated by electric motors. The motors drive the common ring gear which in turn transmits the torque through the drive shaft - located in the centre of the torque tube - to the cutterhead.

5.3.5 Thrust

The thrust is generated by retracting hydraulic cylinders acting between the main body and the gear case. The force is transmitted to the cutterhead by the torque tube.

5.3.6 Steering

As the machine is anchored by four grippers, it is directionally stable. Correction of the tunnel machine's line and grade are therefore done only at reset. The grippers are controlled individually for this purpose.

5.4 Typical Boring Cycle for a 5 m diameter MK TBM

A normal boring cycle can be described as consisting of the following steps.

5.4.1 Step Description

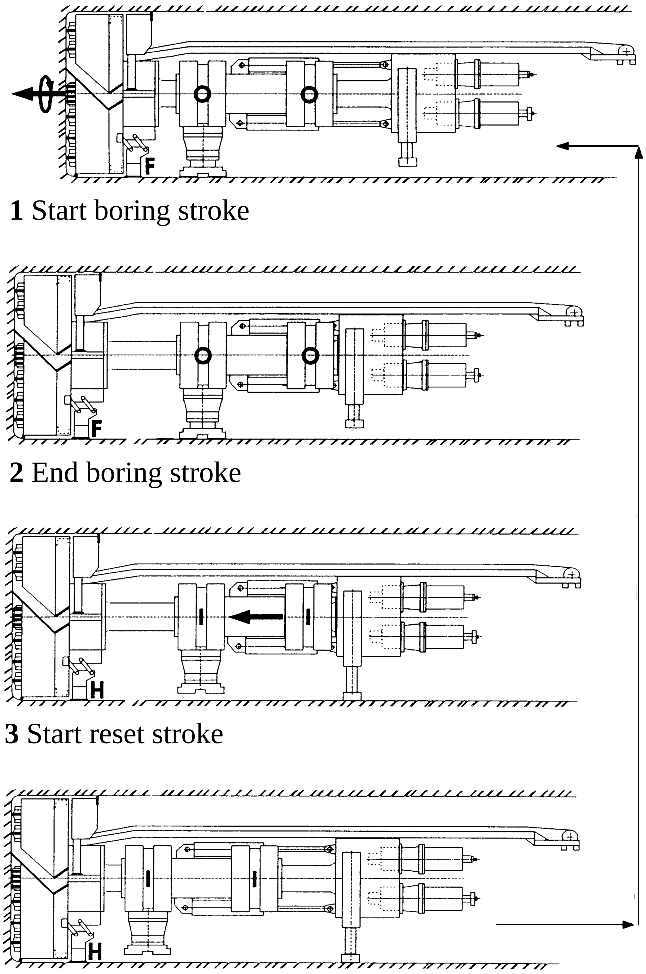
1. At the beginning of the cycle, the anchoring section with grippers are engaged against the tunnel side walls and the front lift leg is in contact with the rock in the invert. The invert scraper is in floating contact with the rock in the invert and the hydraulically operated rear support legs have been pulled up from the invert. With the head rotating, the thrust cylinders are activated to retract and the working section forward the full 1.5 m stroke.

2. At the end of the forward stroke, cutterhead rotation is stopped. The rear lift legs are lowered against the invert, and the invert scraper is brought from the floating mode - which is the normal one for boring - to the supporting mode in order to carry the weight of the front end of the machine.

3. With the TBM thus supported at both ends, the grippers and the front lift leg are retracted. The anchoring section becomes free for resetting which is done by retracting the thrust cylinders.

4. When the cylinders are fully extended once more, the front lift leg is lowered onto the invert. The in vert scraper is brought to the floating mode again, and the TBM is aligned for using the front and rear lift legs. Then the rear grippers are brought into contact with the rock again and the rear lift legs are pulled up. Thereafter the whole machine is set on line by means of the rear grippers. The front grip pers are then extended to anchor the machine in the bore and the rear lift legs are retracted.

1. Then cutterhead rotation is started once more and the TBM is ready for the next boring cycle.



1 Start boring stroke

2 End boring stroke

3 Start reset stroke

4 End reset stroke

F = Invert scraper floating **O** = Gripper out
H = Invert scraper holding **I** = Grippers in

Fig. 6 A complete boring cycle - step by step

5 EARLY TBM PROJECTS

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ABSTRACT: Tunnel boring started in Norway in 1972 in a section of a sewerage tunnel in Trondheim. The first results were not encouraging. The progress was slow and cutter costs were excessive in the tough rock types encountered. Then the TBM was strengthened, mainly by increasing the thrust capacity. The penetration rate improved drastically and the tunnel was completed satisfactorily.

The next tunnel, started in 1974, was the first section of the renewal programme for the Oslo main sewerage system. Tunnel boring was chosen mainly to avoid blasting under the city. The boring itself was successful and was completed in 1976. The method had proven its capability, and within three more years 12 other tunnel boring projects had been started and some even completed.

The tunnelling industry made the results available for analysis and research. Empirically based prediction models were developed, allowing planning and estimates to be based more on thorough preinvestigations and less on guesswork. The basis for a rapid development to the forefront of hard rock tunnel boring was laid.

1 EARLY SHAFT BORING

The first attempt at mechanical excavation of rock in Norway was made by NVE, the Norwegian Power Board, by the raise boring of a hydropower shaft in Vådalen on the Tokke hydropower project. The shaft was 73 metres long, had a diameter of 1 metre and was bored with an inclination of 50 degrees in granitic gneiss. The 195 mm pilot was drilled upwards, and the reaming was performed in several repeated steps with "Søding & Halbach" reamer heads of 305, 406, 610, 813 and 1016 mm diameter. Repeated problems occurred with breakage of the tungsten carbide segmented rings. The boring was completed by cutter rings with tungsten carbide inserts, at a slow rate and relatively high costs.

The conclusion, in spite of the difficulties, was that the method was feasible also in hard rock. Improvements of the machinery were necessary.

2 THE FIRST TBM TUNNEL

2.1 Why tunnel boring?

The first TBM tunnel was a section of the Sluppen to Høvringen main sewerage tunnel in the city of Trondheim. The main reason for choosing tunnel boring instead of the traditional Drill & Blast-method was to avoid rock blasting for the 5 km section situated under residential areas in the suburbs. The rock cover varied from 20 to 60 metres. The client, Trondheim Municipality, preferred tunnel boring considering the risk of damages from blasting vibrations and the cost of extensive vibration control along the alignment, although the bid was higher than for drilling and blasting.

Other expected savings were a reduction in transport costs resulting from lower excavation volume due to the reduced tunnel cross section. Furthermore, a concrete invert deleted, and the rock support expected to be reduced to 50%. The owner shared the cutter costs above a certain level with the contractor.

The first TBM stroke was commenced in the beginning of July 1972 on the northbound face in the Marienborg access.

2.2 Geological conditions

The rock along the tunnel was mostly greenstone deposited as subsea lava flows, therefore frequently exhibiting a pillow structure. In some sections the pillows were flattened and stretched out to a schistose structure with chlorite partings. The tunnel alignment was parallel or had a small angle to the schistosity. Dykes of quartz-keratophyre occurred. Except for the schistosity, the rock had few discontinuities.

The section with 5 km of greenstone coincided with the built up areas. The outer part of the tunnel in quartz-diorite was excavated by drilling and blasting. The soil deposits above the tunnel were mostly moraine and alluvium. The risk of damages due to settlements was small, thus probe drilling and pregrouting was not required.

The geotechnical evaluation report had not recommended tunnel boring, mainly because of the quartz-diorite, but also because of the occurrence of the massive type of greenstone. However, due to the reasons mentioned above, tunnel boring was chosen for the greenstone section. Later it became clear that the contractor's estimates had been based on the manufacturer's testing of samples which included schistose greenstone, but not the massive type.

2.3 The equipment

The contractor, AS Jernbeton Trondhjem, utilised a second hand Demag TVM 20-23H with Söding & Halbach cutters rented from a German subcontractor for a fixed price per meter bored. The subcontractor provided one operator for each 12 hours shift and one repair man.

The 2.3 metres diameter TBM was 17 metres long and the gross weight 55 tons. Installed motor capacity was 220 kW, with 2x90 kW to the cutterhead drive motors. The cutterhead rpm was as high as 14. The thrust capacity was 1200 kN to the cutterhead, providing a maximum of 40 kN per cutter ring. There were a total of 11 cutters, 10 with 3 or 4 rings, and 1 centre cutter with conical rollers. Most of the cutters had steel rings, but a varying number of cutters with tungsten carbide inserts were used depending on the rock conditions. The cutter groove spacing was approx. 40 mm.

Fig. 1. Triple ring cutters, some with tungsten carbide inserts.

The muck was scraped up at the invert of the face and dropped onto a chain conveyer below the TBM. The operator could therefore look at the size of the drill chips from his operator seat. A small amount of water was used for dust suppression and cooling. The transport was done by Hägglund wagons; one was attached to the back-up rig and used as a buffer during boring.

2.4 Costly experience

The first centre cutter, which had steel teeth suitable for soft rocks, lasted only 2 metres. It was replaced by two conical rollers with tungsten carbide inserts, which performed well.

The boring went well with a typical net penetration rate of 1.2 m/hr, varying from 0.6 to 1.8 m/hr. The rock on the first section was a schistose greenstone with a narrow spacing between smooth chlorite partings. After the first 100 metres, the rock became more massive and the net penetration fell to 0.3-0.9 m/hr. On the first 200 metres section the Uniaxial Compressive Strength (UCS) varied from approx. 75 MPa to 220 MPa. The wear on the cutter rings increased because of the presence of epidote and because of the poor chipping between the cutter grooves.

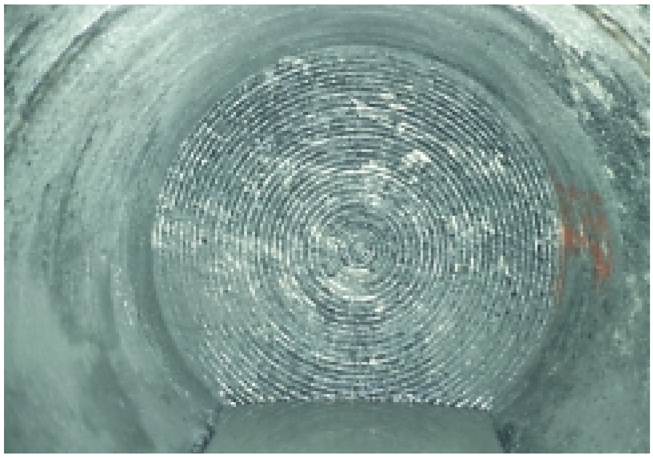

Fig. 2. Tunnel face in massive greenstone

Two quartz-keratophyre dykes were bored through. In the more massive dyke the penetration rate was very low, in the jointed dyke a very good penetration rate was achieved in spite of the hardness.

After another 100 metres of low penetration rates and high cutter consumption, the situation was evaluated. The delivery time for tungsten carbide cutters was several weeks. On the southbound face the rock was schistose and inspections above the tunnel indicated a fair amount of schistose rock further on. It was decided to turn the TBM around, which took 2 weeks.

The first 100 metres southwards went well with up to 16.5 metres progress on a 12 hour shift. Then massive rock was again encountered. While waiting for the delivery of tungsten carbide cutters an experiment was made to blast a couple of short rounds ahead of the TBM. The cracks from the blasting increased the penetration at a further distance of 2-3 metres.

When the new supply of tungsten carbide cutters arrived, 300 metres more were excavated. The cutter consumption was lower, but due to the higher price for these cutters, the costs per meter were still too high. The boring was stopped, and the situation re-evaluated. There was no reason to expect other conditions than the variation that had already been encountered.

By participation of all parties involved, the economic aspects were resolved, and it was decided to increase the TBM thrust. By changing the thrust cylinders, the effective thrust per cutter ring was increased by 40-50 %. The cutterhead was also modified. The improvements took 2 months in total.

When boring resumed, the net penetration doubled. The weekly progress improved to typically 100 metres per week, and in the best week 193 metres was bored. Over the length of the tunnel the boreability varied from very low to good with Drilling Rate Index (DRI) ranging from 34 to 68.

The tunnel boring was organised in 10 shifts of each 12 hours per week. The contractor managed to bore approx. 60 hours per week (50% of available time). The overall average ROP was 45 meters per week. For Drill & Blast, the operating time was restricted to two shifts of eight hours per day. The assumed progress also 45 metres per week. After the increase of thrust, the average weekly progress was 70 metres. The need for rock support was very low.

In the end, the client and the contractor were both reasonably satisfied. Later, the contractor bored two more tunnels in another section of the main sewerage system in the same rock type.

Fig. 3. Breakthrough!

2.5 Lessons learnt

Besides the detailed experiences regarding the penetration rates and the cutter costs, the
following conclusions were drawn at the time, Blindheim (1972, 1973).

1. It is important to perform thorough preinvestigations for the purpose of evaluating the possibility of tunnel boring. Preinvestigations should include detailed mapping of the rock type distribution and variations in schistosity and jointing.
   Representative samples, including the hardest rock types should be tested. The samples should be tested by potential TBM-manufacturers and independent laboratories.
   The investigations should also clarify potential problems with weakness zones, water leakages and rock stresses; a TBM is too expensive for probe drilling.
2. Rough correlations were found between the net penetration rate and the Uniaxial Compressive Strength (UCS) as well as the Point Load Strength Index, but a significant effect of discontinuities and weakness planes was obvious. Work started to quantify this effect.
3. It is important to implement sufficient thrust to achieve efficient cutting, resulting in attractive penetration rates and agreeable cutter costs
4. The possibility for full face tunnel boring should be considered for all tunnels with small cross sections, especially under built-up areas.
5. Adaptation of plans to the special requirements of tunnel boring, such as minimum curve radius, need for access etc. should be considered already at the planning stage.

3 SHAFT BORING SUCCESS

The experience from shaft boring forms an important part of the further development. In 1971, A/S Sulitjelma Gruber, a copper mine in northern Norway, bought a Robbins 61R raise boring machine. By 1974 they had bored six shafts in Sulitjelma, five in Fosdalen mine and one for a civil project in Bergen.

This included shafts with diameters up to 1.6 metres, lengths up to 258 metres and alignment ranging from vertical to 30 degrees from horizontal. The latter was the shallowest sloping raise bore in Europe at the time. It was done with a modified reamer head with scrapers and water flushing for removing drill chips from the face, flushing it down the shaft.

The rock types were quartzitic mica schist, greenstone, quartz-keratophyre and amphibolite. These rocks were bored with good results with respect to penetration rate and cutter consumption, with the exception of an amphibolite with UCS of 300 MPa bored parallel to the foliation.

It is important to note that single disc cutters were used for all these shafts. These were of 13 inch (330 mm) diameter with 1/2 inch cutter edge of hardened steel. This approach to cutter utilisation contradicted common practice at that time. This pioneering attitude gave useful knowledge of single disc cutters, e.g. increased thrust per cutter ring as compared to thrust on multidisc cutters. The increased thrust caused improved cutting process. At the time 160 kN could be applied on a 13 inch cutter ring, facilitated by the development of durable roller bearings.

Encouraged by the results, A/S Sulitjelma Gruber went on with further raise boring with larger diameters. The next chapter in the Norwegian tunnel boring history was opened by the purchase of the Robbins TBM model 105-165 with 3.15 metres diameter and 12 inch (305 mm) single disc cutters.

4 RAPID BORING!

The first lot in the upgrading of the main sewerage system in Oslo was a 4.3 km tunnel section from Majorstua to Franzebråten. This tunnel passes under suburban areas, the main road and railway connection to southern Norway, recreational areas, a cemetery and old city buildings. The rock cover below the main road and the railway was only 2.4 metres, i.e. less than the tunnel diameter.

The rock types were sedimentary rocks; 60 % slightly metamorphic clay schist and 35 % limestone, and 5 % dykes. The dykes had thickness ranging from less than a meter up to 10-20 metres and included diabase, rhomb-porphyry and syenite. UCS values varied between 50 and 100 MPa for the sedimentary rocks and 150-200 MPa for the intrusive rocks.

Several weakness zones were encountered. Probe drilling and grouting were necessary to prevent subsidence damage to buildings above the tunnel.

A reduction in the need of rock support and grouting of 50 % and 25 % respectively was assumed. The tunnel spoil was used by the city for trench filling. A planned access tunnel could be left out, saving a children's playground, and eliminating construction noise in a residential area. All in all, the costs for the tunnel boring alternative were still 10 % higher. The client, Oslo Municipality, considered this to be acceptable for the reduced environmental disturbances, taking also into account that the experience to be gained would be beneficial for the rest of the project. The main contractor was Diplomingeniør Kaare Backer A/S. A/S Sulitjelma Gruber was subcontractor for the tunnel boring.

The boring was successful. After 4.3 km the net penetration rate, calculated as an average over each shift, had varied between 3.0 and 5.6 m/hr in the sedimentary rocks and 2.0-2.7 m/hr in the dykes. The lowest progress recorded was 1.6 m/hr. Average net penetration rate was 3.3 m/hr, max. progress over one day was 40 metres. The best monthly progress was 537 metres, a Scandinavian record at the time. The TBM and the cutters held up well, and only minor problems were experienced with TBM availability.

The probe drilling was performed through the cutterhead, without retracting it, with light drilling equipment. Holes for pregrouting were drilled in the same manner. The pregrouting operations delayed the progress; typical TBM utilisation was 18 %. To improve the overall progress the daily work schedule was changed to tunnel boring on two shifts and post-grouting on the night shift. This improved the TBM utilisation to 37 %. However, the need for more efficient incorporation of probe drilling and pre-grouting with the tunnel boring was clearly demonstrated. This was used as input for the specifications for the later sections of the same project.

5 THE FIRST PREDICTION MODEL

The experience gained while boring the first two tunnels gave an insight into which parameters were of importance. This included rock mass discontinuities, schistosity, partings and weakness planes, rock strength or rather toughness, abrasivity, thrust level, etc. A variety of rocks had been bored, but more information was necessary to allow independent predictions, supplementing the manufacturers' testing.

A survey was performed in Switzerland in a number of tunnels under excavation by TBMs. This study included observations of net penetrations and testing of rock samples.

A picture emerged regarding the test parameters. The Point Load Strength Index was, as expected to be, a good indicator. It was difficult to find useful correlations valid for several rock types. Also UCS did not give any generally usable correlation; it remained as a means of characterisation rather than a boreability parameter.

However, one came to recognise that tunnel boring is a kind of "percussive boring". The cutters are delivering hard impacts to the rock face, crushing the rock under the edge and chipping it between the cutter grooves, much in the same manner as percussive drilling except for the scale. These effects of crushing and impacts were already simulated in the empirical prediction models developed for percussive drilling. For this a large data bank of test results for Norwegian rock types was available.

Based on analysis of the data sets available from the tunnels in Norway and Switzerland, the first prediction model was developed and published in cooperation by the Departments of Geology and Construction Engineering at NTH. Admittedly with a slim database, it did allow a systematic comparison of progress and costs between tunnel boring and Drill & Blast tunnelling on the same basis.

6 THE RUSH IS ON!

6.1 Overview

Encouraged by the promising results from the first two tunnels, and presumably helped by the prediction model, all the main Norwegian contractors went into tunnel boring alone or in cooperation with experienced European partners. Within 3 years from 1976, 13 more TBM tunnels had been started, some of them completed. By 1982, 10 years after the start in Trondheim, a total of 18 tunnels had been completed and 4 more started.

An overview of key data is shown in Table 1.

Following, a brief summary about the experiences are given for some of the tunnels.

Table 1. Overview over TBM tunnels started before 1983.

Place	Year	Machine	Diam., m	Length, km	Rock type
Trondheim	1972-74	Demag	2.3	4.3	Greenstone, greenschist
Oslo	1974-76	Robbins	3.15	4.3	Limestone, schist, dykes
Fosdalen	1977	Robbins	3.15	0.67	Greenstone, keratophyre
Eidfjord	1978	Robbins	3.25	0.35	Granitic gneiss
Trondheim	1977	Atlas Copco Mini-Facer	1.5x2.4	0.12	Greenschist
Kjøpsvik	1978	Wirth	3.3	1.15	Limestone, amphibolite, mica schist
Oslo	1977	Bouygues, two TBM´s	3.0	3.6	Limestone, schist, dykes
Oslo	1979	Bouygues	3.0	3.9	Limestone, schist, dykes
Oslo	1977-79	Wirth	3.35	7.6	Limestone, schist, dykes
Oslo	1979-80	Atlas Copco Mini-Facer	2.1x3.2	1.0	Limestone, schist, dykes
Oslo	1978-81	Robbins	3.5	7.2	Limestone, schist, dykes
Oslo	1978-81	Robbins	3.5	7.1	Limestone, schist, dykes
Aurland	1977-78	Robbins	3.5	6.2	Phyllite, quartzite
Lier	1979	Robbins	3.5	1.3	Sandstone, limestone
Eidfjord	1979-80	Wirth	2.5	2.8	Granitic gneiss
Sildvik, shaft	1980	Wirth	2.5	0.8	Quartz biotite schist
Sørfjord	1980-82	Robbins	3.5	5.9	Mica schist, amphibolite
Brattset	1980-82	Robbins	4.5	7.0	Phyllite, mica schist
Ulla-Førre	1981-84	Robbins	3.5	8.0	Granitic gneiss
Sørfjord	1882-83	Robbins	3.5	3.0	Mica schist, amphibolite
Mosvik	1982-83	Jarva	3.5	5.7	Granitic gneiss
Ulset	1982-83	Robbins	4.5	7.3	Mica schist, gneiss

6.2 The short mini-fullfacer story

The first attempt in Norway to apply the undercutting excavation principle, adopted by Atlas Copco, was made at Duge on a hydropower plant in southern Norway. After less than 20 metres with slow progress and excessive consumption of the drag cutters in a granitic gneiss, the trial was stopped. The undercutting principle was not suitable in hard, massive and abrasive rocks. This trial is not on the above list.

The next attempt was made in Trondheim in 1977 by A/S Jernbeton Trondhjem in cooperation with Atlas Copco. A 120 metres tunnel with rock cover 2-6 metres was excavated to put a small stream underground below a housing development area. The tunnel was 1.5 metres wide and 2.4 metres high.

The rock was a greenschist with a well developed schistosity. The spacing between chlorite partings was typically 5 cm which gave good cuttability. Where the spacing increased up to 50 cm, the rock was significantly harder to cut.

The tunnel was completed in 3 weeks with an average net penetration rate of 1.5 m/hr and acceptable cutter costs. The total costs for the municipality was lower than for drilling and blasting of an open trench.

The advantages of simple mobilisation and convenient cross-section made the method attractive, especially for city tunnels. Both the limitations and possibilities were clearly demonstrated by these two tunnels. In non abrasive, schistose or jointed rocks with low to moderate strength, the method could be an interesting alternative. A midi-fullfacer was later tried on the Oslo sewerage project.

6.3 Tough going in Fosdalen and Eidfjord

After the successful boring in Oslo, the same Robbins TBM was used in Fosdalen mine for a transport tunnel. The TBM was dismantled to get it down the shaft. The boring went well at first, but then a massive tough greenstone was encountered, resulting in very low penetration rates. After 670 metres the job was interrupted.

The TBM was then rebuilt to 3.25 diameter, and the cutter diameter increased from 12 to 14 inches (356 mm). Its next job was a hydropower tunnel on the Eidfjord project, for NVE in 1978. The rock was a granitic gneiss for the first 1.7-1.8 km with UCS up to 270 MPa. Further, phyllite with quartz lenses, quartzite and other gneisses occurred.

The hard gneiss proved to be a tough task. After 3 weeks of boring and 130 metres tunnel, the main bearing broke. The repair took 5 weeks. After 2.5 weeks of boring, the bearing broke again, and this was repeated a third time. A world record the contractor did not appreciate! The problems were connected to the repeated dismantling and rebuilding of the TBM and not to the hard rock. Eventually the TBM was completely rebuilt prior to new assignments.

The net penetration rate in the hard and massive rocks had varied from 0.5 to 2.5 m/hr, and best weekly progress was 70 metres. The need to develop cutters that could sustain even higher cutter loads was highlighted.

The owner was not discouraged, and another contractor, A/S Høyer-Ellefsen, completed the tunnel without special problems in 1979-80 with a Wirth machine.

6.4 Kjøpsvik

In 1978 a transport tunnel was bored in Kjøpsvik in northern Norway in a mostly massive limestone with UCS of approximately 100 MPa. The Wirth TB II machine had 19 double ring disc cutters and two centre cutters with tungsten carbide inserts. The applied thrust was 120-140 kN per cutter and the rpm 10. The contractor A/S Høyer-Ellefsen was satisfied with the results. Average weekly progress was achieved by 83 metres in the access and 73 metres in the transport tunnel. The cutter consumption was high. It is noteworthy that the TBM was ready to bore 1.5 week after arrival on site.

6.5 The Oslo story

The continuation of the tunnel boring in Oslo from 1977 to 1981 is a remarkable story. Tight restrictions were necessary to control allowable water inflow in order to avoid damage due to subsidence in above lying sediments. Based on the experience from the first TBM tunnel the owner issued strict requirements in the bidding documents as to the performance of probe drilling and pregrouting in connection to the TBM operation.

In total 40 km was bored with TBMs from four different manufacturers, with quite different concepts. At one time five TBMs were in operation.

The achievements with probe drilling and pregrouting was remarkable, but valuable lessons were also learnt regarding how to get through fault zones with unconsolidated material, giving acute stability problems at the tunnel face. Further experience was gathered on boreability in a variety of rock types and degree of jointing, supplementing the database of the prediction model.

6.6 Aurland

On the large hydropower scheme in Aurland a 6.2 km water transfer tunnel was bored by the contractor Ingeniør Thor Furuholmen A/S. The 3.5 metres diameter Robbins TBM 116-181 had 25 single disc 14 inch (356 mm) cutters and two double disc centre cutters.

The rpm was 7.2. The rock types were phyllites with smaller and larger lenses of quartz and sections of a metamorphic sandstone with a quartz content of 40%.
Fig. 4. Single disc cutters

The tunnel boring started the summer of 1977 and towards the end of 1978, 5.9 km had been bored. Progress on the best day was 35.2 metres, the best week 181 metres and the best month 651 metres. Average net penetration rate had been 1.9 m/hr and overall progress 108 metres per week. TBM utilisation was 63 %, best week 72 %.

For long sections the schistose rock was bored in the most difficult direction, i.e. parallel to the tunnel axis. A series of tests was performed with the TBM checking the relation between penetration rate and thrust, demonstrating the significant effect of higher thrust. During normal boring 145 kN per cutter was used, which was 80 % of the rated cutter capacity. This proved most economical for disc rings and bearings. In short periods the TBM bored up to 7.3 m/hr, fully utilising the torque of the 4 drive motors.

The cutter costs were higher than estimated, up to USD 20-25 per metre tunnel in the quartzitic sandstone. No rock support was installed during excavation, and the need for permanent support was minimal. The costs per metre tunnel were still higher than for Drill & Blast tunnelling, but the reduced construction time could be used, allowing earlier transfer of water.

The Aurland tunnel became important for the further development of the prediction models. Comparable data sets made it possible to quantify the effect of the angle between the foliation and the tunnel axis. It was found that the net penetration rate increased in proportion to the angle; e.g. in the phyllite a 45 degree angle gave a 45 % increase. This was included in the next revision of the prediction model together with similar data from other tunnels.

6.7 Lier

After completion in Aurland in 1978, Ingeniør Thor Furuholmen A/S put the Robbins TBM 116-181 to use on a water transfer tunnel from Holsfjorden and Lierdalen, near Oslo. The rocks in the 3,600 metres long 3.5 metres diameter tunnel were clay schist, limestone, mudstone and sandstone. After boring 1,262 metres the best daily and weekly progress were 42 and 144 metres respectively. In rock with medium boreability (DRI=53 and BWI=23) an average net penetration of 3.7 m/hr was achieved.

The overall results did not meet the expectations, as the TBM availability was low due to several mechanical, electrical and hydraulic problems. A cave-in of 50 m^3 rock above and on the side of the TBM occurred in a wet zone of blocky sandstone with slickensided joints. Minimal damage occurred, but it took four weeks to clear out the debris manually, install steel lining segments and pour concrete behind the segments to resume boring.

This demonstrated that good net penetration is not enough. Progress depends on TBM utilisation, TBM availability and rock stability problems.

6.8 Sørfjord

In 1981-82, the same 3.5 metres Robbins TBM 116-181 bored the 5.9 km Brynsvatn-Nikkelv tunnel and in 1982-83 the 3.0 km Nikkelv-Vatn tunnel for the Sørfjorden Hydropower Project, through mica-schist, gneiss and amphibolite.

The achieved penetration rate was on the average 2.1 m/hr in mica-schist (DRI approx. 75) and 1.4 m/hr in gneiss and amphibolite (DRI approx. 50). The gneiss and amphibolite had no jointing of significant effect for the penetration, whereas the foliation of the mica schist improved the boreability significantly.

Overall progress was 130 and 105 metres per week for the 5,836 metres and 3,030 metres sections respectively. Best weekly progress was 240.5 metres and best monthly 775 metres. Cutter costs were as expected, at about 2-4 USD per m^3. It was concluded that the results were satisfactory with respect to technical performance, time and costs both for the owner and the contractor.

6.9 Revised prediction model

With results from other tunnels, a large amount of experience data was now available. This was due to the willingness of the owners, contractors and manufacturers to provide access for follow up and testing during and after construction, and the openness with detailed and overall results. The NTH prediction model was again revised, including all relevant parameters, supported by an extensive database.

7 CONCLUSIONS

During the first ten years of tunnel boring, the use had increased to a significant portion of the total tunnelling. Experience was gained regarding a variety of rock conditions and technical problems. The main points regarding what had been achieved and what was to be expected was summarised as follows:

- Disc cutters were effective for both raise and tunnel boring in hard rock, and the importance of utilising high thrust levels had been demonstrated.
- The undercutting principle with drag cutters could be used in weak rock with low abrasivity. Medium hard rock could be bored when jointed.
- Discontinuities and different types of weakness planes in the rock mass; bedding planes, foliation, partings etc., have a pronounced effect on the boreability of the rock mass. Efforts to quantify the effect had made significant progress.
- Because the owners, contractors and manufacturers had showed openness about achieved results, a useful prediction model for TBM performance had been developed. This was available as a planning and follow-up tool, making it feasible to consider the advantages of tunnel boring already at the planning stage by adapting plant layouts to the possibilities of faster tunnelling, longer drives etc. It could also serve as a basis for risk sharing between the client and the contractor.
- Tunnel boring in massive and abrasive rocks had been performed with good results by TBMs with single disc cutters sustaining high thrust levels.
- Systematic probe drilling and pregrouting could be combined with tunnel boring by proper preparation of the TBM.
- Efficient boring through weakness zones and sections with high rock stresses by utilising light support elements and rock bolts had been achieved.
- Several cave-ins in unconsolidated crushed zones causing delays had been experienced. Methods to avoid cave-ins, or to pass through should they occur, had been improved.
- Boring of inclined shafts with TBMs had proven to be an efficient and safe method.
- High TBM utilisation was necessary for good progress. The importance of an effective organisation and good backup systems had been demonstrated and remarkable results achieved.

It was concluded that tunnel boring was more sensitive in respect to organisation and geological conditions than Drill & Blast tunnelling. The use of TBMs with even higher cutter capacity would open the possibility to boring longer and larger tunnels in even harder rocks. The prediction model provided a tool for choosing between tunnel boring and Drill & Blast tunnelling in a balanced manner. The basis for a further development to the forefront of hard rock tunnel boring, mastering also the potential difficulties with high rock stresses, fault zones, water etc., had been laid.

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6 THE VEAS PROJECT 40 KM TUNNELLING WITH PREGROUTING

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ABSTRACT: In 1973 the City of Oslo and the adjacent municipalities on the western side of the Oslo Fjord decided to build a large scale sewage treatment plant approximately 30 km from the city. This meant that a total of 42 km transport tunnels had to be constructed, partly under heavy populated areas. The geological strata shows a very folded rock surface, with an overburden of up to 30 metres of clay. As direct foundation is the most common, this meant that the sensibility to ground water changes was considerable. Along with a close survey of the groundwater table and subsidence, and in order to avoid seepage into the tunnels, an extensive pregrouting programme had to be carried out concurrently with the TBM excavation.

1 INTRODUCTION

The inner Oslo Fjord is, particularly during the summer, one of the most important recreational areas for nearly one million inhabitants of Oslo and the surrounding municipalities. During the fifties and sixties, the marine life in the fjord, as well as the fjord as a recreational asset, was severely threatened by the growing pollution in the water due to sewer contamination. Starting in 1962, the Norwegian Institute of Water Research carried out extensive investigations in the Oslo Fjord. From these results, the conclusion was drawn that a new and state of the art treatment plant had to be built to relieve the many smaller and less efficient ones already in service.

During the second half of the seventies, the city of Oslo and the neighbouring municipalities on the western side of the Oslo Fjord, Baerum and Asker, formed a joint venture to collect, transport and treat sewage using current state-of-the-art methods. To transport the sewage and water to the scheduled treatment plant, approx. 30 km from the town centre, it became apparent at an early stage that the only realistic transport method was tunnels. In 1970 the project was considered to be a typical Drill & Blast job, but in 1973 the project was opened for alternative tendering based on TBM excavation. When the tender conditions were finally settled in 1976, a decision had been made that all the major tunnels were to be TBM excavated tunnels. Conventional Drill & Blast methods were no longer accepted. Over a period of no more than six years, a complete change in the availability of modern excavation methods, caused a significant improvement to the feasibility of the entire project.

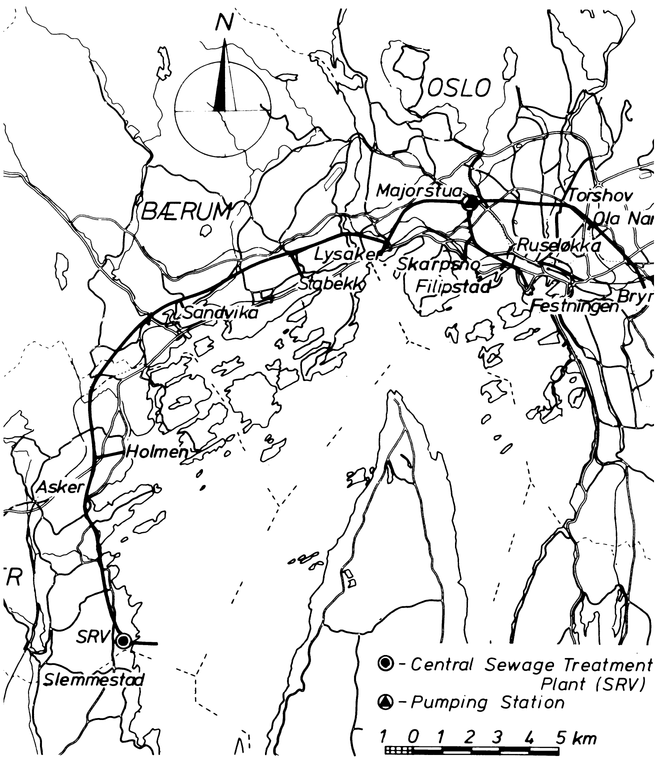


Fig. 1. Map of the Oslo metropolitan area showing the tunnel system for the VEAS sewer project.

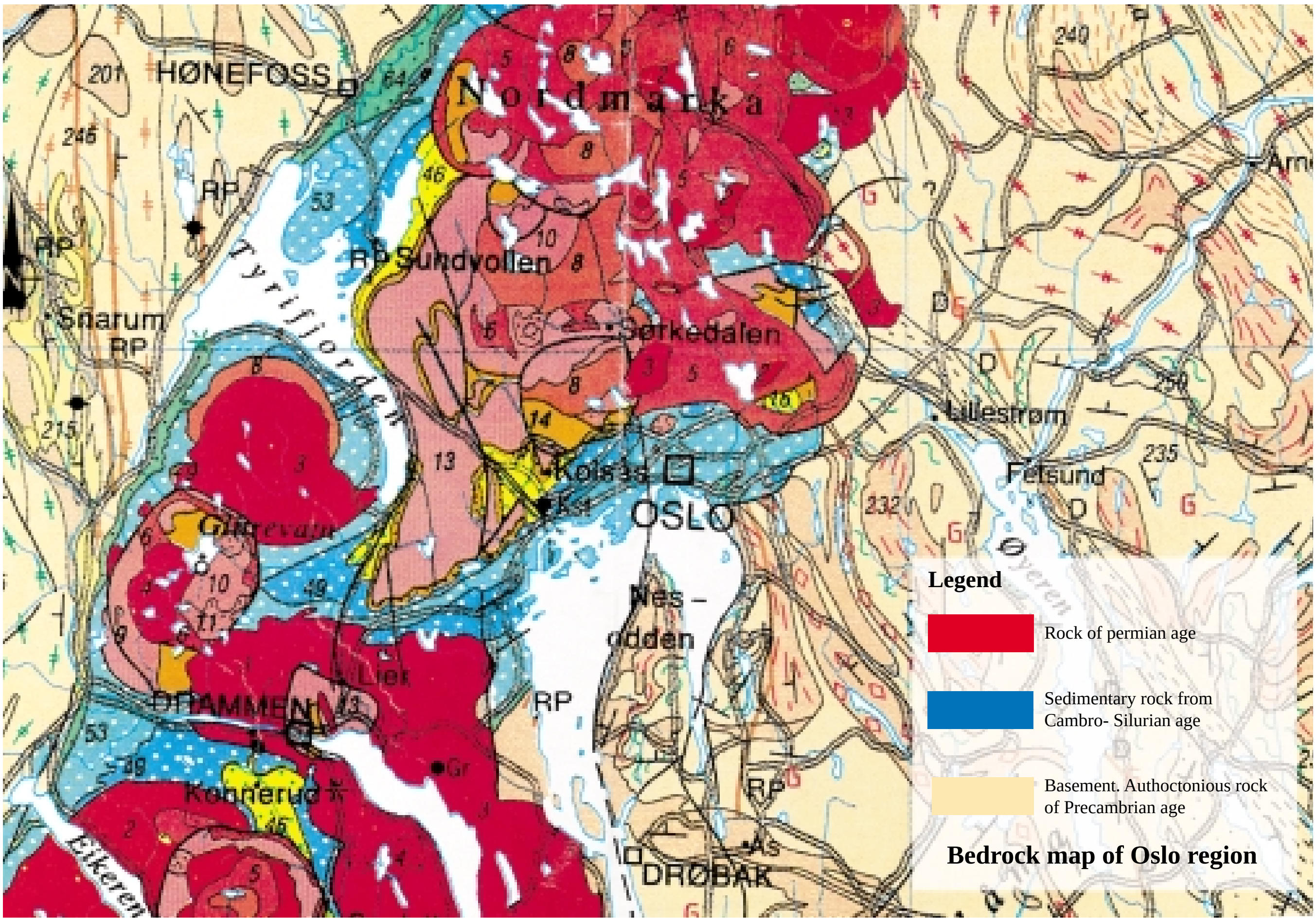


Fig. 2. Geological map of the Oslo area. Scale approx. 1:550 000

2 GEOLOGY

The bedrock in the area is dominated by Cambro-Silurian sedimentary rocks and Permian igneous rocks of the Oslo graben. The Cambro-Silurian rocks include limestone, shale and sandstone, folded around an axis in NE-SW direction, which is also the strike of the bedding. Along the tunnel, alternating strata of shale and limestone prevail in varying thicknesses.

3 SOIL DEPOSITS

It was considered essential to have a detailed knowledge of the soil deposits located on the rock surface above the tunnel. Therefore, detailed soil investigations were performed at an early stage in the planning process. In addition to finding the most favourable alignment for the tunnel, it was also necessary to choose strategic installation locations for the piezometers, and select buildings to be monitored for possible settlement.

The soil along the tunnel mainly consists of soft and compressible clay, in depths of up to 30 metres. Hence, they are very sensitive to settlement when reductions occur in the pore water pressure.

4 GROUND WATER RESPONSES

The tunnels pass under urban and suburban areas where the ground water table is vital for the foundation for both domestic dwellings and larger buildings. If the ground water table dropped due to drainage into the tunnel, subsidence and consequently damage to the buildings would most likely occur.

Prior to the tunnel construction, soil depths were measured or interpolated from previous investigations carried out for other foundation purposes.

In these areas, piezometers for measuring the groundwater table were strategically installed. The ground water levels were monitored on a regular basis for two years, to obtain the normal variations of the groundwater level.

Buildings strategically located within the soil areas were surveyed and had levelling bolts installed to register possible subsidence. The results of these records and registrations were later used in the design of the necessary grouting works ahead of the tunnel faces.

5 SEEPAGE CRITERIA

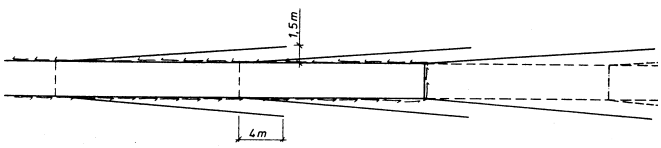
The type and depth of soil, as well as the sensitiveness of the foundation of the buildings along the tunnel, showed significant variations. Hence, the final criteria for maximum seepage into the tunnel changed along the tunnel, and were not exclusively related to measurements of water in probe holes ahead of the tunnel face.

6 PREGROUTING PROCEDURES

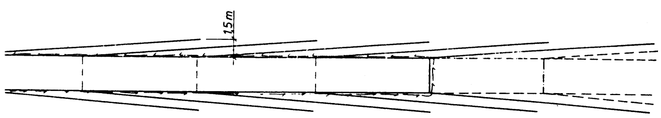
The contractor was instructed to strictly follow certain procedures for probe drilling and pregrouting when performing the tunnel boring.
A certain number of probeholes were to be drilled ahead of the tunnel face. These probeholes were drilled with narrow angles to be later used for grouting to form a single or a double fan surrounding the tunnel.

The permeability of the rock was measured for water leakage by injecting water under pressure. The water seepage into the rock was expressed in Lugeon (L). From the measurements, an indication of the rock permeability was gained, an in turn this was used to decide the type of grouting material and the procedure to be followed.
The probeholes were then grouted until a specific stable pressure was reached.

If the probeholes did not indicate that the seepage level in the rock was reduced to an acceptable level, repeated rounds with control holes and grouting were performed until the maximum seepage requirements were met.



1. Standard injection procedure



2. Injection procedure with «double» fan

Fig. 3. Layouts of different grouting fans.

7 PREGROUTING FROM A TBM

On the VEAS project, three TBM manufacturers were involved. Due to different design and mechanical construction, the need for different layouts of drilling and grouting equipment was recognised, as well as different procedures for drilling and grouting. An outline of the drilling and grouting procedures for different manufacturers is shown in table 1 below.

Maufacturer	Drilling and grouting procedure
Bouyges	The machine was withdrawn approx. 6 m, and an hydraulic jumbo was mounted on one of the TDM arms.
Wirth	A drilling rig with two hydraulic jumbos was installed permanently immediately behind the TBM. The starting points were behind the rear grippers, approximately 6 metres behind the tunnel face.
Atlas Midi	The machine was withdrawn and a jumbo was transported up front to be mounted on the head.
Robbins	A hydraulic jumbo was mounted on each side of the TBM. The starting points were just behind the head, approx. 3 meters behind the face.

Table 1. Procedures for drilling and grouting at the VEAS project for the different TBM manufacturers.

At the time of the tendering, only one tunnel had been bored in the Oslo area. On this occasion, the pregrouting system was unsuccessful, and the contractor faced bankruptcy due to large financial claims. For the VEAS project the contractors therefore went into alliances with foreign contractors to benefit from their experience.

The main goal was to produce a watertight shield of grouted rock as close to the tunnel wall as possible. To prepare for the pregrouting efforts, the TBMs were modified to enable drilling at certain locations along the circumference of the cutterhead, and to produce overlap between each grouting fan. This could mean arrangements to drill directly behind the cutterhead itself, or to drill from starting points further behind on the machine at a narrower angle. One of the requirements however, was to place the packers 1.5-2.0 m ahead of the cutterhead when grouting. When starting further back on the machine, the packer rods had to be considerably longer.

To perform the grouting, a dedicated platform had to be constructed to accommodate grout pump, mixer, agitator etc. Depending on the layout of the TBM and backup, this was located at the TBM itself or further behind, and added another 15-20 metres to the total length of the TBM equipment.

8 GROUTING PROCEDURES

For each stage of the construction different grouting procedures were to be followed. These procedures took into consideration rock conditions, soil overburden depth and buildings or constructions sensitive to settlement.

The main grouting material was Rapid Portland Cement, a special cement mixture with a hardening time of less than 30 minutes. This allowed the tunnel boring to continue as soon as the grouting and water loss measurements were carried out and met the specified criteria. When the water loss was still too high, after cement grouting, chemical solutions with viscosity equal to water were pumped in at high pressures to fill joints and fissures with openings less than 0.1 mm.

The effective grouting length along the tunnel axis was approx. 25 metres. An overlap of 4 metres was required in each round. Hence, the TBM could proceed approximately 20 metres between each grouting round.

In zones with extensive faults, it was experienced that 25 metres is too long for one grouting round. Severe difficulties arose when drilling the probeholes and placing the packers, and grout flew back into the tunnel. The grouting length was therefore reduced to 10 metres or even less when working in heavily fractured rock.

As mentioned, the packer rods were relatively long, approx. 7.5 metres. To be able to flush the packer rod with water, and avoid hardening of the grouting material in the rod, a simple valve and a specially designed plastic hose were used to do this successfully. Hence, the packer rods could be reused.

The cement was transported into the tunnel in standard 50 kg sacks and lifted by hand directly into the mixer. Chemicals used were pumped directly from a transport tank.

For the grouting operation, a crew of five men was needed; The same force doing the tunnel boring.

Naturally, the grouting accounted for a relatively large part of the total construction time for the tunnel.

9 SEEPAGE REDUCTION

The seepage into the tunnels was given in Lugeon (L) when handling the seepage in probe holes ahead of the face during excavation. After excavation the seepage was given in litres pr minute pr 100 m of tunnel. For the TBM excavated tunnels the remaining seepage was in the range of 2 - 5 litres pr minute pr 100 m of tunnel.

In most cases, the installed piezometers showed no drop in the groundwater level. However, a few piezometers showed a significant drop, and temporary infiltration had to be utilised to compensate for the seepage.

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7 BORED ROAD TUNNELS IN HARD ROCK

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ABSTRACT: The paper describes the first full-face tunnel boring and part-face road-header cutting of road tunnels in Norway. The tunnel boring proved feasible from a technical performance viewpoint, and satisfied the requirements of the projects, including reduced environmental impact. At the time, tunnel boring and roadheader excavation could not compete with Drill & Blast with respect to costs on a meter by meter comparison. However, the experience gained has contributed significantly to the development of hard rock tunnel boring under demanding conditions.

1 INTRODUCTION

Because of Norway's topography, most of its population is living along valleys and fjords, and therefore a large number of road tunnels has been built. In the early eighties, approximately ten years after the start of tunnel boring in Norway, the technology had apparently advanced enough to consider TBMs for the large diameters needed for road tunnels.

Two government organisations, the Norwegian Hydropower Authority NVE *) and the Public Roads Administration, pioneered the use of TBMs for excavation of road tunnels in the period from 1984 to 1987.

*) Through a restructure of the managing of the Norwegian water resources, NVE has been divided into (1) NVE, concentrating i.a. on governmental duties like rules, regulations, concessions, controls, safety and (2) Statkraft as the commercial agent of Government within hydro power.

2 HOLANDSFJORD

2.1 Project description

The Svartisen Hydropower Project in northern Norway, was planned to tap the water from the Svartisen glacier. It became the scene for a lot of tunnel boring, described in other papers in this publication.

The advantages of providing a ferry-free access to Holandsfjord for the hydropower development and the small townships of Kilvik and Braset with road connection, which could later become a part of the coastal route, triggered a co-operation between the hydropower and road authorities. This resulted in the 7.6 km Svartisen road tunnel. NVE financed the project for the Public Roads Administration because of the advantages during the development of the hydropower project. NVE also performed the tunnel boring.

The southern 5.1 km section of the tunnel had micaschist changing to gneissic rock towards north, with a section of embedded quartzitic sandstone. Granitic gneiss dominated the northern 2.5 km section of the tunnel.

2.2 Choice of method

The TBM was chosen for the southern section of the tunnel, because the rocks were expected to have good boreability, making it possible to reduce the tunnelling time. The northern section was excavated by Drill & Blast. A private contractor was assigned this duty.

A 6.25 m diameter refurbished Robbins TBM 204-216, equipped with 44 single disc 15.5 inch cutters (not changed as for the other TBM in Kobbelv) and seven 200 hp drive motors, was made available by NVE.

The project management expected the TBM to advance 90-100 m per week, and complete the bore in spring 1985. It was assumed that Drill & Blast tunnelling would have required almost a year more, due to slower excavation rate and more rock support. The tunnel had to be ready in 1986 when the development of Svartisen Hydropower Project was scheduled to start in full.

The reasons for choosing tunnel boring as compared to the Drill & Blast method are summarised as follows:

- Shorter construction period.
- Reduced rock support.
- Possible reuse of TBM for the headrace tunnel to the Svartisen power station

- Reuse of the mobilisation facilities to construct Svartisen power station in Holandsfjord

Other aspects were:

- The type of TBM and the tunnel mobilising set up was similar to that used for another project in the vicinity, hence synergies could be expected
- Initially, it had been decided to plan for single lane traffic in the tunnel. The selected 6.25 metres diameter TBM would accommodate one lane. Reconsideration of the future traffic intensity called for two lanes in the tunnel. The adequate decisions were made and the cross section of the tunnel modified accordingly. To obtain the requested cross section for two lanes of 3.0 metres each, an increase of the 6.25 diameter bored tunnel by means of Drill & Blast took place.

2.3 Experience

The contractor rented the refurbished TBM from the producer for a fixed price per cubic metre. The rent covered capital costs, cutter costs and spare parts for TBM, backup equipment, muck cars and rotadump.

A landing craft was used to bring the TBM and equipment onshore at the remote site in September 1983. Boring started 18.01.84, after assembly of the TBM and back up equipment, preparation of a starting chamber and the construction of the muck dumping station. This included excavation of 200 m of the tunnel by Drill & Blast. Some modifications were done to the TBM, based on the experience encountered in Kobbelv with the other machine.

The back up equipment consisted of a double track system with traverse shifting using 14 m^3 muck cars. A rotadump was installed over a silo system at the portal.

The TBM holed through in March 1985 after 4,300 m had been bored in a total of 62 weeks. The average progress was 80.7 m per week for the weeks actually bored. Average net penetration was 2.0 m/h, typically varying between 1 and 3 m/h. Thrust per cutter was 200-220 kN. The TBM was torque limited on long sections.

The transport facilities kept pace with the boring until a penetration of 2.5-2.8 m/hr was reached. When higher penetration rates were achieved, traffic problems occurred.

In some jointed and karstic zones heavy water ingress was experienced. This resulted in flooding of the dump station and the silo, as well as messy working conditions at the TBM. The wet muck ran over the conveyer belt and manual clearing of the invert was necessary before laying the rails.

In a zone of quartzitic sandstone, several sections with crushed rock were encountered, resulting in difficult conditions for bolting, fallouts over the TBM and problems with gripping. Attempts to shore the gripper pads with timber did not work well. Steel beams were welded to the grippers and attached to the rock behind the zones, allowing the TBM to 'walk' slowly over the worst zones. Boring with very low thrust, resulted in 0.15 m/h net penetration and 10-12 hours per 1.8 m thrust. In a section of 50 m, shotcreting was performed up to the cutterhead roof shield. Loading of fallout was done manually and by a small bobcat below the TBM. It took six weeks to excavate through 200 m.

It was clearly demonstrated that in poor ground it was important to be able to perform rock support, probe drilling, prebolting and pregrouting immediately behind the cutterhead.

After breakthrough, the bored cross section of 32 m^2 was enlarged by blasting.

In spite of the water inflow and stability problems, the TBM performed well. However, a comparison of progress and costs performed afterwards indicated clearly that tunnel boring could not compete with Drill & Blast tunnelling with respect to costs. The total cost was USD 2,600 (NOK 17,000) per m tunnel, whereas Drill & Blast would have cost one third less, even with 10 times more rock support. A larger bored cross section, with no need for enlargement by blasting, would also have been more costly.

The northern section of the tunnel was excavated by Drill & Blast. This granitic part cost less than half of the TBM section (three times more rock support included).

The principal target had, however, been achieved, and useful experience gained on tunnel boring of larger cross sections. When opened for traffic, the Svartisen road tunnel was the longest road tunnel in Norway. The TBM was then moved then to the Kobbelv Hydropower Project, to bore the last 2.6 km of the headrace tunnel towards the pressure shaft.

3 FLØYFJELLET

3.1 Project description

Bergen, Norway's second largest city, frequently named the city between the seven mountains, urgently needed a bypass motorway system through the most congested area of the city. In 1982 it was decided to construct a double tube tunnel, each tube with two traffic lanes under the Fløyfjellet mountain.

The southbound tube would be 3.2 km and the northbound 3.8 km.

The areas around the tunnel alignment are partly built up and there are hospitals close to both portals.

80% of the tunnel passes through mainly hard granitic gneiss (with Uniaxial Compressive Strength UCS approx. 150 MPa, 20% quartz and Drilling Rate Index DRI approx. 50). In the southern part, the gneiss is more schistose (UCS 130 MPa, 15% quartz and DRI approx. 40) and interrupted by layers of jointed hard quartzite (70-80% quartz and DRI 40-50). At the southern portal there is a zone of hornblende- and green-schist (UCS at 100-200 MPa and DRI 40-50).

3.2 Choice of method

The choice of excavation method caused extensive discussions. It was assumed that the boreability of the rock mass would be a challenge, because of the large diameter. It was concluded however, that tunnel boring would be feasible, based on testing of rock samples and field assessment of the influence of jointing.

It was expected that the boring would be relatively easy in the hornblende- and green-schist, and that the boreability would be moderate to low in the granitic gneiss, and very low in the abrasive quartzite, the latter being present only on a limited section. The predictions pointed to an average net penetration of 1.74 m/h.

The choice of excavation method was made with emphasis on the following factors:

- it was urgently needed to relieve some of the traffic congestion; by tunnel boring the first tube could be completed one year earlier
- the total costs would not be higher than for Drill & Blast, and a 10% reduction was expected
- a reduction of up to 75% in the rock reinforcement was considered likely
- the reduction of vibrations and ventilation from blasting was of particular importance for the exter nal environment, due to the hospitals close by both portals
- the work safety and environment would be improved

In addition, the Public Roads Administration had a considerable interest in gaining first hand experience of the technology, which could be of interest for other and especially long road tunnels.

3.3 Experience

The TBM was ordered specifically for the project and came to Bergen a year later. Boring started after two months on 12.09.84 at the southbound tube, boring north.

The Robbins TBM 252-226 had cutterhead diameter of 7.8 m, 57 single disc 17 inch cutters and 6.13 rpm. It weighed 450 tons and was 18 m long, plus a 50 m long backup. The thrust cylinders allowed 230 kN/cutter and 1.83 m strokes. The trailing rig had two sliding “deck” units, the first with workshop, lunchroom and electric equipment, and in the second two silos of 12 m^3 were located. Figure 1 shows the main components of the TBM with backup.

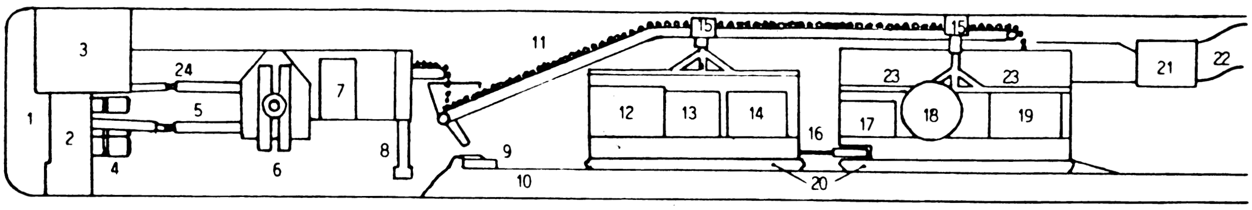
The muck transport was performed by subcontractors using trucks. Each of the two 12 m^3 silos corresponded to about 12-13 cm of tunnel to be filled in 2-2.5 min. under normal boring.

Part of the muck was allowed to fall on the invert from the conveyer belt behind the TBM to provide a temporary sub-base. An 8 inch drainage pipe wrapped in geotextile provided drainage. The rest of the muck was conveyed to the silos. Most of the time, transportation by two trucks kept pace with excavation, four was needed at maximum. The turning niches, located at positions corresponding to the crossover tunnels between the two tubes, were drilled during one weekend and the following nights, and blasted the next weekend.

By 02.09.85 the TBM completed the boring of the first tube. It was then withdrawn through the bored tunnel and made ready to start the 3,800 m drive by 12.12.85. The final hole through took place on 07.12.86.

As typical on TBM projects the work hours consisted of two 10 hour shifts a day five days a week, and one 8 hour shift (6 hours used for boring) on Saturday.

Progress peaked at 119 m/week in the first tube and 152.5 m/week in the second. Utilisation was 48% and 59% respectively in relation to the 106 hours available



1. CUTTERHEAD
2. SIDE SUPPORTS
3. ROOF SUPPORT
4. DRIVE MOTOR
5. FEED RAM
6. GRIPPER
7. CONTROL ROOM
8. REAR SUPPORT
9. LEVELLING PLANE
10. MASS SPOIL
11. CONVEYOR BELT
12. LUNCH ROOM
13. EL. ROOM
14. TRANSFORMER
15. ROOF SUPPORT
16. RAM
17. GENERATOR
18. CABLE SPOOL
19. AIR COMPRESSOR
20. SLED
21. VENTILATION BOX
22. VENTILATION DUCT
23. SILO
24. MAIN BEAM

Fig. 1 Main components of the Fløyfjellet TBM "Madam Felle".

for boring per week. Cutter change took 23% and 20% respectively. The average net penetration was 1.6 m/h in the first tube and 1.45 m/h in the second.

In the second tube, more wear was allowed on the cutters, lowering the penetration in the hardest and less jointed sections. Still, average weekly progress went up from 68 to 81 m, a 20% increase. Cutter life increased from approx., 100 m^3/cutter to 115, also due to better routines for cutter change.

Cutter wear was lower than expected. Cutters were inspected at the end of the last shift every day, typically 5-10 cutters were replaced at the beginning of the next shift, the 4 hour shutdown between shifts allowing the cutters to cool.

The final cross section increase was established by Drill & Blast, in the lower sidewalls with about 3 m per day, as only daytime blasting was allowed in the built up areas. In parallel with this work, the temporary sub-base material was replaced. The theoretical cross section is shown in Figure 2.

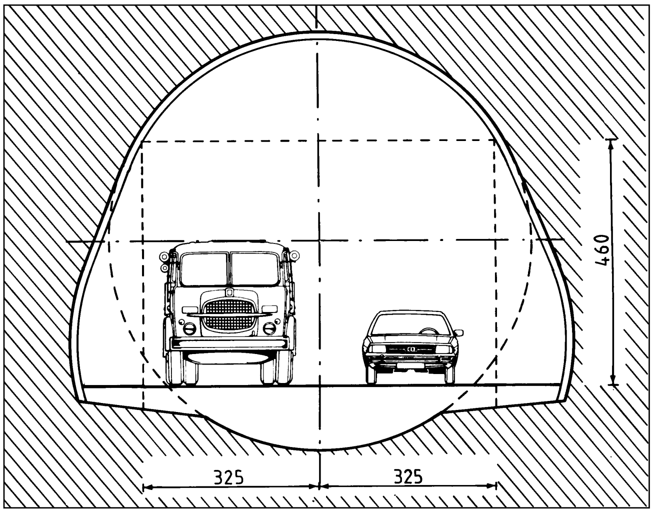


Fig. 2 Theoretical cross section of one of the double lane tunnels

The TBM was equipped with a drilling unit for probe drilling ahead and for prebolting through crushed zones, but the need for these measures was small. Rock bolting was done with handheld jack legs mounted on the main beam behind the cutterhead support. 350 and 106 bolts were installed in the two tubes respectively. No concrete lining was needed. Polyethylene sheets were used for frost insulated water shielding.

The rental of the TBM was based on a fixed price per m^3 bored, based on an assumed cutter consumption, with an upper limit. Deviation from the assumed cutter consumption price was shared.

The crew comprised experienced tunnellers, recruited either from the Public Roads Administration's own tunnelling crews, or hired in to provide TBM experience. A Robbins engineer was present during the whole period.

Total cost for the tunnel was USD 7,700 (NOK 50,000) per metre including the increase of the cross section, two ventilation shafts and installations. Approximately one third was connected to the tunnel boring. The tunnel would probably have been less costly by Drill & Blast, not considering the environmental problems with blasting.

The option to buy the TBM was not utilised, as other road tunnel projects were not available for a continued use, except for the Eidsvåg tunnel described below.

4 EIDSVÅG

4.1 Project description

The Road Administration planned to excavate the second Eidsvåg tunnel tube of 850 meters length. The rock consisted of blocky granitic gneiss. The second tunnel was planned with a spacing of 11 m from the existing first tube. The AADT of 25,000 units caused daily traffic jams during peak hours.

4.2 Choice of method

A major consideration of reducing Drill & Blast as well as the availability of a suitable TBM in the close vicinity were basic factors, while exploring the optional alternatives.

A new rent agreement allowed for a diameter increase from 7.8 meters to 8.5 meters to fit the needs at the Svartisen project.

4.3 Experience

The number of cutters were increased from 57 to 61, new roof and side supports were installed, as well as a gear reducing the rotation from 6.13 to 5.81 revolutions per minute.

The boring took 10 weeks and was completed in December 1987

The construction costs were higher for tunnel boring than it would have been for Drill & Blast. A benefit for the community was an early relief from traffic congestion in the tunnel area.

5 AUGLANDSHØGDA TUNNEL

5.1 Project description

The tunnel consisted of the second of a 350 m double tube tunnel on the main motorway between Stavanger and Sandnes. The first tube had been completed in 1973, and was fully concrete lined with a membrane. It was preferable to carry out the extension from two to four lanes without interruption to the 30,000 AADT traffic, a situation similar to the Eidsvåg tunnel.

The tunnel cross section was 63 m^2. The distance between the two tunnels was 11 m, to avoid damage by blasting to the lining in the first tube.

The rock was phyllite, with UCS 35-85 MPa, uneven foliation and thin quartz layers and lenses. The rock cover was down to 5 m near the portals, partly with poor stability condition.

5.2 Choice of method

The wish to try a method suitable for a non-circular cross section, triggered the co-operation for a trial project between Public Roads Administration and the contractor Selmer-Furuholmen A/S, who made available a Westfalia Lünen WAV 178, powered with 340 kW, 200 kW for the cutterhead.

The agreement was made for 50 m tunnel, to be excavated before the decision about further use would be taken.

5.3 Experience

The initial phase demonstrated the capabilities as follows:

- Excavation capacity: 7.3 m^3/h during cutting
- Utilisation: 59%
- Pick consumption: 0.85 pieces/m^3

The cutting head was utilised to push the muck towards the loading belt.

Two types of picks were tried. The larger tungsten carbide inserts proved best, and it was important to change broken cutters as early as possible, to avoid breaking of neighbour picks and excessive wear on the sockets.

Several rock samples were taken showing UCS values ranging from 40 to 60 MPa, quartz content of 38-51%, DRI of 49 and Bit Wear Index BWI of 31, indicating medium boreability. The rock mass had few joints besides the foliation.

The results were not satisfactory with respect to overall performance, and the approach was changed in the next phase. The inner part of the cross section was blasted. 60 cm inside theoretical contour was left for road header cutting.

The cutting improved due to the blasting cracks, but the costs were still too high. Finally only 20 - 30 cm were left for mechanical cutting. The remaining 300 m of tunnel was completed in this manner.

The total cost by this method was directly comparable to Drill & Blast, approximately USD 3,000 (NOK 19 000) per m, mainly because of a reduction of the rock support by half. A prediction model for roadheader cutting was developed using DRI, BWI and frequency of jointing, as for TBM predictions.

The results demonstrated that roadheader cutting was not suitable in normal hard rock conditions, with medium boreability parameters.

6 CONCLUSIONS

The experience from mechanical excavation of a total of about 8 km road tunnels can be summarised as follows:

- Generally the results were positive with respect to technical performance and progress for hard rock TBMs, including boring of massive, hard and abrasive rocks.
- Problems in weak and water bearing zones were difficult to handle due to the confined space. Valuable experience was gained regarding the efficient handling of temporary rock support behind the cutterhead, and the utilisation of prepared equipment for probe drilling, pre-bolting and pre-grouting.
- The final product was agreeable, except for the disadvantage of the cross section increase by blasting, creating irregular walls and delaying the completion.
- Available methods for cutting of non-circular tunnels were not feasible in normal hard rocks

The costs of a larger bored profile, big enough for two traffic lanes, could at the time only be expected to be competitive on a meter by meter basis under rock conditions favourable for tunnel boring. The possibility of developing methods to adjust the circular TBM cross section, e.g. by using additional drums with disc cutters to cut the lower corners, had been discussed. It was concluded that the large investment for a prototype TBM and the technical risks involved did not justify such approach.

The later development of tunnel boring, notably the utilisation of cutters with much higher thrust capacity, has opened for boring of road tunnels with diameters up to approximately 10 m economically also in hard rocks. Ongoing development of integrated rock support and water/frost insulation, to allow efficient completion for traffic, will also make tunnel boring of road tunnels more attractive.

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8 THE SVARTISEN HYDROELECTRIC PROJECT - 70 KILOMETRES OF HYDRO TUNNELS

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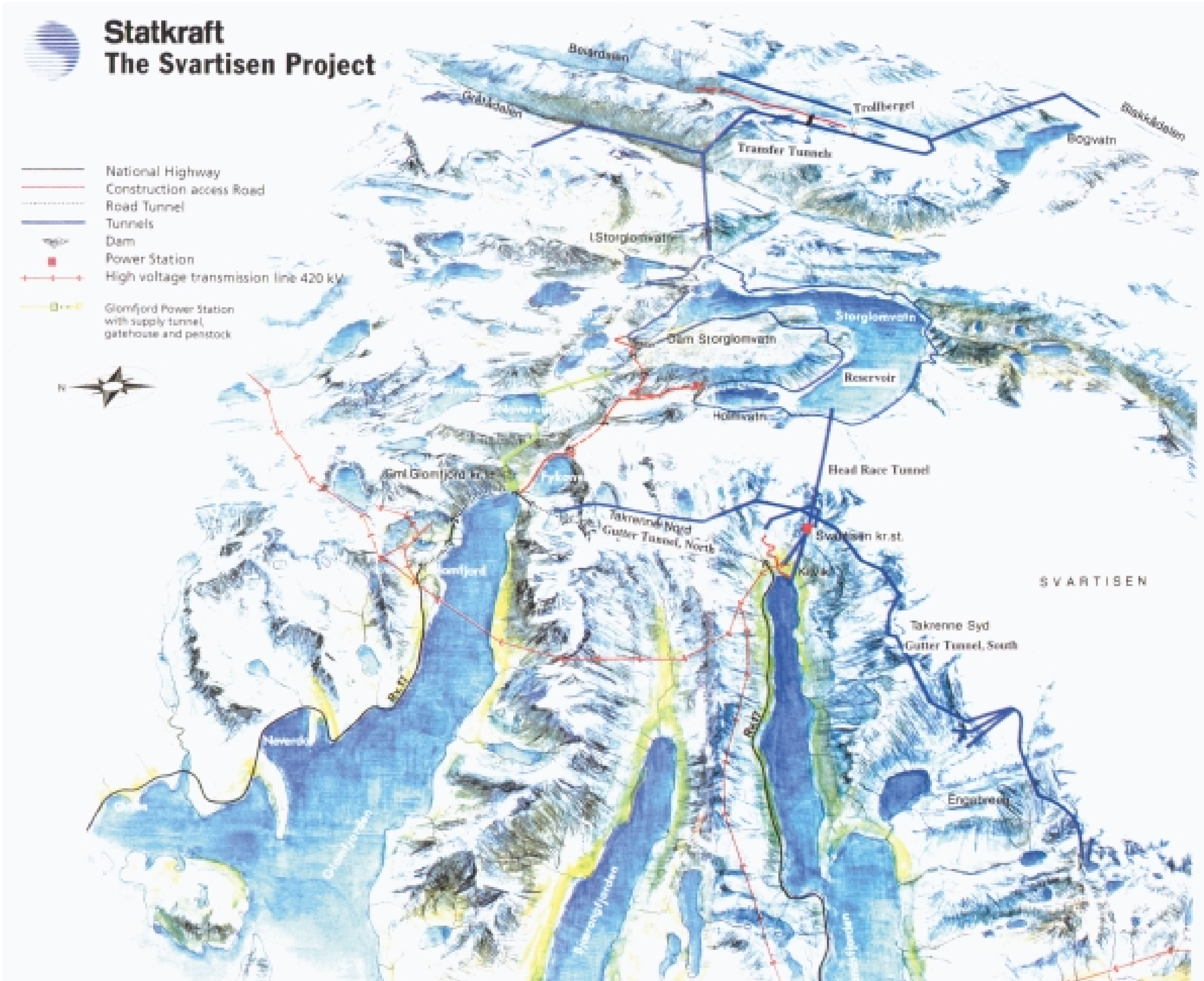
ABSTRACT: The planning as well as the construction works for the Svartisen hydroelectric scheme are carried out by Statkraft Engineering AS and Statkraft Anlegg AS respectively. Both companies are daughter companies of and 100% owned by Statkraft SF, the owner of the plant.
The first plans for the project were presented in the early 70's. Drill & Blast-tunnelling was regarded as the only realistic method, and the technical solutions, operational layout and the precalculations were based on that assumption.
The development of TBMs to stronger and more reliable machines able to cope successfully with the hard and homogeneous rock that could be found in the area, led to altered plans, reduced costs and last, but not least, to reduced damage to the terrain and improved environmental conditions, even for the tunnellers.
The experiences gained at the Svartisen Hydroelectric Project have brought TBM-tunnelling as method another long step forward.



1 INTRODUCTION

Svartisen Hydroelectric Project is a part of the Svartisen-Saltfjellet Hydroelectric Development Project and is situated close to the Arctic Circle, partly under the glacier Svartisen, or the Black Glacier (direct translation).

2 PROJECT DESCRIPTION

A system of transfer tunnels, about 40 km altogether, and shafts lead water from small lakes, glaciers and brooks down to Lake Storglomvatn, the large reservoir - very large actually. 5 TWh in stored energy makes it one of the largest in Europe.
From the reservoir the water flows through a 7.3 km long headrace tunnel down to the turbines in the power station at sea level.

Built about 550 m above the station are two gutter tunnels. The first, about 15 km long comes from the south, from a smaller glacier. The second is about 7 km long and comes from the north. Both tunnels catch water from brooks and glaciers through numerous shafts. Altogether the project comprises more than 90 km of tunnels, of which approximately 60 km are for water transfer. 56 km are TBM-bored.

The power station is planned with two Francis turbines, each of which will generate 350 MW. So far only one is installed, but when the second unit is installed the yearly production will be about 2,170 GWh.

The reservoir has two rockfill dams with asphaltic concrete cores, with volumes of 5.27 and 1.18 million m3 respectively.

The project head office for the civil construction works was established in Glomfjord, near the power station site in Kilvik, which was also the base for construction of the headrace and the tailrace tunnels. The construction site for the gutter tunnels was established at Storjord near the Kilvik site, but about 500 metres higher.

As for the transfer tunnels the construction site was established at Trollberget, situated about 200 km by road from the Glomfjord office.

The Construction Department, Statkraftverkene (Norwegian State Power Board) was assigned the construction contract. (The name of Contractor after reorganisation in 1993 is Statkraft Anlegg AS.)

Statkraftverkene, now Statkraft SF, is also the owner of the Svartisen Hydro Power Plant on behalf of the Norwegian State.

3 CHOICE OF TUNNELLING METHOD

The first plans for the project were presented in 1975. The potential possibilities in the TBM-method had been recognised, but since this was in the early stages of TBM boring in Norway and since TBM in hard rock was unusual and unsure, the Drill & Blast method was regarded as the only realistic method. Project design as well as budget prices were based on conventional method

During the second part of the seventies and beginning of the eighties several tunnel projects in Norway were successfully completed by means of TBM. Statkraft Anlegg AS started its first TBM-operation in 1981 and since the company has obtained a leading position in Norway with regard to development of TBM technology and utilisation for cost efficient tunnelling in hard rock.

From the very first meter bored there was a close cooperation between Statkraft Anlegg AS and the Division of Construction Engineering at the Norwegian Institute of Technology (NTNU) in Trondheim and with the Norwegian Geotechnical Institute (NGI). Follow up studies from all tunnels bored by Statkraft Anlegg AS and other contractors in Norway were carried out and registration of data was systematised and scientifically adapted. Prognosis models were developed and became efficient tools for planning of TBM tunnelling.

The design of tunnel systems and excavation methods for the Svartisen Hydroelectric Project were continuously reviewed and adjusted in order to benefit from the continuous improvements in TBM tunnelling performance. At the time of construction in 1987, a major part of the water tunnels was redesigned for TBM-boring.

The main reasons for the extensive use of TBM are:

- Headrace tunnel: Due to the estimated high advance rate one could carry out the tunnelling works from one face only, and still have the works finished within the time schedule.
  Due to improved ventilation conditions there would be no hazard to the tunnellers health, even at the far end of the tunnel. A hazard that would have made Drill & Blast from one heading impossible.
  With TBM it became unnecessary to open an adit in the other end and consequently unnecessary to build a camp with barracks for the crew, workshop and so on.
  Building a 5 km long road through untouched country for the transport of heavy tunnelling machinery and supplies would not be required. The road would have had to be kept open all year round and the cost would have been considerable. Damage to flora and fauna could be expected.

The hydraulic coefficients in the tunnel improved, by boring instead of blasting the head-race tunnel, and the tunnel area could be reduced from 98 to 57 m^2 without increasing the head loss.
The volume of excavated rock could be reduced accordingly with approximately 270,000 m^3.
The estimated cost reduction by using TBM was in the range of NOK 20 mill (about USD 3 Mill.).

- Transfer and gutter tunnels: Similar effects as described above was achieved in Vegdalen, one of the four tunnels in the eastern transfer system. This transfer system comprises more than 40 km of tunnels. The basic intention was to excavate through five adits, two at Trollberget, one in Vegdalen and one in Beiarndalen, some kilometres further up in the valley from the established adit, and finally one near Storglomvatn.
By utilising the experience gained through seven years and 50 km of hard rock TBM boring combined with the most recent TBM technology, one could manage to excavate the tunnel system from one adit only.

4. Trollberget

Fig. 1 shows the tunnel system. Water is collected from 12 intakes and brook inlets along the tunnels and transferred to the main reservoir Storglomvatn. The only crosscut for the whole tunnel system is situated at Trollberget.

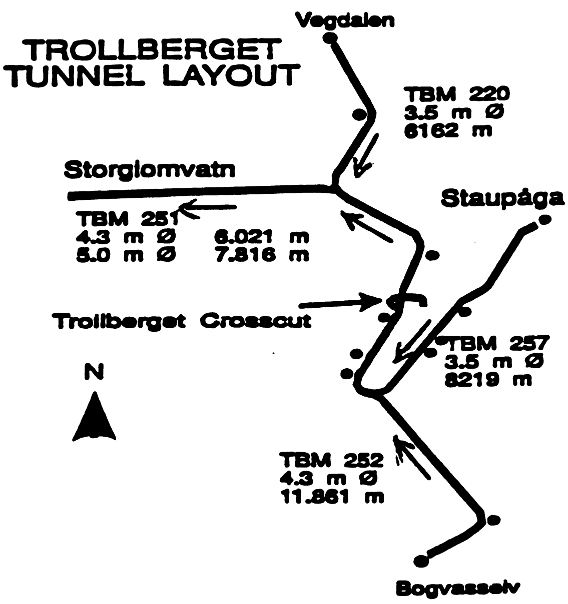


Fig. 1 The Trollberget tunnel layout

4.1 Mobilisation

Mobilisation started in August 1987. Access roads, camp, concrete plant, power supply etc. had to be established.

Excavating of the access tunnels and the underground erection chamber, workshop chamber, service area and silo for reloading muck from train to trucks started in October 1988 and was finished in July 1989. During this period 94,000 m^3 solid rock were excavated, including the 800 m long entrance tunnel and 700 m of the water transfer tunnel where the TBMs and back-ups for the two first tunnels were assembled and made ready to start boring in September 1989.

The design criteria for the layout were:

- The portal of the entrance tunnel is at level 420 m. The entrance tunnel had to be 800 m long with slope 1:8 in order to reach the level of TBM-tunnels.
- The service area should be able to support four TBMs boring simultaneously with total advance rates in the order of 800-1000 m per week.
- The dumping arrangement had to have sufficient capacity to cope with instant penetration rates of 4-6 m/h pr. TBM. There should be a large buffer capacity in order to even out the peak muck-production in the tunnels and the long time capacity of the trucks taking the muck from the silo to the tip-site some 1.5-2 kilometres away.
- The workshop should be able to maintain and do necessary repair works on 10 locomotives, 100 muck-cars, and several service cars, flatcars, remixers and auxiliaries.
- Four TBMs and back-ups had to be assembled and disassembled. Several of these operations had to be performed without disturbing the headings where TBMs were boring.
- A track system with enough switches and lines to handle the rolling stock of more than 150 units. A total length of about 800 m was required.
- The whole arrangement should be complete with cranes, communication facilities etc., in order to save manpower, and should be integrated in the permanent tunnel-system to minimise the excavating cost.

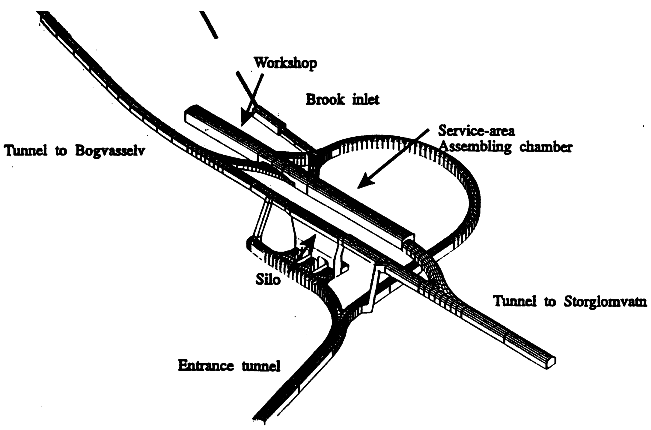


Fig. 2: Layout in the cross-cut area.

4.2 Geology

The area belongs to the Beiarn thrust Nape in the Caledonides of Northern Norway. Most of the rocks are of sedimentary origin and of Cambrian to Ordovician age. They are strongly folded and metamorphosed. The main strike direction of the beds is NE-SW, while the dip varies from horizontal to vertical.

The three main rock types are:

Mica Schist, Mica Gneiss	80%
Limestone, Marble	7%
Metasandstone, Granite, Granitic Gneiss Diorite	13%

The Mica Schist is partly a massive, garnet type and partly a calcareous type with well developed schistosity. Most of the rocks are in-between fracture class 0-I to III.

The Limestone occurs in beds with thicknesses from some centimetres up to more than hundred metres. Karstic features like caves and underground channels are often seen at the surface. The Metasandstone is a pure Quartzite. Lenses of Granite and Gabbro occur in rocks of sedimentary origin. The size of the lenses is from a few metres up to several hundred metres.

The DRI (Drill Rate Index) for the different rock types varies in the range of 35-75 and the CLI (Cutter Life Index) from 4-25. The unconfined compressive strength varies from less than 100 MPa to more than 300 MPa.

Combined with a great variation in jointing or fracturing of the rock mass, this gave net penetration rates from less than 1 m/h in the massive Gneiss to more than 6-8 m/h in the most favourable rock. Also the cutter life showed great variation. In pure Quartzite, or in mixed face condition with solid Gneiss or Quartzite together with Mica Schist, the cutter life might be less than 50 m^3 per cutter. In Mica Schist with well developed schistosity the average cutter life could be more than 400 m^3 per cutter.

4.3 TBMs and Back up-equipment.

Statkraft had in its possession one 3.5 m Ø Robbins TBM-model 117-220. When it was bought in 1981 the 220 TBM was a very powerful machine equipped with 15.5 inch cutters. In the period from 1981 to 1991 when it was taken into use at Trollberget, the «220» had bored four tunnels with a total length of 27 km, mostly in very competent and massive Granites and Gneisses, and it had been refurbished and modified several times. The cutter size had been increased to 17 inch.

Statkraft prepared tender documents for the necessary further three TBMs and obtained quotations from Wirth GmbH and from The Robbins Company.

During the evaluation the investment cost, the calculated performance, cutter cost, spare part cost and resale value were taken into account. The contractor decided on two 4.3 m Ø Robbins HP TBMs for the first two tunnels and one 3.5 m Ø Robbins HP TBM for the third tunnel. (HP = High Performance).

The Robbins High Performance TBM is basically

Tunnel	**Storglomvatn**	**Storglomvatn**	**Bogvasselv**	**Staupåga**	**Vegdalen**
TBM	Robbins HP	Robbins HP	Robbins HP	Robbins HP	Robbins HP
Production number	1410-251	1410-251-1	1410-252	1410-257	117-220
Year of production	1989	1989/1990	1989	1991	1981
Diameter	4.3 m	5.0 m	4.3 m	3.5 m	3.5 m
Cutters, diameter	19"	19"	19"	19"	17"
Number of discs	29	36	29	25	27
Max. thrust pr cutter	32 tons	32 tons	32 tons	32 tons	22 tons
Thrust, total	930 tons	1150 tons	230 tons	800 tons	600 tons
Cutter spacing	100 mm	100 mm	100 mm	90 mm	75 mm
RPM	11,94	11,94	11,94	12,5	10,8
Cutterhead power	7 x 450 Hp	7 x 450 Hp	7 x 450 Hp	4 x 450 Hp	4 x 200 Hp
Stroke	1,8 m	1,8 m	1,8 m	1,5 m	1,5 m
Weight of TBM	270 tons	290 tons	270 tons	180 tons	120 tons
Boring period	Sep.89-Oct.90	Jan.91-Jul.92	Sep.89-Apr.91	Jul.90-Apr.92	May 91-Jan.92
Tunnel length	6,021 m	7,816 m	11,861 m	8,219 m	6,162 m
Boring in curves	1 250 m		2.075 m	911 m	435 m
Bored volume	87,500 m^3	153,500 m^3	172,200 m^3	79,000 m^3	59,200 m^3

Table 1: Specifications for the different TBMs.

designed like a standard open hard rock TBM, but oversized in every way compared with the latter in order to make it able to bore with a thrust of 32 m tons per cutter instead of 20-25 m tons. The HP TBMs have 19 inch cutters and the main-bearing is tri-axial instad of tapered roller bearing installed on the standard version.

The 220 machine is equipped with a single track Muhlhauser back-up trailing a California switch. Also the back-up has been modified since it first came into operation in 1981.

Back-ups for the two Ø 4.3 m TBMs were also from Statkraft. They had earlier been used by Statkraft in two 4.5 m Ø tunnels at the Jostedalen Hydro Power Project and had served as back-ups for a Wirth TBM and an Atlas Copco Foro TBM.

The back-up for 3.5 m Ø HP Robbins was bought from MCS (Mining and Construction Services of Scandinavia).

All the old back-ups were greatly modified so that they all had the same basic outline and could fit the same rail system and equipment.

4.4 Additional Machinery and Equipment.

Basically the system for muck transport is the same for all the headings and can be specified as follows:

Locomotives	Schøma, diesel, 22-24 tons, 180 HP	8 units
Service-locomotives	Levahn, diesel, 9 tons	9 units
Muck-cars	Fosdalen, 10 m^3 hauling capacity,	
	bottom discharge, car bodies hanging	
	between bogies, 8-10 cars in each train.	100 units
Manrider cars	Fosdalen, 6 persons	11 units
Remixers	AMV, 6 m^3	2 units
Flatcars		6 units
Service-cars		2 units
Track	Track-width = 900 mm, rails 33-35 kg/m	
	steel-sleepers of type Trollberget,	
	c/c = 800-1000 mm	
Travelling speed	max. 30 km/h	

The bottom dump muck-cars are discharged into the silo which is 40 m long, 6 m wide and 20 m deep with a storing capacity of 1,200 m^3. The silo will be utilised as a sand trap when the tunnel system comes into function.

4.5 TBM crew.

The operator's cabin on the TBMs is mounted on one of the first sections of the back-up. The operator, one man on each shift, operates the TBM as well as the back-up by remote control. He also moves the muck-cars during boring and loading. The operator gets information from several instruments and from 10 TV cameras mounted on the TBM and on the back-up.

In addition to the operator there is one electrician and one mechanic working at the face, maintaining TBM and back-up and replacing cutters. They also install the rails, ventilation duct, water pipes and high-voltage cable and do rock support if necessary.

The number of loco-drivers taking the muck from the back-ups to the dumping place depends on the penetration rate, the size of the tunnel, the travelling speed of the train, the length of the heading and the time spent discharging the muck-car train. When the boring started each heading had one locodriver at each shift. At the end of the tunnels, when the train had to travel 12-14 km from the back-up to the silo with an average speed of approximately 30 km/h, there were 2-3 drivers per heading per shift. The loco-drivers join the crew at the TBM during cutter change and major repair works.

In addition to the crew directly involved in the boring, there were 4-6 men at each shift working in the service area and at the cutter shop and 2-3 truck drivers taking muck from the silo to the dumping site.

Norwegian regulations state that the average working hour for underground work should be 33.3 hours per week. Thus there are three sets of crew, each crew working two weeks with one week off. This gives a normal week of 102.5 hours available time in the tunnel.

4.6 TBM boring.

The first borer, Robbins TBM HP 1410-251 with diameter 4.3 m, started boring in September 89, and bored 6 km in the direction of the reservoir to the junction for the tunnel branch towards Vegdalen. A new erection chamber was established in the junction and the TBM diameter was increased to 5.0 m. The boring continued with this diameter for another 7.8 km to the reservoir, bringing the total length to 13.8 km

The second borer, Robbins TBM HP 1410-252, Ø= 4.3 m started boring towards Bogvasselv one week later than the 251, and bored 11.9 km without major geological problems.

Robbins TBM HP 1215-257 came into operation in August 1990. The TBM and back-up was erected in the main erection chamber and wheeled on dollies 4 km to an established junction and started boring the 8.2 km towards Staupåga.

In May 1991 at station 4,700 m from the junction the TBM came into a section with very bad rock conditions and a water ingress of more than 0.5 m^3/sec. The bedrock consisted of folded marble with karstic veins and clay zones. This caused a delay in the tunnelling of 3-4 months. Rock bolting, shotcrete, concrete and grouting had to be undertaken in order to advance the tunnel. The bad conditions lasted for some hundred metres. The rest of the tunnel was finished without problems of any kind and with advance rates averaging close to 200 m per week.

TBM No. 4 was the above mentioned 10 year old Robbins 117-220 which bored the tunnel branch towards Vegdalen. The 6.2 km long tunnel was finished without any problems with an average advance rate of 200 m per week

More specific data from the boring is given in the table.

TBM	Diameter m	Length m	Net. Penetr. Rate m/h	Net. Penetr. Rate mm/rev.	Advance rate m/week	Lifetime of cutters m^3/cutter	Boring % of total time	Rock support % of total	Rem.
Headrace 226	8.5	7.334	1.08	3.10	53	135	48.8	7.0	1
Gutter tunnel 220	3.5	9.273	2.78	4.29	127	88	45.3	7.4	2
Trollberget 251	4.3	6.021	3.76	5.25	125	145	32.4	8.8	
251-1	5.0	7.816	2.74	3.82	137	152	48.7	0.6	
252	4.3	11.861	3.55	4.96	181	150	49.6	0.1	
257	3.5	8.219	3.69	4.92	130	138	34.5	13.8	3
220	3.5	6.162	3.85	5.94	200	180	50.7	0.2	
Sum		56,686							

Remarks:

1. Included breakdown and replacement of three main bearings
2. Included three weeks stop due to cast concreting of fault zone
3. Not included 14 weeks stop due to water ingress and changed schedule

5 TBM BORING AT THE SVARTISEN HYDROELECTRIC PLANT

There has been a lot of production-records set for hard rock TBM-tunnelling. The best results are:

* Best shift	61.2 m	TBM 252
* Best day	90.2 m	TBM 252
* Best week	415.0 m	TBM 220
* Best month	1176.0 m	TBM 220

More than 95% of the tunnels have encountered good rock conditions, not requiring immediate rock support.

Some shorter sections have required rock bolting, shotcreting and even cast concrete in front of the cutterhead. The overburden is 600-1000 m along parts of the tunnel. Some of these areas have seen rock burst development several months after the TBM passed without problems. Rock bolting has been carried out in such rock burst areas to obtain the required working safety. There have also been some shorter jointed zones where, even if the TBM passed without difficulties, shotcrete with steel fibres and rock bolts have been applied to prevent rockfall in the future when the tunnel system has come into operation.

4.7 Conclusions

Statkraft had already TBM-bored more than 50 km in hard rock when the boring at Svartisen started, and had gained experience and know-how on geology, machine properties, the correlation between the two, and perhaps the most important of all, how to achieve tunnel metres in practice.

Even so we took a calculated risk when we started boring at Trollberget with a new generation of TBMs and what we considered as a rather optimistic lay-out. The boring at Trollberget has given a lot of new information and experience as input to further development in hard rock boring.

The High Performance TBM is a very strong machine. The main structure, the main bearing and the torque capacity allow high thrust required to achieve high penetration rates in hard rock. Thus the potential in the TBM itself is very great. However, our experience is that the cutters are unable to meet the rated load unless the conditions are almost ideal. No construction is stronger than the weakest link, which in our case and for the time being, is the cutter. There is still a great potential for improvement of cutters and cutter configuration on the cutterhead to obtain better cutter life and to raise the cutter load and thus increase penetration rates and reduce down-time due to cutter changing.

TBM-tunnelling is like running a factory. Machinery, equipment, crew and management are joined in one unit, which has the goal of doing the job in an optimal way. Very often too much attention is paid to the TBM as the main commodity in the process while the rest of the system gets less attention, and the result being a TBM without sufficient support to be able to utilise the TBM-capacity. The upfront planning and the quality of the crew and the management are as important as the TBM.

As a government-owned company, Statkraft has always felt the obligation to play an active part in testing new machinery and methods, and in accordance with give-and get traditions in the field, to share the experiences openly. Thus, the experiences gained at Trollberget has brought hard rock boring one step forward and has already been utilised in other Norwegian projects, proving useful in the struggle for higher tunnel production and improved economy.

9 ROCK STRESS PROBLEMS IN BORED TUNNELS

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ABSTRACT: The paper summarises the experience of rock stress phenomena in TBM tunnels in Norway. These range from serious problems with intense rock spalling in hard and massive rock, causing significant safety hazards and delays in work progress, to occurrences of mild buckling of schistose rock, or long term slabbing during operation of unlined water tunnels.

Possibilities exist for predictions of such problems, although this is not discussed in detail in this paper. The need to be prepared is highlighted. Especially, it is important to be able to perform rock bolting efficiently immediately behind the cutter head in order to maintain work safety and firm gripping for continued boring.

1 INTRODUCTION

Norwegian tunnelling experience includes a significant number of tunnels with high rock stresses. The problems are typically related to rock spalling due to anisotropic stresses below steep valley sides. This may be encountered during excavation of hydropower plants and tunnels in areas with dramatic topography, or in road tunnels along or between fjords under high cover. The experiences from deep level mining with the need to minimise pillar size have also brought significant contributions to the understanding of the rock mechanics involved.

The tunnelling community in Norway has a thorough understanding of the phenomena of high rock stresses. This includes the precautions during planning and the practical measures during construction to maintain safety and progress when rock spalling is encountered. Methods for predictions of rock stress problems had been established on the basis of experiences in blasted tunnels, Russenes (1974). Methods for rock stress measurements were well developed and frequently applied when problems were foreseen or encountered.

The rapid development and application of tunnel boring resulted in the need to handle rock stress problems also in bored tunnels. The experience should be of interest to anyone planning TBM projects where such problems may be encountered.

2 SILDVIK

The first TBM project in Norway to meet rock stress problems was the inclined shaft at Sildvik Hydropower Project. The headrace pressure shaft had an inclination of 45 degrees, 2.5 m diameter and length of 760 m. The rock was a quartz-biotite schist. The experiences are summarised below based on Skjeggedal (1980 and 1981) and Dahle & Heltzen (1981).

Problems with intense spalling were experienced in the access tunnel and in the power station area. This made it necessary to perform extensive rock support by rock bolting and sprayed concrete. In the access tunnel, rock stress measurements showed the major principal stress of approx. 21 MPa, the intermediate of 18 MPa, both close to horizontal. The minor principal stress was 8 MPa. The rock cover was about 400 m under the steeply sloping valley side. The rock had an uniaxial compressive strength of 150 MPa.

From this rock stress problems were expected, especially in the lower parts of the shaft. From the orientation of the major principal stress in relation to the shaft, it was assumed that the spalling would occur in the area between the roof and the face, giving problems with work safety during cutter change. It was foreseen that spalling in the left side of the roof and the right side of the invert could result in loss of support for the TBM gripping pads.

Although the configuration of the Wirth TBM left very little space for activities immediately behind the cutterhead, the contractor A/S Høyer-Ellefsen was well prepared. From the reports the following can be noted:

- The shaft had a relatively favourable orientation to the stresses. The intense spalling was therefore limited to the lower part of the shaft and diminished upwards.
- An effective rock support was performed with radial bolts, straps and wire mesh. The systematic support was co-ordinated with the progress without causing significant delays. This was made possible by a work platform behind the extra anti-sliding supports, and the efforts of 2-3 men in addition to the normal crew of 5-6 men. By providing galvanised materials, the installed support served both as primary and permanent support.
- It was not necessary to install the support just behind the cutterhead, as the spalling did not develop immediately.
- The work safety had a high priority. The preparations, which had been planned for even worse situatons, paid off by an efficient execution.

In conclusion, the conditions were as expected or better, and the thorough preparation allowed for an efficient completion of the shaft, with an improved work safety compared to Drill & Blast.

3 BRATTSET

In the transfer tunnel on Brattset hydropower project, some interesting observations were made during the tunnelling period. These are briefly summarised blow, Blindheim (1982, 1987). The overall experiences with tunnel boring on the project is described in other papers.

The transfer tunnel was excavated by a 4.5 m diameter Robbins TBM at moderate rock cover of 100-200 m, with the alignment close to parallel to the foliation in phyllite and mica schists.

In some sections the phyllite had an especially well developed schistosity, including smooth slickensided weakness planes. This resulted in buckling of layers in the upper left roof and lower right invert, but caused little disturbance during excavation, and the need for some scaling only. This phenomena may not even have been attributed to rock stresses in a blasted tunnel although the need for scaling could easily have been larger. The lack of blasting cracks in the bored tunnel obviously allowed the stresses to come closer to the tunnel contour, resulting in this mild buckling.

In the same paper, Blindheim (1982) summarised experiences with tunnel boring under high rock stresses from a number of visited projects abroad, such as deep level mines in South Africa, as well as tunnels in South and North America. The need to be able to perform rock support immediately behind the cutterhead and to maintain the gripping for the TBM was discussed. It was also recognised that in massive and strong rocks the spalling can occur in a more concentrated and intense manner in bored tunnels than in blasted tunnels, as the stresses can remain closer to the periphery of the tunnel. The advantage of utilising TBMs with an open configuration, and not shielded TBMs, was also highlighted.

4 KOBBELV

4.1 Project description

The experience from the tunnels at the Kobbelv Hydropower Project deserves special attention because of the magnitude of the problems due to rock stresses. A comprehensive investigation program was performed during construction to illuminate the causes of the problems and to help determine the practical measures to counteract the difficulties.

The tunnels were parts of a large hydropower development with the Norwegian Hydropower Board, NVE, as both owner and contractor. The scheme utilised an elevation difference of approx. 600 m, and included eight dams, 33 km of headrace and transfer tunnels, a 900 m inclined shaft, and a power station in rock. The project and experiences are summarised from Johansen (1984 and 1985), Østby Lyng (1989) and Myrvang & Johansen (1995).

Tunnel boring was chosen for about half of the transfer tunnels for the following reasons:

- The expected faster progress rates made longer branches possible and reduced the number of adits with connected mobilisation in an environmentally sensitive area with a very rough terrain.
- The total construction time could be reduced.
- The rock support could be reduced considerably.

The TBM tunnelling was performed by three Robbins open hard rock TBMs. The smaller TBM was already owned by NVE and had previously bored 8 km tunnel in hard rock on the Ulla Førre Project. The larger TBMs had bored successfully on the Walgau Project, and was hired from The Robbins Company. The specifications for the two TBMs that experienced rock stress problems are shown in Table 1.

Robbins model	204-215	177-220
Diameter, m	6.25	3.5
Cutter diam., mm	432	432
Number of cutters	44	26
Thrust per cutters, kN	230	220
Effect, kW	1050	600

Table 1: TBM key specifications

4.2 Geological and mechanical conditions
The rock types in the bored tunnels were mostly massive gneisses and granites. Some sections had quite extensive surface exfoliation, i. e. surface parallel extension cracks. Such phenomena are good indicators of high horizontal in-situ stresses.

Both in the bored and the blasted tunnels extensive spalling occurred in the roof, and in the bored tunnels also in the invert. In a niche excavated by blasting, two tunnel workers were killed due to a sudden large roof spalling. No spalling had occurred in the adjacent TBM tunnel.

A comprehensive rock mechanical investigation program was carried out, which comprised in-situ rock stress measurements, laboratory testing of mechanical properties, geological and structural mapping in the tunnels and on the surface, as well as air-photo studies. Stress measurements by overcoring of triaxial cells were performed in six tunnels, Myrvang (1993). The typical mechanical properties are shown in Table 2.

Compressive strength, MPa	89
Tensile strength, MPa	9.5
Young´s modulus, Gpa	18.5
Poison´s ratio	0.13
Density, kg/m^3	2640
Sonic velocity, m/s	3000

Table 2: Mechanical rock properties

Compared with granitic rocks normally found in Norway, all the parameters show relatively low values. This is thought to be the result of the coarse grained texture of the rock material. Under otherwise equal conditions, a low elastic modulus allows the rock to store more elastic energy when highly stressed. This may result in more violent spalling when the energy is released.

The rock was also tested for long term deformation or creep. Provided that the applied stress level during testing was approx. 75 % of the uniaxial compressive strength, the granitic gneiss showed a pronounced tendency to creep.

The in-situ stress measurements at the six locations showed a very consistent pattern dominated by high horizontal stresses. In general the principal stresses were as outlined in Table 3.

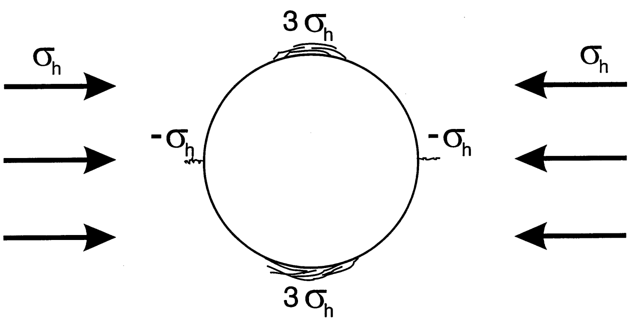
Major stress, MPa	27, horizontal
Intermediate, MPa	15, horizontal
Minor stress, Mpa	5, vertical

Table 3: Principal stresses

The horizontal stress field did not seem to change much with depth. Even at very shallow depth, the stress pattern was the same. In one case, quite intense spalling occurred in the roof of a Drill & Blast tunnel at only 25-30 m rock cover.

In theory, a circular tunnel in an uniaxial horizontal stress field will have a compressive stress concentration in the roof and invert of three times the horizontal stress. In the walls the tangential stress will be tensile with the minus value of the horizontal stress. See Figure 1. In this case, it gives a tangential stress of 3 x 27 MPa = 81 MPa in the roof and invert, and -27 MPa in the walls. The compressive stress in the roof/invert is by this only slightly lower than the uniaxial compressive strength determined on 62 mm cores. This indicates the probability of heavy spalling, as was observed. The tensile stress in the wall by far exceeded the tensile strength, and a tensile crack was actually observed at springline in each of the walls in one of the tunnels. These horizontal cracks typically started to develop 20-30 m behind the face, and extended for several hundred meters. They did not cause any stability problems.

Fig. 1 Tangential stress concentrations around a bored



tunnel

In general, if the vertical stress due to rock cover is introduced, the superposition to the horizontal stress field will reduce the tangential stresses in the roof and invert and the tensile stress in the walls. This should give less violent or reduced spalling in the roof and invert with increased depth, as was definitely observed in reality.

As the contour of a TBM tunnel is less disturbed than in a Drill & Blast tunnel, the concentrated tangential stresses may occur closer to the contour. This may result in more violent spalling than in a Drill & Blast tunnel where the high stresses are redistributed further away from the contour due to the blasting cracks. On the other hand, the rock mass in a TBM tunnel contour will have a higher remaining strength, thus spalling may not take place at all at relatively low stress levels. This was demonstrated in several areas where comparisons were possible in adjacent bored and blasted sections. The blasted sections needed more rock support.

4.3 The Tverrelvdal tunnel

The diameter of this tunnel was 6.25 m and a total length of 6,500 m was bored with TBM 204-215.

As a precaution, the TBM was fitted with rock bolting equipment on each side of the main body.

The first 1,700 m was bored virtually without stability problems. After the break-in period the net penetration was approx. 1.3 m/h and the weekly progress 70-80 m. The rock material had good boreability (DRI at 65-75), but the rock mass had little jointing. As boring with high cutterloads in the massive rock resulted in heavy vibrations of the relatively light body of this TBM, the rock bolting equipment required extensive maintenance and was eventually removed.

After about 1,700 m of advance, increasingly intense spalling occurred. Some spalling had been anticipated, but the intensity was unexpected. On some sections the rock at the contour was more or less crushed, and the progress of tunnelling was slowed down due to:

- the installation of rock support
- gripping problems due to overbreak in the walls
- hand clearing of debris and rock fragments

Fig. 2 Overbreak due to crushing and spalling

During the first critical phase, rock bolting was performed by jack-leg drilling. Later, working platforms were added making it possible to install rock bolts during operation of the TBM, as originally intended.

Mechanical expansion shell anchored bolts with length 1.5-3.0 m were used more or less systematically. The bolts were installed immediately behind the cutterhead roof shield. Experience proved that it was necessary to install the bolts as early as possible. In areas where the spalling removed the tunnel wall needed for gripping, time consuming use of steel beams and wooden supports became necessary to allow the TBM to move forward.

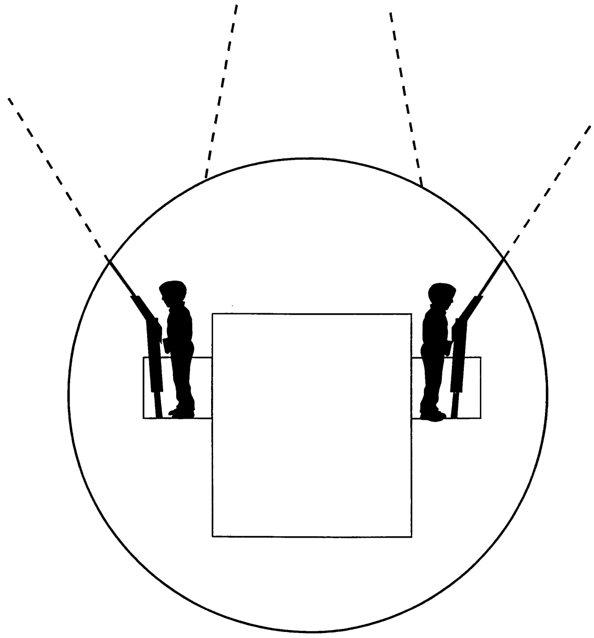


Fig. 3 Rock bolting from work platforms on the 6.25 m diameter TBM

The spalling also created a lot of debris and rock fragments that had to be removed. In particular this happened when the TBM advance was halted due to rock bolting and problems with gripping, and the spalling had time to develop. The use of a Bobcat 400 mini front-end loader, which could be operated under the TBM body, eased the mucking. The spalled and crushed material was dumped in small cars, which were pulled into the track laying area under the bridge conveyer. The normal TBM crew of four men was reinforced with two for rock support etc. Seven weeks were needed to pass through the worst section of 200 m.

Further ahead, the intensity of the spalling decreased, probably because the rock cover increased, thus counteracting the effects of the horizontal stresses. Still, on some sections the rock spalling could develop fast enough to occur 20-30 m behind the face, above the TBM backup where it was difficult to install rock support and clear debris. Systematic bolting behind the cutter head was then performed until conditions became more stable.

Out of the total section of 6,500 m, 650 m had extensive spalling requiring systematic bolting. A total of 2500 bolts were installed. This increased the construction time by 7-8 weeks, which could have been reduced to half the time with better preparation.

4.4 The Reinoksvatn tunnel

In this transfer tunnel a total of 9,500 m was bored with the 3.5 m diameter TBM, first 6.5 km south from Reinoksvatn access towards Linnajavri and then 3 km north towards Lievsejavri.

During the Drill & Blast excavation of the access tunnel, heavy spalling occurred in the roof, and dense bolting combined with sprayed concrete was necessary. The rock pressure problems started already at low rock cover due to high subhorizontal stresses induced from the nearby high rock massifs.

The TBM operation started without problems with a net penetration of 2.5 m/h. However, after some time, heavy spalling started to develop behind the machine. Overbreak occurred in the roof and as the TBM progress slowed down due to the support works behind the TBM, the spalling caught up and started to occur already at the TBM. Because the installation of support at the TBM was time consuming, the spalling had time to develop more intensely.

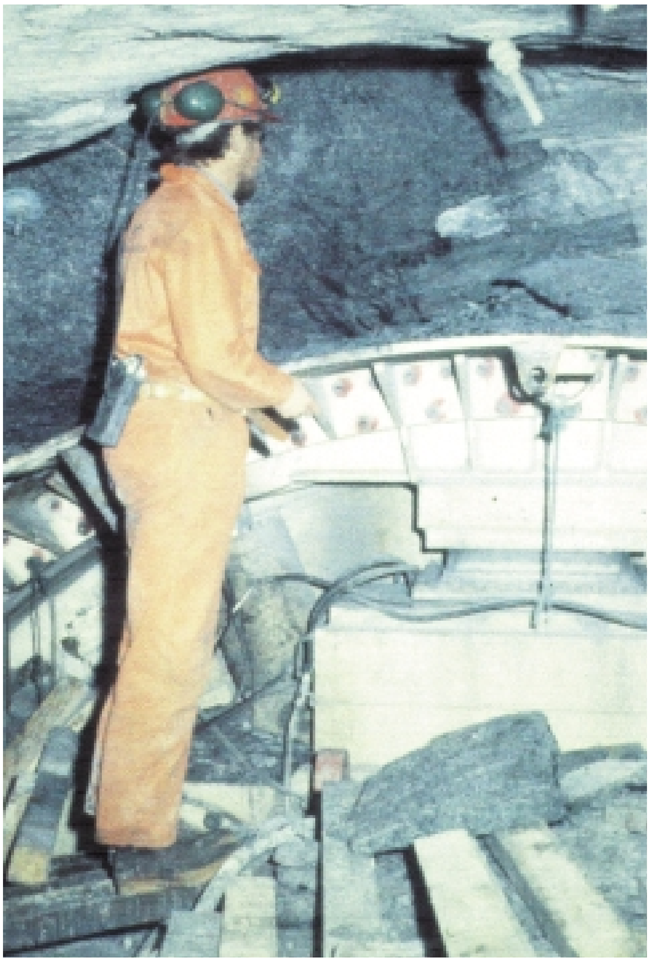


Fig. 4 Rock spalling overbreak in roof

The rock support, and the mucking out of debris in the cramped space of this smaller tunnel, was a straining experience. A procedure of applying approx. Six mechanically anchored bolts (diam. 20 mm, length 1.2 m) per 1.25 m tunnel length was adopted. The bolts were combined with straps and wire mesh to collect loose fragments for work safety. After the modification of the equipment, and the training period, the weekly advance could reach 100 m, including the installation of 500-600 bolts with straps and mesh.

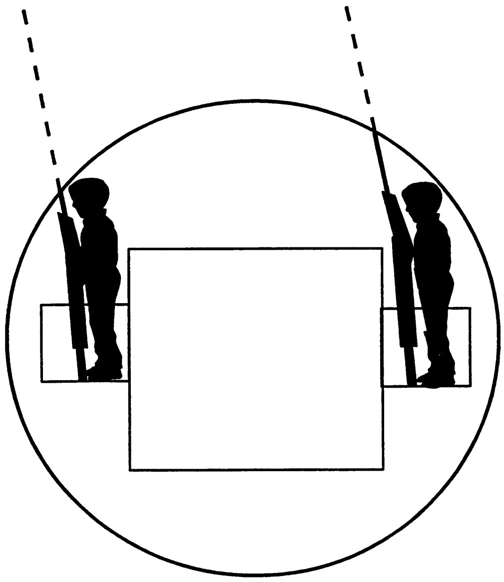


Fig. 5 Rock bolting from work platforms on the 3.5 m diameter TBM

In addition to the critical stability problems in the roof, problems were also experienced in the invert. The high stresses crushed and lifted the invert, and with it the rail sleepers. The rail became uneven resulting in derailing and transport delays. The rail was replaced in some areas, after concreting of a new invert.

The problems with spalling occurred in the first 1000 m of the total 9500 m bored, and delayed the construction by eight weeks. The average weekly progress during the period when spalling occurred was 52 m, as opposed to 120 m in the rest of the tunnel. As in the Tverrelvdal tunnel the spalling more or less ceased where the rock cover increased. Also here, the delays could have been reduced by half with better preparation. Rock stress problems had not been expected in this tunnel.

4.5 Evaluations

The high horizontal stresses in the Kobbelv area resulted in extensive spalling for sections of the TBM tunnels. This made it necessary to perform rock support immediately behind the cutter head. Problems with gripping and the time consuming task to clear out the debris in the confined space around the TBMs added to the delays. With detailed preparation, it is possible to reduce the impact of such problems on the overall progress. It is crucial that the necessary rock support for work safety is performed in an efficient manner without interfering too much with the operation of the TBM.

Compared to the Drill & Blast sections of the other tunnels in the project, and the problems experienced in the adjoining access tunnels and niches excavated by blasting, it is obvious to the authors that the extent of the spalling would have been worse if the tunnels had been excavated by Drill & Blast. The intensity might have been less, but the sections of tunnels affected would have been longer.

5 SVARTISEN

The tunnel boring of the Svartisen Hydropower Project is described in other papers. Here it shall only be noted that some rock stress phenomena were encountered. These mostly consisted of mild spalling behind the face without interfering too much with the progress of works. Although sections with high rock stresses were anticipated, little disturbance was actually experienced.

However, the observations reported by Vinje & Drake (1989) are of interest. After standstill of the TBM, during a weekend or for other reasons, it was frequently noticed that the net penetration rate would be higher for a few meters, probably due to fissures created by the stress relief ahead of the face.

6 ULSET

The tunnel boring on the Ulset Hydropower Project, as for the Brattset Project, is described elsewhere. Observations of the long term effects of presumed relatively high horizontal stresses will be briefly described.

With a diameter of 4.5 m the 5 km long headrace tunnel was bored in 1983-84 in mica schists and mica rich gneisses, with close to horizontal foliation.
The tunnel was inspected after two years operation as part of a large research program of checking the adequacy of rock support in hydropower tunnels, Bardal & Bruland (1986).

On a number of locations in the tunnel, mild long term spalling or slabbing had taken place. Adjacent to joints, in the otherwise rather massive rocks, loose blocks were found hanging on rock bolts or laying on the tunnel floor. On some locations these blocks were present in the form of long slabs of typically 0.2-0.3 m thickness, with widths about 0.5-1 m and lengths of up to several meters.

Apparently, the long term deformations had exceeded the strain capacity of the rock in the most unfavourable direction. Although a shallow keel was formed in the roof by this slabbing, it had no significant impact on the operation of the tunnel, as the head loss would be negligible except may be for some dislocated transversely oriented slabs obstructing the flow of water. The overall tunnel stability was not threatened by this slabbing.

Most of the fall-out could have been avoided by the installation of a few more rock bolts in the tunnel crown adjacent to the joints giving stress concentrations.

7 CONCLUSIONS

Rock stress problems in the form of intensive spalling may cause significant delays to tunnel boring. The impact can be reduced with careful planning and practical preparations.

It is possible to make predictions about the occurrence of rock stress problems, based on evaluations of tangential stresses and rock strength. As such predictions may not be accurate, the preparations will have to take the possible variations into account.

Open TBMs, with the means to perform rock support close to the face, can be utilised efficiently also under rock stress problems. The most efficient procedure will normally be to install rock support (with short bolts and straps) immediately behind the cutterhead for work safety and to maintain the contour. Heavier support can be installed from a platform behind the TBM, by supplementary bolting and sprayed concrete as necessary.

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10 SIX CASE HISTORIES

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ABSTRACT: The article presents a series of cases from the early period of Norwegian TBM tunnelling. Rock conditions favourable for TBM excavation were encountered mainly in these early projects. The main data for the boring of these projects are summarised in table 1 below.

Project	Year	Tunnel length, m	Main rock type	Dia. m	Thrust, kN/cutter	Cutter diameter mm(inch)	Average advance m/week	Average net penetr. m/hour	Average prod. rate
Brattset	1980-81	8,200	Phyllite, micaschist	4.5	178	394 mm (15.5")	122	2.85	43 %
Brattset	1981-82	3,200	Phyllite, micaschist	4.5	178	394 mm (15.5")	127	2.5	47 %
Brattset	1980-81	3,800	Phyllite, micaschist	4.5	178	394 mm (15.5")	113	2.5	47 %
Mosvik	1982-83	5,400	Gneiss, amphibolite	3.5	186	394 mm (15.5")	147	2.45	55 %
Stølsdal	1981-82	7,800	Granitic gneiss	3.5	178	394 mm (15.5")	67	1.65	40 %
IVAR	1989-91	8,100	Phyllite, micaschist	3.5	216	413 mm (16.2")	159.3	4.3	37 %

Table 1. Six early case histories of hard rock tunnel boring. Main boring data.

In the latest of these projects, the IVAR project, adverse boreability conditions were encountered. In spite of this, a very good performance was achieved with the TBM, due to available improved cutter technology, and layout of TBM and backup systems, as well as better understanding of boring conditions to be encountered due to improved prediction models.
The improvement of this technology has to a large extent taken place during the five earliest projects mentioned here.

1 ORKLA - GRANA HYDROPOWER PROJECT. HEADRACE TUNNEL FOR THE BRATTSET POWER PLANT

1.1 Project Description
With two Robbins TBM, 4.5 m in diameter, 15.2 km of tunnel was bored for the Brattset Hydro Power Project, comprising all together three TBM jobs. One TBM bored 8.2 km and the other bored 3.2 and 3.8 in two jobs. At an early stage in the planning process, the alternatives were 25 m^2 Drill & Blast or 16 m^2 fullface bored tunnel. Tunnel lengths, adits and other construction details were the same for the two alternatives. The full-face boring, however, showed significant advantages when looking at time consumption and costs. The total construction time for the entire power project could be reduced by six months, and a 75 % reduction in the need for rock support works was foreseen. This turned out to be a correct assumption.

1.2 Geology
The rock in the area consists of phyllites and micaschists with dykes of Trondhjemite (a tonalite). The foliation of the rock was steeply oriented and approximately parallel to the tunnel.

1.3 Equipment selection
The contractor used two Robbins TBMs with a diame-

ter of 4.5 metres. The first of the machines bored 8.2 km on a 0.2 % incline. The second bored 3.8 km on a 0.2 % incline and then 3.2 km on a 0.2 % decline. A sketch of the TBMs is shown in the figure below.

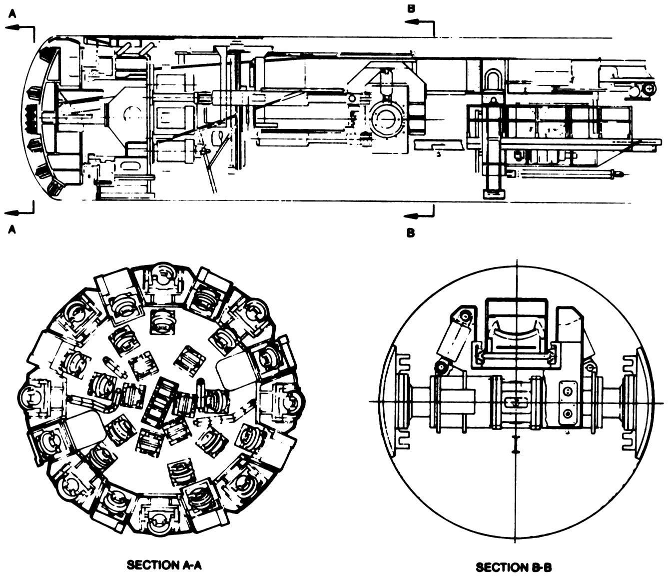

Fig. 1. Robbins TBM 148-212/213, employed at Orkla - Grana Hydropower.

The backup arrangements were delivered by LEWA, and manufactured in Switzerland. The backup consisted of 16 units of a length of 6 m on a 750 mm track. There were two tracks, allowing both trains to be loaded at the same time. During loading, the waggons are manoeuvred by a chain mechanism. Hence, only one locomotive was sufficient for the transport.

1.4 Results/Experiences
The production results for the two machines were almost similar, and reached a total of 122 and 113 m/week. The results obtained for best day, week and calendar month, were for the two TBMs 59.2/49.5 m, 200.2/176.0 m and 690.8/661.3 m. The average net penetration rates were 2.85 respectively 2.5 metres per machine hour. Achieved production rates, including rock support works, were 43 and 47 %.

2 MOSVIK HYDRO POWER

2.1 Project Description
The Mosvik Hydro Power Project is located in central Norway, near the Trondheim fjord. At the bidding stage for the 5,400 m transfer tunnel in hard rock conditions, a 16 m^2 area Drill & Blast tunnel was presented as an alternative to a 9.6 m^2 TBM tunnel. The 3.5 metres diameter TBM alternative was chosen, as it had significant advantages, such as:

- reduced total tunnel length, as one adit could be omitted
- reduced volume of rock support
- reduced mass volume, that was to be disposed in the sea
- reduced project time consumption on the intake side, particularly as winter operations were avoided.

In total, the TBM alternative had economic advantages compared to traditional tunnelling.

2.2 Geology
The dominating rock types along the tunnel were different types of gneisses, such as granitic, amphibolitic and micaceous gneisses with crossing quartz dykes. The rock later turned out to be relatively homogenous. Hence, the fissuring rate was low and gave only a small contribution to the penetration rate.

The main uncertain aspect of TBM tunnelling in this case, was the fact that a part of the tunnel would traverse a length of 500 to 1,000 metres of extremely hard rocks. The rest of the tunnel was situated in rock that was considered favourable for TBM excavation.

2.3 Special conditions
The job was carried out with a 6 man crew per shift on the boring and mass transport activities, with additional 2 men at the stuff workshop. On a 2 shift of 10 hours each per day, and an 8 hour shift on Saturday, the weekly total was 108 hours. Having three crews on a 2 weeks on and 1 week off basis, the working hours rules were followed.

2.4 Equipment selection
For the job, a new Atlas Copco Jarva MK 12 T was employed. The most important machine specifications were:

- diameter: 3.5 metres
- cutters: 22 15.5'' roller chisels and 1 12 '' centre chisel (four rings)
- feeding force: 19 tons per cutter
- rotation rate: 10.55 rpm
- stroke: 1.25 metres
- weight: approx. 140 tons

2.5 Results/Experiences
After an initial operation period of approximately 6 weeks, a production of 500 metres was completed. This gave an average weekly production of 86 metres. Throughout the next 4,900 metres, the average weekly production was 147 metres, with a best week of 246 metres.

Net penetration as measured over one week was 3.8 metres per machine hour. The accumulated average over the entire project was 2.45 metres/hour.

The machine availability during the running in period was 35 %. Problems with the backup were the most frequent cause of delay. During the production period, the average availability was 55 %, compared to the all high week of 67 %.

As for the entire job, the availability average was 52 %, with machine dependent stops (cutter changes, machine stops etc.) of 29 % and non machine dependent stops (back rig, rock securing activities etc.) of 19 %. Compared to the total hours available for boring, the average availability was 90 %, which is regarded as very satisfactory.

2.6 Problems encountered

The cutter ring wear was considerable, due to competent and abrasive rock types. A total of 930 cutter rings were used, and the life time of each cutter ring varied from 60 to 480 metres. The maximum wear was located to the 4 cutter locations inside the peripheral cutters, a phenomena which must be linked to the shape of the bring head.

2.7 Conclusions

The TBM performance on this job shows that fullface boring in hard rocks is a competitive alternative. There are still economic improvements to be made when working under these rock conditions. The efforts to reduce cutter costs will be of key importance.

3 THE STØLSDAL TUNNEL

3.1 Project Description

At the Ulla-Førre hydropower project a 7.8 km diversion tunnel was bored with a 3.5 m TBM, as were 4 penstocks of about 150 m each. In addition, two intake shafts with a diameter of 1.4 m and lengths 260 and 300 m were raise drilled.

3.2 Geology

Based on sampling and laboratory tests, the rock was characterised mainly as massive and fine to coarse grained gneisses with only a faint extent of foliation. The drilling rate index (DRI) ranged from 40 to 60, i.e. low to high, but mainly in the high area, which indicated good boreability. The bit wear index was in the range of 25-35, indicating low to medium abrasiveness of the rock.

The assessed weak zones were expected to require heavier rock support works than normal for TBM tunnels. As a conclusion, it was considered feasible to excavate the tunnel with a TBM at a lower cost than conventional Drill & Blast excavation.

3.3 Equipment selection

When going for such a comprehensive job in hard rock, it was of vital interest to have guarantees from the TBM manufacturer linked to the rock parameters for boreability and cutter lifetime from the Norwegian Institute of Technology (NTH). This was accepted by the manufacturers, even though the NTH-index was not a well known standard at that time.
The TBM was bought from Robbins. The most important machine specifications were:

- diameter: 3.5 metres
- cutters: 27 15.5'' including 2 double centre cutters (four rings)
- nominal thrust force: 18.2 tons per cutter
- stroke: 1.25 metres
- torque: 58 tons m.
- rotation rate: 10 rpm
- mean ring track distance: 6.5 cm
- weight: approx. 92 tons

The reason why the TBM diameter of 3.5 m was preferred to 3.0 m diameter, was to give retain the possibility of continuing the tunnelling with conventional methods. If further boring should turn out to be impossible and had to be terminated, the loss would not be too high, since the total price was not much higher than for a 3.0 m machine.

3.4 Experiences

The achieved rates of advance and cutter lifetime for this project, reflect the hard, massive and abrasive rock mass which was encountered. The rock mass properties were indeed at the limit of what had been experienced within hardrock tunnel boring. Hence, the economical benefit from the choice of TBM excavation in this particular case, was marginal. However, useful experience was obtained, particularly in terms of identifying rock mass properties which were essential in the improvement of the prediction models for advance rates and cutter lifetime.
The following table shows some of the production data for this project.

Maximum weekly advance,m	120.8
Average weekly advance,m	67.1
Maximum advance pr. shift	20.4
Net penetration rate, m/hour	1.45 - 2.80
Time consumption, boring	40.2% (of total)
Time consumption, cutter change	21.6% (of total)

Table 2. Experienced production data for the Stølsdal hydropower tunnel.

The basis for prediction of cutter wear and advance rates, were the index parameters DRI and BWI (drilling rate index and bit wear index) obtained from laboratory testing on rock samples collected at the surface along the tunnel. These prediction procedures proved to be inadequate as a tool for characterisation of relevant input data for the assessments for tunnelling. Variations in the jointing characteristics of the rock mass turned out to be the most governing geological feature.

4 THE IVAR WATER TREATMENT PROJECT, STAVANGER

4.1 Project Description

IVAR is a joint venture of several municipalities in the northern part of Jaeren in Rogaland County, south of Stavanger. The organisation was responsible for building a transportation system and a treatment plant for sewage water. The plant is situated approximately 10 km west of the city centre of Stavanger. The system will serve an equivalent to 240,000 persons, and is designed for a dimensioning water flow of 1.5 m^3/s, and a peak flow of 4.0 m^3/s.

The transportation system includes an 8.1 km tunnel from the city centre to the treatment plant, as well as a 4.1 km outlet tunnel from the plant to the sea.

4.2 Geology

The tunnels are located entirely inside an allochtonous unit in the Caledonian orogeny consisting of sediments with medium to high metamorphic grade. Phyllite and micaschist are the dominating rock types. The phyllite showed significant variations in mineral composition and structures. Frequently it appeared as a greyish green, fine grained micaschist with a strongly foliated curled structure. In local zones the phyllite occurred with well developed planar foliation with slickensided discontinuity surfaces, sometimes with graphite joint fillings.

Inside the phyllite, quartz appeared as a fine grained base structure and as irregular veins. The quartz content normally varied between 10 and 50 %.

4.3 Equipment selection

The contractor chose to purchase a new Atlas Copco Jarva MK12. The most important machine specifications were:

- Diameter: 3.5 metres
- Cutter head
- Effect: 900 kW
- Rotation rate: 12.2 rpm
- Torque: 705 kNm
- Available thrust force: 6,350 kN
- Stroke advance: 1.5 m
- Engines: 4 x 225 kW
- Gripper force: 18,870 kN
- Gripper configuration: T-shaped, front and rear
- Capacity for muck handling: 6 m tunnel per hour
- Machine weight: approx. 160 tons
- Cutter diameter: 413 mm

4.4 Experience

The boring of this tunnel turned out to be a major success, both in terms of achieved advance rates and improved cutter technology.

The phyllite turned out be extremely variable in terms of abrasivity. Hence, extremely high degrees of cutter wear were experienced. This was explained by the rare type of phyllite which was encountered in parts of the tunnel. This abrasive phyllite consisted of quartz grains in the clay fraction. «Normal» phyllites, with high content of clay minerals, were interbedded with the quartz phyllite, giving rapid changes in advance rates and cutter wear. Under these conditions the weekly advance rate varied between 40 and 210 m (2 - 4 meters per hour net penetration).

Cutter rings with a heavy duty alloy proved to be favourable for these adverse boreability conditions.

In the «normal» phyllite, record advance rates were achieved several times.

Maximum advance pr. shift (10 hours)	53.7 m
Maximum advance per week (106 hours)	350.8 m
Average advance per week (106 hours)	159.3 m
Average net penetration rate	4.3 m/hour
Net penetration rate in «normal» phyllite	3.7 - 4.9 m/hour
Average utilisation grade	37.2%

Table 3. Production data from tunnel boring at the IVAR project.

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11 THE MERÅKER PROJECT - 10 KM OF TUNNEL IN 12 MONTHS

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Scandinavian Rock Group AS

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Atlas Copco Robbins Inc., Seattle, USA

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ABSTRACT: New powerplants, tunnels and dams have been built at Meråker in Central Norway. A total of 44 km of tunnels with cross sections varying from 7 m^2 to 32 m^2 have been excavated in hard rock formations. Ten kilometres were excavated by a High Performance Tunnel Boring Machine (HP TBM). This paper gives special attention to the TBM drive and equipment selection, including planning, site organisation and performance.

1 INTRODUCTION

Meråker is located 80 km east of Trondheim. Five small hydropower stations were built in the years 1890 to 1915 to provide electric power to the Meraaker Smelter. In 1987 it was decided to upgrade and replace four of the generating facilities, increasing the annual output from 200 GWh to 590 GWh.

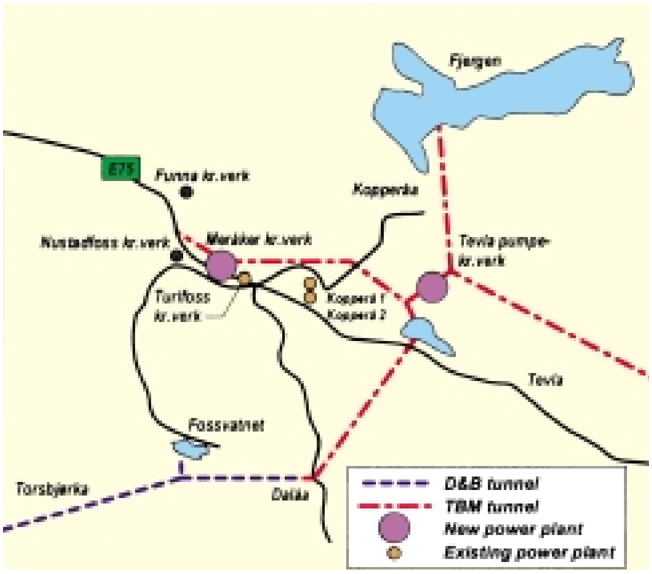

Fig. 1 Overview of the Meråker Project

2 TIME SCHEDULE

The construction started in September 1990, and the power plants, tunnels and dams were commissioned 1994. By the end of 1992, all 44 km of tunnels had been completed, approximately nine months ahead of schedule.

3 CONTRACTS

The client, Nord-Trøndelag Energy, invited bidding, and the civil works were divided into two contracts: The Tevla Power Station contract including 17 km of tunnels and two rock fill dams, was signed with the VSF-Group, a joint venture consisting of A/S Veidekke and Selmer A/S.

The Meråker Power Station contract with 27 km of tunnels, was awarded to Merkraft, a joint venture consisting of Eeg-Henriksen Anlegg A/S and A/S Veidekke.

4 GEOLOGY OF THE TBM DRIVE

The rock types expected in the project area consisted mainly of Cambrian and Ordovician metamorphic sediments with metagabbro intrusions. The metagabbro was considered extremely hard and massive with unconfined compressive strength up to 300 MPa.

Six different rock types were anticipated along the tunnel.

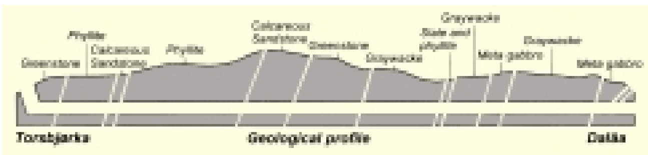

Fig. 2 Geological Profile.

To obtain the best possible basis for the tender, Merkraft supplemented available geological information with its own comprehensive survey before and during the tender period. The survey was carried out in cooperation with the Norwegian University of Science and Technology (NTNU) and the Norwegian State Power Board (Statkraft).

The tests from the 10 km Torsbjørka - Dalåa indicated that the rock parameters varied from a relatively soft phyllite with a Drilling Rate Index (DRI) of 60 and Fissure Class II+ or more, to a very hard metagabbro with DRI=32 and Fissure Class 0+; according to the NTNU Classification System of Report 1-88.

The formations of graywacke and sandstone appeared as mixed face conditions. Hence, the high rock strength and the wide variation in boreability characteristics became important issues in the selection criteria for a TBM.

5 SELECTING THE TBM

During the 1980's several major projects in Norway were completed in hard massive gneiss and granite by the TBM method. These were all excavated using state of the art machines, designed with capacities up to 222 kN per cutter. Though most projects were successful, technically and financially, actual field tests and studies showed that the projects could have been done even faster.

Experience showed that the machines including hydraulics, electrics, main bearing, cutters and cutter housings could not sustain the high thrust levels needed in the massive, abrasive and hard rock types. It became obvious that a substantial ROP gain would be realised if machines and cutters were designed to take higher thrust.

In the softer rock formations several machines also showed torque limitations at a rate of penetration of about 5 m/h.

In 1989, Statkraft decided to put into operation a new generation machines for the Svartisen hydropower project. During the period 1989-1992, three Robbins HP machines were purchased for boring more than 30 km of tunnels, with diameters of 4.3, 5.0 and 3.5 m respectively.

With the above mentioned experience in mind, the contractor specified that the TBM should:

- Be able to efficiently bore through hard massive metagabbro, greenstone, graywacke, mixed face conditions as well as the softer phyllite.
- Be able to increase the diameter to 4.2 m without changing the basic HP concept.

These requirements lead to a Robbins HP TBM, designed to bore with an average cutter load of up to 312 kN per cutter using 483 mm cutters.
Close co-operation between Statkraft and The Robbins Company lead to further development of the HP concept and ideas, eventually to the final design and manufacture of Model 1215-265.
Brief Machine specifications were as follows:

- Maximum recommended cutterhead load: 7,900 kN
- Cutterhead power: 1,340 kW (335 kW x 4)
- Cutterhead RPM: 13.4
- Number of cutters: 25 single discs
- Machine weight: approximately 200 tons

6 MACHINE DEVELOPMENT CHANGES

The experience at Svartisen with the 3.5 m diameter HP TBM 1215-257, indicated unusually high cutter wear and cutter bearing seizures in the transition area of the cutterhead when the average cutter load exceeded 265 kN per cutter. This clearly limited the practical and effective use of the machine at full thrust, and meant extra costs and downtime for cutter changing. The cutter profile and cutter spacing for the 1215-265 were therefore revised and changed according to Fig. 3, to provide a better load distribution and a closer spacing in the critical zone.

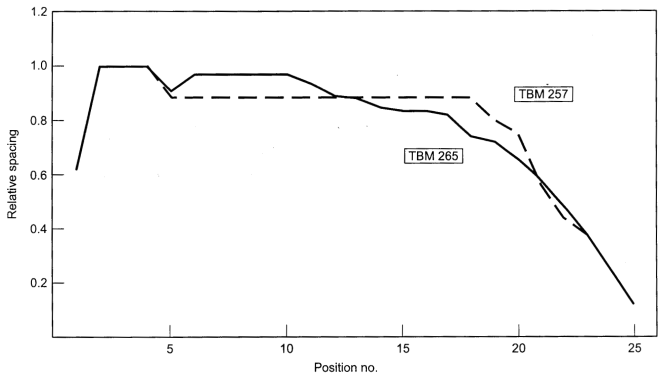

Fig. 3 Spacing 1215-257 and 1215-265

The modification turned out very satisfactory, and made it possible for the first time to bore with a sustained load of 312 kN per cutter.

The Svartisen Project and the Hong Kong Cable Tunnel Project experiences were transferred, and lead to additional modifications.

7 BORING PERFORMANCE

Fig. 4 shows that the make 111-265 bored 10 meters per hour and more during certain periods. The torque did not cause any problems and the muck handling system performed well.

ROP turned out to exceed the contractors scheduling prognoses. From the figure 5 one will also see that the ROP out-performed the NTNU 1-88 prognoses.

During the first four-week period of boring more than 1,000 meters tunnel had been achieved. The entire tunnel was finished six months ahead of schedule.

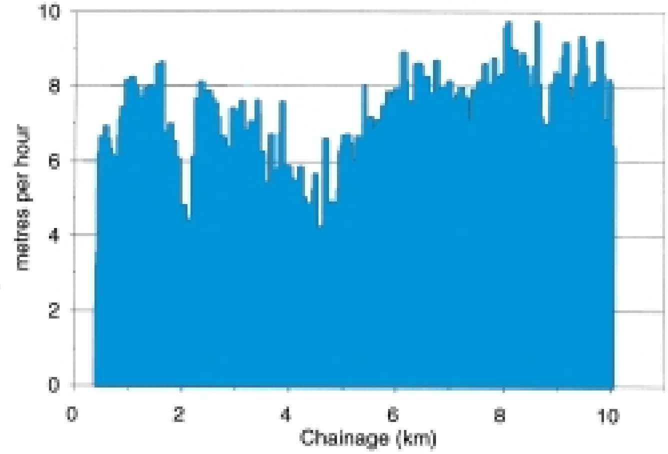

Fig. 4 Actual ROP over the Tunnel Length

7.1 Advance Rates and Utilisation

Norwegian regulations allowed Merkraft to operate 100 shift hours per week only. In spite of this limitation, the actual advance rates at Meråker averaged 253 meters per week, or almost 100 meters more per work week than ever before achieved in Norway. During the first four-week period of boring more than 1,000 meters tunnel had been achieved. The entire tunnel was finished six months ahead of schedule.

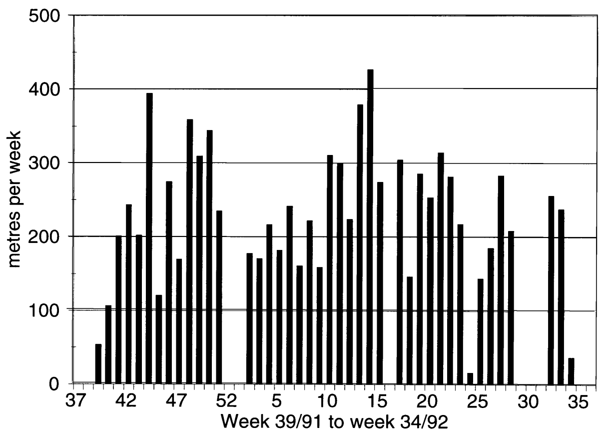

Fig. 5 Weekly Advance Rate

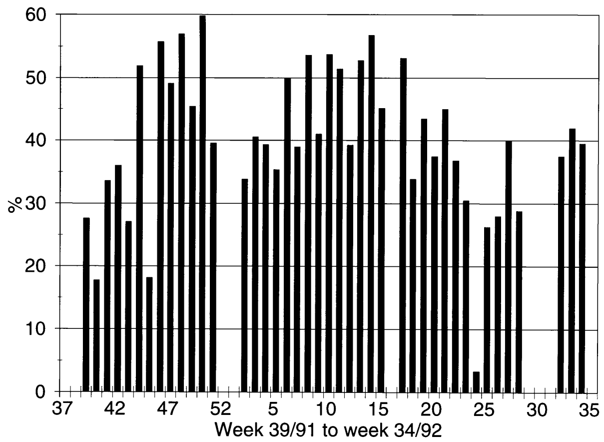

Fig. 6 Machine Utilisation

The TBM utilisation rate, defined herein as actual boring hours in percent of available shift hours, are shown in Fig. 6. The reduced utilisation towards the end was due to in-sufficient muck transport capacity over the last four km (two muck trains and one locomotive).

Increased average ROP and thus reduced construction time had been well possible if one additional locomotive and one California switch had been made available for the phyllite section towards the end of the tunnel.

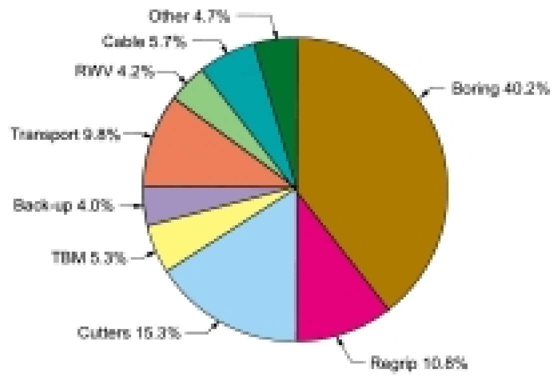

Fig. 7 Average Machine Utilisation

The results from Meråker can be summarised as follows:

- Best ROP over one single shift: 9.54 m/h
- Best shift (10 hours): 69.1 m
- Best day (2 x 10 hours shift): 100.3 m
- Best week (100 shift hours): 426.8 m
- Best month (430 shift hours): 1,358.0 m
- Average ROP: 6.4 m/h
- Average weekly advance rate: 253.0 m

8 CUTTER WEAR

The greenstone turned out to be the most difficult to bore. It caused the greatest amount of cutter wear and downtime for changing cutters. The variations of the mixed face conditions were also major factors for the high cutter consumption.

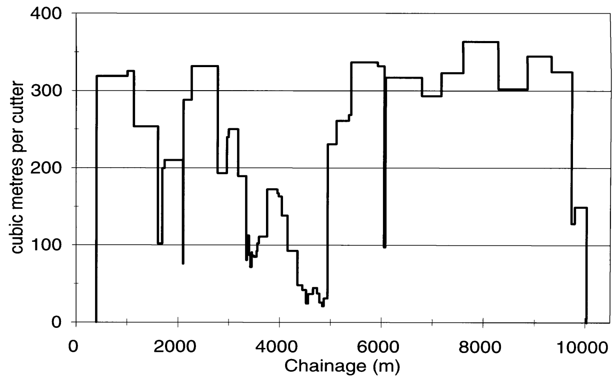

Fig. 8 Cutter Life

The great variation in cutter life was surprising. The TBM never had a ROP lower than 4 meters per hour. The cutter life varied from about 300 m^3 to 30 m^3 per cutter ring.

The overall downtime due to cutter changes was kept on a reasonably low level. This result was partly due to the improved load distribution made possible by the modified cutter profile and spacing. The modifications also meant changing more cutters in series, hence saving time.

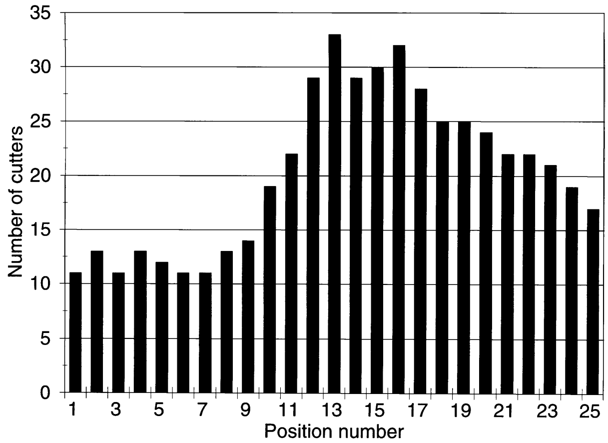

Fig. 9 Cutter Changes

9 MUCK TRANSPORT

The transport arrangement from the heading to the silo unloading station consisted of two muck trains, each with nine 10 m^3 bottom dump mine cars. The two trains were moved in shuttle service, maintaining an average speed of 25 km/h turn-return to the heading sloping 0.2-0.8% uphill. Each train load had a capacity of three boring strokes, equal to 4.5 m tunnel production. A rail bound back-up with towed California switch ensured a high production system.

The mine cars dumped the muck into a 300 m^3 silo, placed within the mountain. A subcontractor with commercial type trucks transported the muck up the 1:8 decline access tunnel and to the disposal area approximately 500 m outside the portal.

Due to high penetration rates of more than 10 m/h, the tunnel muck haulage itself lacked adequate transport capacity on the last 4 km of the 10 km long drive. However, the cost to install a bypass switch, an extra locomotive and extra personnel was estimated to be higher than the actual cost of waiting. The weekly advance rates were anyway far ahead of schedule, and no special bonus was offered by the client for early completion.

10 ASSEMBLY/DISASSEMBLY

The 265 TBM was shipped from Seattle to Norway in components not exceeding 87 tons. The shipment arrived at the site in August 1991 and the TBM was assembled underground. Three and a half weeks later the TBM started boring. Detail planning, experience and staff training, including extensive training for key personnel at the manufacturer's plant, made such short assembly period possible.

Once the tunnel was completed, the cutterhead was disassembled and removed through an existing 100 m long intake tunnel. The rest of the machine and back-up travelled out on the tunnel rails.

11 SITE ORGANISATION AND STAFF

Norway has long been recognised for its cost efficient tunnelling. Some of the main reasons may be the low number of staff, crew flexibility and capability, and the use of modern and well maintained equipment. At Meråker, 16 men covering three shifts were employed, each working the regular 33.6 hours per work week.

This crew covered all operations including boring, rock support installation, mucking, work shop and cutter repairs. Small crews should only be considered when experienced, flexible, dedicated hands are available. Fair pay and bonuses do the rest.

The crew at the face worked on a rotation system work face to improve teamwork. One operator controlled the TBM and the filling of trains from the cabin mounted on the back-up. One mechanic, one electrician and one locomotive driver handled all the other duties.

The crew was paid based on actual production. This meant that the machine had to be properly maintained and repaired to prevent downtime.

The TBM site management included five persons. These also supervised the 5 km long 20 m^2 Drill & Blast tunnel and the tunnel intake construction.

12 ROCK SUPPORT

The contractor's site investigation during the tendering period indicated only a minor need for rock support when using the TBM method. The amount actually needed ended up even less. Over the 10 km length, only 140 bolts and 44 m^3 of shotcrete were used for support. The client's original estimate included 900 bolts, 300 m^3 of shotcrete and 200 m^2 of liner plates for the TBM tunnel. On the Drill & Blast tunnels, the bid documents turned out to be nearly accurate with an average of 250 bolts and 33 m^3 shotcrete per km.

The average production rates for the Drill & Blast tunnels were approximately 80 meters per week per tunnel heading.

Tunnelling in Norway by tradition utilises the bearing capacity of the rock itself, using support only as and when needed. The smooth excavation provided by the TBM, the small cross section and the rock quality were major reasons for the small amount of rock support required at Meråker.

13 SUMMARY

The Meråker Project has demonstrated a new standard in tunnel production. The high advance rates and reduced need for ventilation give possibilities for longer tunnel drives, reduction of overall construction time and cost savings.

The production planning, the choice of machine and back-up, and the experienced and dedicated staff were the main ingredients to successful hard rock tunnel boring performance.

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12 NORWEGIAN TBM TUNNELLING. HEALTH AND SAFETY

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ABSTRACT: This article describes experiences from different investigations on health and safety related to TBM tunnelling in Norway. These investigations include a majority of the TBMs used in Norway from the start in 1972 up to 1993. Stresses on the environment are evaluated and compatibility with the existing environmental demands, and recommendations about choice of solutions and measures are given.
One of the main conclusions is that significant environmental improvements can be obtained with relatively small adjustments and modification of current operating methods. This relates in particular to dust, noise and ventilation.

1 CONCLUSIONS AND RECOMMENDATIONS

TBM operation has reduced/eliminated typical environmental stress (blasting fumes and diesel exhaust) which are dominant in conventional tunnel blasting. In spite of this, TBM still represents a significant stress on the environment for the operators, especially where it concerns dust, noise and to some extent vibration whilst boring in hard, quartz-rich rock types.

Correctly dimensioned and adjusted ventilation, correctly designed dust cleaning system and use of water at critical points are crucial for the dust situation.

«Critical points» mean in front of and behind the cutterhead (hosing down walls), loading point, conveyor belt, and walkways. The dust cleaning system (wet washer, Turbofilter or electrofilter) must be adequately dimensioned and be suitable for the type of rock, quartz content, particle size and additional local conditions. There must be a correct balance between in-going and out-going air volume. Recommendations about dimension criteria are given in the different site reports (available from SINTEF). The noise picture is dominated by low frequency noise (up to 114 dB, frequency < 250 Hz). Most current hearing-protectors have poor damping in low frequencies. Enclosure, screening-off and generally good sound-damping of rest-rooms and operator's cabins and workplaces are therefore important. Vibrating constructions can be damped with rubber lining or rubber/steel springs. Operator's cabins and rest-rooms are placed on the back-up equipment. Routines for cleaning, use of slip-free underlay and best possible lighting of walkways need closer attention. Satisfactory job-rotation, training, information and motivation are important key words in reducing the psycho-social demands.

2 PROJECT BACKGROUND.

The first TBM started in Norway in 1972. Since then, close to 300 km have been bored. Use of TBM has, compared with conventional tunnel blasting, reduced or completely eliminated environmental pressures such as blasting fumes, diesel exhaust, block-fall; but at the same time has resulted in increased pressures elsewhere.

Questionnaires show that TBM operators perceive the following environmental situations as most onerous:

- noise
- dust
- vibrations
- heat (change of cutters)
- repetitive work
- ergonomic pressures

A clear majority fear health risk on account of long-term effects of dust, noise and vibration more than the risk of accidents of acute, mechanical character or pressure injuries.

The working environment can be a limiting factor for the use of TBM for example in hard and quartz-rich rock types. Previous investigations have shown that substantial environmental improvements can be achieved through focusing the environmental issues.

The following environmental conditions have been evaluated to a greater or lesser degree in the investigations: dust, noise, vibration, temperature and relative humidity, radon, natural radioactive background radiation, electromagnetic radiation from electromotors, methane gas, scraping/stumbling dangers, monotony, ergonomic conditions and lighting.

3 BASIS MATERIAL

The project contains a review of environmental data from some 20 Norwegian TBM tunnels investigated in the period, in addition to a smaller amount of TBM data from abroad.

4 RESULTS

The operators of older TBMs were placed in the proximity of the cutterhead. They were without sound-proofed cabins and exposed to very high noise and vibration levels. The noise picture is dominated by low frequency, continual and monotonous noise, which can be difficult to dampen. 114 dB is not unusual in the low frequency region, but is about 800 times louder sound than 85 dB. The equivalent noise level outside the operator's cabin placed at the cutterhead is measured at 100 - 110 dBA. Inside a reasonably well insulated operator's cabin the noise level is measured at 64 - 80 dBA. Here we must be aware that a noise level of 100 dBA for 15 minutes corresponds to 85 dBA for 8 hours.

Recommended limits outside «danger to health» from vibration are often exceeded after 1 - 4 hours. Periodically, higher vibration levels are measured on older TBMs in the operator's seat than on the frame of the machine itself where the seat is mounted. Badly maintained TBMs have a higher noise and vibration level than well maintained machines. Work rotation can be used to reduce environmental pressures for the operators. On newer TBMs the operators are better placed environmentally, and at the same time operator's seats and cabins are well noise and vibration proofed.

Boring with TBM in quartz-rich rock types has great potential for development of silicosis with long-term exposure. Measurements show that the threshold limit values (TLV) are exceeded in 50% of the cases. With the halving of the TLV for quartz-holding dust, which was passed in 1990, there must be expected an increase in the number of excesses, if the necessary measures are not taken. Ventilation and vacuum-cleaning apparatus are shown as a principle in Figure 1.

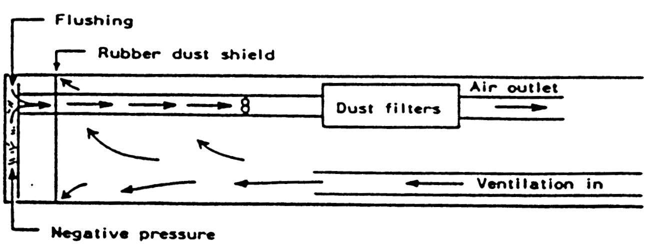

Fig. 1. General layout of ventilation and dust extraction unit.

Dust problems with TBMs in Norway are not primarily linked to incorrectly dimensioned main ventilation, but in most cases are caused by poor function and working level of the dust remover. The dust problem must be tackled at the source, and with a combination of measures. It is of little effect if one, to a too great extent, concentrates attention on a single measure, and forgets or is careless with the others.
Important measures are as follows:

- Adequate supply of fresh air, and correct distribution of air along the TBM and back-up equipment.
- Good packing between the dust shield and the tunnel walls. Minimal contact area against the cutterhead area.
- A pipe/duct system from the cutterhead to the dust separator designed according to flow mechanics. Avoid pipe bends and restrictions.
- Suction opening for dust from the cutterhead should be placed in the upper third part of the dust shield to reduce the proportion of coarse particles.
- Operator's cabins and rest-rooms with overpressure ventilation.
- Local encapsulation and vacuum cleaning can be possible in special cases.
- Use of water as prescribed.

If a dust separator is chosen the supplier's specifications must be closely followed. If not, reduced separation will result. A wetwasher is cheaper than a Turbofilter, but has a higher energy requirement, so that operating costs are normally higher. A low-pressure washer of the Sepax type has poorer separation than the Rotovent high-pressure washer.

An optimal Turbofilter, because of its working-method, will have a better separation efficiency than a wetwasher (the former uses a stopfilter). Separation efficiency for wet washers is good down to a particle size of 3 micron, but falls off quickly thereafter. For electrofilters the separation degree is critically dependent on the speed of the air over the filter. The working method, separation degree, energy requirements and trouble sources for particular types of dust separator are discussed in the site reports (available from SINTEF). Recommendations are given for dimension criteria on fresh air capacity and suction capacity.

Experience shows that with concentrated and systematic efforts, and adjustments and modifications of the older TBM set-up, dust, noise and other environmental conditions can be substantially improved. TBM represents a highly mechanised operations type, where often physical environmental pressures are reduced and psychological ones increase. Satisfactory instruction, information, motivation and work rotation are important keywords for a reduction in psycho-social stresses.

Emission of radioactive radon gas, or methane gas which can ignite and explode, are often local in effect and difficult to predict in advance. Both radon and methane are of secondary interest in Norway.

Electrical installations and motors can cause high electromagnetic fields with risk for health injury. Investigations show that exposure is low, but a little higher than one is exposed to in a normal living situation.

The project on TBM Health and Safety has given useful information and knowledge about a series of work environment conditions involving TBM operations, of which only a few dominate, i.e. dust, noise, vibration.

13 TBM VS DRILL & BLAST TUNNELLING

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ABSTRACT: In Norway around 5,000 km of tunnels have been excavated. Out of this, 29 tunnels with a total length of 173 km have been bored with TBMs. Most of the bored tunnels are in hydro power construction. Based on this experience, pros & contras for choosing TBMs or not is discussed in this article.
Today TBM tunnelling is a very competitive alternative for long tunnels also in hard rock, and first of all for water tunnels. TBM is, however, also a realistic alternative for other tunnels, like road and railway tunnels.

1 CONSTRUCTION TIME

1.1 Excavation time for different types of rock and cross section

A general statement is that the progress of TBM boring is much higher than with Drill & Blast (D&B). However, the progress in TBM boring is much more dependent on the rock conditions than D&B. The progress of D&B may vary some 20% due to drillability and necessary number of holes, while the progress of TBM boring may vary more than 500%. In some very special cases the progress with D&B may even be higher than with TBM.

Internationally, the compressive strength is the most commonly used rock parameter when assessing a TBM-project.

In this country however, a set of parameters as developed by the Norwegian University of Science and Technology, NTNU, are commonly taken into considerations.

These are:

- Drilling Rate Index
- Fracturing. Type, orientation and spacing of fissures
- Abrasiveness and porosity

The main machine parameters for TBM are:

- Cutter thrust
- RPM
- Cutter size and spacing

For very hard rock the penetration when boring may be drastically reduced if the machine parameters do not meet the actual rock parameters.

For D&B the modern Jumbos with hydraulic hammers will be of sufficient strength, and the boring time for one round does not differ very much.

A good weekly production for D&B is 80 m for cross sections of 50 m^2; and more than 100 meters for smaller cross sections. For TBM a good production may be in the range between 150 and 400 meters dependent on rock conditions, machine parameters and diameter.

1.2. Time for erection and dismantling

A new drilling jumbo has a short delivery time and is operational when it arrives on site.

For TBMs, it takes more time. If a new TBM has to be purchased, the delivery time is 6-12 months. If a second-hand TBM is to be used, it has to be modified to fit to the actual job, and a general overhaul may be required.

While a boomer can easily be brought to a workshop and repaired, repairs of TBMs, after boring has started, have to be done in the tunnel. To bring in a spare boomer is normally possible, while bringing in a spare TBM may take a year or more. Therefore a thorough pre-operation overhaul of TBMs is more vital than of a boomer.

A boomer is flexible regarding cross sections. The diameter of a TBM can be changed to some extent, but once it's in the tunnel it is a very time-consuming operation to do diameter modifications. It is, however, possible.

A TBM and backup have to be transported in pieces. Erection time varies dependent on local conditions, crane support etc. and takes normally 3-6 weeks (small & medium sized hard rock TBMs).

1.3. Time for rock support

Rock support is normally reduced by TBM boring compared with D&B. Time for rock support depends very much on how the TBM is equipped with rock support equipment, like drilling equipment for bolting and grouting, shotcrete equipment, liner plate erector etc. If well equipped, bolting and shotcreting may be carried out during boring. For D&B, rock support at front can not be done during excavation.

Rock support is adapted to the actual needs. Systematic rock support is never specified. TBMs are hence equipped for bolting "as decided" only, sometimes with grouting equipment. Additional equipment, e.g. shotcreting equipment, has to be brought in when needed.

2 COSTS

2.1 Transport, Rigging and Erection

Equipment for D&B can normally be transported as it is, and costs for transport and erection are a very small part of the total cost.

For TBM it is different. The weight of the TBM is much higher than of a boomer and is in the range of 200-1,000 tons plus the weight of the back-up equipment. The TBM and backup have to be dismantled, and the heaviest piece will be 50-100 tons. Strengthening of roads and bridges may be necessary, and big cranes are required for erection.

Rigging is also different. Rigging for TBM is dependent on the muck handling system and if the TBM rigging is done in a cavern or outside the tunnel. Total costs for transport, specific TBM rigging and erection are often in the range of 5-10% of the excavation costs, but will of course vary a lot.

2.2 Excavation

The excavation costs for D&B are to some extent dependent on the number of holes required, consumption of explosives and wear of drilling steel, but do not vary very much.

For TBMs the excavation costs vary a lot and are mainly influenced by the following factors:

- Capital costs which are much higher than for D&B.
- Net penetration that can vary from less than 1 m/hour to more than 6 m/hour depending on rock conditions and TBM specifications.
- Cutter wear that can vary from next to nothing to 50 USD/m^3 or even more.

2.3. Rock support

Need for rock support is drastically reduced with TBM, under normal conditions to 30-50%. The reason is obvious; rock stresses from the detonations are eliminated.

If very serious rock problems like rock slides occurs, it may be more time consuming and costly to carry out the rock support, especially if the TBM is not equipped with the necessary devices. The reason is that there is limited space for carrying out the work and that it is difficult to bring in the necessary equipment and material through the back-up.

2.4. Comparison of costs

Erection and capital costs are higher for TBM, while marginal costs for the excavation phase are less. Therefore, the tunnel has to exceed a certain length to be economical with TBM; minimum length for choosing TBM for economical reasons is 5-6 km. If there are restrictions regarding D&B like vibrations from the blastings, TBM may be the right alternative also for shorter tunnels.

The optimal length for a D&B tunnel is normally approximately 3 km and for a TBM tunnel 8 km. The marginal unit rate per meter increases more for a D&B tunnel than for a TBM tunnel. The high TBM capital and installation costs can be distributed on more tunnel meters. For D&B tunnels additional length may cause additional ventilation at prohibitive expenses.

3 GENERAL LAYOUT

3.1 Curve radius

For D&B there are normally no practical limitations regarding curves. For TBMs, narrow curves may cause problems.

The TBM itself can pass a minimum radius of 40-80 m, but the backup equipment determines the minimum radius. That can be from 150 to 450 m when boring. It is, on the other side, possible to transport a TBM through an already excavated tunnel with reduced radius. The Project layout therefore has to take this matter into account.

3.2 Tunnel slope

The TBM itself can bore any slope, for steep slopes extra grippers have to be installed. The limitation is again the backup and transportation systems. In principle the options regarding transportation are the same for TBM and D&B. That is; normal railbound transport is limited to 2% and preferably to 0.7%. For TBM diameters of 7 meters or more trackless transport is feasible, while the diameter for D&B may be smaller because meeting niches are much cheaper, and the cross section is more flexible.

Conveyor transport of spoil is a good alternative and the slope may be up to approximately 30 degrees. It is required to have vehicles to bring in operators, cutters, spare parts, rock support equipment and materials etc., and that limits the slope to 1:5. For steeper slopes a winch operated waggon may be utilised.

For shafts the slope is normally 45-60 degrees. The rock debris has to be taken care of in a tube or channel.

3.3 Niches and branch tunnels

The TBM is unable to bore a niche, and the excavation has to be carried out by D&B. Extra equipment for boring and mucking have to be brought in. Interference will occur. D&B close to the TBM may halt boring. Especially for small diameters the downtime in boring may be significant.

4 CROSS SECTIONS

4.1 Hydro tunnels

The difference in head loss between unlined TBM tunnels and D&B tunnels is substantial, and a reduction in cross section of 40% is normal. The ratio is dependent on cross sections, rock conditions and accuracy of the contour blasting in D&B. Fig 4.1 shows the ratio. The head loss in a D&B tunnel with smooth contour compared with one with rough contour may vary 25-30%.

A TBM tunnel does not vary very much but a shisty or blocky rock may decrease the difference.

In Norway 99% of the hydro tunnels are unlined or shotcreted. International practise is, however, to have concrete lined hydro tunnels. The quantity of concrete for the lining is reduced in a bored tunnel, but the friction in a concrete lined tunnel does not differ very much from a bored tunnel, and since the cross section is reduced by concreting the total head loss is higher. Concrete lining should be done only if rock conditions make it necessary,.

Sewerage tunnels requires a smooth invert. If bored, it is not necessary to concrete the invert.

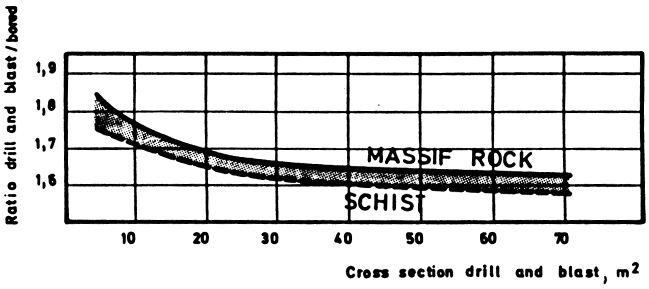

Fig 4.1 Equivalent cross section

4.2 Road and railway tunnels

The circular cross section is not optimal for traffic tunnels. The cross section will hence be somewhat bigger than that of a D&B tunnel. A possible solution for reducing the TBM cross section, is to do some D&B excavation in the lower part of the profile.

5 ENVIRONMENTAL ASPECTS

5.1 Noise and vibrations

In urban districts noise and vibrations from the blastings is a problem for the neighbourhood. The disturbance may be psychological or structural like cracks in foundations. Therefore restrictions are often required:

- Reduced quantity of explosives each round, either by reducing the length of each round or by dividing the cross section in separate blastings.
- Blastings to be executed at fixed times, and often totally restricted during night-time.

Vibrations have to be monitored and eventual cracks in buildings and foundations to be recorded. By use of TBM these problems are negligible.

5.2 Impact on environment

Progress rates are higher with TBM. That means more tunnelling can be carried out at one front without increasing construction time. For long tunnels the distance between adits can be increased or adits may be omitted. Roads and powerlines in sensitive areas can be omitted, and the project will be easier acceptable from an environmental point of view

6 HEALTH AND SAFETY

6.1 Air pollution

Air pollution from the blastings is a problem in D&B tunnels, because of toxic gases and reduced sight. The tunnel ventilation has to be dimensioned to solve or at least reduce the problem. This problem does not exist for TBM tunnels.

Mucking in D&B tunnels is normally executed with diesel engine loaders. Loaders with electrical motors are an option, then the exhaust pollution is eliminated. In TBM tunnels the mucking is carried out by the TBM itself and the TBM is always electrical driven.

Transporting of the muck out of the D&B tunnel is normally carried out by diesel engine trucks or trains. Electrical engines or conveyor belts are alternatives. Conveyor belts are more common in TBM tunnels and pollution from transportation is often a lesser problem in TBM tunnelling.

The main pollution problem in TBM tunnels is dust, especially if the quarts content in the rock is high. The content of fines in the muck is higher than in the D&B muck, and it is the fines that represent the health risk.

The solution for reducing the dust is water spraying. This has to be done at several places; at the front, at the tunnel perimeter behind the front, where the muck is poured from one conveyor to another or to a waggon. Adding too much water makes the TBM muck adhesive which may create problems in emptying transport units.

6.2 Physical stresses
The operators of a TBM or drilling rig are exposed to noise and vibrations. This can be reduced to an acceptable level by installing an insulated and vibration dampened operator's cabin. The problem for the crew occurs when additional tasks have to be done outside the operator's cabin, like rock support, rail erections, charging during boring etc. The noise from a jumbo is still higher than from a TBM, but on the whole there is no significant difference between the methods regarding noise.

A special problem in TBM boring is cutter change, due to high temperature and disagreeable working conditions.

6.3 Risk for accidents
The risk for serious accidents from handling explosives is eliminated by TBM tunnelling. There are no statistical support or evidence for one method being safer than the other.

14 Future Demands and Development Trends

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ABSTRACT: Hard rock tunnel boring has seen great improvements over the last years, and the state of the art is impressive. However, there are several areas of the technology that need improvement to make TBM tunnelling more competitive. Topics such as cutter and machine design, back-up equipment and other service functions behind the face, rock support and contract, are treated in this paper.

1 INTRODUCTION

The TBM technology has existed for approximately 50 years. Over the years, the tunnel boring technology has responded to a wide variety of demands from contractors and developers or clients. The technology is currently able to cope with ground conditions from very soft clay, sand, etc. to hard rock. At the hard rock end, the limits of economic boreable rock conditions have been moved considerably over the last 10 years or so. This paper will concentrate on demands and development of hard rock related tunnel boring in the future, based on Norwegian experience and tunnelling tradition.

2 STATE OF THE ART

Technically, all hard rock conditions may be bored by modern TBMs, with tunnel diameters from less than 3 m to more than 12 m, although the economic result of a project that stretches the limits may be less favourable. To our knowledge, rock types with a compressive strength of more than 300 MPa (DRI value less than 20) have been bored through. To accomplish this, the use of so-called High Power TBMs has been necessary. The HP TBMs are designed to be able to bore with a thrust level of up to 330 kN/cutter.

In medium hard rock, 3.5 m diameter TBMs have achieved an average net penetration rate of 6 m/h and an average weekly advance rate of 250 m in a 100 hours week. The economic result of such productivity is of course very favourable, with a total cost of less than USD 1,100 per tunnel metre.

The maximum tunnel length feasible to bore from one adit is basically decided by ventilation requirements. When using conveyor belt transport, the possible (or theoretical) tunnel length in hard rock is approximately 30 km for a 3.5 m diameter TBM. For a TBM diameter of 8 m, the possible tunnel length is substantially longer. Hence, the economic feasible tunnel length is more or less decided by excavation time requirements.

3 LIMITING FACTORS

Currently, hard rock tunnel boring is limited by other factors than available thrust and torque. The most important factor is the material quality of the cutter rings. The quality of the cutter ring steel limits the thrust level of 483-500 mm cutters to an average of 260-280 kN/cutter, resulting in a situation of not being able to utilise the HP TBMs as intended.

The shortcoming of the ring steel quality may be aggravated by the cutterhead design, i.e. the number of cutters and the placing of cutters on the cutterhead. Especially for small diameter TBMs, the placing of cutters is a compromise between cutter size and available space, leading to a less favourable cutterhead design with regard to penetration rate and cutter life.

In the competition with Drill & Blast tunnelling, large diameter TBMs have more difficulties than TBMs of 3.5-5 m diameter. One reason for this is that the net penetration rate (and the weekly advance rate) depends on the cutterhead RPM, which again is inverse proportional to the TBM diameter. In Drill & Blast tunnelling, the relative decrease in weekly advance rate with increasing tunnel area is less than for TBM tunnels, due to the use of the largest possible equipment admitted in the cross section area.

In the planning and design of tunnel projects the use of TBM excavation is disregarded. If a TBM alternative is presented, it is often a modified Drill & Blast design with regard to tunnel alignment, inclines/declines, location of adits, rock support method and quantity, limiting the overall potential of the TBM method. The main reason for this is the lack of knowledge and know-how among consultants and developers or clients. It seems that the planners are the last ones to obtain information on the TBM tunnelling possibilities. An important task will be to get this kind of information across from the contractors and manufacturers to the developers or clients and consultants.

There are of course several other factors which reduce the economic feasibility of TBM tunnelling, such as rock support methods, available machines, geological risk and risk sharing, etc.

4 MACHINE DESIGN

The design criteria of future TBMs may very roughly be divided in two:

- Hard rock machines, where increased net penetration rate and cutter life will result in increased ability to compete with the Drill & Blast method. This means that improved cutter technology and cutterhead design are the areas with largest potential.
- Medium and soft rock machines, where the rock support, machine utilisation and total construction time are decisive factors. Hence, focus should be on the rock support system, the machine and back-up system's ability to install rock support while boring with high net penetration rate, and solutions making it more efficient completing the final tunnel installations (e.g. road or railway) parallel to the boring.

4.1 Cutter Technology

The development in cutter technology for hard rock TBMs has concentrated on increasing the cutter diameter to be able to sustain the cutter loads required to break the rock. In this process there are two main topics:

- Ring steel
- Bearings

For the largest cutters (483-500 mm diameter), experience shows that the ring steel is not able to utilise the thrust capacity of the machine. To get an acceptable cutter life, the average thrust level has been reduced to 80-85 % of the design thrust of the machine. In homogeneous rock, where the exponent of the penetration curve shown in Figure 1 is e.g. 4, the penetration rate will be reduced by 50 % if the thrust level is reduced by 15 %. This indicates a large potential for reduced excavation costs by only small improvements in the ring steel quality.

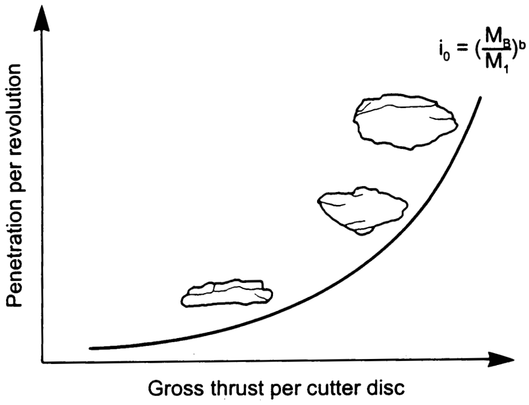

Fig. 1 Penetration curve.

On the other hand, the cutters in the range of 394-432 mm diameter, and even small cutters of e.g. 200 mm diameter, have shown improvement in thrust capacity and wear resistance. Using the model in /1/, estimated penetration rate of a 3.5 m diameter TBM is as shown below.

Cutter diameter, mm	432	500	500
Average cutter spacing, mm	65	70	70
Average thrust, kN/cutter	230	270	320
DRI	40		
Degree of fracturing	st I		
Angle	20 degrees		
Penetration rate, m/h	3.45	3.58	5.25

This may indicate that a machine with 432 mm diameter cutters and as many cutters as possible on the cutterhead, should be considered even for hard rock projects as long as the average cutter thrust is limited to approximately 270 kN/cutter for 500 mm diameter cutters.

The table also shows that materials technology should be focused to be able to make cutter rings with increased thrust capacity. There is no need to concentrate on the machine power before the "cutter problem" is solved.

4.2 Cutterhead Design

The cutterhead design may aggravate the cutter thrust problem. Most cutterheads for hard rock conditions are made as flat cutterheads with a relative small transition area towards the gauge. When boring in hard rock, the transition area has the highest portion of blocked cutters or cutters with oil leakage. This indicates that those cutter positions are exposed to the highest loads over the cutterhead. As many cutters as possible should be placed in this area; at the same time as the cutter spacing of the inner face is increased. In other words, the transition area is relieved while the inner face would be suffering higher individual cutter loads.

On smaller machines (e.g. 3.5 m diameter), it is very difficult to place each cutter in an optimum position, due to the size of the cutter housing, cutterhead balance, required space for buckets and manhole, etc. When designing a machine for 483-500 mm cutters, it might be advantageous to re-think the basic layout of the cutterhead, concerning placement of buckets, manhole and other features.

5 MANUFACTURING AND ASSEMBLY

Since a TBM represents a large investment and more or less an individual machine design and manufacture, the general rule is that the actual TBM that shall bore the tunnel must be refurbished, rebuilt or manufactured after the tunnel excavation contract is signed.

For short and medium long tunnels, TBM excavation has a great disadvantage concerning the time needed between the contract is signed and the boring can start, compared to Drill & Blast tunnelling. The time needed for manufacturing or refurbishing of the TBM and back-up, and transport to and assembly at the site may take from four months to one year. The equivalent time for the Drill & Blast method is often as low as 2-6 weeks.

There is no obvious solution to this problem, but at least three subjects should be addressed.

- Refurbish or manufacture "at site".
- The developers or clients, contractors and manufacturers should jointly consider the various machine alternatives as early as possible in the design and bidding phase. This calls for an interactive process between the developers or clients and the contractors.
- The developers or clients may be less rigid concerning the specifications in the contract, e.g. open up for a tunnel diameter range, and how that should be handled in the bidding and construction process.

The above may result in more cost effective reuse of machines. A TBM will generally be able to bore more than one tunnel in its economic useful life. For a given tunnel project, there will most likely be one or more used TBMs suitable for the job, but not to the exact specifications.

6 ROCK SUPPORT

Generally, a hard rock TBM is built for tunnels with little rock support. The philosophy has often been that weakness zones must be handled individually requiring relative long stops in the boring operation. This has lead to some untimely incidents and ad hoc solutions, which again results in unforeseen time consumption and costs.

Small diameter TBMs are particularly difficult to equip for immediate and effective installation of various support methods such as rock bolts, shotcrete lining, ring beams etc. The reason for this is the limited space near the cutterhead. A good concept seems to be the use of a specially designed rock support platform between the TBM and the back-up. This has been used at some tunnel projects, but needs further development concerning simultaneous installation of rock support and boring, transport and placing of shotcrete, and improvement of other support methods to avoid the use of shotcrete. An improved system for probe drilling and grouting ahead of the tunnel face is also demanded.

Large diameter TBMs have more space near the cutterhead, but have also to some extent been subject to ad hoc solutions when difficult ground conditions are encountered.

The problem should be focused from different angles, such as:

- Improved and adapted rock support methods.
- Training of the tunnelling crew in how to handle poor rock conditions, and when and how to install rock support.
- Equipment for continuous surveillance of the rock conditions in front of the tunnel face while boring.

7 GEOLOGY AND CONTRACT

The geological risk of TBM tunnelling is large compared to Drill & Blast tunnelling. The reasons for this are obvious:

- For Drill & Blast, only 0.2 to 1.0 percent of the rock volume is drilled, the rest is blasted. Hence, variation of the rock drillability has little influence on the total costs. For TBM tunnelling, the rock mass boreability is by far the most important factor for time consumption and costs.
- An open type hard rock TBM is less flexible in handling extremely poor and unexpected ground conditions.

The risk may be reduced through various measures. There is one characteristic risk which should be addressed in all contracts: It is not possible to make an exact model of the geology along the tunnel through the pre-investigations. The contract should therefore contain regulations on how to handle variations in the geological conditions with respect to time consumption and excavation costs. One example is shown in principle in Figure 2. "Rock properties" may be valid for one or more geological parameters used to estimate penetration rate or cutter wear. Preferably, "rock properties" should express the combined effect of all geological parameters used to estimate time consumption and costs of the tunnel excavation.

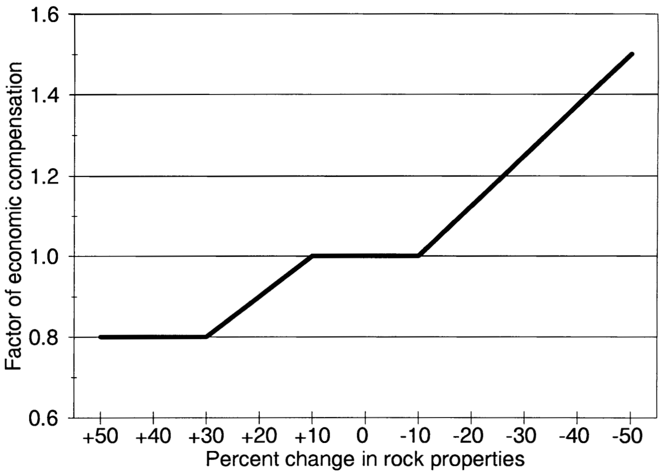

Fig. 2 Example of predetermined rate of compensation for variation in the geological conditions.

8 CREW

The TBM and back-up equipment represent a very complex and valuable system of mechanics, hydraulics, electronics, logistics, etc. To get the best performance and economy of the system as a whole, a skilled and motivated crew is a decisive factor. In the Norwegian tradition, self governing crews have shown very high productivity. This type of organisation may be developed further, with focus on training, experience and decision making.

It is important that the crew has procedures and methods available to handle the ordinary machine and geological problems. But, it is also important that the crew members know when outside expertise should be consulted. Outside expertise includes professionals among the management on site and if necessary, external consultants.

9 OCCUPATIONAL ENVIRONMENT

The work conditions of TBM tunnelling are very much like those of Drill & Blast tunnelling, with some exceptions, the most important being:

- In a TBM tunnel, the blast fumes are not present.
- The changing and handling of cutters.

Compared to the advanced technology of the TBM itself, changing and handling of cutters, especially on HP TBMs are all but satisfactory. Great improvements are needed if the general intentions of the occupational environment regulations are to be met.

10 FUTURE PROJECT TRENDS

The tunnel market of the future will probably have a larger portion of tunnels under cities and other densely populated areas. This will mainly be three types of tunnels:

- Traffic tunnels; subway, railway, roads.
- Infrastructure tunnels; electricity, gas, heating, water, communications, etc.
- Raw and potable water, and sewage tunnels.

The traffic tunnels will typically have a diameter of 9-10 m. The two latter types of tunnels will typically have a diameter of 3.5-5 m, or even less. Tunnelling under populated areas has one special feature: the third party that needs the services provided by the tunnel, but does not want to be "disturbed" by the excavation of the tunnel(s). Hence, focus should be on improved solutions for areas where the third party is in direct contact with the excavation, such as:

- Adit tunnel or shaft, and required surface installations.
- Muck reloading and transport.
- Discharge of polluted water and air.

For the tunnelling itself, the developers or clients and consultants want "flexibility", meaning:

- Back-up and muck transport systems for small curve radii.
- TBM, back-up and transport systems able to handle boring on large declines and inclines.

As small an area as possible for assembly, operation and disassembly of the equipment.

In hard rock, the major part of the tunnels bored until now, is in the 3.5-5 m diameter range. For some recent road and rail tunnels (9-10 m diameter), one has seen that the HP TBM concept gives total project cost estimates close to the cost estimates of the Drill & Blast method, but the risk connected to the TBM method has been evaluated as considerably higher than that of Drill & Blast tunnelling. Models for risk sharing should be tailor made to handle the geological risk of TBM projects.

Due to third party and environmental considerations, and the constantly improved productivity of tunnel excavation compared to other solutions, tunnel headings are increasing in length. This calls for well prepared solutions and installations at the start of the tunnel to be able to keep a high productivity (and low costs) throughout the tunnel.

On the other end of the length scale, there is a growing demand for short, small diameter (1.5-2.5 m) tunnels in hard rock. This calls for machines that may be transported in pre-assembled units, providing very fast assembly at the site. Furthermore, development of methods for personnel transport, muck transport and handling, ventilation, rock support, etc., suited for the small cross section are needed.

11 CONCLUSION

The most important step of future TBM technology will be to develop cutter technology to be able to bore hard rock with cutter loads of 320 kN/cutter or more. This will make the TBM tunnelling more competitive and also increase the tunnelling market as a whole.
It is also important that tunnel project planners at an early stage of the process utilise the special features and advantages of hard rock TBM technology to get an optimum solution for the project as a whole.

LITERATURE

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