1 THE GEOLOGY OF OSLO REGION, SEEN FROM THE ENGINEERING GEOLOGISTS' VIEW

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ABSTRACT: The geology of the Oslo area is rather complex, with rock types from precambrian to permian age. The area is a classic ground in geological research, consisting of precambrian basement rocks, cambrosilurian sedimentary rocks, and of permian plutonic rocks and dike rocks. The caledonian folding and the permian and younger block faultings made the Oslo graben, with the younger rocks laying in between the older basement. The differing rock properties with different need for support is therefore a challenge for the geological engineers and for the constructors.

1 INTRODUCTION

The region around the inner Oslofjord is the most densely populated area of Norway. Because of all the subsurface work carried out in this area the geological conditions are well known, as well as the engineering aspects.

This article describes the bedrock conditions in the Oslo region, as seen from an engineering geologist's point of view. The bedrock map, Fig. 1.1,

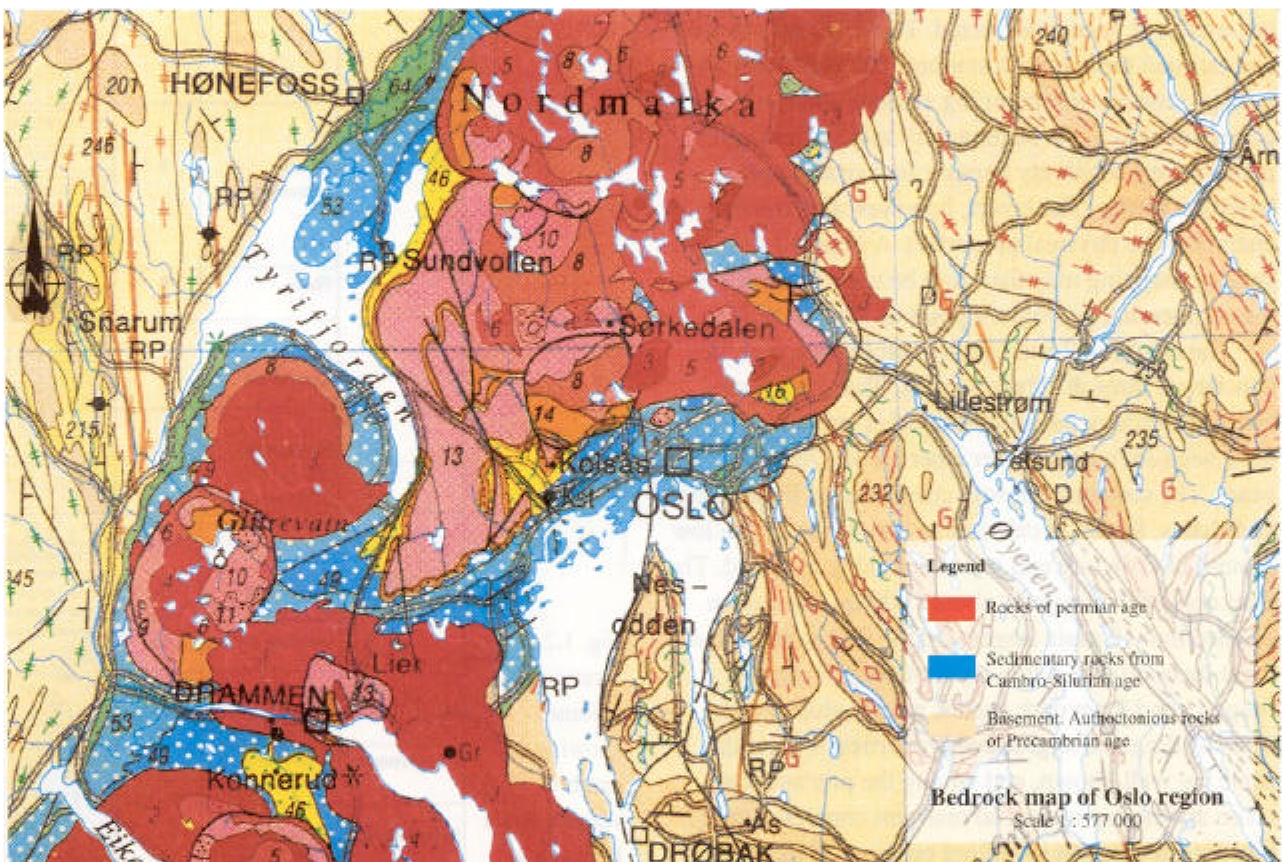

Fig. 1.1 Bedrock map of the Oslo Region

shows that several rock formations form the area. In the western and eastern parts of the map there are Precambrian rocks. In between, there are cambrosilurian sediments, which in the lower parts are intruded by Permian volcanic rock, both plutonic rocks and lavas.

The Oslo region is in fact a graben, with fault zones along the borderlines of the Permian rocks. The fault zones mostly have the direction north-south. They are often crushed and squeezed, and should be avoided for underground construction works. The fault zones can be quartz breccias, but are also crushed zones filled with clay. Tunnels and caverns have crossed these fault zones several places in the eastern and central parts of Oslo, where oil and ware- house caverns are built, as well as road and railroad tunnels.

The Oslo Tunnel traverses one of the large zones, consisting of several meters of clay, with a small rock overburden. Tunnel construction through this zone required freezing of the zone before crossing.

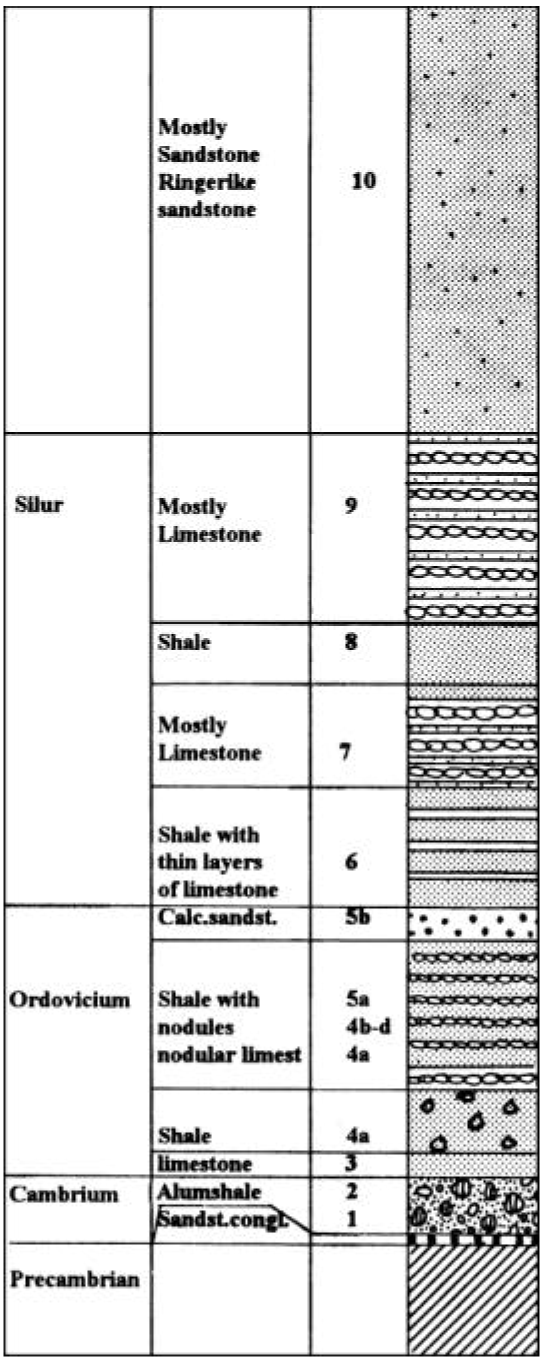

Fig. 1.2 Stratigraphic column showing the Cambrosilurian strata in the Oslo area

2 CAMBRO-SILURIAN ROCK; LAYERS OF DIFFERENT PROPERTIES

In the cambrosilurian layers, several tunnels and caverns have been constructed. These are mostly road and railroad tunnels with stations, but also water and sewage tunnels with pumping stations and treatment plants situated in the sedimentary rocks. The sediments have a relatively complicated stratigraphy, based on fossils. The layers, however, also have their characteristic physical properties. A detailed stratigraphic mapping may therefore be useful also from the engineering geologist's point of view.

The cambro-silurian geological sequence in fact consists of three different periods. That is the cambrian, the ordovician and the silurian strata, which have been sediment over a period of 170 mill. years. The sequence, mostly consisting of limestones and shales, folded in Caledonian age, make up the low ground of Oslo city and westward along the fjord. The thickness of the layers is about 1000 m, and can be divided into 10 main stages. On the precambrian basement there often is a thin layer of conglomerate. Stages 1 and 2 are dark shales, mostly alum shales. The swelling and aggressive properties of the alum shale are well known, and among the precautions taken are the use of special resistant concrete, and to take into account the swelling pressure of the rock when exposed to air.

The alum shale is found in the random parts of the graben, that is for instance in the lower eastern parts of central Oslo. In these areas, Permian dikes often may have intruded the alum shale. They occur as sills with remnants of alum shales as thin layers in between.

The alum shale is a relatively stable rock, but in thin layers between Permian sills, where swelling pressures are high and in contact with concrete, it can cause construction problems. Necessary precautions have to be taken.

Stage 3 is shales with thin benches of massive limestone. The so-called orthoceras limestone is one of the layers in stage 3. This stage is not known to give stability problems, but water leakage often occur between the layers of shale and limestone.

Stage 4 is a stack of alternating layers of shale and nodular limestone. The shale can be relatively weak, but this does not cause severe stability problems. The nodular limestone is of good strong quality. In some horizons in stage 4, thin layers of bentonite clay occurs. These are layers of volcanic ashes sedimented in between the clays. The layers mostly have a thickness of some few centimeters, and may cause stability problems in a tunnel when they occur parallel to the tunnel. The bentonite may have gone through a metamorphosis making it more like the surrounding shale and, therefore, not causing problems for tunnel constructors.

Stage 5 consist of gastropod limestone and calcarious sandstone. This is a nodular limestone with layers enriched with sand. Tunnel stability is good in this stage, but some of the sand layers may cause a considerable drillbit wear.

Stage 6 is a stack of layers of shale with a lot of thin sandy benches in between. The shale weathers easily and along the sandy benches the water leakage is considerable.

The stages 7, 8 and 9 are limestones and shales of good tunneling quality.

Stage 10 is a sandstone, called the Ringerike sandstone. This is a brittle rock, and has considerable fracturing. Along the fractured zones the fracturing is extreme, and the fracture planes may be filled with clay causing poor tunneling stability.

In the layered rock of cambro-silurian age it is important to find the optimal direction for tunnels and caverns. That is a direction deviating from the strike of the rock formation. In the Oslo region the strike is NE-SW.

The types of sedimentary rock of the cambro-silurian age have a bad reputation among the contractors and engineers. This is not because of poor stability, but due to weathering the surface seems to be very fractured. Weathering often occurs in a thin surface layer only, and the shale may therefore be massive and strong at a shallow depth under the surface. These sedimentary rock types are not suitable as construction material, e.g. for top layers in roads or concrete fillings.

3 FAULTS AND PERMIAN DIKES IN THE CAMBRO-SILURIAN ROCK FORMATIONS MAY CAUSE STABILITY PROBLEMS

Generally, the cambro-silurian rock types cause no greater stability problems in normal tunneling, but there are always anomalies. First and foremost, these anomalies are faults zones. In places there can be zones of several meters, fractured and often with clay fillings on the fracture planes. These zones may require heavy rock support such as bolting, shotcreting and even concrete lining.

The major fault zones in the Oslo area often have a N-S direction. However, there are also zones along the main strike direction NE-SW. The N-S zones are the easiest to discover in the terrain, as they form depressions across the strike direction.

The strike faults are more difficult to detect, because the folding and the variety of rock types have shaped hills and valleys along this direction, regardless of the fault zones.

The clays in the fault zones in cambro-silurian rock usually have a uniform composition. They mostly consist of clorite and mica (illite). Smectite (swelling clay) occurs only in small quantities, and causes therefore minor problems due to swelling pressures.

Igneous dikes occur particularly in the cambro-silurian sediments adjacent to plutonic bodies, with a heavy concentration in certain areas around volcanic necks. The thickness of a dike can be from some decimeters up to many 10's of meters. From the engineering geologist's point of view it is of great importance to map these dikes. They often have heavy fracturing, and may have undergone metamorphism that may have made them unstable. They also can be water bearing and cause inflow of water into tunnels. The most numerous dikes are in the area from the western part of Oslo to the western suburb of Sandvika. Nevertheless, the dikes may also occur here and there in all rock formations, as well as in the basement rocks. The main direction is as for the main faults, N-S or NE-SW.

There are many types of dikes. The most usual ones are the maenaites and the diabases. The maenaite is a light, massive rock and has usually not undergone any metamorphism. It therefore causes few engineering problems. The diabase is darker and is often fractured and methamorphised. Swelling clay may be a problem in the diabases.

Other types of dike, such as quartz porphyry, syenite and rhomb porphyry are more rare, but they can occur as dikes of several tens of meters thickness. Some of the problems following the diabases also occur within these dikes.

4 PERMIAN LAVAS GIVE STABILITY- AND WATER PROBLEMS IN TUNNELS

The Permian lavas are mainly basalts and rhomb porphyries. They were formed as a series of lava streams one on top of the other. The lavas are hard and strong rock types, and well suited as gravel for road fill and concrete aggregate. The conditions in tunnels due to the lavas can vary. In each lava layer the top of the stream often is porous and water bearing, and there can be ashes that are methamorphosed. A tunnel crossing lava layers may have both water and stability problems, and since the lava streams are often horizontal they can affect the tunnels over long distances.

5 PERMIAN PLUTONIC ROCK WITH SWELLING CLAY

Permian plutonic rock comes in a variety of types. The most common in this area are the Drammen granite west of Oslo, the syenite (nordmarkite) north of Oslo and the larvikite south west of Oslo. These rock types are well suited as gravel, and also as building stone.

Zones in these rock types easily methamorphose and form swelling clay minerals. Tunnels built through these rock types, especially the granites, have had severe stability problems.

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2 UNDERGROUND CONSTRUCTION IN THE OSLO REGION

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ABSTRACT: The life of the city of Oslo and the sea has always been closely tied together. However, the two have gradually been separated through an increase in the road and railroad surface traffic. For the city planners of Oslo, it has therefore become a challenge to work for development that again would tie the city and the fjord closer together. This is now about to be achieved through locating major transportation developments under ground. At the same time this is a powerful contribution to improving the environmental conditions in the city center, as is the localization of land consuming and environmentally damaging installations in rock caverns.

1 INTRODUCTION

Surrounded by hills and forests, the city of Oslo is situated at the bottom of the Oslo Fjord. The inner part of the Oslo Fjord is, particularly in summer, one of the most important recreational areas for nearly one million inhabitants of Oslo and the surrounding communities. During the rest of the year, the possibilities for hiking and skiing in the hills and forests around the city are among favorite leisure activities of the citizens.

The built up areas of the central part of Oslo consist mainly of 4-5 stores high buildings, built in the second half of the 19th century. The greater part of these buildings is founded on clay deposits. Newer buildings in the city can be considerably higher, and are mostly founded on rock.

As in most cities, the planning and building of new houses, highways, public transport systems, fresh and waste water transportation and treatment plants and other infrastructure facilities confront the city planners with a number of dilemmas.

In the 1980's, an architectural contest was arranged under the name: "The City and the Fjord - Oslo towards the year 2000." The main conclusion drawn from this contest was that the city and the fjord should be more closely tied together by the removal of some of the barriers built up by heavily trafficked main roads, railway lines and stations. As part of the main city plans, the removal of these barriers has been an on-going process. The docks and traditional works connected to sea transport have been allocated in the outskirts of the city, and the main roads and the railroad constructed under ground in tunnels.

In the 1990's, politicians and city planners focus more strongly on environmental aspects. One of the major elements in town planning today is to reduce the use of private cars in the city, as well as avoiding parked cars filling up the streets. The development of better mass transport facilities will encourage the people to use the public transportation systems, and is therefore an important means to reach this goal.

2 GOING UNDERGROUND

In the Oslo region, working underground started nearly 1000 years ago by mining to extract silver, copper and lead from veins in the cambro-silurian sediments. There is a legend about mines under the Old Aker church as far back as the 1100's.

In the 1500's, the mines were still in operation, and were described in several documents by such well-known mineralogists and metallurgists as Agricola and Ziegler. Not until late in 1800 and early in 1900 was underground space utilized used for other purposes. Gradually the underground was more and more utilized, firstly for water regulation. The lakes in the surrounding area north of Oslo were joined by tunnels, and the water supply for the town was led through hills and heights to utilize gravity. Finally, waste water was also led through tunnels to treatment plants to avoid pumping.

Favorable construction costs, easy to obtain building space and reduced maintenance costs have motivated municipal builders and others to take their construction plans underground.

3 RAIL SYSTEMS

The first metro or subway in Oslo was opened for traffic in 1928. It was a 2 km long twin tunnel from The National Theatre in the city center northwards underneath the Royal Palace to the Majorstua suburb.

The original construction works started as early as in 1912. At that time, the knowledge of ground subsidence was limited. As the tunneling works proceeded, a number of buildings in the vicinity experienced several decimeters of settlement. The geotechnicians joined efforts in studying the phenomena, arguing the reasons why and how to solve the problems.

The construction works halted and legal processes were initiated against the subway company for economic compensation for the damage caused to buildings along the tunnel route. This crushed the financial schemes for the project, and the subway was therefore delayed and not put to operation until 1928.

In 1954 the city government decided to start building a new subway, called the "Tunnel-bane". The "T-bane" system, including four new lines from the eastern parts of town joined in one tunnel terminating at the main railway station near the city center, opened in 1977.

Until 1980, the old and the new T-bane systems were not directly connected. As for the railways, the only connection between the lines for southern and western destinations and the lines to the North of Norway as well as abroad, was a street line, making the car traffic halt every time a train was moving slowly across the Town Hall square. There was an obvious need for an underground connection for the two systems.

The central section of the two systems had a near parallel route, and the city transport authorities and the national railway company NSB joined forces in building the part of the link that could not be placed in a rock tunnel. Through a very challenging construction job, a two stories high tunnel was constructed through a deep soft and quick clay deposit in the city center. This linked the new and the old subway systems together, giving the city a total of 95 km's of mass rail transportation, of which more than 15 km are under ground, and opened for rapid and frequent railway transport through the city.

As in most European countries, the main objectives in the long term plans for railways in Norway are faster trains, shortened lines and fewer stations. In these new plans, tunnels will be extensively used. The railway line from Oslo city to the new airport under construction north of the city, will have considerable distances under ground. The longest tunnel is 14.3

km, and will be described in another article in this volume.

The southward railway line through the Østfold county, south east of Oslo, will be dimensioned for velocities of 200 km/h also will have long distances under ground between Oslo and Moss. This is the railway connection to Sweden and the rest of the European railway network.

Through the construction of the "Ringeriksbanen", the travel distance to Bergen, on the west coast of Norway, will be shortened with 60 km. An extensive part of the 45 km distance from Oslo to Hønefoss will be in rock tunnels. The national railway's plans for the years to come are ambitious. They will be a total investment of more than NOK 7 billion over a period of four to five years.

4 ROADS

The noise and fumes from road traffic are elements disturbing the environment. The traffic is increasing dramatically in Oslo as well as the surrounding areas. In addition to occupying valuable areas in the city, traffic causes both air pollution and noise, and is also the cause of many deaths and injuries. The traffic impact puts restrictions on the use of areas that the planners and the inhabitants of Oslo would prefer to use for other purposes.

Removing a large number of the private cars from the streets and into tunnels has several advantageous effects. In addition to a considerable reduction in pollution and noise, street areas previously occupied by vehicles can be utilized for other purposes. Going underground also allows for extended traffic capacity without claiming new and precious city center areas.

Oslo is an old town with ruins dating to around years 1000 A.D. To preserve these old town ruins and restore the areas around them as a cultural heritage, the highways and railroads through the old town are now located underground. As for the environmental impact, road tunnels have the advantage that polluted air from inside the tunnel can now be cleaned before release to the atmosphere. All the new tunnels constructed in the Oslo area today have filter and cleaning systems installed.

Over the last 10 to 15 years, extensive construction activities have taken place in developing of the highway and main road system in the Oslo region. This is about to catch up with the need for improved standards. This development has included several new tunnels, as do the plans for future development. After having had only one short road tunnel of less than 200 m length on the Ring-road, several major tunnels were built during the 80's and 90's. The most important of these being the Vålerenga Tunnel, the Oslo Tunnel and the Granfoss Tunnel, of altogether more than 8 km. The city now has more than 10 km of road tunnels, and approximately the same amount is found in the nearest communities outside Oslo. A comprehensive technical development has taken place since the Vålerenga Tunnel was built in 1988, with a rather simple water and frost insulation and with only axial fans for ventilation. The tunnels now have smooth and easy to clean lining elements with frost insulation. The new ventilation systems include electrostatic filters and use water for cleaning the air, as described in a later article.

Road toll charges has made it possible to finance this renewal of road works, and the immense increase in road capacity over the last years.

5 WATER AND SEWAGE TREATMENT

5.1 Drinking water supply

The modernizing of the fresh water treatment and distribution in Oslo started in the 1960's. The Oset treatment plant, opened in 1971, has a distribution net of tunnels and main pipelines in trenches. The plant, consisting of caverns with a volume of 350,000 m^3 built in hard syenitic rock, was an outstanding project at the time.

The distribution system from the plant, located a few kilometers north of the city center, consists of several kilometers of tunnels and pressure chambers. In the city of Oslo there are more than 35 km of freshwater tunnels reaching to the outskirts. The tunnels mostly have a small cross section area -less than $10m^2$. To a great extent, the water is led through pipes in the tunnels to reduce the risk of pollution.

In 1995, a new freshwater treatment plant is taken into operation in the eastern part of Oslo, producing freshwater satisfying the EU rules. Both the plants will be described in a later article.

5.2 Sewage and waste water treatment

The wastewater tunnels of Oslo are from the period from the 1950's and up to the 1980's. Oslo now has more than 57 km of wastewater tunnels within its borders. From Oslo, a 23 km long bored tunnel leads to the main treatment plant on the west coast. The collecting tunnels to this plant include a total of 42 km with 40 inlets.

Plants for treating waste are often situated in the vicinity of the coast. In Oslo and the surrounding communities the waste water has to be treated in the most efficient way, because of the recreational areas along the coastlines of the Oslo fjord.

The wastewater is transported to two major treatment plants, one on each side of the fjord. Because of the hilly landscape and the intention to transport the waste water to the treatment plants by gravity, using as few pumping stations as possible, there has been an extended use of tunnels.

These plants would occupy vast areas of land if constructed above ground, and may produce fumes and smells that are not acceptable to the public. Treatment plants for waste therefore are situated underground at several locations in the Oslo suburban areas.

The VEAS treatment plant at Slemmestad, south west of Oslo is the largest one with a volume of caverns of 400,000 m^3. It handles sewage from 315,000 m^3 people in the region, in addition to commercial and industrial wastewater equivalent to approximately 250,000 people.

6 ROCK CAVERNS FOR DIFFERENT PURPOSES

A city founded on solid and sound rock can utilize the underground for nearly any purpose. To some extent, rock caverns have also been used as storage and warehouses.

At the dockside in the eastern part of Oslo, a complex of 48 caverns with a total volume of 23,000 m^3 was opened in 1965. All kinds of goods are stored in these caverns, which have temperatures from freezing to temperate. Old air-raid shelters are also used for different storage purposes. Deep freeze stores are built in rock for storing ice cream, meat and fish. The volume of such storage in the Oslo area is 160,000 m^3.

Oslo also has huge rock caverns for storing oil products. The actual volume of these caverns is not released, but at the eastern harbor there are five caverns and more than 4 km of tunnels for the storage and distributing system.

Several sports halls and swimming pools have been built in rock caverns in the Oslo region. Located in densely populated areas, these caverns are also utilized to serve as bomb shelters. Oslo has two such halls and the nearest surrounding communities also have a couple of these combined sports and swimming halls. A typical hall has a volume of 35,000 m^3.

The Civil Defense Authorities has provided shelters for more than 50% of the population. Because rock installations give good resistance against attack, rock caverns are often used. Many are dual purpose constructions, in peacetime serving as sport halls or subway stations. Single purpose air-raid shelters in rock have also been built in Oslo, but many of these are in bad condition due to poor maintenance.

Electricity transformers, operating centers for telecommunications and other technical centers are also often placed underground for protection against war and sabotage. The Oslo Energy Board has several transformers in rock caverns, and plans are under way for more.

Several other purposes, such as parking facilities, heath pump facilities for remote heating systems, traffic control centers, multipurpose tunnels and shooting ranges are typical examples of facilities built underground.

7 FUTURE

The exploitation of the underground is still in its youth. To be in front of future development, the planning authorities of the city of Oslo are preparing legislation for regulating underground construction, in the same way as above ground.

As construction techniques continuously improve and an increasing number of activities are attracted to already densely developed areas, a prosperous future can be seen for the underground construction.

3 WATER SUPPLY FOR THE CITY OF OSLO

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ABSTRACT: The City of Oslo is supplied with drinking water from lakes in the surrounding forest areas. The water is treated in several plants before being transported to the municipal distribution network. This article describes two of the major treatment plants, in service from 1970 and 1994, thus representing different stages in technical development. Technical solutions for each plant are presented, together with a design comparison of the two.

1 INTRODUCTION

The water supply to the City of Oslo is based on the utilization of surface water from the forest areas surrounding the city. At present, the city has water sheds at its disposal covering a total area of 330 km^2. Because of the rugged terrain and suitable rock conditions around the city, water supply schemes have been designed based on water transfer through rock tunnels. Owing to restrictions on land use, the plants and reservoirs have been built underground in rock caverns.

This article describes two different water treatment plants administered by Oslo Water and Sewage Works (OWSW). Both plants play an important role in the water supply of the city:

- Oset Water Treatment and Pumping Plant, in operation from 1970
- Skullerud Water Treatment Plant completed in 1994. At the end of the article, a comparison is made between the two plants with respect to design principles.

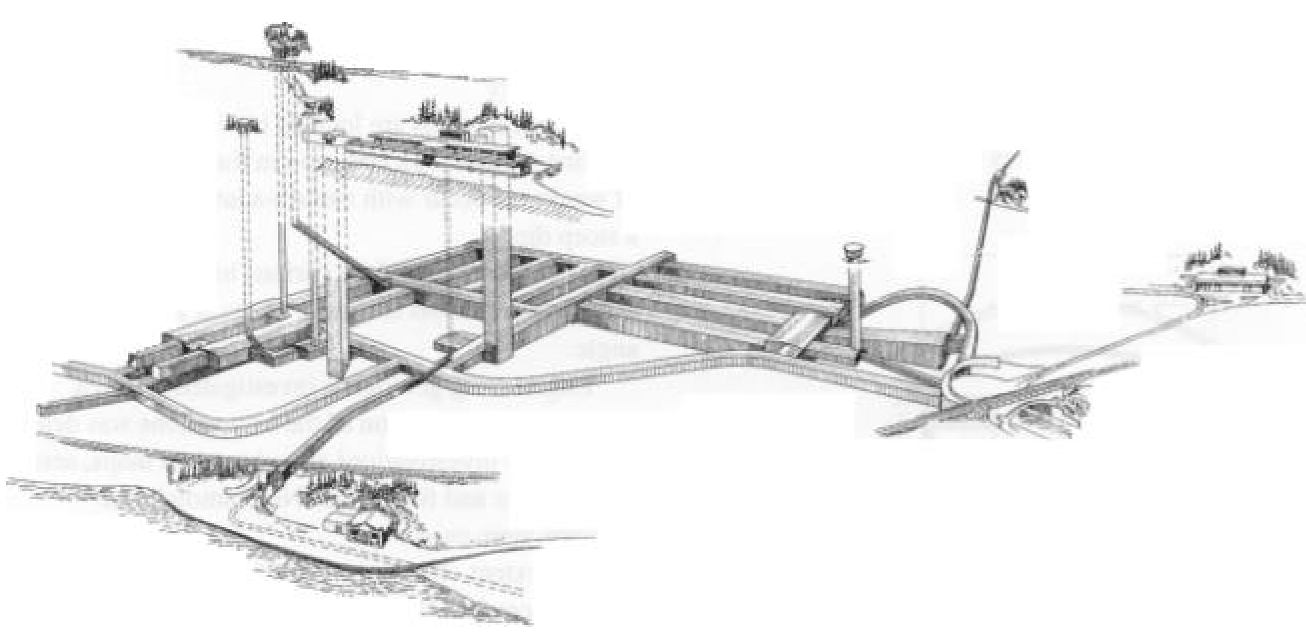

Fig. 3.1 3D drawing of Oset water treatment plant

2 OSET WATER TREATMENT AND PUMPING PLANT

2.1 General description of the plant

The Oset Water Treatment plant is based on the Maridal water shed with a total area of 250 km^2. Lake Maridalsvann is the intake reservoir.

The plant has a design capacity of 6 m^3/sec. It consists of a network of basins and channels. A large part of these channels are covered by concrete slabs designed to carry vehicular traffic loads. The total floor area is close to 30,000 m^2.

The size and proportions of the halls as well as the layout were determined mainly by their functions in the water treatment and pumping processes. In principle, the plant has been built as two parallel units, each of which can operate independently.

The plant comprises the following elements (taken in the direction of flow):

- Water intakes at depths of 30 m and 12 m, with tunnels, 22 m^3 and 10 m^3 respectively, leading to the intake chambers. Chlorination point is located at the tunnel inlet
- Raw water pumping station lifting the water to the operating level. Capacity 2 x 6 m3/sec.
- Aeration chambers where fresh air is injected into the water. Chlorination after aeration.
- Retention basins of 19,000 m^3 volume.
- 30 Micro strainers.
- 4 pure water basins with a total volume of 50,000 m^3. The third chlorination point before entering the basins. The basins act as regulation chambers and chlorine contact chambers.

The design of the basins at Oset is laid out with a concrete floor on a drainage layer of coarse-crushed stone, free standing concrete walls and a free span arch. This design minimizes the risk of contamination from the groundwater.

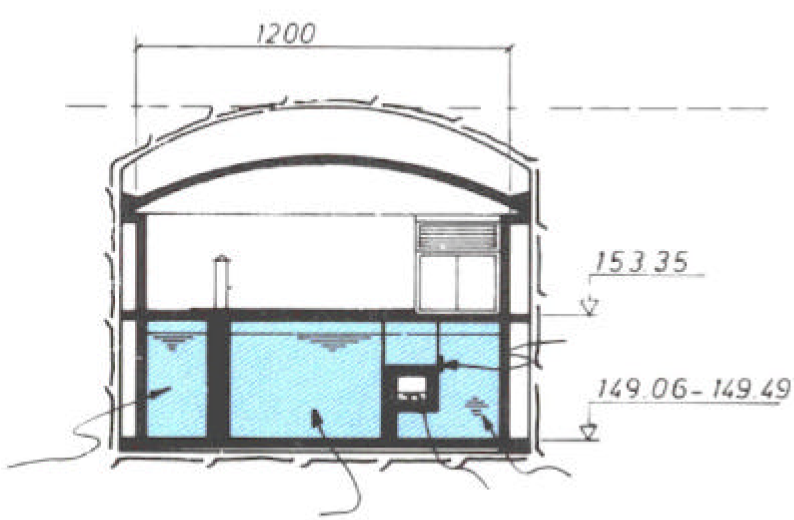

Fig. 3.2 Typical sections of micro strainer, with concrete slab

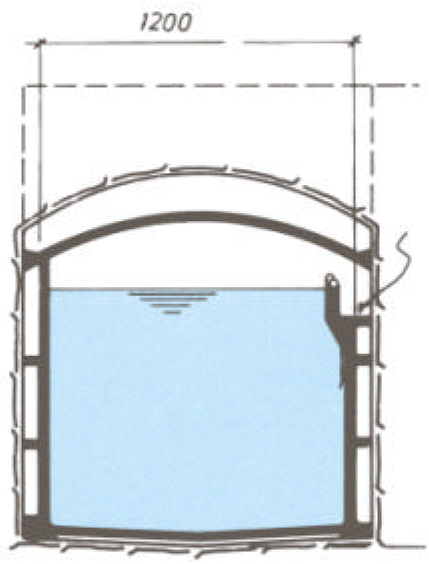

Fig. 3.3 Typical sections of retention basin

2.2 Distribution

The water is distributed through three mains to the different areas of the city. Two are gravity mains while the third is a riser (Grorud main) fed from a high pressure pumping station with a total capacity of 1.3 m^3/sec. against a head of 115 m.

From the pumping station, the water is pumped through a 1,500 mm diameter steel pipe embedded in concrete in a 2.4 km long tunnel to Årvollåsen reservoir.

Årvollåsen reservoir has a cross section of 100 -110 m^2 and a length of 670 meters. Only the floor has been concreted at this stage, but if it becomes apparent that pollution from external sources is increasing, concrete walls and roof may be constructed.

2.3 Geology

All the excavations are located in Nordmarkite, an alkaline syenite rock common in the Oslo area. Zones of weakness occur with a north-south orientation and a steep dip.

The orientation of the various halls was chosen such that they cross the weak zones at a favorable angle.

Engineering geologists investigated the area before the final location of the excavations was determined. The investigations were based on maps, aerial photographs and field work. No diamond core drillings were used.

The extent of support work was determined as the work proceeded. Concrete lining was needed at some particularly weak zones, but the main part required no other support than a few bolts.

2.4 Construction

The main halls at Oset have a free span of 13.2 m and a height of 16 m, i.e. a cross section of approximately 200 m^2. The halls were excavated in three sections. The top section was 8 m high, giving an initial cross section of approximately 100 m^2. Two 4 m benches were used to attain full depth. The Årvollåsen reservoir was excavated full face. Most of the excavated rock was crushed and utilized by the municipality.

The work comprised the following approximate quantities:

- Excavated rock volume 400,000 m^3
- Formwork 150,000 m^2
- Concrete 50,000 m^3

2.5 Future developments

The existing water treatment processes at the Oset plant are not sufficient to meet the existing water quality requirements. The plant will be upgraded with additional treatment processes, which will include filtration. Three different pilot filtration plants have been in operation for some time in order to arrive at an optimal solution. However, there are still overall considerations to be made concerning the role of the plant in the water supply master plan for Oslo. The future capacity and priority for this plant are not yet determined.

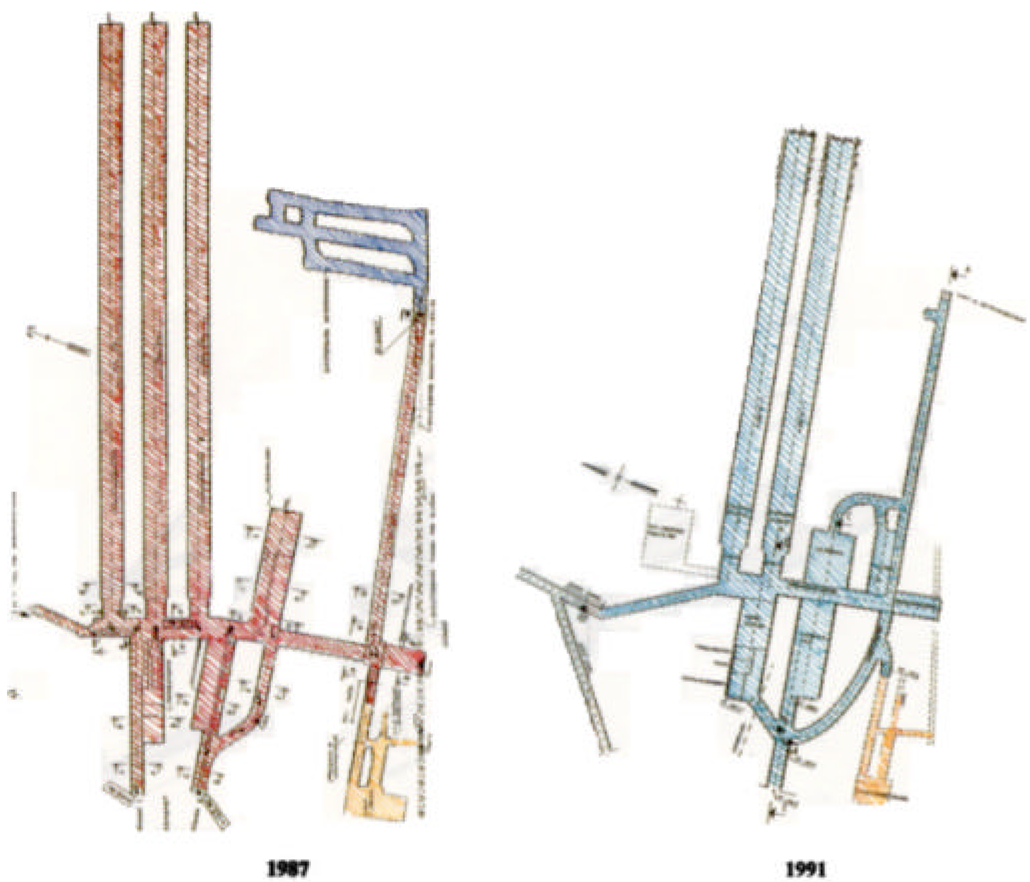

Fig. 3.4 19871ayout versus 1991 layout.

3 SKULLERUD WATER TREATMENT PLANT

3.1 General Description of the plant

After operating the Oset W.T.P. for 20 years, the Oslo Water and Sewage Works instituted comprehensive action to upgrade the water quality. As a first step, the Skullerud Water Treatment Plant has upgraded the water quality for the south of Oslo. This plant will be an auxiliary to the main supply of water from the Oset plant.

A preliminary design for this project was finalized as early as in 1987, and bidding documents for the rock excavation and concrete structures contract were completed. However, this was postponed due to funding problems.

In November 1990, the work was resumed. Plans were turned around and the papers from 1987 was to be sent out after a short review.

Reviewing the papers from 1987, there were a few important aspects of the plans that needed a more thorough examination based on the elapsed time and the changes in general requirements and technical developments that had taken place.

A new preliminary design was undertaken and completed in March 1991, taking into account changes in general requirements and new technology.

The following changes are of interest:

1 The plant has a more compact design.

2 There is only one adit.

3 The adit is situated on plant property.

These changes allowed the construction to start in the summer of 1991, as no lengthy property acquirement was needed.

3.2 Rock excavation

The rock quality of the site is exceptionally good and there were no technical problems with the rock excavation process. The contract was formulated to be an incentive to the contractor to blast to fine tolerances. However, the rock excavation contract was sublet to another contractor, lacking the same incentive.

This resulted in an excessive excavation and consequently the use of a much larger volume of concrete to be placed against rock surfaces.

Inaccurate rock excavation also caused problems as the positioning of the process equipment had to be adjusted, at extra cost to the contractor.

The ceiling of the caverns was secured with fiber reinforced pneumatically applied mortar. Water leakage problems, as always, are the problem in rock caverns. They were solved by drip shields of corrugated aluminum plates and are working well in the process areas.

3.3 Concrete and steel structures

The concrete structures in the caverns consist mostly of water holding channels and basins and require special water tight concrete. The reinforcement was complicated and the geometry of the structures difficult to form. The end result demanded a relatively small amount of grouting to tighten up the structures.

The preparation tanks for lime solution was of such a complicated form that it was decided to change from concrete to steel which saved three months in construction time and gave a more flexible solution that works well to day.

3.4 Layout

As mentioned, the revised layout is compact and the one adit (wide enough for two way traffic) that presented a logistic problem during construction is more than adequate in the operation phase. All vital areas in the plant may be reached with truck or fork lift truck. The delivery of all chemicals is received as close to the adit as possible.

3.5 Piping

A considerable amount of piping has been fitted in the plant. In fact the piping contracts are only surpassed by the construction contracts in size.

The main supply piping consists of up to 1,600 mm steel piping, polyethylene coated and cement lined. The piping is welded and bolted in single pieces so that all temperature forces are taken up by concrete structures. This provide us with a minimum of supports. Process piping up to 1,200 mm is of stainless steel material. Overflow piping up to 1,200 mm is of fiberglass material.

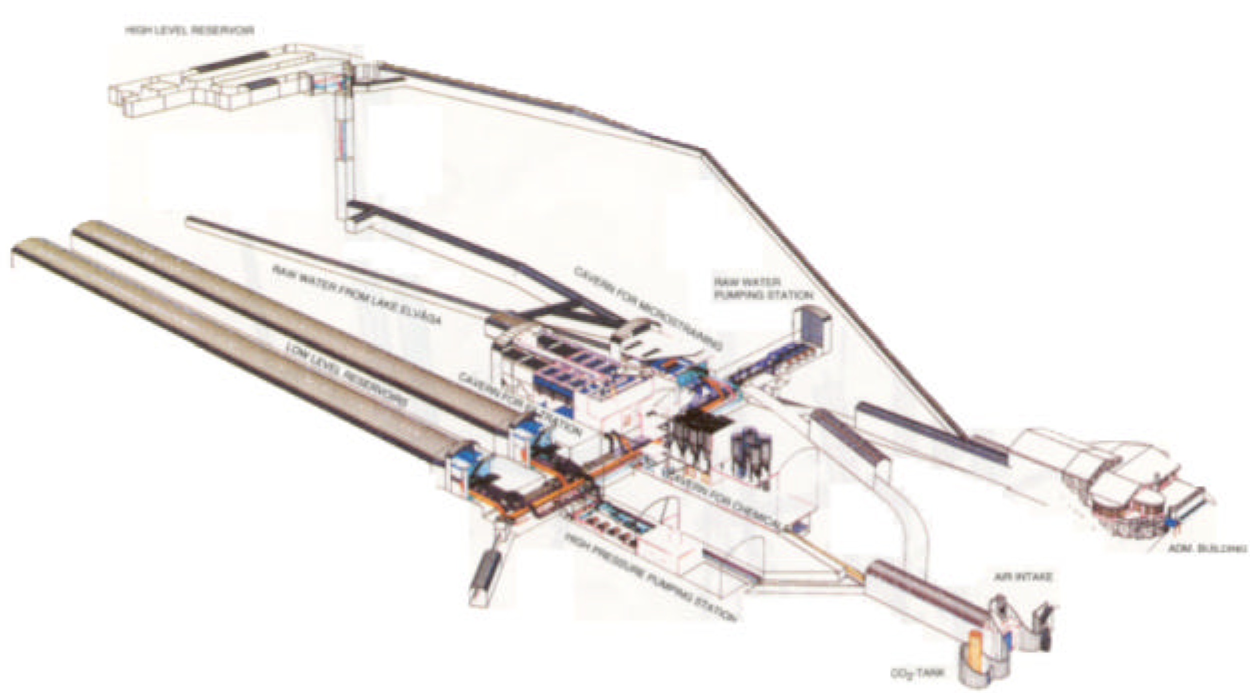

Fig. 3.5 General view of the plant. (3D CAD drawing)

3.6 Process

The process has shown to be very robust and gives excellent results. The treated water satisfies all requirements for good drinking water and also all EU requirements. Operating normally, the plant delivers 500 l/sec. by direct filtration which is adequate for the area it has been planned for. However, if the main plant at Oset should break down, Skullerud Water Treatment Plant has an auxiliary function to provide up to 2,300 l/sec. with a less comprehensive water treatment process. The process is controlled by an automatic control system that is now working satisfactorily after a period of testing and debugging.

3.7 Cost

The total cost of the plant that was finished in late 1994 is NOK 193 mill.

3.8 Plant data

The plant was official1y in operation on 24.11.1994. The raw water from lake Elvåga (195 meters above sea level) flows partly through a tunnel and partly through a 1,000 mm steel pipe to the Skullerud W.T.P.

3.8.1 Capacity 0-500 l/s (Normal operation)

Direct filtration (with Aluminum sulfate) in a 3-media filter (6 ea). Use of lime and carbondioxide for corrosion control. Disinfection with sodiumhypochlorite. Back flushing of the filters is done automatically. The sludge (5%) is delivered through the municipal sewerage system to the Bekkelaget sewage treatment plant, while decanted "clean" water is reintroduced to the raw water side of the plant.

3.8.2 Capacity 500-2,300 l/s

Microstrainers (3). Mesh openings 64 µm. Alkalizing with lime. Disinfection with sodium hypochlorite.

3.8.3 Quality requirements

In accordance with the recommendations by the National Institute of Public Health and the EU Directive 80/778.

3.8.4 Operating and monitoring

The operation of the plant is fully automatic, consisting of two independent production lines. The central control facilities are located in the control room in the administration building, while several control satellites are placed at strategic sites in the plant. The water treatment plant is operated by five persons during daytime. At off-duty hours, emergencies are received at a 24 hour alarm central and automatically relayed to the operator on duty. The operator has a portable computer with modem connection to the control system at the plant.

3.8.5 Power supply

Two transformers, each of 1,600 kVa, are installed. Voltage is 3 x 400 V. An emergency power plant of 400 kVa will keep vital parts of the process in operation in case of power failure. Equipment that requires continuous power supply is supplied with UPS (uninterrupted power supply).

3.8.6 Quality control

Bacteriological tests by an accredited laboratory. Chemical water analyses at OWSW's central laboratory. Daily analyses at the plant laboratory (temperature, pH, Al, turbidity, color, chlorine residual).

3.8.7 Civil work

A volume of about 110,000 m^3 rock was excavated. This includes two low level reservoirs 2 x 15,000 m^3 (220 m x 12 m), 178 m above sea level, and a high level reservoir 7,000 m^3, 236 m above sea level. The rock caverns are secured by bolting and fibre reinforced pneumatically applied mortar. Water channels and reservoirs have been an epoxy coating where they are exposed to water, and are painted or dust treated on other surfaces. All floors are epoxy treated.

3.8.8 Chemical storage tanks

Sodium hypochloride 2 x 5 m^3 Lime 2 x 44 m^3 Aluminium sulphate 2 x 36 m^3 Polymer Delivered in bags Carbon dioxide 1 x 40 m^3

4 DESIGN COMPARISON

Oset and Skullerud water treatment plants represent construction methods with a time difference of 25 years. Some differences in the design and construction methods are observed as a result of the technical development in this period. Some differences are mentioned below:

- Generally, the choice of solutions tend to move against more economic design principles.
- Excavation methods are more or less the same. However, the equipment used to day is more efficient with larger excavation capacities as a result and benches are made higher.
- The development in rock support methods has been significant.

In the Oset project the rock support was not extensive. For the major portion of the rock surface area the support was limited to rock bolting and some shotcrete in minor areas. Fault zones were supported by in situ concrete lining. As an additional security measure roofs of free standing concrete shell arches were constructed in practically all the halls and basins of the plant. This solution was also chosen to take care of groundwater leakage from the rock. Experience from this solution is that where the rock is free standing without shotcrete or concrete lining, the rock surface tends to loosen with time, causing a need for rather extensive scaling and support work above the concrete shell arches. This has been done 2-3 times in the period after the completion of the plant.

As for the Skullerud plant the rock surfaces, in addition to spot bolting, were shotcreted to a much larger extent to make a permanent and complete rock support. Also weakness zones are supported by shotcrete combined with rock bolts. Rather large amounts of steel fibre reinforced shotcrete have been applied to the rock arch surfaces. No in situ concrete lining has been used. To catch ground water leakages, light arch roofs of corrugated aluminium plates are installed.

- In the Oset plant the water is conveyed through the different parts of the plant by concrete channels, while in the Skullerud plant large diameter pipelines are used. Piping is not practical at Oset, due to the large capacity needed here.
- Water channels and basins at the Oset plant are all constructed using free standing concrete walls. The reason for this was to have as much control over water leakage (in and out) as possible. As for the Skullerud plant, the water basins in the treatment sections are constructed with rock contact concrete walls. It may be commented that this difference in the design is not a result of development of design, but just as a result of individual considerations of the differences for the two plants.

4 WESTERN OSLO FJORD SEWERAGE SCHEME

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ABSTRACT: Due to the combination of densely populated areas and narrow inlet of the Inner Oslo fjord, the fjord has become more and more polluted and less attractive as a recreation area. To improve the water quality, a sewer transportation tunnel system and an underground sewer treatment plant were constructed during the 1980's. Except from the pumping station at Frognerparken, the flow through the tunnel system is by gravitation. Today's technology at the underground treatment plant is a mechanical/chemical process which has been extended to include a fixed film biological removal system. To meet the new requirements to reduce outlets of phosphorous and nitrogen components, process improvements and extension to the plant were necessary. Despite heavy underground construction works, the plant has been in continuous operation with only a 10-20% reduction of capacity during the extension works.

1 INTRODUCTION/ GENERAL REQUIREMENTS

Oslo is situated at the end of a 100 km long fjord with a narrow inlet. The exchange of fresh seawater from the Outer Oslofjord and the North Sea is thus limited. Pollution from approximately one million people ends up in the fjord. The fjord is a popular recreational area for a large number of people, for fishing, boating and swimming in the relatively warm summer climate of the Oslo area.

During the 1950's and -60's the pollution of the fjord from sewage was a growing problem. The West- fjord Regional Sewerage Authority (YEAS) was formed in 1978 with the mandate to plan, implement and operate a trunk tunnel system and a regional treatment plant for the Western Greater Oslo Region.

The preliminary design of the trunk tunnel system was carried out during the late 1970's. The main requirements for a sewage collecting system were to intersect a large number of existing mains and some existing plants discharging into the sea, and transport the sewage to a planned regional sewerage treatment plant. The design criteria included sewer for a total of 600,000 population equivalents. The existing sewers included both old pipelines containing mixed foul sewage and surface runoff as well as new separate systems.

2 CONCEPT DEVELOPMENT

2.1 Transportation system

During the early design stage, the main principles of an efficient transport system had to be developed:

- o a near-surface trunk pipeline system, or
- o an underground tunnel system excavated in rock.

The near-surface system would include a large number of pumping stations and several local overflows. The tunnel system located close to the shore line along the fjord would make it possible to intersect most of the sewers by gravitational flow. Other possibilities including intermediate solutions were also studied.

An important basis for the choice between alternatives was the ground conditions of the area. The specific conditions are described in a later section. In addition, the concept development involved a number of other parameters to be considered:

- operation and maintenance cost
- available technology
- environmental matters
- time of completion

Based on thorough evaluations and considerations a tunnel system was selected for the conceptual design. The design followed a few simple and basic principles:

- The flow should be by gravitation, and the intersecting system should consist of only a few main pumping stations. Existing old pumping stations should also be abolished, if possible.
- The main part of the system should consist of tunnels close to the shoreline, and at a level low enough to catch most existing main sewers by gravity.
- Self-cleaning effect should be achieved.
- To keep costs at a reasonable level, the tunnels should be situated in solid rock.
- With the selected tunnel dimensions, reservoir apacity for levelling out the daily variation and peak flows was achieved.
- The method of excavation based on tunnel boring machines (TBM) had the following advantages:
  - hydraulic gradients and cross-sections
  - short construction time
  - better conditions for making the tunnels watertight
  - favorable cost
- Principally the tunnels should be unlined. Rock support and reinforcement design should be based on "design-as-you-go" principle, using mainly rock bolts and shotcrete.

2.2 Sewer treatment plant

During the design stages and feasibility studies, aspects such as land requirements, environmental aspects, construction and operational costs were assessed, and a possible site at Bjerkås hill was selected. The area and volume of the Bjerkåsen hill was relatively limited, and to find an optimal location, orientation and design of the caverns was a challenge.

From topographical and geological factors and parameters, an optimal location and orientation of caverns and interconnecting tunnel system was found and design parameters as maximum width, length and height were assessed. The maximum span width of the caverns is 16 m and some caverns have a height exceeding 20 m. Distance between caverns varies from 12 to 14 m.

From thorough studies and evaluations, location of the outfall for treated effluent was selected at a distance of 700 m from shore and at a depth of 50 m. Basically, two different concepts were evaluated:

- Subsea polyethylene pipes anchored to sea bottom.
- Subsea tunnel with cross section of 10 m^2, and vertical shaft to sea bottom at 20 m depth and 3 polyethylene diffusor pipes from a manifold further down to 50 m depth.

As construction costs were approximately the same for the two alternatives, the tunnel alternative was selected to avoid anchoring difficulties for vessels.

3 GEOLOGY

Reference is made to a separate article on the geology of the Greater Oslo Region. The bedrock in the western Oslofjord area is mainly of sedimentary origin and consists of shales and limestones of medium strength. Generally, the sedimentary rocks are densely jointed. Igneous dykes are quite frequent, in some areas with thickness up to 20 m. The igneous rocks are much harder than the sedimentary rocks. The sedimentary rocks are intensely folded, mainly with a northeast-southwest strike, and a predominant set of north-south trending weakness zones exists. During glacial periods trenches and valleys have been formed in the bedrock, and they have been filled with marine and glacial deposits. The soil deposits are of recent geological age, and include marine clays.

The clays may exceed 30 m in thickness and are partly soft with a low shear strength. The soft clays are very compressible and a lowering of the groundwater level results in settlements. The magnitude of the subsidence from lowering of the groundwater level may reach substantial values and represents a potential hazard to buildings with foundation in the soil.

Fig. 4.1 The tunnel system from Oslo to the VEAS treatment plant

4 CONCEPTUAL LAYOUT AND ARRANGEMENTS

4.1 Transportation system

The tunnel system is shown in Fig. 4.1. The total length of the tunnels is approximately 42 km. Flow in the main tunnel from Oslo East (Fagerlia) to the treatment plant (YEAS), a distance of 33 km, is by gravity. Sewage from the central parts of Oslo collects in a lower branch of the tunnel system and is pumped to the westbound tunnel at Frognerparken pumping station. The pumping station is situated deep in rock. Access to the two levels of tunnels is through a 500 m long spiral tunnel (Fig. 4.2).

The main sewer pipelines in the catchment area carry the sewage to a number of feeding points to the tunnel. In most cases the feeding points are connected to the tunnel by raised drill shafts of 500 to 1,200 mm diameter. Each feeding point or inlet has been equipped with facilities for inspection, flow measurement, lighting, water for cleaning and valves for diverting the flow in case of shutdown of the system.

The longitudinal section of the system shows the inclination of the main tunnel of 0.75 m per 1000 m. Frognerparken pumping station, designed for a maximum capacity of 2.4 m^3/s, lifts the sewage 31 m. Eight large, submersible pumps are installed in the station. At the central treatment plant the sewage is lifted against a head of 22 m by dry-mounted pumps in a dry pump room.

Facilities for overflow have been installed at two points in the tunnel system. Remotely controlled gates at Engervannet and Frognerparken may retain sewage water at peak flows or divert the flow to the overflow arrangements.

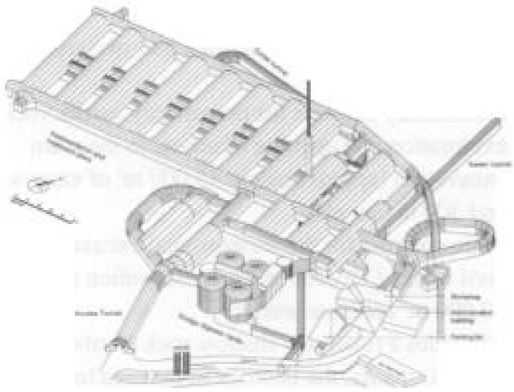

Fig. 4.2 3D drawing of the VEAS treatment plant

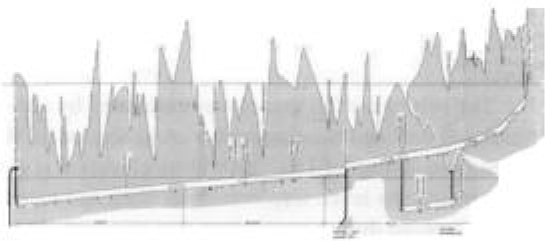

Fig. 4.3 Longitudinal section of the tunnel system

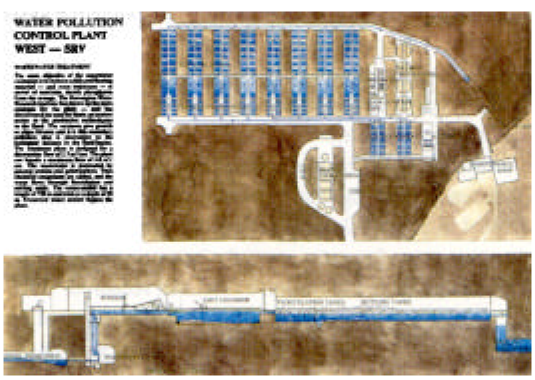

Fig. 4.4 The VEAS plant layout

A ventilation system has been installed to prevent odor discharge in the vicinity of the feeding points and to ensure an acceptable environment for inspection and maintenance crew.

4.2 Treatment plant

Fig. 4.4 shows the layout of the system. The underground area is divided in two physically separated areas:

1. Traffic area where all transport and loading/ unloading of chemicals, sludge, grits and screenings takes place. This is obtained by roundabouts for trucks and arrangements with conveyor belts.
2. The plant itself with settling tanks, thickeners, ventilation, screens and grit chambers has ready access to the maintenance workshop and administration building. The elevation of the plant itself is 6 m above sea level, while the pumping station receiving effluent is located at 15 m below sea level. The pump station is equipped with 8 pumps, which -with a capacity of 7,2 m^3 per second - pumps the effluent 22 m up to the treatment plant.

Approximately 25,000 m^2 of plant area was developed, giving 400,000 m^3 of rock volume from cavern excavation. Disposal of rock spoil from excavation was a minor problem. This was utilized as landfill and fill for a small boat harbor in the immediate vicinity.

Outdoor construction was limited to an administration building, workshop and an auxiliary power plant, beautifully set in a small bay in the Oslofjord. Lime storage tanks and a pier for receiving chemicals are combined with transport activities for the industrial activity in the area.

5 CONSTRUCTION

5.1 Transportation system

The construction work was carried out under five contracts, involving a total number of six TBMs and a "Midi Facer". Drill and blast was carried out where geometry was not suitable for full face boring. The tunnel diameter varied from 3.0 to 3.5 m.

The rock conditions for TBM excavation were excellent due to the medium strength rocks (UCS generally between 60 and l00 MPa), with low contents of wearing minerals and extensive jointing of the rock mass.

Due to the particular geology of the area, with a number of buried valleys filled with soft marine clays, it was of utmost importance to prevent lowering of the groundwater level. Since lowering of the groundwater could not be accepted, neither temporarily nor permanently, it was decided to use grouting as a basis both for initial and permanent tightening of the tunnels. Criteria for water tightness in each area of tunneling was evaluated, based on the potential risk of damage to nearby buildings and structures.

The TBMs had to be specially equipped for a systematic probing ahead and pre grouting procedure. The combination of tunnel boring and extensive systematic grouting was unique at that time, and equipment and procedures were further developed during the early construction period. Satisfactory results of water tightness were achieved on most of the tunnel lengths. Additional grouting was carried out to some extent. A full concrete lining was required for some 400 m tunnel in central Oslo. Recharge wells were installed in some areas for extra control of groundwater, but permanent installations have not been required.

Valuable experience from the extensive use of the various techniques and materials has been gained from the project. To summarize, the conclusions were:

- It has been possible to reduce leakages down to 1-2 liters per minute per 100 m length of tunnel
- Pre-grouting is far more efficient than post-grouting .
- The use of cement suspensions under high injection pressures, gives an efficient large water leakage reduction
- The use of chemicals or micro cement suspensions was necessary to satisfy the tightness requirements in some areas.

The pre grouting of the tunnels in some areas was the dominating activity. Up to 2/3 of the costs and time consumption was associated with this work.

Rock support and reinforcement works were necessary only on 7% of the total tunnel length. Shotcrete and grouted rebar bolts account for most of the support works. In weak rock, reinforced arches of shotcrete have been used and, as previously mentioned, only 400 m of the total TBM tunnel length required an in situ concrete lining. The rock support works were mainly carried out in weakness zones, often in conjunction with the igneous dykes.

5.2 Treatment plant

The rock excavation was divided into two separate contracts:

1. Preliminary contract for preparatory works, rock exploration and opening of access to the main excavation (approximately 45,000 m^3 of excavation and 300 m tunnels)
2. Main contract for the underground excavation and civil works (350,000 m^3 rock excavation and 10,000 m^3 concrete works)

Besides a relatively shallow rock overburden and poor rock quality, the bedrock in the area (nodular limestones and shale) is subject to erosion and disintegration when exposed to air.

Thus, rock support by systematic rock bolting, reinforced mesh (approximately 50% of the roof area) and shotcrete has been necessary. Fibre reinforced shotcrete was not adequately developed at that time. Rock support accounts for 30% of the excavation costs.

The subsea outfall tunnel was excavated by conventional drill and blasting with a cross section of 10 m^2 and inclination of 1:6. The bottom elevation is at 90 m below sea level. Exploratory drilling and extensive follow-up during excavation was required. At the end a vertical shaft of 50 m height was excavated and a steel manifold was installed at the outlet opening.

6 EXTENSION OF THE TREATMENT PLANT

6.1 Introduction

The original technology at VEAS is a mechanical/ chemical process by which 97% of the phosphorous and 65% of the organic matter is removed, whereas only 18% of the nitrogen compounds is removed.

Norway and the other North Sea countries have signed an agreement to reduce outlets of phosphorous and nitrogen compounds in 1995 by 50% from the 1985 level. In order to meet the strong requirements from the authorities, improvement of the purification process to include biological treatment was necessary.

In co-operation with the SINTEF group at the Norwegian Institute of Technology, and other experts, VEAS has developed a new total concept for a compact solution for nitrogen removal, reducing the need for volume increase in rock caverns to a minimum.

The extension included construction of four new sludge digester tanks above ground, 700 m tunnel excavation, removal of 4,800 m^3 of concrete structures, volume increase for six rock caverns by totally 54,000 m^3 rock excavation, civil works and technical installations.

There was an overall requirement to keep the treatment plant in full operation during the construction period, and a very high degree of quality assurance including monitoring and instructions for careful blasting was required.

The extension works started in 1991 and will be completed in 1995.

6.2 Sludge digester tanks

The tanks are constructed in an open trench, excavated in Bjerkåsen hill outside the treatment plant. The site geology is generally of a similar type and quality as the rock caverns with limestones and shales, partly folded and faulted. The soil cover is relatively shallow in the area.

The trench width was approximately 50 x 50 m at the bottom and the design inclination was approximately 70° The maximum slope height was approximately 30 m. The excavation works included approximately 15,000 m^3 of soil excavation and 60,000 m^3 of rock excavation.

During excavation of the open trench, a very poor rock quality was encountered and a serious rock slide occurred in the northern slope. The slope inclination was decreased to approximately 55 degrees and considerable rock support by cable anchors, rock bolts and wire mesh was required.

The support was designed for a temporary period only, since the trench was to be backfilled by rock fill after construction of the concrete tanks.

6.3 Tunnel system

The tunnel system included approximately 200 m of new access tunnel, interconnecting adits, service tunnel for drain pipes underneath the caverns (elevation -15 m b.s.l.) with short, vertical shafts for drain pipes to the caverns, small adits between the caverns and transport and access tunnel to the rear end of the caverns. Altogether, a tunnel system of 700 m was excavated with cross sections of 20-25 m^2.

Due to a generally poor rock quality and partly shallow rock overburden, the tunnels were supported by systematic rock bolting and fibre reinforced shotcrete.

6.4 Rock caverns

From the total of eight caverns, volume increase and reconstruction was required in six of the caverns.

The volume increase was performed by bottom benching of 3.5 m, 7 m and 9 m depth down to a deepest bottom elevation of -6 m b.s.l. Removal of reinforced concrete floor and support of the remaining wall structures was necessary before the rock excavation.

With a minimum distance of 12 m to existing structures, installations and research equipment, procedures for careful blasting were necessary. Vibrations during blasting should not exceed a value equivalent to 20 mm/s measured at the administration building, 30 mm/s at critical instruments and installations and 50 mm/s at pipes and concrete structures. Vibration levels up to 150 mm/s was measured at less critical points.

Systematic rock bolting of rock walls, partly combined with shotcrete, were necessary as rock support.

In order to prevent leakages from neighboring caverns during emptying and cleaning operations, the concrete basins constructed after excavation were impervious.

As well as the volume increase of the six caverns, a small adit with separate exit tunnel to the administration building was excavated for installation of a gas engine for additional power supply. The engine was operated on gas from the sludge digesters.

The extension works have so far been successfully performed without any damage. There is a slight delay related to the completion of the eight caverns, and there has been a capacity reduction of approximately 10-20% on the treatment plant.

7 OPERATION EXPERIENCE

The treatment plant and the western parts of the tunnel were commissioned in 1982 and the rest of the tunnel system was finalized in 1986. Inspections have shown that no problems of solid waste sedimentation exist and the system has worked satisfactorily since the start of operation. The operation control, surveillance of the flow rates and the overflow arrangements have been functioning well even for large storm flows, which have been experienced on some occasions during the years of operation. The assumed maximum is estimated to 9.5 m^3/s, registered at a flow of 35 m^3/s.

In 1990 parts of the main tunnel were inspected prior to excavation of a nearby road tunnel. In this area a weakness zone had necessitated relatively heavy support work in the form of reinforced shotcrete arches. It was concluded that the rock reinforcement (bolts and shotcrete) was in good shape and that no deterioration of the materials had taken place.

The overall experience with the underground plant at Slemmestad has been good. There has been a minimum use of land above ground and there is full control of discharges to the air without any odor problems.

Minor water seepage into the rock caverns has been experienced and there is a certain seasonal variation due to the shallow location. These leakages are partly taken care of by local drains, but do still cause some minor problems.

The shotcrete has been inspected after 10 years and is generally in good condition.

By transferring direct outlets via the tunnel system and the VEAS treatment plant, the water quality in the Inner Oslofjord is improved significantly. Visibility in the water during the summer has increased and the fjord is today attractive for swimming, beach activities and other kinds of recreation activity.

5 ROAD TUNNELS IN OSLO

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ABSTRACT: As the result of an explosive growth in traffic density and decades of very low investment rates in the road network, Norway's capital had become the traffic congestion of the nation. The introduction of toll charges speeded up the development of a modern road system with adequate capacity, reduced accident rates and damaging environmental impacts. The main link in the system is the Oslo Tunnel, a dual tunne16-lane carriageway beneath the town center. Three Ring Roads serve the through traffic and has a function as feeders for radial distribution to the town districts. A road network to serve the future needs in Oslo is about to be completed.

1 INTRODUCTION

Over the last 30 years, Oslo has seen an explosive growth in traffic density. As this development was foreseen, plans for large-scale urban motorway construction were made as early as in the 1960's. However, only fragments of these plans were actually realized. Both the costs of the motorways and their impact on the urban structure became a hinder for political acceptance.

Instead, the public transport network was improved. At that time, Oslo was the only city with less than one million inhabitants to have a subway system.

Today the Oslo public transport system takes a large portion of all journeys. About 30% of all passengers who cross the toll-ring are traveling by means of public transport. Closer to the center, across the inner ring, this percentage rises to about 80%.

Because so much investment was made in the public transport network, road building came in second place. Nevertheless, traffic continued to grow, and at a tempo that confirmed the prognoses of the 1960's. The result was queues, jams and chaos most of the day. The peak of traffic on Oslo's city center ring came in 1986, when, in practice, capacity was exceeded all day long.

It was therefore decided to finance new roads in the Oslo area by tolls, in the same way as in Bergen, Norway's second largest town, and a toll-ring surrounding the central city was introduced in 1990.

2 PRESENTATION OF THE ROAD SYSTEM

The design of the preliminary road network now being built is very different from that planned early in the 1960's. Highway standards have been relaxed in order to achieve a better match with the surroundings. The premises for road network design are now to a greater extent dictated by the structure of the city. In the 1960's it was the other way around. The city had to adapt to the road network. At that time urban motorways were also meant to serve as a tool for urban transformation in line with the functionalistic philosophy- a dissected city, divided into zones and with a strictly segregated transport system.

The pattern of the main road system in Oslo was established many years ago. Today's task is to improve parts of the road network that are environmentally unacceptable, and/or lack sufficient capacity.

Heavy traffic on older roads and streets have the effect of destroying many urban areas. The new primary roads are not a supplement to the existing roads and streets. They replace those that are overloaded with traffic. Moreover, tunnels are being more frequently used than previously, to prevent the destruction of the original dense city structure. As much as 70 % of the new road network in Oslo will be tunnels.

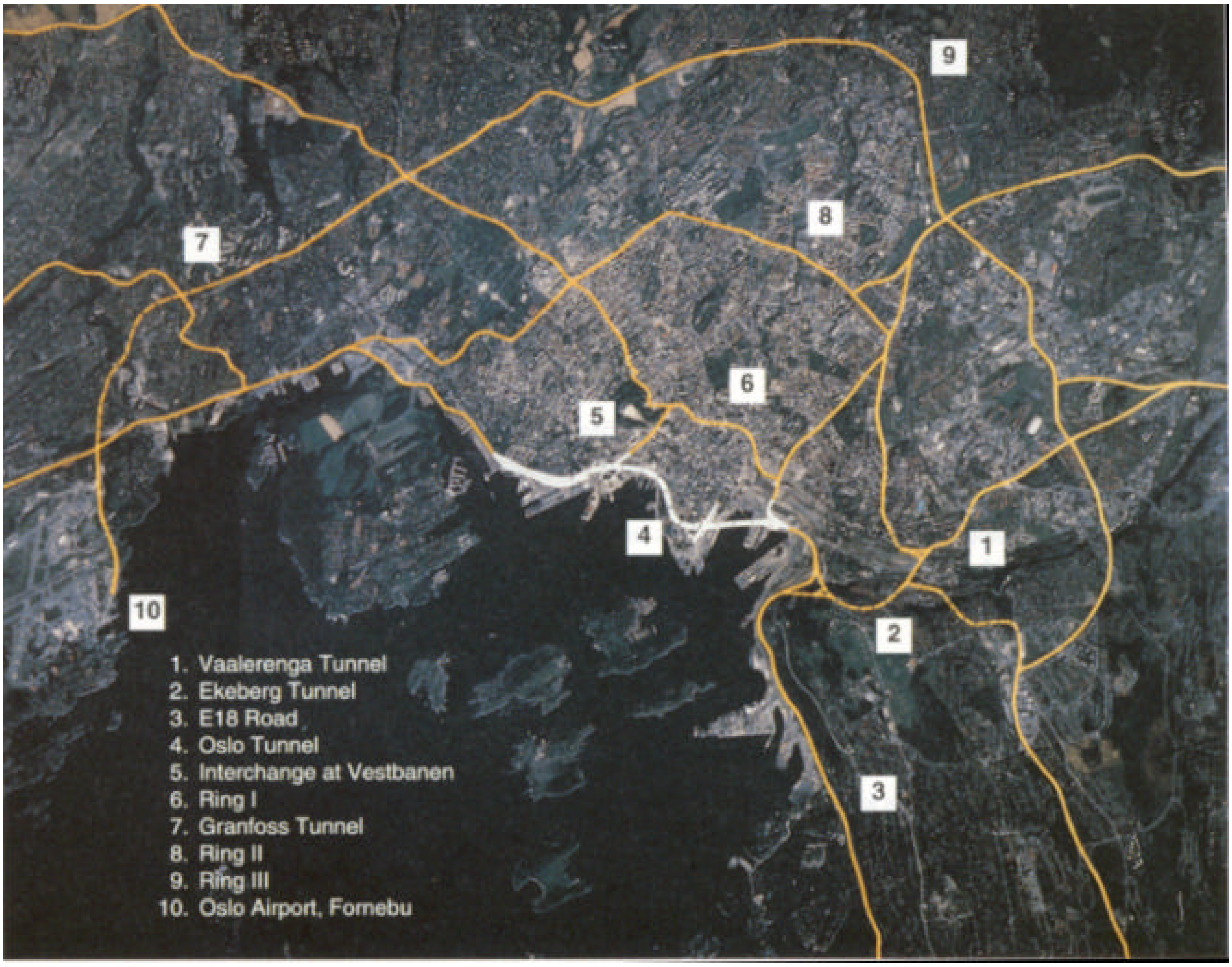

Fig.5.1
Central Oslo Road Network
Yellow: Stage 1, the Oslo Tunnel (dotted) Red: Stage 2, the Vestbanen Interchange Green: Stage 3, the Bjørvika Bay Crossing

The most important link in the main road system in Oslo, is the E18/E6 through the central parts of the city. Traffic on this link, more than ADT 100,000 at some sections, had to use old streets through the city and along the seafront. These old roads are now replaced by a new link, approx. 6.5 km long, with three tunnels making a total of 4.6 km.

Moving from east to west we find the first project to be completed, the 0.7 km long Vålerenga Tunnel (1). This tunnel was opened for traffic 1987/88, removing through-traffic from the old main road, which now is closed for through-traffic.

The Vålerenga Tunnel is now followed by the 1.4 km long Ekeberg Tunnel (2), leading traffic around the Old Town -the original Oslo, which for a long time has been ravaged as a result of the traffic impact. Roads, railroads and harbor constructions have wiped out most traces of medieval Oslo. New roads and tunnels under construction will remove traffic from the Old Town. Around the ruins of the Maria Church a park will be established. A new era will start in 1995, when the Ekeberg Tunnel is opened for traffic.

Between the Ekeberg Tunnel (2) and existing E18 (3) a temporary connection will be constructed. In the future, this connection will be replaced by an 800 m long immersed tunnel across the bays of Bjørvika and Bispevika. This link will eliminate the barrier between the city and the sea front represented by the existing roads, and open up for a massive city development.

From Bjørvika and through the city center, we find the most complex project, the Oslo Tunnel (4). Stage one, finished in January 1990, embraces construction of a new six-lane highway through the central part of the city. The total length of this road section is about 3,000 m, with 1,520 m tunneled through the bedrock and 270 m in cut-and-cover tunnels. Stage two was the construction of a major subterranean interchange west of the central business district, at Vestbanen (5). Ramps in bedrock tunnels and cut and covers will give connection between the Oslo Tunnel and the Henrik Ibsen Ring Road, Ring I, which encircles the central business district. This ring is partly completed, but some tunneling remains. Plans are made for a tunnel under the park at the Royal Palace.

Fig. 5.2 The Oslo Tunnel with adjacent road sections.

Before the construction of the Oslo Tunnel, most of the through-traffic in Oslo had to use roads crossing the area between the town hall and the seaside. Eight lanes carrying 80,000 vehicles a day, made it impossible to utilize the town hall square as intended; a place for people to mingle and enjoy this popular area where the city meets the sea.

The Oslo Tunnel also replaces the road along the sea front by the old Akershus Castle. This area was of high environmental value before the destructive impact of heavy through-traffic. The traffic and the harbor activities gave the area an industrial image.

From the city center and westwards the El8 is extended from 4 to 6 lanes, plus lanes for busses in both directions. Near the Fornebu Airport, about 5 km from the city center, interchanges are constructed for connection between the E18, the Granfoss link (Ring 3) and local roads.

The Granfoss (7) link was completed in 1992. Two bedrock tunnels, altogether 2 km, of which one is constructed under the Lysaker river, lead the traffic from the E18 and the Fornebu Airport to the major ring road (Ring III) around Oslo. On this ring road construction works are still proceeding to eliminate capacity problems in some junctions. A section of the ring road passing through a densely populated area will be replaced by a 1.5 km 4-lane tunnel in 3-4 years.

3 THE OSLO TUNNEL

3.1 Project

The Oslo Tunnel was constructed from March 1987 to January 1990 as two parallel 1800 m long tunnels of 3 lanes each, running through the central parts of Oslo from east to west. The Tunnel is part of the project E18 route through Oslo, with a total cost of USD 200 mill. As the intention was to link up the center of Oslo to the waterfront, without a traffic barrier, this necessitated the location of the main east-west connection below ground.

Fig. 5.3 Geological map showing different rock qualities alonthe route.

Light green: Nodular Limestone Green: Clay shale
Dark green: Limestone
Grey: Alum shale
Brown: Meanitt
Red: Diabase/syenitt
Pink: Rhombic porphyry

A rock tunnel was chosen to reduce costs and avoid excavation of large sections of the center of Oslo with enhanced temporary traffic problems as result, access difficulties to downtown buildings as well as possibilities of serious settlements of the same. Driving of the tunnels started at 12 m b.s.l. To prepare for this, 181 m and 92m cut and cover tunnels were built at the west and east end respectively.

The rock tunnels are 1,520 m and 1,527 m long, south and north tunnel respectively. Due to the rock level the tunnel had to dip down to 47 m b.s.l. At that point the rock surface is only 4.8 m above the tunnel roof. The gradients were 45-50 ‰. Three cross tunnels were established for vehicles and three for walking in case of emergencies. Bedrock tunnel ramps near both tunnel entrances link the tunnel traffic to the city center. The cross section area of these branch-offs is 215 m^2.

The cost of the tunnel was USD 81 mill, i.e. USD 53,500 per meter, ventilation, lightning, traffic control and emergency equipment not included.

3.2 Technical challenges

3.2.1 Vibrations

Standard cross-sections vary between 88 m^2 and 153 m^2 with a corresponding volume in normal blasting rounds of 350-600 m^3. Nevertheless, the aim was to keep the vibrations at a level where damage to buildings was avoided. This was very important as quite a few very old buildings worth preserving were situated just above the tunnel, particularly in the grounds of Akershus Castle.

The contractors were required to apply blasting techniques that produced vibrations below 80 µm (0.08 mm) and velocity amplitudes less than 40 mm per second. These limits were halved in certain areas, and in special cases even lower. Normally, buildings are not expected to sustain damage at vibrations below about 200 m. There were no damage registered to buildings due to blasting.

3.2.2 Settlements/leakages

Buildings in Central Oslo constructed on overburden deposits, are supported partly on piles and partly on rafts . The possibility of seepage from these overburden deposits into tunnels, and the subsequent damages to buildings, have produced very stringent specifications for sealing the tunnels, both during construction and when in permanent operation.

The location of the Oslo tunnel project determined the choice of construction methods and linings to preserve the groundwater and pore water pressure at constant levels. Valuable experience gained from several major underground facilities built earlier in central Oslo, has been utilized on the Oslo tunnel project, reducing the risk of damage even further.

Sealing specifications are expressed in terms of maximum permitted seepage into the tunnels, varying from 1.5 to 2.5 litres per. minute for every 100 meters of tunnels. After having been in operation for more than 4 years, the total amount of measured leakage is 2.31/min for every 100 ms of tunnels, which agrees well with the presupposition.

As far as settlements are concerned, no subsidence exceeding 10 mm has been measured on any building. On the other hand, high grouting pressures have raised some buildings up as much as 16 mm, but with only small damage as a result.

3.3 Construction techniques, bedrock tunnel

The Oslo Tunnel was constructed by conventional drill and blast techniques. Full profile boring and road header machines were found not to be competitive. A concrete lining was cast along the full length of each tunnel. This lining will function in accordance with different principles.

In the west, where the tunnel runs through shale and limestone, the concrete lining provides the permanent seal against water penetration, and the tunnel invert is therefore also concreted. Concreting took place not more than eight weeks after or 100 meters behind blasting operations. Temporary water sealing was done by grouting.

In the eastern end, where the tunnel runs through gneiss/amphiholite and menaitt, only the sides and roof were concreted as a drained safety lining. Here, the permanent watersealing was done by grouting. Mechanical stability of the rock is secured by bolting and shotcreting.

3.4 Grouting

The stringent set of specifications established for water sealing in all phases of construction was met by systematic pregrouting prior to blasting. This was done through 20 meters long holes bored in a fan shape ahead of the tunnel face. The distance between each grouting section was 12 meters in the west and 8 meters in the east, giving a minimum overlap of 8 meters for each section. The difference in overlap reflects the fact that pregrouting in the east provides the permanent seal, while it is temporary in the west until the watertight concrete has been cast.

To prevent longitudinal seepage behind the concrete, grouted sections were established every 50-100 meters. In addition, contact grouting between concrete lining and rock was carried out.

3.5 Watertight concrete

The concrete lining was cast in direct contact with the rock or with shotcrete cover. On levelled invert, one or two layers of sheets filled with bentonite clay were laid prior to casting. These sheets expanded on contact with water and sealed any cracks that might occur in the invert concrete.

The concrete was of C40 quality, with 350 kg cement and 20 kg silica per m^3 of mix. The water cement ratio was 0.45 and the permeability was max. 1.10-12 m/s. The concrete met the requirements for a very aggressive environment. Before the tunnel was opened, grouting of concrete joints and cracks from the shrinkage was carried out. Experience shows that grouting as far down in the cracks as possible is important.

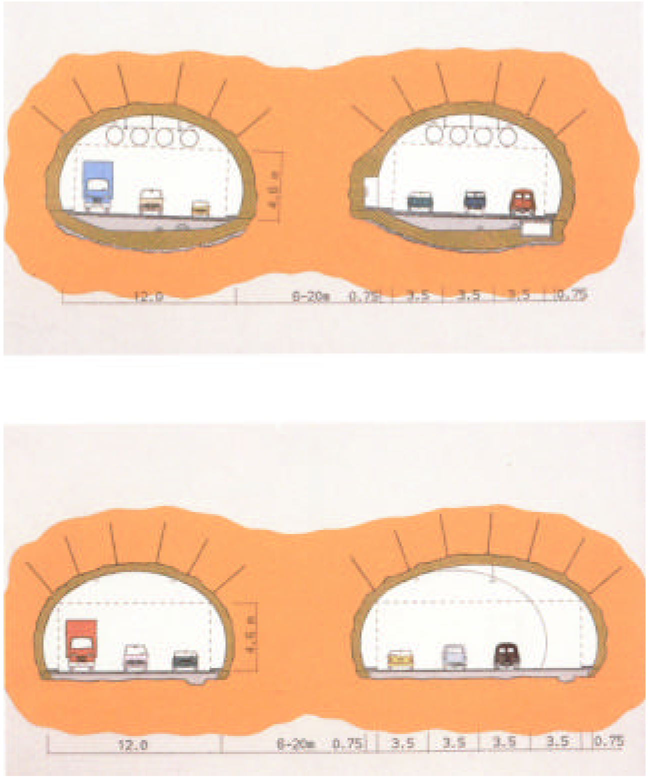

Fig. 5.4 Watertight lining (top), with sealed concrete around the whole cross-section included the base. Total tunnel width is 12 meters. Drained lining (bottom). The concrete lining is drained and the base is not concreted.

3.6 Fault Zone

At its lowest point the tunnel crossed a fault zone. The rock overburden at this point was only about 4.7 m. The rock was highly fractured in 6-7 meters length of the roof in the northern tunnel, and in 13 meters length in the invert in the southern tunnel. The fractured zone consisted of very water sensitive alum shale. The tunnels spanned 14.3 and 15.5 meters respectively.

Lengths of 55 m in the northern tunnel and 60 m in the southern tunnel were defined as problem zones and dealt with as follows:

A 25 m^2 pilot tunnel was first driven through 42 m of the zone in the northern tunnel, from the west end, then enlarged to full area, all the time using reduced rounds. The remaining meters were driven full face from the eastern side. The rock support consisted of bolts ahead of the working face, and radial bolts in combination with steel fibre reinforced shotcrete arches.

Pregrouting, excavation, rock support etc. was completed within five months.

Based on experience from the northern tunnel and the expectancy of much worse rock conditions, it was decided to freeze 28 m of the problem zone in the southern tunnel. The whole works with the mobilization, the freezing operation, the excavation and the concreting of this part of the zone took four months.

4 CONCLUSIONS

The experience from construction of the Oslo Tunnel show that:

- The water tightness of the tunnels gained was satisfactory.
- There was minimal damage to the surroundings .Rock grouting was an effective method to seal the tunnels.
- Grouting and cracks in the concrete lining was much more comprehensive than assumed.
- Also in sensitive urban areas it is often technically feasible and less expensive to seal off tunnels by use of extensive rock grouting, as compared with watertight concrete lining.

6 VENTILATION AND AIR CLEANING TECHNOLOGY FOR ROAD TUNNELS

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ABSTRACT: In Norway, we have long experience with longitudinal ventilation of road tunnels. Longer tunnels and heavy traffic tunnels in densely populated areas, however, require sometimes new solutions. In 1989, research was initiated to evaluate the possibilities to clean polluted road tunnel air. Using electrostatic filters, we have developed the technology to extract polluted particles, with a very high extraction rate. The article will also present how the Norwegian authorities plan to use air cleaning technology (dust/soot and gas) combined with longitudinal ventilation for road tunnels.

1 INTRODUCTION

On the national highways in Norway, 620 tunnels have been constructed amounting to a total distance of 477 km. The longest is the Gudvanga Tunnel, situated in Sogn og Fjordane in the west of Norway, with a length of 11.4 km. Another tunnel of 24 km is now being designed for the same area.

The majority of the tunnels are built with the most simple form of longitudinal ventilation, namely portal to portal ventilation. Lately, however, environmental considerations have caused the construction of tunnels of a greater length, as well as many new tunnels in urban areas.

If necessary, a tunnel is divided into ventilation sections by use of shafts or side adits. This makes it possible to renew the air inside the tunnel. New technology also makes it possible to clean the polluted air in stages along the length of the tunnel.

In 1989, a research program was started in Norway to determine the possibility of cleaning polluted tunnel air. As a result of this, the technology for extracting particle pollutions (soot and dust) with a high extraction rate is now known.

The tests for a pilot system removing NO_2 gas is going on, and has been installed in the Oslo tunnel since 1992. So far, after running for more than two years, the cleaning system for removing NO_2 gas seems to be very promising, with a high extraction rate. We are now working with cleaning technology for NOx-gas.

A proposed road tunnel, 24 km long, is now in its final design stage, with longitudinal ventilation and no shafts. In this case, it is planned to use new technology for cleaning polluted air in a cleaning circuit inside the tunnel, for both particles and NOx-gas.

2 LIMIT VALUES FOR DESIGN AND CALCULATION

The following limits are used in the planning and calculation of ventilation requirements in tunnels in Norway:

- CO 200,0 ppm
- NO_X 15,0 ppm
- NO_2 1,5ppm
- Visibility 1,5 mg/m^3

Max air velocity in tunnels with

- unidirectional traffic 10 m/sec.
- bidirectional traffic 7 m/sec.

Outside the tunnel, we have the following limits:

Mean values over a period of time				
	1 hour		8 hour	
Units	mg/m^3	ppm	mg/m^3	ppm
CO	25	21	10	9
NO_2	0,20-0,35	0,10-0,17	-	

Table 6.1 Recommended limit values for outdoor air

Public health authorities have suggested a reduction of the limit for NOx (in table 1) to 0.1 mg/m^3 as one hour value. The reason for this is that NOx-gases seem to create health hazards other than those earlier anticipated.

With the limits given in this chapter, either visibility or NO_2 will in most cases determine the necessary capacity for a ventilation system. A reduction of the limit for NO_2 will increase the requirements of fresh air and, accordingly, the requirement for ventilation power.

3 VENTILATION SYSTEMS

In Norway, we always use longitudinal ventilation systems. If necessary, a tunnel is divided into ventilation sections by use of shafts or side edits. The most common solutions of longitudinal ventilation are shown in Fig. 6.1.

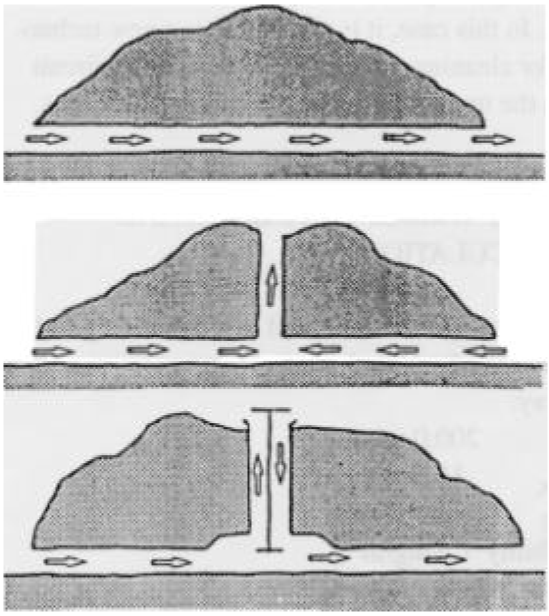


Fig. 6.1 Common solutions for longitudinal ventilation

4 CLEANING TECHNOLOGY

For most tunnels, the effect of catalyst cleaning and lower emission from petrol fuelled vehicles, will result in visibility and NO_2 becoming the important factors governing the necessary ventilation capacity. There are several reasons for cleaning tunnel air, such as:

- the concentration of polluted air or the air velocity in the tunnel is too high.
- the concentration of polluted air outside the portal is too high.
- the process of land acquisition to build a ventilation tower becomes complicated, and
- economic reasons.

4.1 Particle Cleaning

In 1989, research was started in Norway to determine the possibility of cleaning polluted tunnel air with a combination of mechanical and electrostatic filters. As a result of this research work, we have now technology for extracting polluted particles (soot and dust) with very high extraction rate. So far, we have installed electrostatic filters in three tunnels in the Oslo area -the Oslo Tunnel, the Granfoss Tunnel and the Ekeberg Tunnel.

4.2 Gas Cleaning

In 1992, research was started to determine the possibility of gas cleaning. This was done by installing a gas cleaning system in the Oslo Tunnel. So far, after running for approximately three years, the extraction rate for NO_2 is very good. The conclusion from this is that the NO_2 cleaning system will function over a long time span. This is because the process for removing NO_2 gas from the polluted tunnel air is a catalytic reduction process using a special type of active coal.

We therefore do not need to reactivate the coal filter. The limiting factors are the soot and dust particles that will clog the coal filter.

So far, if we wish to remove NO_2 gas from the polluted tunnel air, we have to reactivate the coal filter after a relatively short period of time, because this is an absorption process. We are however looking for another process as well.

Preliminary measurements on the active coal filter, based on VOC (Volatile Organic Carbon), show that the extraction rate on these components is also high.

5 EXPERIENCE FROM USE OF PARTICLE CLEANING SYSTEMS

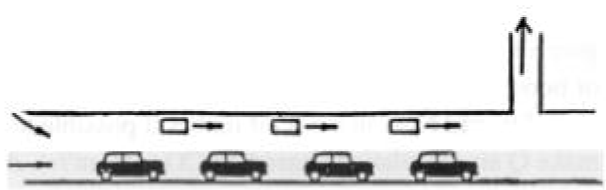
So far, we have installed equipment for particle cleaning in three tunnels in the Oslo area -the Oslo Tunnel, the Granfoss Tunnel and the Ekeberg Tunnel.

5.1 The Oslo Tunnel

The Oslo Tunnel is a highway tunnel under the center of Oslo, consisting of two tubes with six traffic lanes. The length of each tube is 1,800 m. When opened in January 1990, the annual average daily traffic was about 60,000 vehicles/day.

The tunnel has a longitudinal ventilation system with vertical shafts, connected to 20-30 m tall towers for releasing the polluted air. The ventilation capacity is about 1,000 m^3/s for each tunnel. This requires the erection of ventilation towers in order to dilute the polluted air from the tunnels.

In order to improve the air quality in the city, polluted air from the tunnels is cleaned before leaving the towers. In the Oslo tunnel, both mechanical and electrostatic filters have been installed. Norwegian experience from high traffic road tunnels indicates that dust production is considerable during certain periods of winter/spring season, primarily because of the extensive use of studded tires. Even with ventilation towers for letting out polluted air, the spread of particles is a problem in the outside area around the towers.



Length section

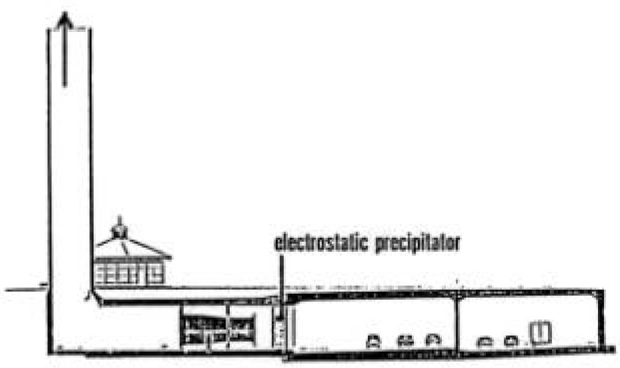


Fig. 6.2 Ventilation system of the Oslo Tunnel in principle

Since the opening of the tunnels, the pollution has been monitored into and out of the tunnel. We have also monitored pollution intensity at different points in the city. These measurements have given us information on the concentration of particles before and after the tunnels were opened.

Experiences so far show that the cleaning system has a positive effect on the area around the ventilation towers.

The test in the Oslo tunnel shows that it is relatively easy to extract particles before emission through the ventilation towers. This in turn implies a clear positive effect on the environment surrounding the emission points.

Research till now on different electrostatic filters show that certain filters have a high extraction rate with an air velocity as high as 7 m/s. By increasing the air velocity through the filter, without renunciation of cleaning effectiveness, the necessary filter area to clean the same volume of polluted air can be reduced. This will reduce the construction and installation costs. Increased allowable air velocity will in some cases also make it acceptable to install the cleaning units in the tunnel ceiling, provided that the air volume to be cleaned is not too large, as described under 5.3.

5.2 The Granfoss Tunnel and the Ekeberg Tunnel

In these tunnels, the equipment for particle cleaning is installed in a cleaning circuit, as shown in Fig. 6.3.

The Granfoss Tunnel is approx. 1,000 m long, with an AADT of 15,000 veh/d. The Ekeberg Tunnel is approx. 1,500 m long, with an AADT of 45,000 veh/d. Both the tunnels have one way traffic.

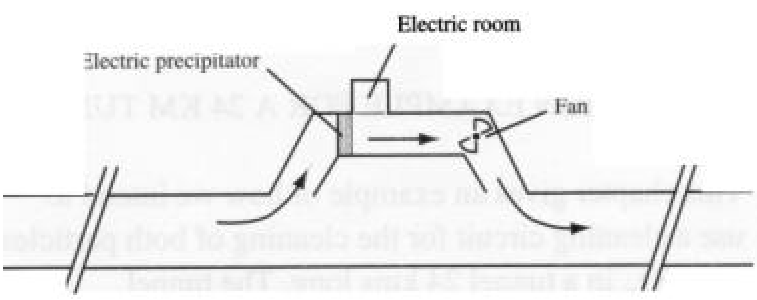


Fig. 6.3 Cleaning circuit for tunnel air cleaning

The main purpose with this type of solution is to achieve better visibility conditions in the tunnel, but also to reduce the pollution and emission of particles (dust and soot) to the surrounding area of the tunnel.

In these tunnels, the cleaning circuits are constructed with sufficient length to install a gas removal system (NOx) as well.

A gas cleaning installation may become necessary if the limit value for NO_2 is reduced in the future. In many cases, a solution like this will be a good alternative to a ventilation tower.

5.3 Particle cleaning at points along the tunnel

In this case, we will install the electrostatic filters in the tunnel ceiling, as shown in Fig. 6.4.

In 1995, a similar system will be installed in tunnel with a length of 3,800 m. This tunnel has two-way traffic, and a traffic volume of approx. 10,000 veh/day. The tunnel will be opened for traffic in October 1995.

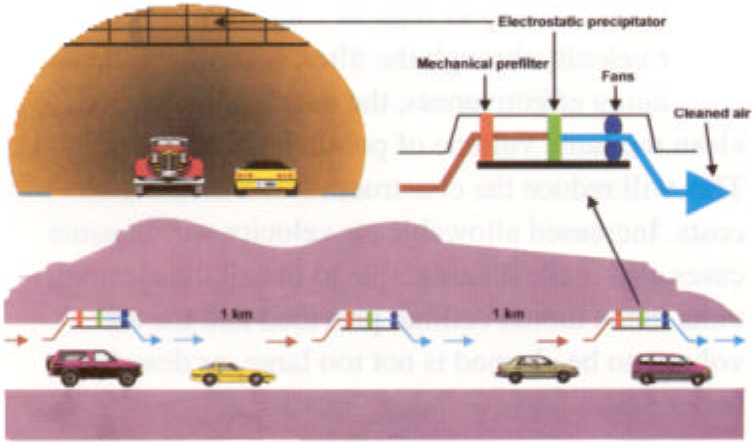


Fig. 6.4 System for cleaning along the tunnel

Cleaning equipment will be installed at three stations along the tunnel, making the distance between the stations approx. 1,000 ms. This is a concept that we regard with great expectations, as it will improve visibility in the tunnel and reduce the emissions to the environment.

6 A DESIGN EXAMPLE FOR A 24 KM TUNNEL

This chapter gives an example of how we intend to use a cleaning circuit for the cleaning of both particles and NOx in a tunnel 24 kms long. The tunnel, between Aurland and Lærdal in western Norway, is now in its final design stage.

For this project, so far we have decided to work with a concept to use the new technology for cleaning particles and NOx- gases. An alternative to this solution could be to construct a ventilation shaft rising approx. 1,100 m high. A section of the tunnel between Aurland and Lærdal is shown in Fig. 6.5.

The tunnel will have bi-directional traffic. Construction will start in 1995.

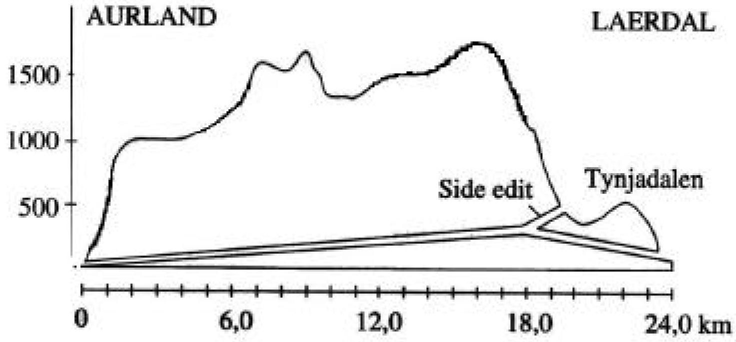


Fig. 6.5 Section of the 24 km long Aurland-Lærdal tunnel

Other data for the tunnel:

- Tunnel cross section 47 m^2
- Hourly traffic design 500 veh/h
- Traffic velocity 80 km/h
- Heavy traffic 10 %

Necessary fresh air volume (Q air) based on CO, NO_2 and visibility for the distance, 0-18 km (Fig. 6.5) is calculated to:

Q air Co = 150 m^3/s
Q air NO2 = 250 m^3/s
Q air visibility = 300 m^3/s

If we calculate the ventilation capacity in the traditional way, based on visibility (300 m^3/s), this would give a high air velocity in the tunnel and require a lot of booster fans. Installation of cleaning equipment for visibility and NOx in a circuit makes it possible to make Q air calculations based on CO (150 m^3/s). A system for this, with a cleaning circuit in position A,
is shown in Fig. 6.6.

A cleaning circuit for both particles and NO_2 will cost about USD 4-5 mill.

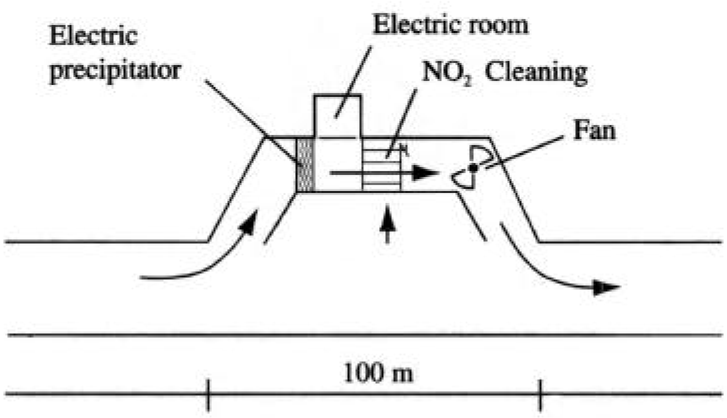


Fig. 6.6 Cleaning circuit in the Aurland- Lærdal tunnel

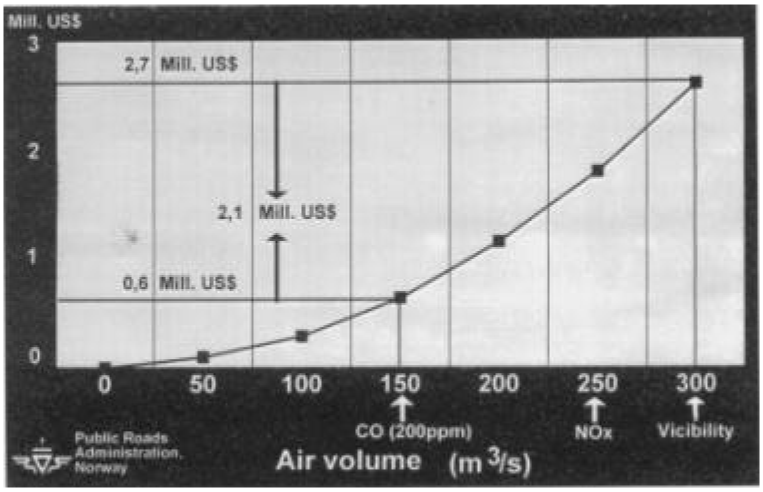


Fig. 6.7 Purchasing and installation costs for ventilation system

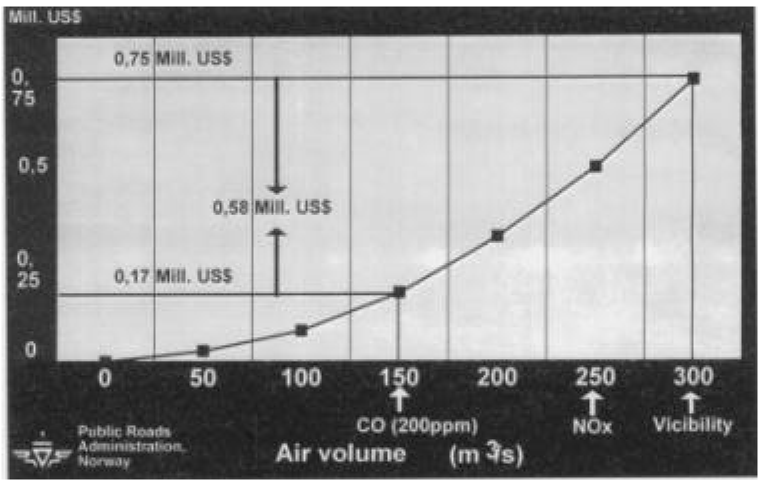


Fig. 6.8 Annual running costs based on electricity consumption

In this case, we must install a cleaning circuit with cleaning equipment for particles and NO_2. The costs for a cleaning circuit with installation as described will be about USD 4 to 5 mill.

Fig. 6.8 also shows annual running costs if the calculations had been based on Q air visibility (300 m^3/s) and Q air CO (150 m^3/s) From Fig. 6.8 we can see that the annual running cost will be reduced by about USD 0,58 mill. if the calculations are based on Q air CO. It has been decided to install one or two cleaning circuits with cleaning equipment for particles and NO_2 in the tunnel between Aurland and Lærdal, provided that the tests with cleaning technology for NO_2 give good results and acceptable costs.

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7 THE GARDERMOEN RAIL LINK, FROM OSLO TO GARDERMOEN AIRPORT

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NSB Gardermobanen a.s

ABSTRACT: In October 1992 the Norwegian Parliament decided the location for the new Oslo international airport, nationally serving the eastern region of Norway. Gardermoen is located 50 km east of Oslo. At the same time, it was agreed upon that the main feeder system to the airport should be a rail link. The new link, now under construction, runs both parallel to the existing track and at some places makes shortcuts. The Gardermoen Line is supposed to serve both airport and intercity traffic. Hence the line is to be completed all the way to Eidsvoll, 66 km north east of Oslo. This was also the destination of the very first Norwegian railway built as early as 1854.

1 INTRODUCTION

Today there are approximately 770 tunnels in the existing railnet of the Norwegian State Railways (NSB) with a total length of 260 km. There are 14 exceeding 3,000 m and a couple with a length more than 10,000 m. In order to achieve an overall high speed standard, tunnels may be the only solution in the future to manage the demand of curve shape, due to the Norwegian landscape recognized by high mountains cut by valleys and fjords.

This rail link has the potential of becoming the most profitable railway project ever launched in Norway. In addition to providing by far the fastest passenger service to Gardermoen airport, the line, as proposed by NSB, will be incorporated as part of the existing main rail network.

The Gardermoen Line is the first step of upgrading the Norwegian rail net. Although 200 km/h does not compare with European high speed train standards, it represents an upgrade to modem rail technology in Scandinavia. The travel time from Oslo to Gardermoen will be 19 min non-stop. The line will be in operation from October 1998 when the airport is to be opened.

In order to meet modern requirements of speed, comfort, capacity and convenience, both planning of the route and the technical solutions require extraordinary resources. Planning and construction are led by NSB Gardermobanen A/S, a project organization founded in 1992 and dedicated only to this task. The cost estimate is NOK 4,76 billion (1995). Most of the route will cross through open farm land. An exception to this is the first 20 km out of Oslo. The trains will be running through a 14 km hard rock tunnel, then cross through a heavily inhabited area, before entering an area with soft ground conditions. From Lillestrøm to Gardermoen there are soft, marine clays cut by many rivers and ravines. North of Gardermoen there will be a short hard rock tunnel of 1.5 km. The first soft soil tunnel to be built in Norway for a half century is located at Eidsvoll and has a length of 500 m. Safety and environmental considerations are aspects which are given overall, major concern throughout the whole project.

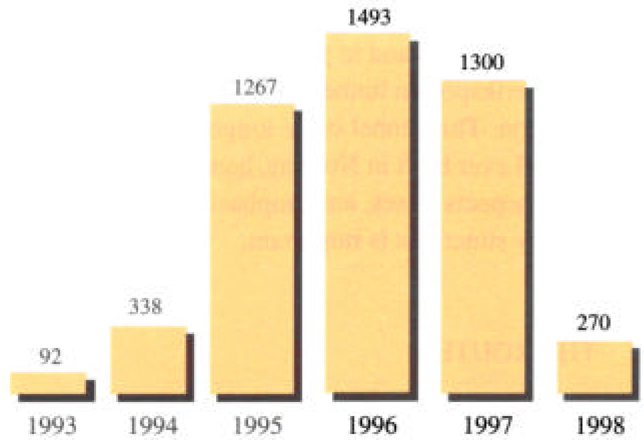

Fig. 7.1 The total budget for the Gardermobanen Rail Link is NOK 4, 760 million (1995)

3 PROJECT PRESENTATION

The two projects, including both the airport itself and the rail link, have an overall estimated budget of app. NOK 20 billion. The ground works for the Gardermoen Rail Link alone, including all but ballast, sleepers and rails, are estimated to NOK 2,2 billion. The existing railway was built in the 1850's where hand tools decided the route. Today the new line will cut sharp curves, increase limited speed potential and remove major capacity bottlenecks. To achieve the speed level, gradients must not exceed 15 m/1,000 m track. Curves must be relative moderate with a minimum radius of 2,400 m.

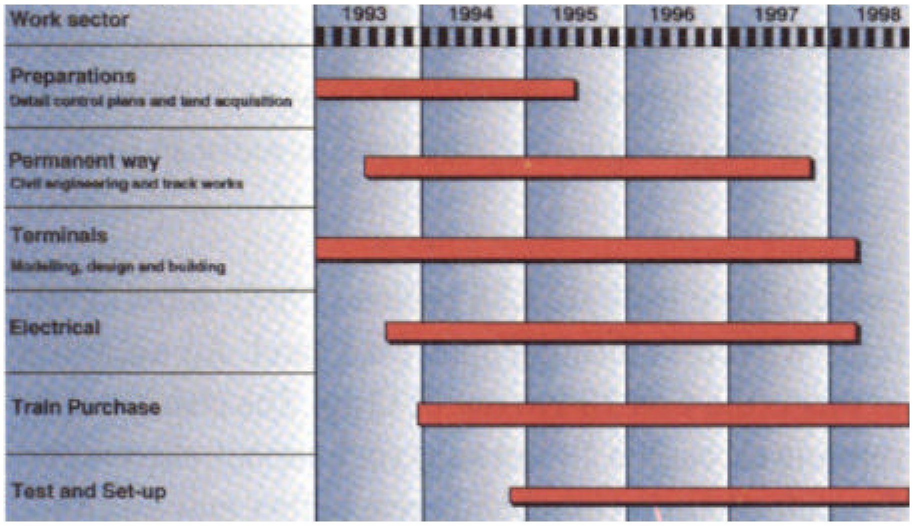

Fig. 7.2 Principal work schedule

The line will provide not just a satisfactory airport feeder service, but also an excellent commuter service to the suburbs it passes through. Hence the passenger anticipation for the first year of operation is 8 million passengers, or as much as 53 % of the airport passengers.

The project involves two hard rock tunnels and one soft soil tunnel in addition to 10 cut and cover tunnels and 16 bridges. This paper is mainly dedicated to the tunnel works and in particular the 13,900 m long Romeriksporten tunnel, which is already under construction. This tunnel is the longest public transport tunnel ever built in Norway, hence many new technical aspects arises, and emphasis on safety and emergency situations is important.

3 THE ROUTE

NSB prepared studies of two principally different solutions in the early stages of planning. One alternative involved a direct line from Oslo to Gardermoen in which the primary objective was minimum travel time. The second principal solution places more emphasis on incorporating the Gardermoen Line with existing rail traffic, thus serving existing traffic junctions and densely populated areas. Both solutions have been presented in several alternatives. The second alternative was recommended by NSB and is now under construction.

The route is double tracked and follows the existing line the first 2,5 km from Oslo Central Station before the train runs through the Romeriksporten tunnel, passing close under both densely populated areas and important recreational areas. In the eastern end, the tunnel is extended with a 500 m cut and cover tunnel and a railway junction system of connecting exit/entry ramps to the existing railway, in total six tracks in section. At Lillestrøm there are plans for a large bus terminal connected to the main train terminal. The line is planned as an open line parallel to the existing tracks. From Lillestrøm the new double track is to be laid northwards, following the present main line with certain deviations, until the Gardermoen Line branches off south of Jessheim. Here the Gardermoen Line runs north west under the airport and northwards where there will be another rock tunnel. Just before Eidsvoll, there will be a short soft soil tunnel in order to, among other reasons, preserve an old and important historic site

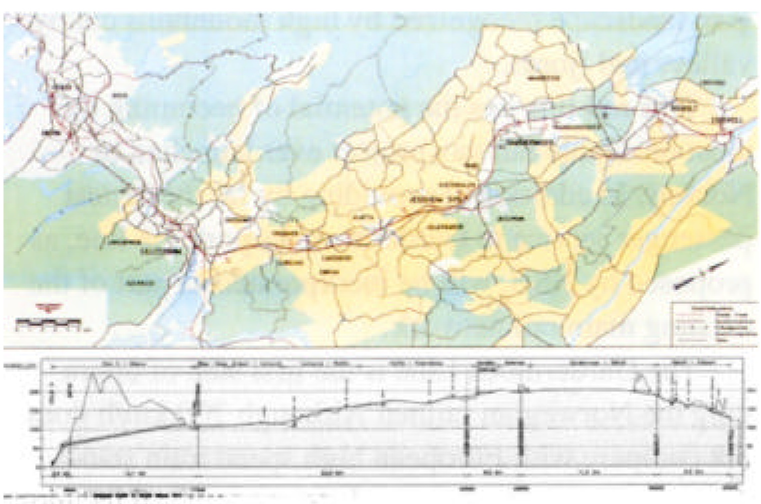

Fig. 7.3 Map of the route

At Gardermoen Airport, the train station will be placed beneath the airport terminal. The station will have four tracks traversing the concourse, and travelers will be taken to the check-in area by escalators and lifts. There has been an effort in planning the airport with as short walks as possible.

Size and length are the two major factors which make this tunnel unique.

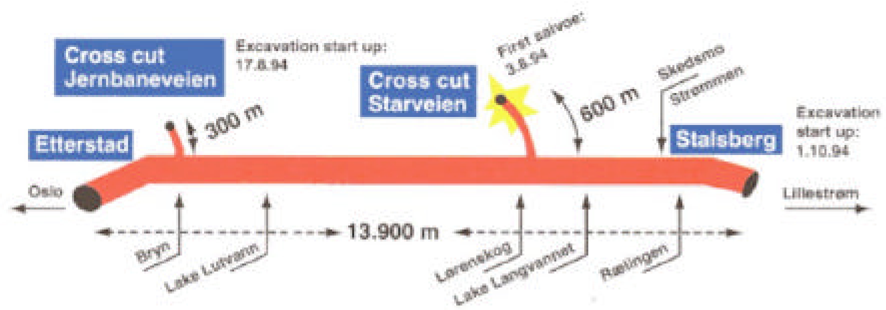

Fig. 7.4 The Romeriksporten tunnel

4.1 Geology

The bedrock along the tunnel consists mainly of a variety of gneisses. At the west end of the tunnel there is a Cambro-Silurian strata where the route runs through alternate beds of clay shale and alum shale before it crosses the main fault in the Oslo region at Bryn. Both sides of the fault are heavily disturbed, hence there are taken special precautions. The plan is to cross this area in a slow pace with extensive pregrouting, short salvoes, shotcrete and a full in situ concrete lining to be casted shortly behind the tunnel face. Only 265 m of the tunnel length is planned with a full concrete lining.

Existing buildings are endangered by settlements due to ground water drainage into the tunnel. The methods used are pregrouting from the tunnel and water injection both from the tunnel and from the surface. Minor water seepage will be taken care of by the tunnel lining. The overburden varies from 6-120 m.

A joint venture of four companies was awarded the contract in June 1994. The partners involved are Fagbygg A.S, Målselv Anlegg A.S, Nor Entreprenør A.S, all of Norway, and PEAB Entreprenad Väst AB of Sweden. The project group name is SRG, Scandinavian Rock Group. The contract sum is NOK 541 mill (1994) and is an unit price contract. The ground work contract involves drilling, blasting, support, lining and preparation of the tunnel base with drainage and ballast mats. Ballast, tracks and sleepers are done in one operation for the whole line. Electrical installation is also separate contracts. Dr.Ing Aas-Jakobsen is the main consultant for the ground works on this part of the project.

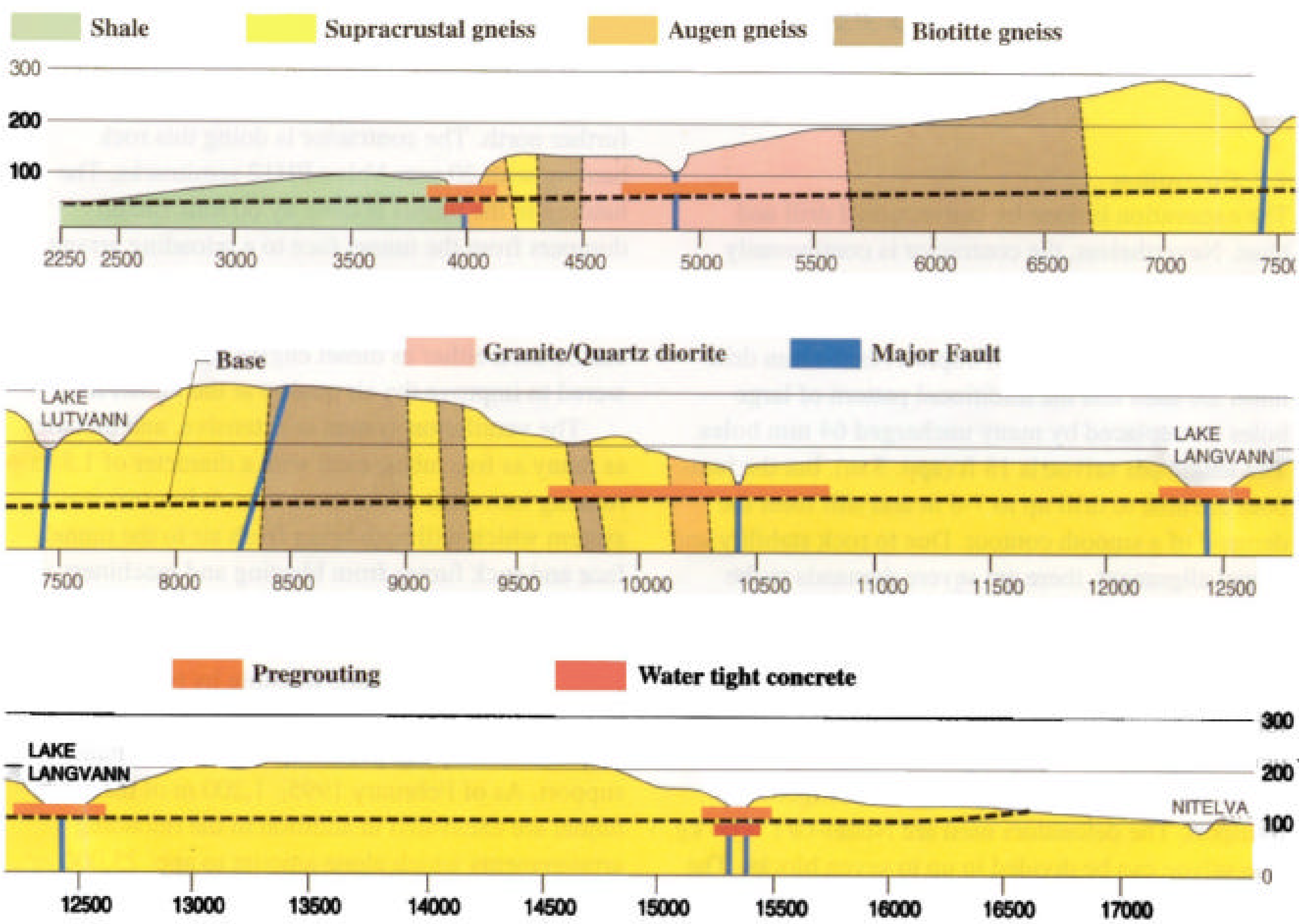

Fig. 7.5 Geological profile for the Romeriksporten Tunnel

4.2 Environment

Strong emphasis is put on environmental planning, both for the construction environment and for the surroundings. Among others, there is a recycling system for the water used for drilling in order to use as little water as possible, and to let little or no contaminated water out in the sewer systems. The Client, GMB, is responsible for a continuous measuring program during the construction phase in order to take care of vibrations and potential damage to nearby buildings. Settlement surveying and video inspection before construction start up is also done on approx. 3,000 buildings along the tunnel. Existing wells along the route are located and tested for quality and inflow.

4.3 Excavation

The excavation is done by conventional drill and blast. Nevertheless, the contractor is continuously working on improving the methods. This involves research with different drill patterns, various drill hole sizes and different types of explosives. 64 mm drill holes are used and the traditional pattern of large holes are replaced by many uncharged 64 mm holes. The length per round is 18 ft (app. 5 m), but the jumboes are able to drill up to 7-8 m and still meet the demand of a smooth contour.

Fig. 7.6 The Jumbo drilling rig

Due to rock stability and tunnel alignment, there are severe demands to the rock surface. At the bottom, the boreholes may not deviate more than 0,5 m from the contour. In the contour the explosive concentration is reduced by use of tube charge, smaller amount of Anolit in the holes, or detonating fuse. The major part of explosive used is Anolit , which is mixed at site to avoid dangerous transport. The detonators used are Nonel-G/T 0-60 kg. The round can be divided in up to seven blocks. The drilling is done by three new, Norwegian, Jumbo computer aided drilling rigs AMV 21SGBC from Andersen Mekaniske Verksted. The drifters are Montabert HC90.

An extensive research and development concerning the drill and blast techniques in long tunnels is going on in Norway in general. Among others, there is put great emphasis on slurry explosives which liberate less nitrogenous and poisonous gases. Another unique aspect by the Norwegian drill/blast method is drilling done simultaneous to charging. The highly experienced tunnel workers surely make their contribution to an effective round.

The support is done mainly by means of rock bolts, scattered and in systematic patterns, and shotcrete to be used in co-ordination with the rock bolts. Estimated amount is 65,500 rock bolts and 16,000 m^3 of shotcrete. It is used both rock bolts fully grouted by cement based mortar and polyester anchored rock bolts. All rock bolts are galvanized steel bolts and, when needed, additionally protected by powder epoxy. The shotcrete is a C40, both with and without steel fiber reinforcement.

The tunnel cross-section is 105 m^2 which makes 1,600,000 theoretically, solid m^3 to be excavated. The rock mass is transported to an old gravel pit and dumped for further refinement before it is used as construction material for the railway along the line further north. The contractor is doing this rock hauling with 20 new Volvo FH12 semi trucks. The hauling in the tunnel is done by 60 tons Euclid dumpers from the tunnel face to a reloading arrangement located at the bottom of the cross cuts. The loader itself is a new hybrid Brøyt XMed 53 which can be used either as diesel engine or electrically powered to improve the air quality at the tunnel face.

The ventilation system is extensive, and includes as many as five tubes, each with a diameter of 1.8 m running into each of the cross cuts. It is a two way system which will both bring fresh air to the tunnel face and suck fumes from blasting and machinery out through separate tubes.

The excavation is done simultaneous on five tunnel faces. This is made possible by two cross cuts of respectively 360 and 590 m. Plan of progress is estimated to 50 m/week per tunnel face ex. permanent support. As of February 1995, 1,200 m of the main tunnel are excavated in addition to the reloading arrangements which alone amount to app. 25,000 m^3.

4.4 Cross-section

The cross-section is 105 m^2 in general, but exceeds 170 m^2 at Stalsberg where there is a railway junction. Several factors influence the cross-section. There must be structural clearance enough for two trains to meet, with the pressure which occurs both on train passenger and on tunnel lining. Experience so far indicate the more space available the better. The Gardermoen Line will be served both by modern high speed trains and existing trains, hence there are many considerations to be taken. There must be sufficient space for tunnel lining, interiors and electrical systems. And there must be space enough to evacuate the tunnel in case of an emergency. Research in this area is being worked on.

Fig. 7.7 Railway junction at Stalsberg

4.5 Safety

In Romeriksporten the two cross cut tunnels will serve as evacuation tunnels for emergency and for maintenance work. Along both tracks there will be a sidewalk with a continuously hand grip. Fire is regarded as the worst threat to safety and all the new trains will have fire extinguishing equipment and a possibility to overrun the emergency brake. The latter, to in any case be able to run a burning train out of the tunnel. Already in the early construction phase, major planning is done for the safety alertness when in operation. An expert panel consisting of members from the local fire departments, hospital rescue teams, police, GMB and the NSB's safety personnel, is already working on preparedness measures.

4.6 Ventilation

The tunnel has a gradient of two to four per. thousand, which is anticipated to give an airflow in eastward direction of 1 ,5 m/s caused by the chimney effect. Because of the amount of frost in this area, it is expected that 2,5 km of the west end of the tunnel must have an insulated lining, whereas there is only need for 0,5 km insulated lining at the east end. These figures are connected with uncertainties, and airflow will be measured soon after breakthrough. Fans will be installed in order to overrun natural ventilation, among others, in case of fire.

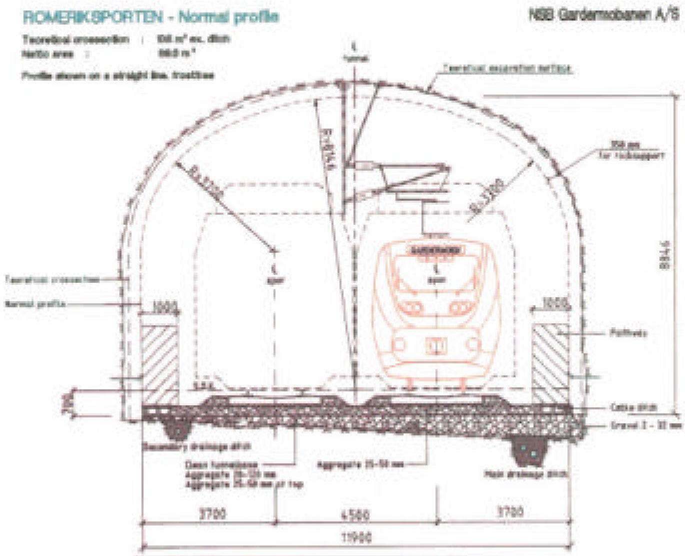

Fig. 7.8 Typical cross-section

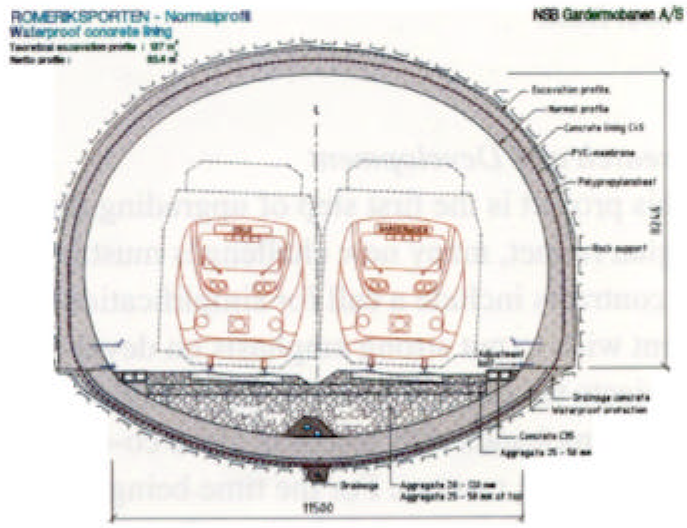

Fig. 7.9 Full concrete lining

4.7 Tunnel Lining

Water leakage problems are not expected in the tunnel in general. Nevertheless, water seepage is not allowed on the rails. Hence it is expected that most of the tunnel must have a waterproof roof lining which also can serve the whole cross-section when needed. This lining is planned as an insulated shotcrete lining with steel mesh. The insulation mostly used today is extruded polyethylene (XPS also referred to as PE-foam) and serve both as insulation and as formwork for the shotcrete. The lining is bolted with rock bolts in an even pattern. A disadvantage is the number of perforations made by the rock bolts, a convenience is the flexibility of the system. The insulated lining in the entry zone is planned as precast concrete elements which will have both an insulation layer and a membrane to prevent water seepage and hence ice building problems. The tunnel lining amount to 30% of the construction cost. There is extensive work done in this area today, both with research concerning shotcrete and development of an optimal water and frost protection, costwise and technically. In such matters, NSB uses the same set of rules as the Highway Department has done for some years, even though the pressure situation is quite different.

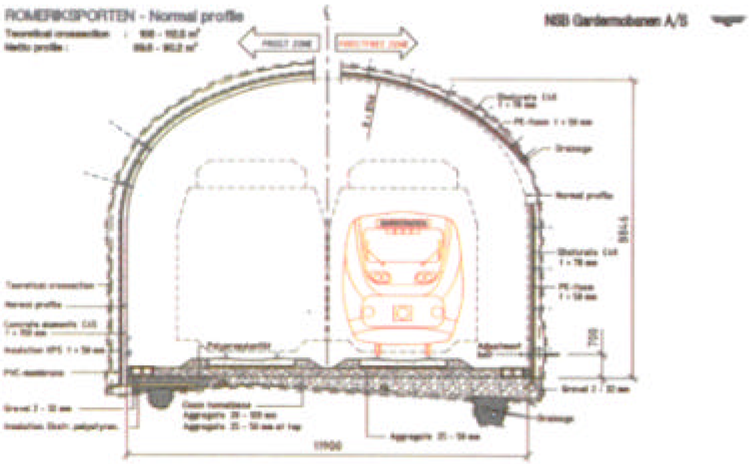

Fig. 7.10 Principal solution for tunneling in zones with and without frost.

4.8 Research and Development

Since this project is the first step of upgrading the Norwegian rail net, many new challenges must be met. All the contracts include a call for simplifications and the Client wish to put strong emphasis on development projects which might lead to new solutions. NSB Gardermobanen A/S works in close cooperation with NSB in such matters. For the time being, several development contracts are worked on. One comprise new materials, using rockwool as ballast mats to prevent vibrations and structure born noise. Another comprehensive project is connected to ballast bed contra asphalt bed in the tunnel. A third is linked to water and frost protection.

5 THE BEKKEDALSHØGDA TUNNEL

The tunnel is double tracked with a cross-section of 111.5 m^2. The length is 1,590 m and the bedrock consist of grey gneiss, typical for the area. The northern end of the tunnel runs under populated areas with an overburden down to 10 m. Special action will be taken concerning drill, blast and rock support. The contract will include the same work as Romeriksporten, in addition to several rail bridges, cut and cover tunnels, retaining walls, crossover bridges and passways. The contract is now out for bid, and will be awarded in May 1995.

5.1 Typical data for the Bekkedalshøgda tunnel:

Excavation ……………………………………….…177,000 m^3
Support, rock bolts ……………………………..….9,000 piece
Support, shotcrete ,……………………………..…..…3,000 m^3
Concrete lining ..……………………………………….70 m
Pregrouting w/sement …………………………...…120,000 kg
Tunnel lining w/ concrete elements ,……………...… 26,000 m^2
w/ PE-foam w/shotcrete ……………..16,000 m^2

6 THE EIDSVOLL TUNNEL

The location of the tunnel is in a landscape of deep and steep ravine valleys. The bottom of a ravine is 40-50 m lower than the mesa above. Max. slope is 1:2,25. This area is formed by the glacier pulling back some 10,000 years ago.

6.1 Ground conditions

The ground in the area consist mainly of medium fast to fast silty clay, with pockets, lences and layers of silt and some fine graded sand. The silt layers are not expected to be continuous, but might be enclosed in the clay.

The ground water table is 5 m below the surface. The pore pressure decreases with the depth. This means that the measured pore pressure is below hydrostatic pore pressure. Free groundwater was not found while doing the prospecting drills, but it's existence in silt-layers, which might be found, cannot be totally excluded.

6.2 General concepts

The tunnel has a total length of 492 m. About 460 m will be excavated as a tunnel while the portals will be cut and cover. The tunnel is single tracked with cross-section app. 50m^2 The tunnel will be excavated through three steps, each time closing a complete ring of shotcrete. Stage 1 is a side gallery of the top heading. Stage 1 will be excavated in steps of 0,8-1,5 m. Stage 2 follows 10 to 50 m behind stage 1, and opens the rest of the top heading. Stage 1 and 2 will be completed all through the tunnel before the excavation and closing of the bench in stage 3. After completion of the excavation and the shotcrete-steel rib lining, a complete reinforced 0,4 m thick concrete inner lining is cast in place all through the tunnel.

The contract was awarded to a joint venture AFSHT Group consisting of the Norwegian contractor AF Spesialprosjekt a.s and the German contractor Hochtief AG in December 1994. The contract is a design/build contract amounting to NOK 117 mill. Construction start up is autumn 1995, completion date July 1997.

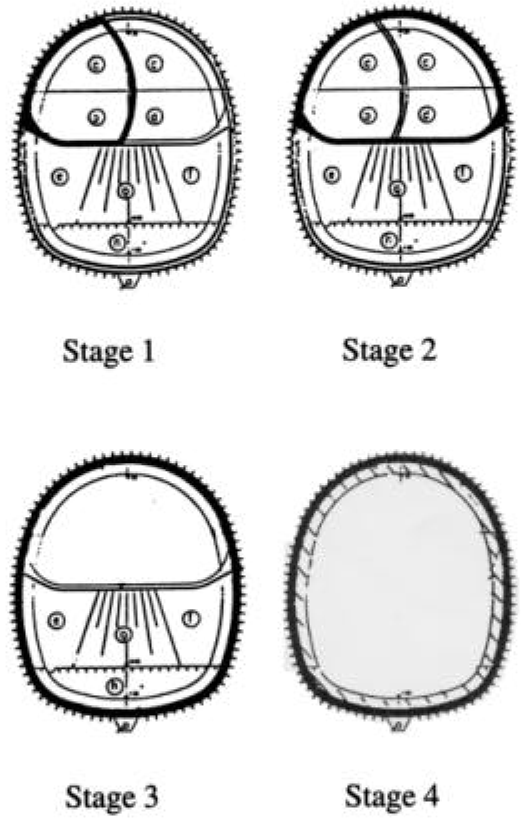

Fig. 7.11 Excavation of the Eidsvoll tunnel

Fig. 7.12 Ravine landscape at Eidsvoll

REFERENCES:

NSB Gardermobanen A/S: The new Gardermobanen Line, Oslo Central Station to Gardermoen Airport. 19 min – all aboard.

8 HOLMLIA SPORTS HALL & SWIMMING POOL IN ROCK

Magne Dørum
Fortifikasjon A/S Consulting Engineers

ABSTRACT: Holmlia Sport Hall and Swimming Pool is one of several large installations in rock in the Oslo region. The facilities are serving in peacetime as gymnasiums, swimming pools, bowling halls, shooting ranges etc., and can in very short time be converted into wartime civil defense shelters.

1 INTRODUCTION

Holmlia is a new residence suburb in the south-eastern part of Oslo. At an early stage of the planning phase, it was decided to build a modern sport center, including a swimming pool. The sport center should also be used as gymnasium for the junior high school in the area and for other social activities.

According to Norwegian regulations, air raid shelters for at least 20 % of the population had to be built in the area. Large volumes of rock material were needed for construction of roads, parking areas and as aggregate for concrete.

All these demands could be solved simultaneously by constructing the civil defense air raid shelter in rock, designed and equipped for gymnasium and swimming pool. In addition, surface areas could be saved for other sports and leisure activities, and ugly and dangerous open quarries could be avoided.

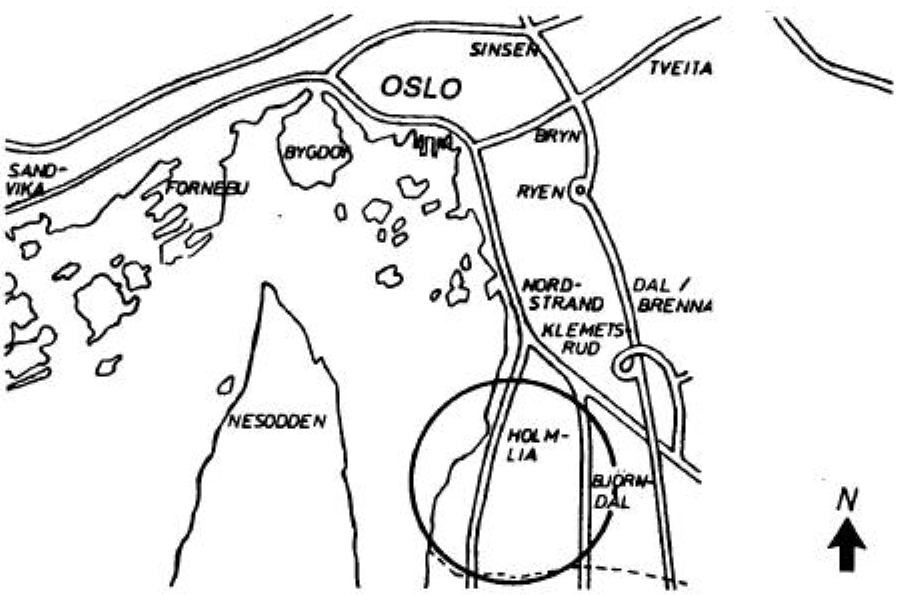

Fig.8.1 location in the Oslo area

Blasting and excavation of the facility started in September 1979, and was completed 18 months later, keeping pace with the development of the outside area.

Structural works and installations were carried on for another 24 months, and was finished when the people started to move into the new town.

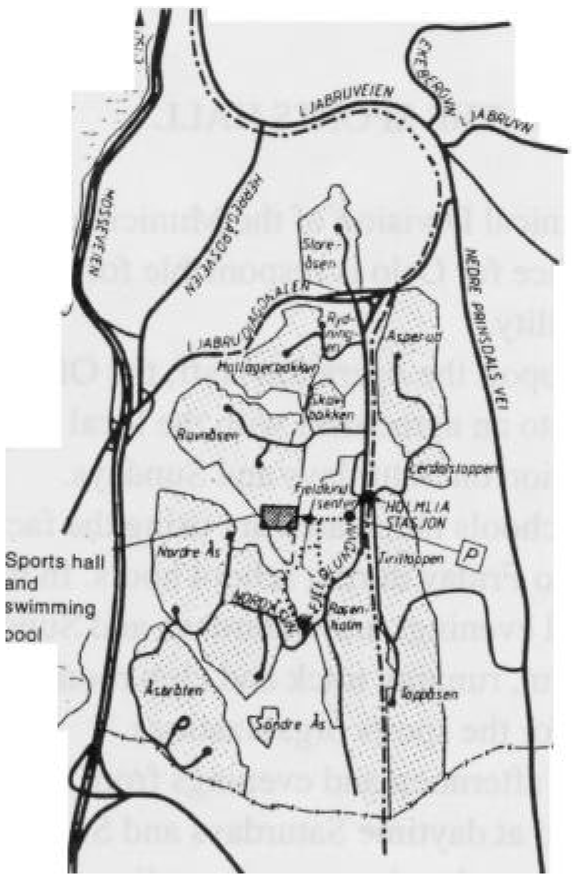

Fig.8.2 Local area map with location of hall

2 SITUATION AND ADMISSION

The sports hall is located in a small hilltop named Ravnåsen. The main entrance is from the east side of the hill, and connected to the area center by pedestrian and bicycle lanes. The walking distance from Holmlia Railway Station is only 250 meters. Another entrance is from the south-west side of the hilltop, serving as entrance for the Holmlia Junior High school and for disabled persons. A parking lot is at disposal for disabled persons.

Fig. 8.3 General view from Holmlia, with railway station and shopping center.

3 USE OF THE SPORTS HALL

The Technical Division of the Municipal Park and Sport Office for Oslo is responsible for the operation of the facility.

To support the operation staff, the Office has entered into an agreement with the local sport clubs for operation on Saturdays and Sundays.

The schools in the area are using the facility from Monday to Friday during school hours. In the afternoons and evenings and Saturdays and Sundays, the gymnasium, running track and club rooms are at disposal for the sports organizations.

In the afternoon and evenings from Monday to Friday and at daytime Saturdays and Sundays, the swimming pool and saunas are at disposal for the public.

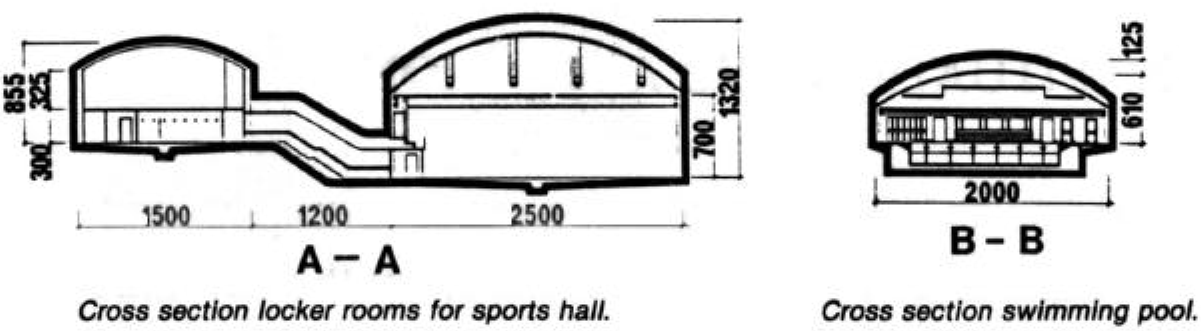

Fig 8.4 Plan-Basement/1st floor Lower part

The installations also includes trim rooms (health club) and solariums. It has been considered as very important that the gymnasium can be utilized for other public activities, as big assemblies and concerts. Big emphasis has therefore been put on acoustic treatment of the hall. In connection with the swimming pool, there is a privately operated health club, located on second floor above the locker rooms. The guests and instructors are using the swimming pool with locker rooms, saunas and shower rooms. On second floor, above the locker rooms for the gymnasium, 600 m^2 is used as activity room for the local sport club. The entrance from south-west is combined with a 60 m running track. The facility is authorized as a civil defense air raid shelter for 7,000 persons.

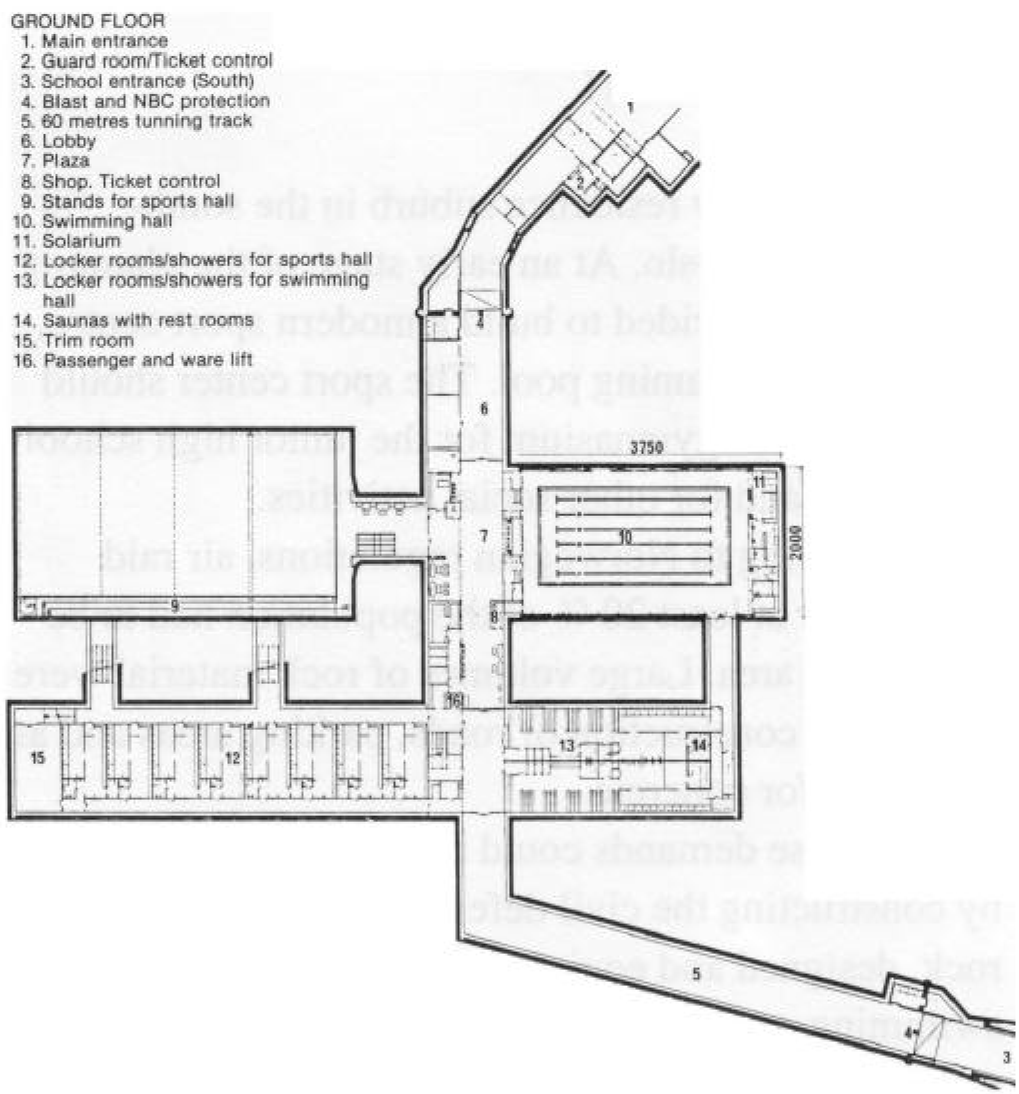

Fig. 8.5 Plan -1st floor

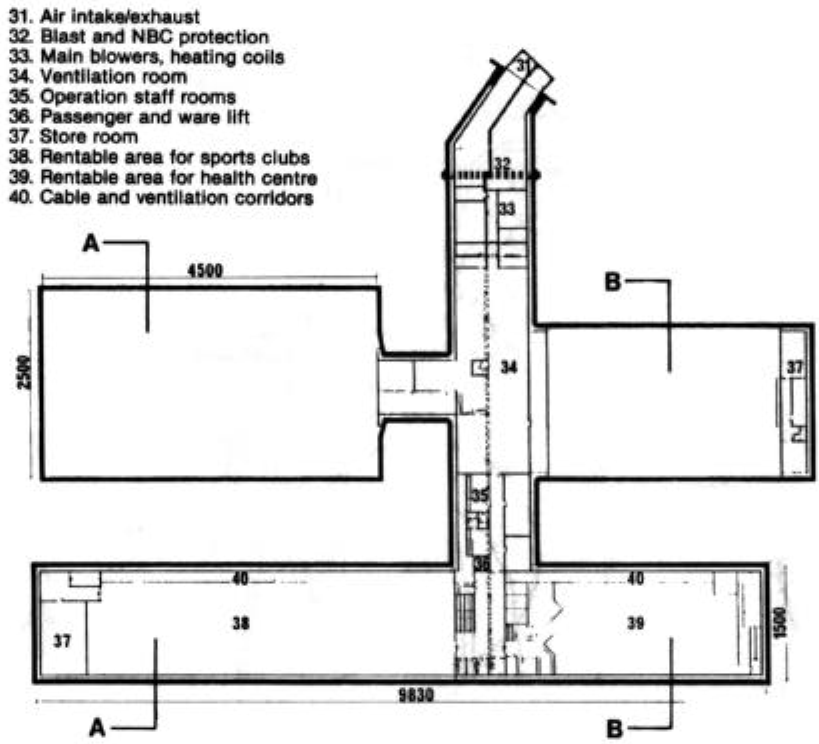

Fig. 8.6 Plan 2nd floor

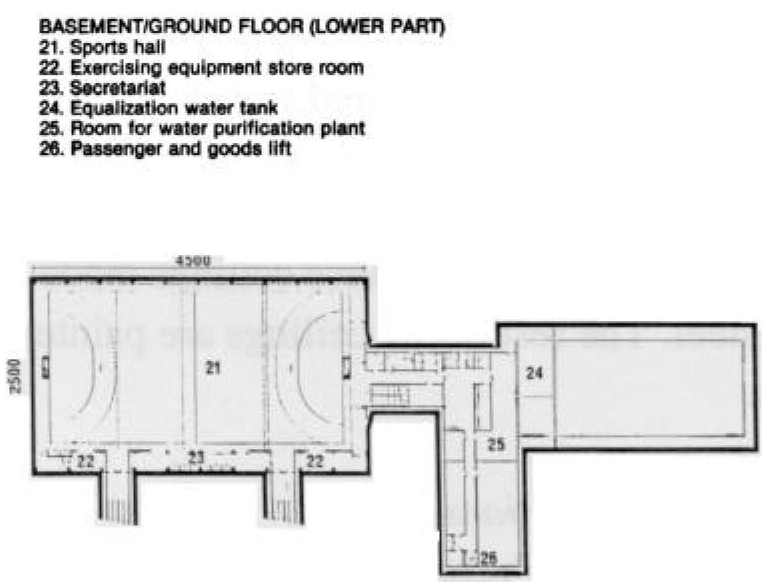

Fig. 8.7 Cross sections

4 PLAZA

From the plaza it is admission to the locker rooms for gymnasium and swimming pool. In the plaza, ticket sale, technical control room, rest groups, shops, public phones etc. are located.

Fig. 8.8 Plaza

Fig. 8.9 Gymnasium

5 GYMNASIUM

The hall is 25 m wide and 45 m long, with a maximum height of 13 m. The hall is giving sufficient space for one handball court, 20 x 40 m. The hall is in addition equipped for gymnastics, basketball, volleyball and badminton The hall can be divided in up to 4 separate courts by folding partition walls. One of the partition walls is a heavy acoustic type. The two other partitions are mesh type. The gallery gives room for 350 spectators. Under the gallery there are store rooms for exercising equipment, rooms for referees, leaders and reserve players, and access stairs to the locker rooms. One of the stairs is equipped with a special lift for disabled persons.

6 SWIMMING POOL

The swimming hall is 20 x 37 m with a 25 x 12.5 m swimming pool. The pool has 6 tracks, and is constructed according to the Deck Level principle. The depth is varying from 0.9 m to 1.8 m. Normal operating temperature is 28 -29°C in the pool and 30°C in the air. In direct connection with the hall are first aid room, solarium and equipment store.

The men's and women's locker rooms have a capacity of 115 persons. In connection with the locker rooms are showers, saunas and lavatories and separate locker rooms/shower rooms for the instructors. Two separate family rooms with all facilities are situated with direct entrance from the plaza to the swimming pool.

Fig. 8.10 Swimming Pool

7 CONSTRUCTION AND MATERIALS

In 1979, the span of 25 m in the gymnasium was considered to be close to the maximum from a technical and economical point of view. The geotechnical conditions in the area were therefore thoroughly checked on beforehand by core drillings.

The rock is mostly consisting of grey gneisses with some variations in structure and compound. The surface of the hill is very weathered. Both entrances had to be secured by bolting and on site cast concrete. The tunnels and the halls were excavated without any special problems.

The roof in the gymnasium is secured by reinforced concrete arched beams, cast in contact with the rock. The spacing between the beams is 5 m. Between the beams, the rock is secured with steel bolts. The concrete beams have a cross-section like an inverted T, and are used as supports for precast concrete slabs installed between the beams.

The roof in the swimming hall and on 1st floor above the technical rooms is secured by steel bolts and by monolithic cast concrete arched roof.

The roof above the 2nd floor of locker and shower rooms is secured by steel bolts and steel fibre reinforced concrete.

The roofs are installed to protect against rock down fall and dripping from rock. The roofs are covered with asphalt roofing. The leakage water is collected in gutters and conducted to the drainage system.

Totally the rock roof was secured by 2,000 bolts, varying in length from 1.5 to 4.7 m. Approximately 1,500 m^2 walls are covered with shotcrete.

7.1 Walls and partitions

Because of the Civil Defense Regulations and safety aspects, use of combustible materials is very limited. Generally reinforced concrete is used for the interior construction. However, in the gymnasium, partition walls and acoustic screens are made of wooden materials. The walls in the shower rooms are covered with ceramic tiles. The other walls are painted.

7.2 Floors

The floors in the sport hall and the south-west entrance have a special sport covering. The swimming pool and floors in the swimming hall are covered with ceramic tiles. The rest of the floors are covered with homogenous vinyl. In the locker rooms a special no slip vinyl covering is used.

7.3 Ceilings

The ceiling in the sport hall is covered with woven glass fibre material and painted. The swimming hall has an aluminum rib ceiling. The rooms for rent on 2nd floor above the locker rooms have ceilings made of corrugated steel. The rest of the ceilings are painted concrete slabs.

7.4 Doors and glass partitions

Doors and glass partitions have generally an impregnated wood framework.

In the swimming hall and locker and shower rooms, the doors are made of glass fibre reinforced plastics. The rest of the doors are made of steel.

Doors in blast and gas barriers are made of concrete.

7.5 Protection level

The shelter area, has minimum 20 m rock overburden and is considered to withstand direct hits of large conventional bombs and blast wave overpressure from nuclear explosions at a distance of a few hundred meters.

The shelter will also be fully protected against effects of all known poisonous war gases and radioactive fallout from nuclear weapons.

8 TECHNICAL INSTALLATIONS

8.1 General

The technical installations are satisfying the demands on climatic conditions and internal milieu for peacetime use as gymnasium and swimming pool.

In addition the installations shall satisfy the demands for use as a shelter for 7,000 people

The stable temperature inside the rock is very favorable for the energy economy. Especially, it is favorable for the swimming hall, where there are no problems with humidity and condensation, contrary to a conventional swimming hall.

Big emphasis has been put on energy saving installations, as heat recovery from exhaust air, from waste water heat pump dehumidification of swimming hall etc.

8.2 Ventilation and Air-conditioning System

The ventilation and air-conditioning system is located on 2nd floor above the plaza. Fresh air is supplied from outside through a second floor above the main entrance tunnel.

The fresh air volume is variable depending on the use of the facility and the number of visitors. The fresh air is preheated during winter operation (precooled during summer) in a fluid circulation heat recovery system. The system also makes possible to utilize radiator heat from the emergency power plant.

The preconditioned fresh air is supplied to a number of air-conditioning units, serving different areas of the facility. The air-conditioning units control temperature and humidity.

For the swimming hall, a dehumidification system is installed in connection with the corresponding air-conditioning system. The condenser heat from the refrigeration machine is used for reheating of air after dehumidification and for heating of the pool water, thus covering the evaporation heat loss from the pool and the hall by utilizing the heat pump effect.

The ventilation system for shelter operation is blast protected and has filtration system for radio-active fallout, biological and chemical agents. (NBC- filters).

Fig. 8.11 From the air-conditioning plant

8.3 Heating System

The heat for operation of the facility in peacetime is supplied from the district heating system. In the heating central in the basement, heat exchangers for heating of utility water, heating of hot water for air conditioners and swimming pool are installed.

From the heating central, the heat is distributed as secondary heat to the different parts of the facility.

8.4 Swimming Pool

The swimming pool is equipped with a deck level circulation system, with water supply through nozzles in the bottom of the pool to ensure an even distribution of filtered and conditioned water. The water is purified in sand filters, and active carbon filters, and is chlorinated, PH conditioned and heated before supplied to the pool.

Fig. 8.12 Swimming pool water purification plant

8.5 Automation and Monitoring

It is installed equipment for fully automatic control of the conditions in the facility. From the operation engineers office, all technical installations can be monitored with reading of all important temperatures and humidity levels, registration of water quality in the swimming pool and recording of malfunctions.

8.6 Sanitary System

The waste water is transported to a pumping pit in the basement floor and is pumped out of the facility. Black water from toilets and drain water from the showers have separate piping system. The drain water from the showers are pumped through a heat exchanger for preheating of utility water. The showers are of water saving type (push-button with timer). To simplify the cleaning of shower rooms, a central high-pressure washing system is installed.

8.7 Electric and Electronic Systems

The main electric power is supplied from the main municipal grid via a transformer station outside the main entrance. Emergency power for shelter operation is supplied from an automatically operated diesel engine powered generator.

The illumination fittings are generally fluorescent. The illumination level is approximately 400 lux. The saunas have electric heaters. In the locker and shower rooms, electric floor heating is installed, and in addition electric hair and body driers.

All electric installations can be remote controlled from the control room in the lobby. Also the low voltage systems can be controlled from the control room.

There are installed door telephone system, emergency communication system, antenna system, TV watch-over system, amplifier system, fire alarm system etc.

A passenger and goods lift is installed between the floors. In addition a chair lift for disabled is installed between the gymnasium and the locker rooms.

9 EXPERIENCE

Since start of operation in 1983 until end of 1994, only regular maintenance has been performed on the technical installations. For the building constructions the expenses for maintenance have been zero.

The energy consumption is far below that of comparable sport halls and swimming pools owned and operated by the Municipality of Oslo.

The annual energy consumption is average 1.92 mill kWh. Compared, a conventional aboveground facility has an average annual energy consumption of 3.0 mill kWh.

10 PROJECT INFORMATION

- Excavated rock volume, total	53,000 m^3
- gross floor area basement	1,620 m^2
- gross floor area ground floor	3,740 m^2

- gross floor area 1st floor	2,190 m^2
- total floor area	7,550 m^2

(The floor area is including swimming pool, stands, gallery and 1st floor above entrances)

- normal floor interval	3.0 m
- sport hall, maximum height	13.0 m
- sport hall width x length	25 x 45 m
- sport hall free height	7.0 m
- sport hall, court	22.4 x 45 m
- sport hall, stand capacity, persons	300-350
- swimming hall width x length	20 x 37 m
- swimming pool	12.5 x 25 m
- swimming pool, lanes	6 pcs
- swimming pool, depth	0.9 –1.8 m
- filter capacity, purification plant	150 m^3/h
- showers, sport hall	70 pcs
- showers, swimming pool	28 pcs
- showers for operating staff	4 pcs
- fresh air blower capacity 43	200 m^3/h
- NBC filter system capacity	25,600 m^3/h
- emergency power unit capacity	300 kVA
- shelter capacity, persons	7,060

Costs:

Engineering fees, supervision, administration	NOK 4,600,000
Excavation, rock securing and building constructions	NOK 36,000,000
Heating, sanitary, water purification installations	NOK 4,100,000
Ventilation, cooling inst.	NOK 4,200,000
Electro technical installations	NOK 4,800,000
Total cost A =	NOK 53,700,000

Approximately 20 % of the cost is for extra installations for use as public shelter.

9 EKEBERG OIL STORAGE AND EKEBERGTANK - CENTRAL OIL STORAGE FACILITIES FOR NORWAY

Asbjørn Føsker

Ekeberg Oil Storage ANS

SUMMARY: After the Second World War the Norwegian Authorities started considering a safe storage system for fuel in the Oslo area. Oil storage facilities in rock caverns were already known from Sweden. The rock under the Ekeberg hill in Southern Oslo was of good quality, and close to the existing oil terminal at Sjursøya. The storage facilities built in rock caverns under the Ekeberg hill are now the main oil storage facilities for refined oil products in Norway.

1 INTRODUCTION

The main oil storage facility for refined products in Norway is located in rock under the Ekeberg hill in Southern Oslo in the harbor area. This storage facility was built in rock caverns of good quality close to the existing oil terminal at Sjursøya.

The facilities are used for temporary storage by the five largest oil companies operating in Norway, and for strategic oil storage by the Norwegian Government. Each oil company has separate storage chambers for different qualities of gasoline, diesel and kerosene.

In comparison with similar storage facilities around the world, the installations in the Ekeberg hill are huge. Approximately 50% of the total annual demand for petroleum products in Norway passes through the caverns. For strategic reasons the detailed layout and information on total storage capacities are not available to the public.

2 HISTORY

Shortly after the Second World War the Norwegian Authorities started considering a safe storage system for fuel in the Oslo area, secure against acts of war and sabotage. The final conclusion was that the most suitable solution was storage in rock caverns.

Fig. 9.1 The Ekeberg oil storage and the Ekebergtank are located in the mountain next to the Sjursøya Oil Terminal

Similar storage facilities had already been developed in Sweden. It was found that by locating the storage close to the oil terminal at Sjursøya the functions of strategic storage and distribution terminal could be combined.

The planning started in the early 1960's with SENTAB as consultant, and the construction phase was in two stages, carried out by Norwegian contractors and suppliers.

The first stage, Ekeberg Oil Storage, was constructed in 1966-69, and the second stage, Ekebergtank, was added in 1975-78. The construction costs were NOK 42 million and NOK 74 million, respectively.

Both facilities consist of a series of excavated unlined caverns located below the groundwater level. The presence of groundwater prevents leakage of volatile petroleum products, thereby eliminating the need to line the caverns. The storage principle for the two facilities are different, as shown in Fig. 9.2 and Fig. 9.3 and further explained below.

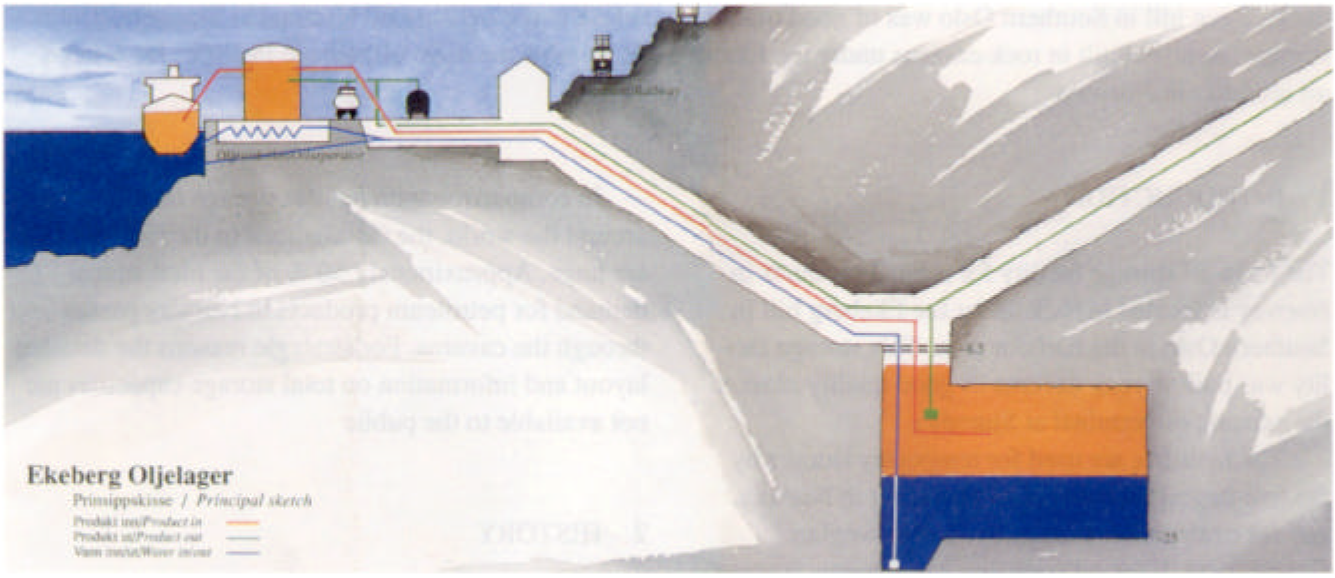

Fig. 9.2 Storage principle Ekeberg Oil Storage

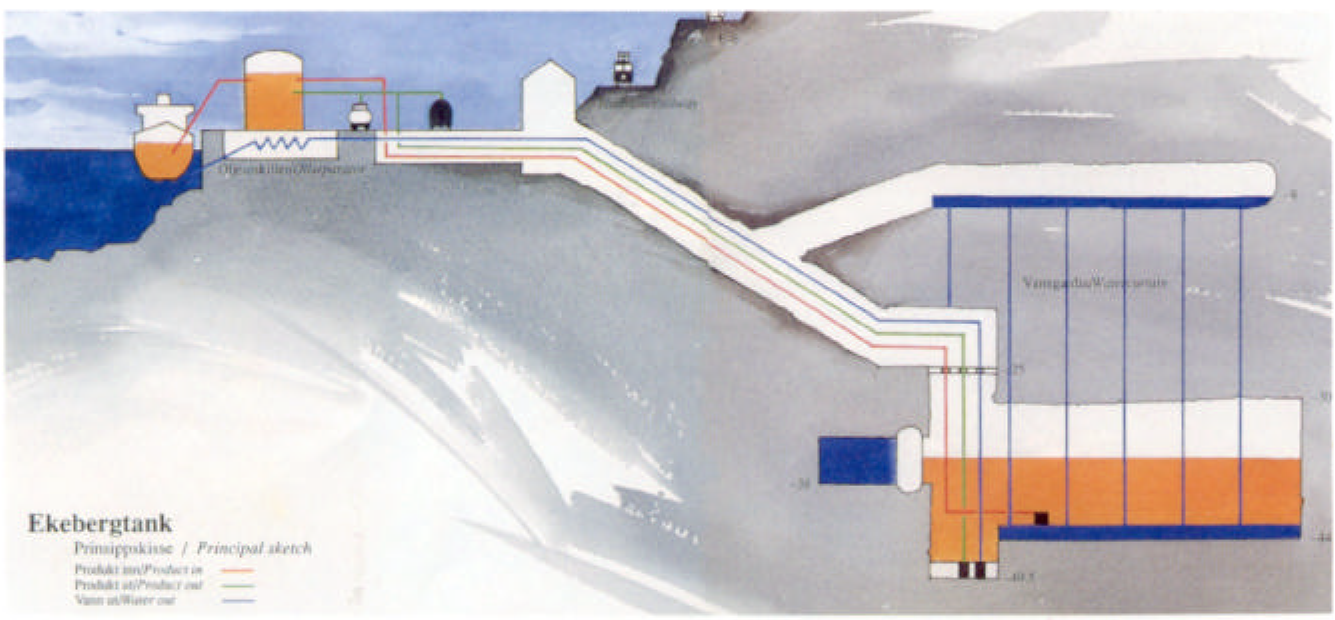

Fig. 9.3 Storage principle Ekebergtank

3 STORAGE SYSTEM AND GROUNDWATER CONTROL

In Ekeberg Oil Storage the top level of the stored oil product is constant while a water bed in each cavern is constantly adjusted. The top of the caverns is formed as a bottleneck with a constant cross-section to minimize the surface level of the oil and reduce the evaporation loss. The waterbed consists of sea water pumped in from the fjord. When receiving oil products, water is pumped out from the bottom of the cavern to avoid oil pollution. As a safety measure this water passes through an oil separator before being discharged into the sea.

In Ekebergtank the volume of the waterbed is constant while the surface level of the stored oil product rises and falls depending on the stored quantities. Use of this principle is possible because Ekebergtank is used to store aviation fuel and gasoils which are far less volatile than automative gasoline.

The caverns are located well below the sea and groundwater levels with the deepest caverns extending down to about Elev. -45m.

Extensive control measures have been implemented to ensure that the groundwater is always at a sufficient level. A water curtain consisting of drilled holes connected to an open canal in a rock tunnel has been installed to avoid any interconnection between the caverns of Ekeberg Oil Storage and the lower elevated Ekebergtank.

Groundwater leaking into the caverns of Ekebergtank is collected and pumped to the harbor basin through an oil separator.

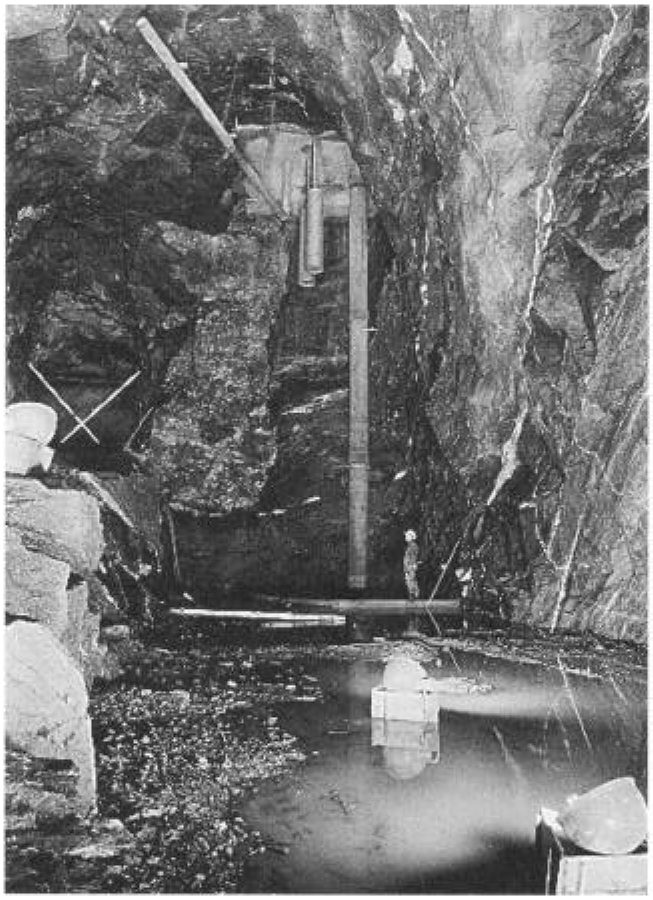

Fig. 9.4 Storage cavern at Ekeberg oil storage. The import pipeline is coming in at the top, the product pumps are hanging at the top. The water pipeline goes all the way down to the bottom. The picture is taken during construction, before the cavern was filled with water and gasoline.

4 ROCK CONDITIONS

The rock consists of massive precambrian gneisses of good quality. Blasting operations and excavation of tunnels and caverns were carried out without major technical problems.

Limited rock support was needed and was carried out by bolting and shotcreting. None of the facilities required concrete lining.

5 TECHNICAL INSTALLATIONS

Pipelines have been installed in rock tunnels with easy access and adequate space for maintenance. In total there are 4 km of walkways and 35 km of pipes transporting petroleum products.

All operations are highly automated. The control center is located underground, from where the entire facilities are operated and controlled, and is manned continuously.

High-voltage electricity is supplied to transformers placed in rock. Separate large diesel aggregates can supply sufficient electricity in case of breakdown of the external supply.

6 EXPERIENCE FROM OPERATION

The experience from 25 years of operation confirms the efficiency and safety of the facilities. There has hardly been any stop in the operations and no major accidents. This is remarkable as tankers arrive almost every day, approximately 350 ships per year, and products are pumped out from the caverns continuously. The storage complex requires few people during operation, and the operating costs are very low.

10 LARGE HEAT PUMP FACILITY IN ROCK CAVERN -EFFICIENT SYSTEM FOR DISTRICT HEATING AND COOLING

Margaret Matte
Berdal Strømme a.s.

ABSTRACT: A district heating and cooling plant in Sandvika represents new ideas and solutions in the Norwegian energy sector. The plant produces and distributes energy to buildings in the town of Sandvika, a suburb 10 km west of Oslo. The energy is produced by heat pumps placed in a rock cavern located close to the sewage tunnel from Oslo West (the YEAS tunnel, ref. Article No.4), utilizing the sewage water as the main energy source. The project has a favorable environmental impact:

- The centralized heat pump system delivers energy with a minimum use of primary energy sources.
- Noise and air pollution is virtually eliminated.

1 INTRODUCTION

The idea of creating a district heating and cooling system in Bærum Municipality emerged together with the plans to develop 300,000 m^2 of new building land close to the town of Sandvika.

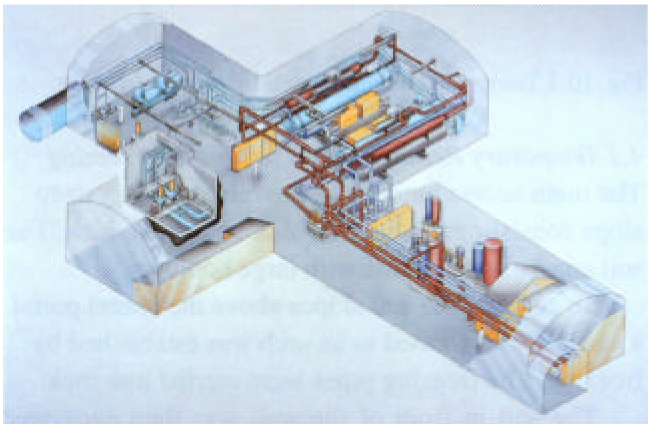

Fig. 10.1 Rock cavern with heat pump system

The system was developed and implemented by the utility company Bærum Energy, which is fully owned by the municipality and with Hafslund Engineering as the main consultant. Hafslund Engineering is now a part of Berdal Strømme a.s. After extensive studies it was concluded that a district heating system based on heat pumps and utilizing sewage water as the energy source would provide the most economic solution. By choosing heat pumps, district heating could be combined with district cooling through a separate cooling distribution network. The economy of the project suggested that both the new area and the existing buildings in the town should be connected to the system.

Preliminary design started in 1985 and the implementation was decided one year later. The time schedule was very tight. Following detailed design, construction took place in 1987-88.

Delivery of heating and cooling to the first customer started already in July 1988.

2 THE PROJECT

The energy production plant consists of two heat pumps installed in a rock cavern which is located close to the main sewage tunnel YEAS from Oslo West. The plant produces hot and chilled water for heating and cooling, respectively, and consists of a base load unit and a peak-load and emergency unit. The heat pumps constitute the base load unit while the emergency unit is a separate facility of oil-fired boilers. The latter unit also provides peak-loads during the coldest winter days.

The hot and chilled water is distributed to subscribers through a piping network placed in trenches and partly in concrete culverts. Heat exchangers in the buildings transfer the heating and cooling to the internal systems in each building.

Fig. 10.2 Heat pump installations

3 ENERGY PRODUCTION PLANT

The two heat pumps each have a capacity of 7 MW for heating and 4.5 MW for cooling. Average energy production per year is 60 GWh for heating and 14 GWh for cooling. By using the same heat pumps for production of hot and chilled water, the economy of the project was enhanced both regarding investment, operation and maintenance costs.

The main energy source for the heat pumps is raw sewage water in the YEAS sewage tunnel. Heat from the sewage is transferred to the heat pumps through two tube-heat exchangers. Electric energy is supplied to the compressors to support the heat pump process. One kWh of electricity is required to produce 3 kWh of heat energy.

The heat pumps are placed in a rock cavern which also has space reserved for a third heat pump unit. The emergency unit consisting of three oil-fired boilers is placed above ground in a former boiler house belonging to an old paper factory that had been closed down.

4 THE ROCK CA VERN

The heat pumps are installed in a rock cavern complex comprising a car park and a public air-raid shelter, located in the hills of Hamangaasen close to Sandvika. The complex has two halls, crossing each other perpendicularly. The lengths of the halls are 60 meters and 40 meters with span widths of 12-14 meters, giving a total area of 1,000 square meters. Two access tunnels lead to the hall system, one for cars and the other for pedestrians only.

The bedrock in the cavern area consists of lime/sandstone and claystone in alternating layers. A few weakness zones cross the hall system, but due to variable jointing of the rock some overbreak in the cavern roofs occurred.

The caverns were therefore systematically supported with fibrecrete and bolting. The cavern is located below residential buildings of 4-5 storeys. Careful blasting was therefore required with strict requirements to the allowable vibrations.

The two access tunnels were constructed through overburden and weathered rock and immediately below existing structures. To secure these structures it was decided to strengthen the soil close to the tunnel portal by ground freezing for temporary support in the construction period as described below. Planning and implementation of the ground treatment was done by GEOFROST A.S., a Norwegian company specializing in ground freezing.

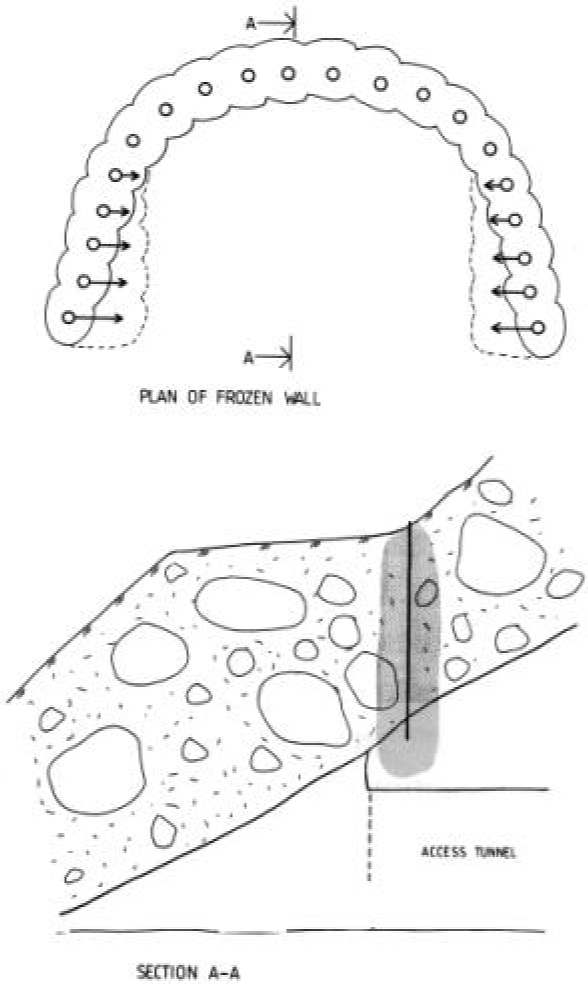

Fig. 10.3 Temporary retaining wall by ground freezing

4.1 Temporary Retaining Wall by Ground Freezing

The main access tunnel was established into a steep slope consisting of about 10 m of soil above rock. The soil consists of moraine with large boulders.

To stabilise the soil slopes above the tunnel portal a vertical wall formed as an arch was established by freezing. The freezing pipes were carried into rock.

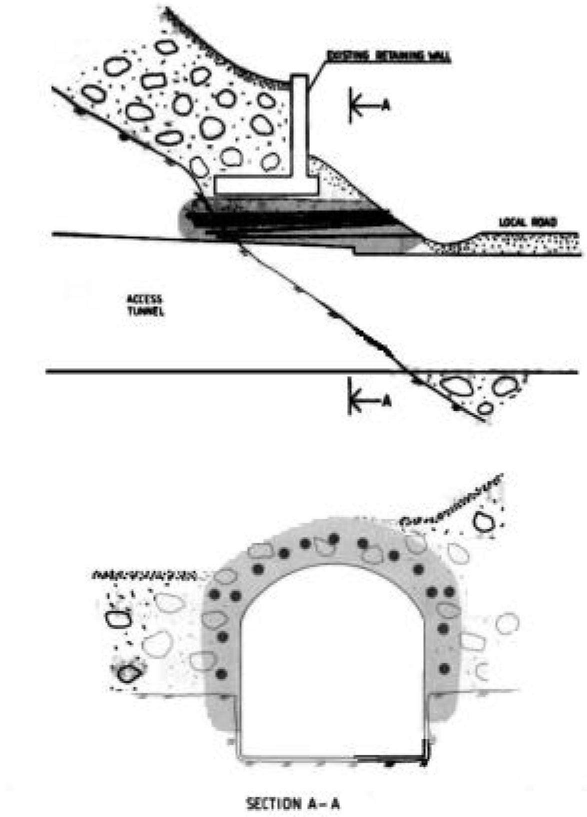

Fig. 10.4 Freezing pipes and excavation in front of tunnel portal

Fig. 10.5 Main access tunnel under construction

The soil in front of the wall was then excavated and the rock portal was blasted. A large diameter, 6 m, corrugated steel pipe was placed to form the access before soil was backfilled to re-establish the slope.

Time required for the ground treatment was about one month: one week for drilling and placing of pipes, one week for initial freezing, and two weeks for maintaining the freezing while the access was constructed.

4.2 Temporary Tunnel Support by Ground Freezing

The secondary access tunnel crossed a soil slope of moraine below a retaining wall which was founded in the soil.

Temporary support was established by freezing of walls and roof by horizontal freezing pipes. The pipes were carried into rock for anchoring of the support structure.

The access tunnel was then blasted and excavated. It was assumed that the frozen soil arch could only be exposed for one day before a steel shield had to be in place. During this time about 5-10 cm of the frozen ceiling could disintegrate, because the moraine was unsaturated and contained little fines.

Drilling and placing of pipes was done in two weeks, the initial freezing required one week only, while the maintenance freezing lasted for another two weeks for completion of the access tunnel.

Underneath a secondary road the access tunnel was constructed during a weekend. Excavation was performed and concrete elements were put in place and backfilled for re-establishing the road above the access tunnel.

Fig. 10.6 Temporary tunnel support by ground freezing

Fig. 10.7 Secondary access tunnel under construction

5 ENERGY DISTRIBUTION SYSTEM

The heating and cooling are distributed to the consumers through separate pipes laid in trenches or in concrete culverts. In the new town area the pipes are placed in prefabricated concrete culverts below street level, together with other utilities for electricity, water, sewage and telecommunication. Four parallel pipes are required for the district heating and cooling, two for leading hot and chilled water to the consumers and two for the return water to the plant.

In the trenches the heating pipes consist of an inner pipe of steel and a casing of polyethylene with an insulation layer of polyurethane in-between. The cooling pipes were made of polyethylene without any insulation. In the concrete culverts, both pipes are mineral wool insulated the steel pipes.

Prior to designing the system the actual heating and cooling requirements of each building to be connected were thoroughly investigated. Computerized methods were used for efficient planning and design, for selection of pump strategy and pipe sizes, analysis of heat losses and dynamic analysis of pressures and flows.

Implementation of the distribution system was complicated due to extensive construction activity, and topographic conditions which imposed restrictions to lay-out of the system and scheduling of the construction work.

6 COMPUTER CONTROL SYSTEM

The entire facility is managed and controlled by an ambitious master control system, that minimizes the operation and maintenance costs and optimizes the operation conditions. The control system continuously monitors the facilities and produces systematic operation reports and other documentation.

The master control system communicates with substations and metering stations in each building through a separate communication network which is placed together with the distribution pipes. A main computer in the control center communicates with programmable control in the buildings to ensure that the use of electricity and oil is minimized.

7 HEAT PUMP IS A CHOICE OF THE FUTURE

The energy systems commonly used to-day are not generally energy efficient as concluded by the World Commission on Environment and Development under the United Nations. This commission set up in 1983 was chaired by the Prime Minister of Norway, Mrs. Gro Harlem Brundtland, consisted of policy makers and scientists from 22 countries, and was given the brief to formulate "a global agenda for change".

The "Brundtland Report" proposed seven goals for the future, one of them is to meet essential needs for energy. Within this sector the report recommends the adoption of soft energy systems which it describes as the best way towards a sustainable future. The most appropriate methods of energy conservation were found to be:

- combined heat-and-power (CHP, cogeneration),
- heat pumps for both space heating and cooling,
- small scale hydropower plants in suitable sites.

The energy efficiency of electrically driven heat pumps is remarkable. For every kWh of electricity fed into the system the output is 3-5 kWh. Primary energy sources are therefore extremely well utilized by heat pumps which are particularly suited and economic for space heating and hot tap water production. When the same heat pump is utilized in addition for space cooling, the energy conservation as well as the environmental effect of the complete system is unique.

The heat pump will reduce emissions of carbon dioxide (CO_2) when oil is substituted for heating purposes.

A district cooling system eliminates air-conditioners in buildings and is therefore an important contribution in the struggle to minimize the release of hlorfluorcarbon gases (CFCs) which can deplete the ozone in the upper atmosphere.

11 UNDERGROUND AIR TRAFFIC CONTROL CENTER

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ABSTRACT: The most important element within a programme to undertake a modernization and enhancement of the air traffic control services in South East Norway is the establishment of a new Air Traffic Control Center for Oslo FIR (Oslo ATCC). The new air traffic control center is situated underground. Administration and training facilities are in buildings at ground level, connected to the air traffic control center by a tunnel. The article presents the design and construction principles for this advanced computerized and vital installation in the Norwegian air traffic control system.

1 INTRODUCTION

Civil Aviation Administration of Norway (NCAA) is currently undertaking a comprehensive programme for the modernization and enhancement of the air traffic control services in South East Norway. The most important element within this programme is the establishment of a new Air Traffic Control Center for Oslo FIR (Oslo ATCC), which will provide both area and approach control services within the area.

A national ATS Academy will be establish in conjunction with the ATCC. The ATS Academy will be using the ATCC Simulator for both basic and advanced training of air traffic controllers. The facility will also provide for maintenance, development and testing of application software and system enhancements. The air traffic control center is situated underground, with tunnel connection to the administration and training facilities in buildings at ground level. Major operation systems supporting the ATC activities include the operational ATC system encompassing radar data and flight plan data processing subsystems, the ATS simulator system, the Voice Communication System and the operational Information System. Sub-system integration and control will be managed by a control and monitoring system. All sub-systems will be highly automated, and will meet strict availability requirements.

The inherent system capacities and flexibility, coupled with expansion capabilities, will ensure that the new Oslo ATCC will have the capacity to handle projected growths in air traffic in a safe an efficient manner well into the next century.

2 THE OSLO ATCC SYSTEM MISSION

The objective of Oslo ATCC system can be described as following:

- To enhance the safety of air travel through the timely acquisition and presentation of flight related data for use by air traffic controllers and support staff.
- To support the training of air traffic controllers and support staff.

- To support the maintenance, development and test of application software as well as evaluation of revised operational procedures. The Oslo ATCC system will be composed from the following sub systems:
  1. ATCC Opertational System, comprising a central complex and interactive ATS-units.
  2. ATS Simulator System
  3. Control and Monitoring System
  4. Software Maintenance and Development System

3 PROJECT DESCRIPTION

Air traffic in Norway, measured in numbers of passengers, has increased by 82% in the course of the last ten years. A total of 16.1 million passengers traveled through Norwegian airports in 1991. NCAA owns, operates and maintains 19 primary airports in Norway, some of them in cooperation with the Ministry of Defense. In addition, the CAA is responsible for overall planning of non-government owned airfields (secondary airfields, short runway airfields and others).

The new control center built at Røyken, named ATCC (Oslo Air Traffic Control Center) will replace the present control center, located at Oslo Airport Fornebu.

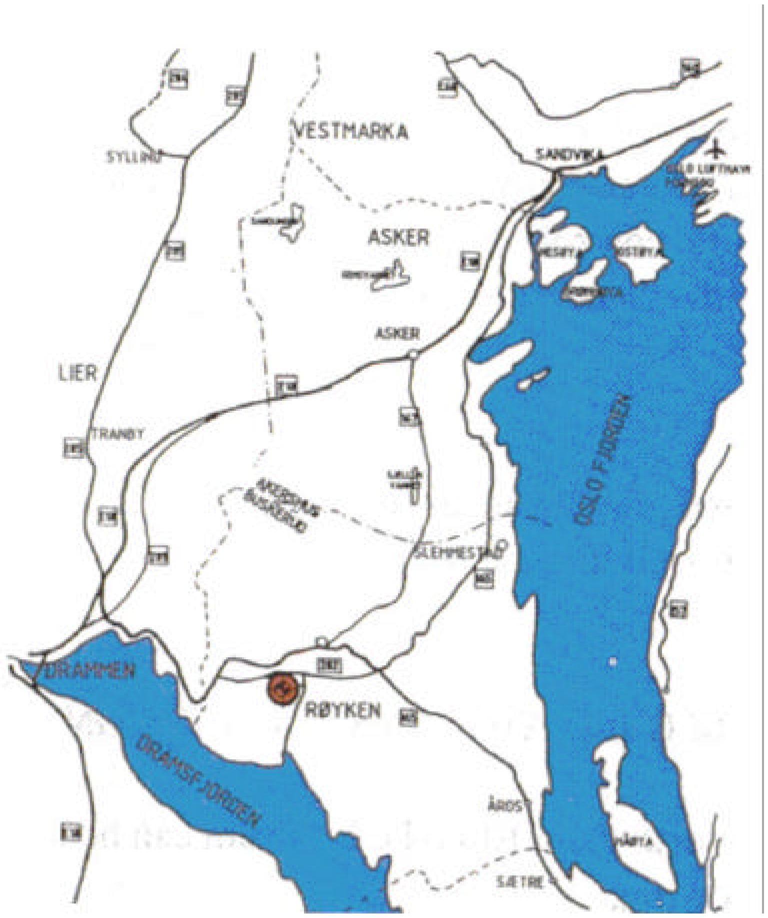

Fig. 11.1 Map showing location of the new Oslo ATCC at Røyken

4 PROGRESS REPORT ON THE CONSTRUCTION PROJECT

In November 1987, the Ministry of Transport asked SBED to undertake the construction of Oslo Air Traffic Control Center, Oslo FIR. The assignment of constructing the ATCC was based on room requirements, prepared by PABAS on the instructions of the Civil Aviation Administration of Norway (NCAA).

A project team was constituted on October 1988. The formal application for building license was forwarded to the authorities in March 1989.

The extensive blasting and excavation works started in August 1989, and the building license was issued in January 1990. The building operations started in June 1990 and were completed and approved in March 1992. The NCAA took possession of the buildings in April 1992.

NCAA is responsible for the radar, data, and communication installations in the buildings. The installation of these systems commenced in the autumn of 1992 and the initial trial run of the simulator took place in January of 1994. Normal operations are planned to commence in March 1995.

5 CONSTRUCTION WORKS

5.1 Basis for planning of facilities

Oslo ATCC, Røyken, comprises six different administrative and operation functions. These functions are:

a) Administration Region Røyken, Air traffic control
East Norway.
b) Oslo ATCC, including educational and training facilities in this connection.
c) ATS-school (Air Traffic Control School for Norway).
d) ETATS-School. (School covering all activities within the CAA).
e) CAA Air Navigation Department (operation and maintenance of data, radar information and communications equipment)
f) Operation and maintenance functions related to the facilities.

5.2 Location

At the time when the location for the new Oslo ATCC was decided, the political discussions for the location of the new Oslo Airport were intense. Independent of the final airport location, the ATCC was located at Røyken, west of Oslo.

5.3 Design of buildings

Design of facilities has been based on cost effectiveness studies, consideration of investment cost as well as costs of operation and maintenance. Recognized and tested technical solutions for construction materials and finish have been chosen. Buildings have been designed with construction flexibility in mind; i.e. expansion possibilities as well as rearrangement of the layout, within given limitations. The rock cavern has been excavated large enough to allow for considerable expansion. Main access is through the central hall of the administration building, with connection to the control center in rock through an access tunnel.

5.4 Underground building

The main functions of the ATCC, i.e. the control room with associated support facilities, are located in rock for protection purpose. Base area of the cavern is approx 2600 m^2. The building in rock is in three stories.

5.5 Administration building

The parts of the ATCC that do not need fortification protection are placed in the administration building located outside rock, adjacent to the protected facility. The administration building has a base area of approx 1810 m^2, and is constructed in three stories.

5.6 Personnel quarters

A building with accommodation facilities has been constructed for external CAA personnel staying at the center for longer periods. This building is located near the entrance to the site. The building has a base area of approx. 665 m^2, and is constructed in two stories.

5.7 Connection tunnels

To ensure that the ATCC functioned according to intentions and requirements, it was necessary to construct connection tunnels between the different parts of the facility. Main access to the ATCC is a tunnel from the administration building. This access is made as a corridor with an internal ambient.

Access to support functions has been arranged as a tunnel directly from the outside, to prevent maintenance and repair work from interfering with normal operation of the control center. The maintenance tunnel can be used by cars.

In addition to this maintenance tunnel, other tunnels have been constructed for ventilation purposes.

6 ROCK CONDITIONS AND UNDERGROUND CONSTRUCTIONS

6.1 The rock cavern

The main rock formation of the area is Drammensgranitt, a strong red granite. The strength of the rock has been decisive for the choice of width and shape of the cavern.

The main cavern was placed in a massive formation without any observed weak zones. The main direction of the excavation is approx. 45º to the direction of the main cracks.

6.2 Rock support

The permanent rock support is based on systematic rock bolting, combined with 10 cm of shotcrete. There was no need for heavy concrete lining.

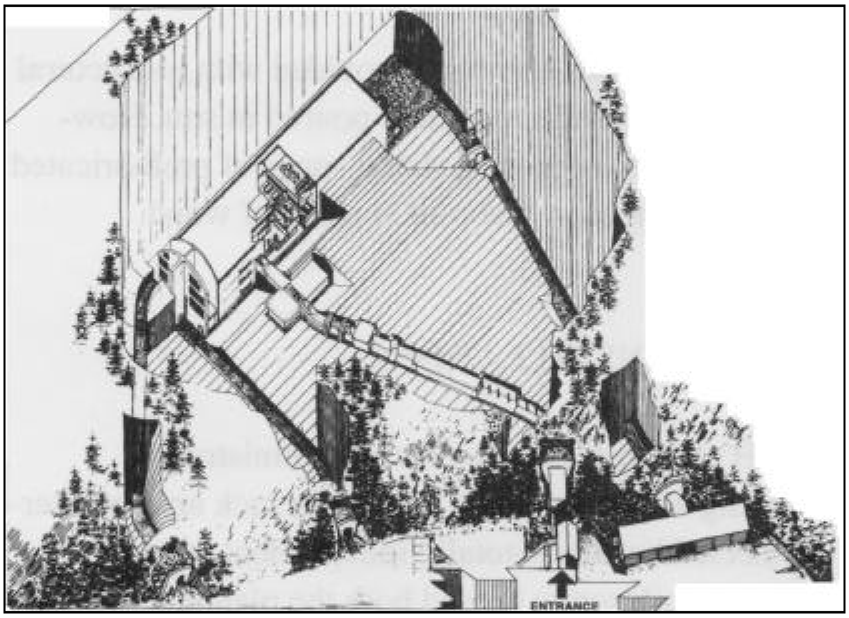

Fig. 11.2 View of the building in rock cavern,

6.3 Underground building – structural system

Columns and beams in the middle of the building are made of prefabricated concrete elements. Intermediate slabs and roof slab are made of prestressed concrete elements. The entire building is enclosed in an envelope of Corten-steel to obtain full EMP-shielding.

6.4 Fire risk evaluation of building in rock

The continuous functioning of the facility is especially important to air traffic safety, in peacetime as well as in war.

Generally, the building is fireproof and the furniture and equipment do not represent a large fire hazard. The interior to the largest possible extent is made of non combustible and fire-retarding materials, which will not release toxic gases in case of fire.

The building has been provided with a fire alarm system, and the most important rooms are equipped with halon fire extinguishing systems. Electric cables are of non combustible and non toxic type.

There are four exit tunnels which can be used for fire escape.

7 STRUCTURAL SYSTEMS

7.1 Administration building

The structural system is made of prefabricated concrete elements. Intermediate slabs and the roof slab area are supported on beams and columns. Wind forces are distributed by the slabs to walls around stairways and end walls of the building.

The auditorium building is made of reinforced concrete poured in situ. The roof is supported on frames of laminated wood.

7.2 Personnel quarters

This building has also been provided with a structural system of reinforced concrete poured in situ. However, the roof supporting slab is made of prefabricated concrete elements. The roof is made of wood.

8 HVAC INSTALLATIONS

The HVAC installations for the administration building and the building located in rock are not inter-connected but act as totally independent units.

The philosophy behind both the planning process and the choice of HVAC systems has been formed in close cooperation with the user.

The most important objective was to create a technical installation that would ensure continuous operation, and at the same time provide flexibility regarding future alterations. Other considerations were easy maintenance and energy efficiency. Flexibility has to a great extent been achieved by separating ventilation, heating and cooling systems, and by using standardized components.

Heat recovery and free cooling facilities have been introduced where feasible. Waste heat from water chillers is used to cool ventilation air.

8.1 Cooling system

Ventilation air for technical facilities in the rock cavern building is cooled by means of chilled water produced in three chiller units.

Chilled water is distributed to cooling coils, fan coils, cooling ceilings and room coolers throughout the building, in areas where cooling is necessary.

The cooling of condenser circuits for water chillers takes place by using three axial fans with variable pitch angles and drawing outside air through the condenser coils. A similar cooling system, with two chiller units, has been installed in the administration building.

8.2 Air-conditioning

The OPS Center is totally air conditioned, with fully developed temperature and humidity controls. Ventilation air is supplied at a constant rate and condition (20 ºC and 50% RH).

Heat loads in the different rooms are removed by use of chilled water recirculating through fan coils or cooling ceilings. The generator room, the UPS room and other power rooms are cooled by means of large room coolers, recirculating and cooling the room air.

8.3 Heating system

The administration building is heated by convectors installed on perimeter walls, and supplied with hot water of 90- 70ºC. The heating water is produced in a combined electrical/oil fired boiler. The rock cavern building is heated by small electric panel heaters.

8.4 Sanitary installations

The buildings are connected to the domestic water and sewage system in Røyken. A new connection to existing facilities in Røyken (2 km long) center was included in the project. Sanitary installations and water supply are provided in accordance with standard Norwegian regulations.

8.5 Ventilation system

Conditioned air is supplied to the various rooms in accordance with Norwegian regulations. No recirculation of air takes place. Heat recovery is performed by means of water circulating between air inlet and outlet coils.

The administration building has four air handling units supplying conditioned air. The ventilation system is divided, to satisfy different requirements in the various parts of the building.

Air is supplied to the different rooms using ceiling mounted diffusors. In operation and control rooms, the air is supplied using low impulse method, i.e. slightly chilled air supplied through floor mounted diffusors.

The ventilation system for the rock cavern building is designed for secured operation. A reduced amount of air is supplied through gas tight dampers and NBC filters. Most of the air is then revitalized and recirculated.

8.6 Fire protection

A separate smoke evacuation system has been installed in the rock cavern building. Its main task is to prevent smoke from entering the emergency exits. This is achieved by forced evacuation of air from a fire zone, and by pressure control of the rest of the building.

Technical rooms and power supply room are equipped with a halon fire fighting system. At the time of construction this was the only suitable type of installation, as the power supply was not allowed to be disconnected in the event of a fire. In addition, a selection of fire resistant and non-toxic materials has been used throughout the installation, to prevent the occurrence of poisonous gases.

Fig. 11.3 Entrance to the main tunnel

8.7 Building automation

Electronic control devices of direct digital type (DDC) have been installed to provide building automation. The autonome subcentrals are connected via LAN to a central monitoring system, with one work station in each building. More than 2,000 points are connected to the system and made available on the monitoring screen.

The central monitoring system also contains a programme for automatic status and alarm transmittal to external subscribers via modem.

9 ELECTRICAL INSTALLATIONS

9.1 Power supply

Electricity for the installations is provided by two substations. The substation for the installations in the rock cavern building has two transformers, each rated at 800 kVA. The substation for buildings outside has one transformer rated at 800 kVA and one at 630 kVA. The latter provides power for heating purposes. Distribution voltage is 400V, TN-S system.

An emergency power plant, consisting of three diesel-electric units, has been built for the installations in the rock cavern building, and important technical equipment in buildings outside. Each aggregate is rated at 700 kVA and covers 50% of the load. This emergency plant also contains a UPS-installation for the supply of power to communications and computer equipment. The installations consist of three UPS-units connected in parallel. Each unit covers 50% of the load. The UPS units have a common battery bank, with a 100% load capacity or 530 kVA for 30 minutes.

For the main power distribution panels, circuit breakers of plug-in type are used. The main power panel for UPS-power supply has a double bus bar system to make servicing possible without cutting the power to any circuit. Capacitors have been installed for correction of power factor and a harmonic absorber filter is placed on the UPS' primary side.

9.2 Shielding

For EMP-protection, the vital installations in the rock cavern building are within a steel shielded building, and all metallic cables are protected by EMP-filters. For this reason, the installations have an extensive earthing system.

9.3 Power distribution

All main cables inside the buildings are placed on cable ladders or are contained in cable ducts. The main equipment rooms have data floors wherein cable ladders and ducts are installed.

9.4 Lighting installations

Throughout the planning process, great emphasis has been placed upon the importance of minimizing the operating expenses regarding all lighting installations. For operation and simulating rooms, and rooms where data terminals are installed, special care has been taken to avoid glare and reflection on screens.

9.5 Cable installations for tele and data equipment

A flexible cable system covering a1l the buildings has been installed.

9.6 Supporting communication systems

A public address system covers all the buildings, both for warning and general information purposes. The maintenance department has been provided with an additional internal telephone system, which makes it possible for the manager to reach personnel whenever or wherever they are working.

9.7 Fire detection and fire extinguishing system

All buildings are controlled by an automatic, analogue, addressable fire detection system. Each building has its own central unit, which can alert the local fire brigade directly. Internally, an alarm from the central unit in one of the buildings also activates the control panels in the other buildings so that alarms are sounded throughout the installations. Displays are installed at all central places to advise people where a fire is located. For the most important technical rooms, automatic halon extinguishing systems are installed. These are, however, limited to a minimum and are only installed where other systems are not available. The halon systems are connected to the fire alarm center, so that the alarm system cover the whole installation.

9.8 Control system

An access control system covers the entrance and other important doors. This is connected to an alarm center. In addition an ITV-system has been installed.

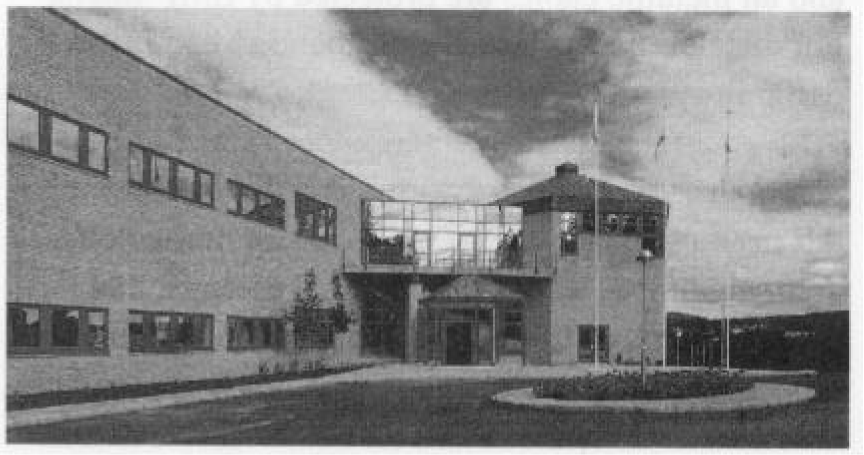

Fig. 11.4 The Oslo ATCC administration building

12 SUPPORT METHODS AND GROUNDWATER CONTROL

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ABSTRACT: The paper describes rock support methods which are used in the Oslo-region. Design of rock support is often carried out using the Q-system as a useful tool. The most important method for temporary and permanent support is based on use of rock bolts and wet process fibre reinforced shotcrete. This method allows for greater flexibility in design and it does not influence the rate of drive as significantly as more rigid methods. Examples of use of rock support are given.

Underground excavations in Oslo require strict control of ground water leakage due to the existence of marine clay that is very susceptible to consolidation settlements and alum shale which can undergo swelling. Permanent waterproofing is taken care of by pre-grouting rock mass or by in situ cast concrete lining. In road tunnels, different types of element cladding are in use. The most important ones recently constructed, are described.

1 INTRODUCTION

The most frequently used rock support methods in the Oslo region are the same as are used in Norway in general, i.e., rock bolts and wet mix steel fibre reinforced shotcrete. These methods allow for a greater flexibility in support design and save time, even in poorer rock mass. In general the sedimentary rocks with sills and dykes, gneisses and granites of the Oslo field need some support, at least spot bolting. Heavier rock support such as reinforced ribs of shotcrete or in situ cast concrete arches, are often necessary when traversing fault zones. Underground structures are usually located at a shallow depth, and depressions in the bedrock formed by weakness zones often result in little rock overburden.

Depressions are generally filled with a soft compressible marine clay. These deposits are very susceptible to consolidation settlement as a result of groundwater leakage into a tunnel or cavern. Alum shales
can also be found in Oslo which, when drained, can undergo considerable swelling due to oxidation and a breakdown of diagenetic bonds. Groundwater that leaks through alum shale can be extremely aggressive. Underground excavations in Oslo therefore require strict control of groundwater leakage.

Attention has focused on tunnel cladding for road tunnels more and more over the last 4-5 years. The different types used in the Oslo region are always insulated to prevent the build-up of ice in tunnels during the winter.

2 ROCK SUPPORT

2.1 Methods and Materials

Typical support is usually bolting and/or steel fibre reinforced shotcrete, S_{fr} Bolting is the dominant form of rock support since it mobilises the strength of the surrounding rock mass in the best possible way. A combination of bolting and S_{fr} is the most versatile support method yet devised. It can be applied to any profile as either temporary or permanent support simply by changing the thickness of the shotcrete applied and adjusting bolt spacing. In poorer rock masses a thick load-bearing ring can be made of shotcrete, i.e., reinforced ribs of shotcrete, RRS, allowing for further flexibility in spacing, thickness, combination with bolts, etc. Cast concrete arches, CCA, are still in use but not as frequently as before. The use of steel arches and ground freezing are other methods occasionally used for stabilizing the ground.

Both end-anchored and fully grouted rebar bolts are often used at the same site. End anchored bolts are usually installed close to the face where immediate support is needed. If expansion bolts are used they are normally grouted later to serve as permanent support. Resin anchored bolts are used as permanent support with immediate effect at the face. Bolting combined with the use of steel straps between the bolts and eventually shotcrete is also used to some extent.

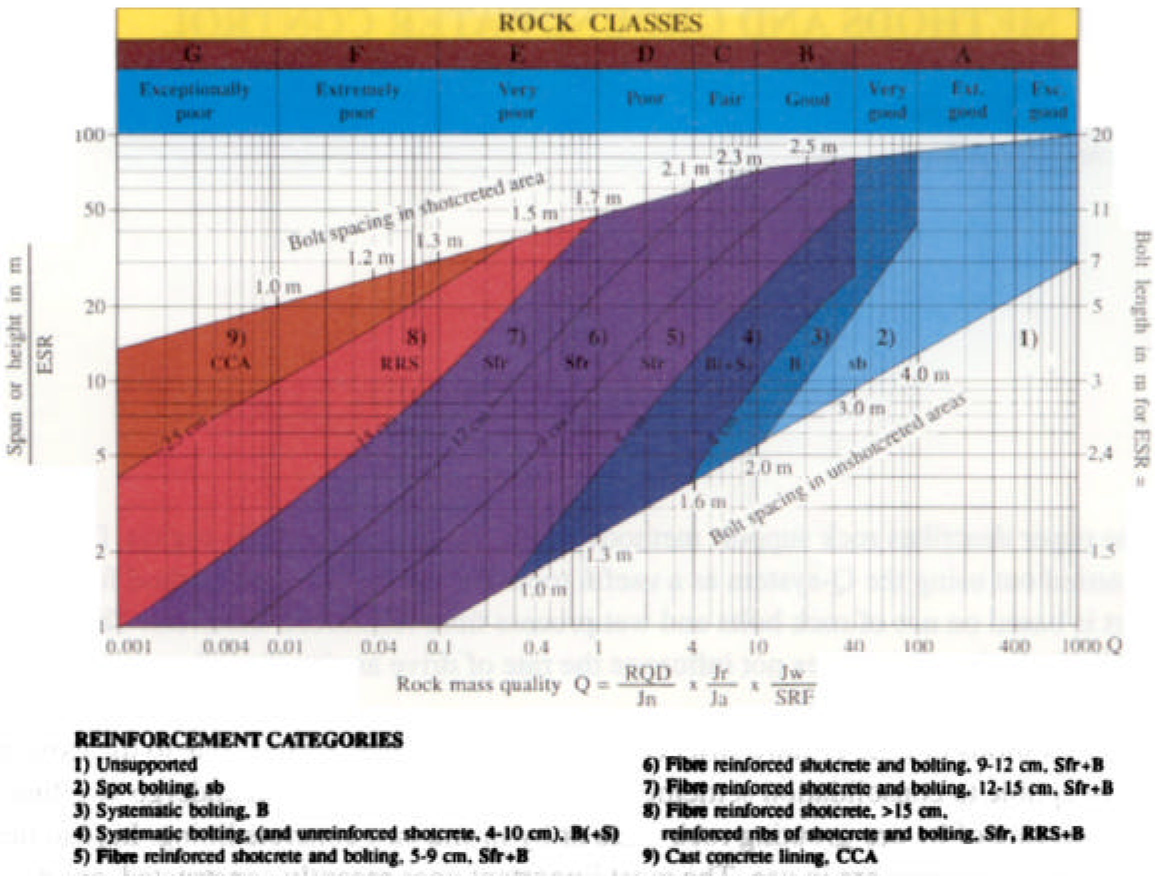

Fig. 12.1 Rock mass classification and recommended support.

Wet process shotcrete replaced the dry process in the early 1980s. Steel fibre began to replace mesh as reinforcement at the same time. Today a typical shotcrete mix looks like this:

Portland cement (c) 450-550 kg/m^3

Silica fume (s)	3-10% of cement weight
Aggregate	0-10 mm
Plasticizer	0.3-1 % of cement weight
Superplasticizer	0.3-1 % of cement weight
Steel fibre	50-90 kg/m^3
Water/(c + 2·s)	0.40-0.45
Slump	15-18 cm
Air content	< 4 %
Temperature	15-20°C

2.2 Designing Rock Support

The Q-system has frequently been used to design underground excavations in the Oslo region since its origin in 1974. An important feature of the Q-system is that it describes the rock mass quality in terms of numbers and this enables the user to design support for an underground excavation dependent on the rock mass quality, dimensions and type of excavation.

Rock mass quality is governed by several different parameters. The most important ones significant to stability are represented in the Q-system and the Q-value is expressed by the formula:

$$Q \; ? \; \frac{RQD}{J_n} \; ? \; \frac{J_n}{J_a} \; ? \; \frac{J_w}{SRF}$$

where the block size is given as

$$\frac{RQD}{J_n} \; ? \; \frac{\text{degree of jointing}}{\text{number of joint sets}}$$

the inter-block shear strength is given as

$$\frac{J_r}{J_a} \; ? \; \frac{\text{joint roughness}}{\text{joint alteration or filling}}$$

and the active stress is given as

$$\frac{J_w}{SRF} \; ? \; \frac{\text{joint water pressure or leakage}}{\text{rock stress conditions}}$$

Each parameter is rated according to a set of tables. Q-value ranges from 0.001 for exceptionally poor quality squeezing ground up to 1,000 for "exceptionally good" quality rock which is practically unjointed.

The rock mass classification is associated with support; recommendations originally based on 212 case records. The support recommendations were updated in 1993 when the base was expanded by 1,050 new cases, mainly from road tunnels constructed during the last 10 years. The new cases include widely distributed rock mass qualities between Q-values of 0.003 (exceptionally poor) to 200 (extremely good). The updated and simplified diagram for the design of support is shown as Figure

12.1. Bolt length for systematic bolting, given on the right hand side of the diagram in Figure 12.1, is based on the equation:

$$L ? 1{,}4 ? 0{,}184 ? B$$

where L = bolt length and B = the span or wall height. The Excavation Support Ratio, ESR, on the left hand side of the diagram is a factor which takes the purpose of the excavation into account. For example, a road tunnel would rate an ESR of 1 while a hydropower water tunnel would have an ESR of 1.6.

2.3 Examples of rock support in different projects

Several support methods were used in the Oslo tunnel, two parallel highway tunnels with a cross section of 88-185 m^2 under the central part of Oslo. Typical support was S_{fr} and bolting as shown in Figure 12.2.

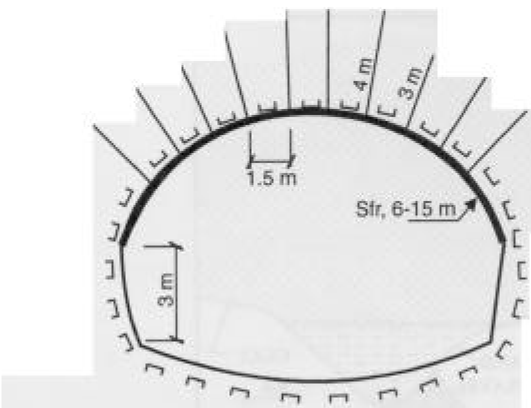

Figure 12.2 Typical support in sedimentary rock.

Table 12.1 shows the use of S_{fr} and bolting and the Q-value distribution in different rock types for the Oslo tunnel. The same information for the Granfoss tunnel (cross section 62-120 m^2) and Nordby tunnel (cross section 61 m^2) is also shown in Table 12.1. An indication of the effectiveness of S_{fr} and bolting is shown in Figure 12.3. Even though the results from plotting the rate of drive against Q are scattered, they indicate that the drive is not considerably reduced until Q is 1 or lower. In this project only rock bolt spacing and shotcrete thickness were varied to follow the variation in rock mass quality.

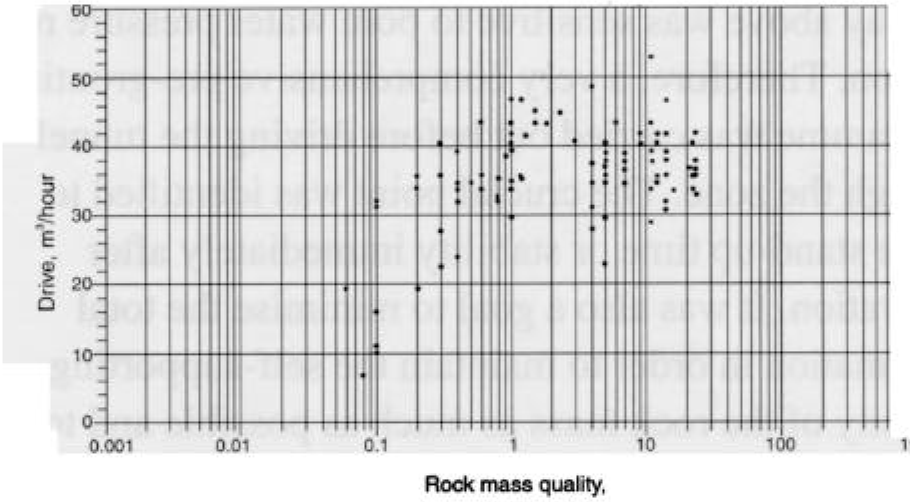

Fig. 12.3 Correlation between Q and rate of drive in the Nordby tunnel

Rock	*Except. poor [%]*	*Extrem. poor [%]*	*Very poor [%]*	*Poor [%]*	*Fair [%]*	*Good [%]*	*Very good [%]*	*Bolts No./m*	S_{fr} m^3/m
O-s	2	5	21	45	22	5	0	6,4	3,1
O-g	0	2	6	21	39	30	2	2,3	1,1
O-m	0	0	0	39	57	3	1	3,5	1,3
G-s	0	1	15	59	25	0	0	7,2	2,1
N-g	0	2	16	13	35	34	0	4,7	0,8

O = Oslo tunnel, N = Nordby tunnel, G = Granfoss tunnel,
s = clay shale, alum shale and nodular limestone with sills and dykes, g = gneiss and m = meanitt with layers of alum shale.

Table 12.1 Q-value distribution and average use of Sfr and bolting per tunnel meter.

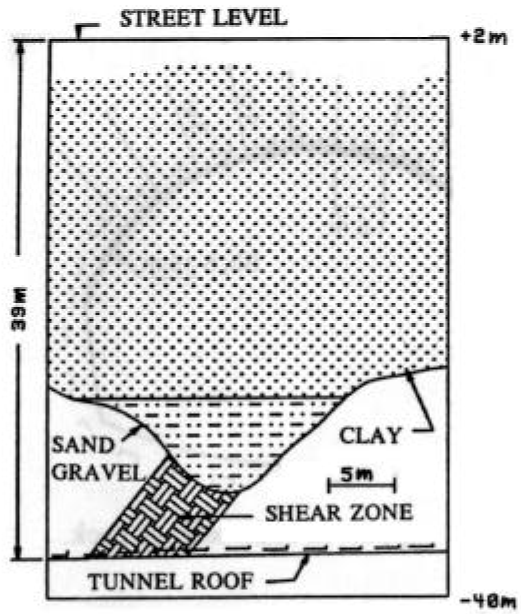

Fig. 12.4 Vertical sketch through the fault zone.

A support method based on shotcrete and bolting was also used in the Oslo tunnel when crossing the main fault zone at the lowest point of the tunnel. Figure 12.4 gives a schematic view of the geological conditions. The total length of this tunnel section was 40 m and the cross section was 110 m^2. Rock mass quality was "extremely poor", Q = 0.01, for 25 m of the section, "poor" and "good" for the remaining 15 m. The crushed alum shale which constituted the central part of the fault zone was sensitive to water and the clay above was sensitive to pore water pressure reduction. Therefore, a very comprehensive pre-grouting programme was carried out before driving the tunnel through the zone. The crucial point was identified to be the stand-up time or stability immediately after excavation. It was also a goal to minimize the total deformation in order to maintain the self -supporting capacity of the rock mass as much as possible and to avoid water leakage. Extensive use of spiting was considered essential to achieve these goals as well as immediate shotcreting after excavation. Reaction curves for ground and support, exclusive use of RRS, were calculated and are shown in Figure 12.5.

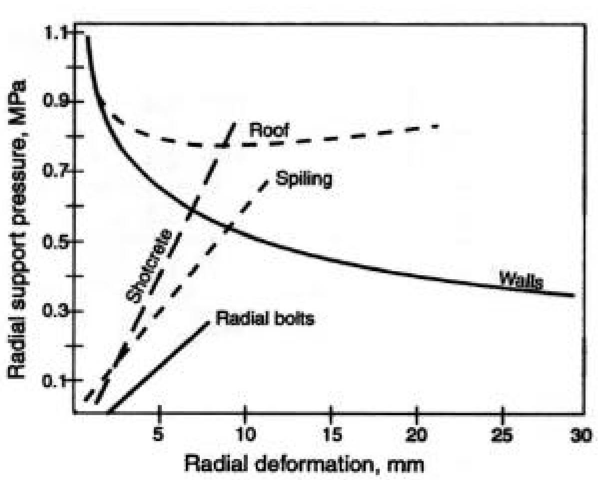

Fig. 12. 5 Reaction curves for ground and support.

Maximum support pressure in the roof given in MPa was estimated from the formula:

$$P_{roof} ? \frac{2J_n^{\frac{1}{2}} ? Q^{?\frac{1}{3}}}{30J_r}$$

The resulting design is illustrated in Figure 12.6:

- o The cross section was divided into 4 parts.
- o Part 1 was excavated through the whole section, then part 2 and 3, and finally part 4. Excavation lengths were 2-3 m. Spiting (c/c=0.3-0.6 m), radial bolts (c/c=0.9-1.35 m) and 20 cm of fibre reinforced shotcrete were applied after excavation of each step. Reinforced ribs of shotcrete (c/c= 1-2 m) were installed consecutively during excavation of parts 2 and 3, and were prolonged down the tunnel walls during excavation of part 4.
- o The total number of RRS was determined by the progress of radial deformation.
- o Radial deformation stopped at 18 mm.

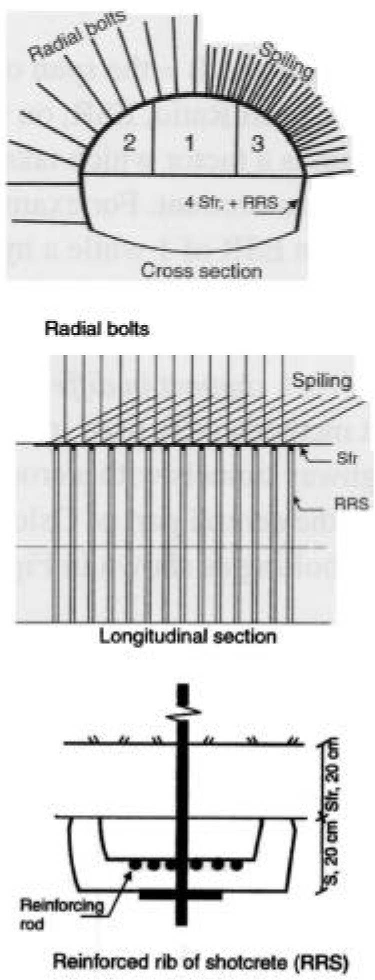

Fig. 12. 6 Excavation and support in "extremely poor" rock.

Ground freezing as temporary support has successfully been used in some critical cases in Oslo. The freezing operation can be carried out both from inside an excavation and from street level depending on which is appropriate. In the twin tunnel crossing the main fault zone mentioned above, the freezing operation was executed from one tunnel face, and the tunnel was driven from the opposite face. The placement of the freezing tubes is shown on Figure 12.7. The thickness of the frozen zone was designed to be at least 2 m with a temperature lower than -15°C. In terms of rock support, this is roughly equivalent to 20 cm of shotcrete.

Immediately after each blast (3 m), the roof and the walls were lined with 10 cm of shotcreted for temporary support. The shotcrete on the face was 5 cm thick. The face was also bolted in the poorest part of the weakness zone. The permanent support, which was applied as soon as possible after the temporary support, was a cast fibre reinforced concrete arch 65 cm thick.

Tunnel Boring Machines (TBM's) have been used in the Oslo region to construct sewer tunnels. The support needed in the TBM tunnels is based on analyses of 14 km of tunnel with a diameter of 3.5 m, and can be summarized as follows:

- Q larger than 10: Generally no support. Spot bolting and some shotcrete used for support of small wedges.
- Q in the range 5-10: Usually no support. Only rock bolts were used when support was needed.
- Q in the range 1-5: For the most part shotcrete was used. In a few cases shotcrete was combined with bolts.
- Q in the range 0.5-1: For the most part shotcrete combined with bolts was used.
- Q lower than 0.5: Normal support was mesh reinforced shotcrete combined with bolts. In some wider weakness zones, steel rod reinforced ribs of shotcrete were used with a spacing of 1.0 m.

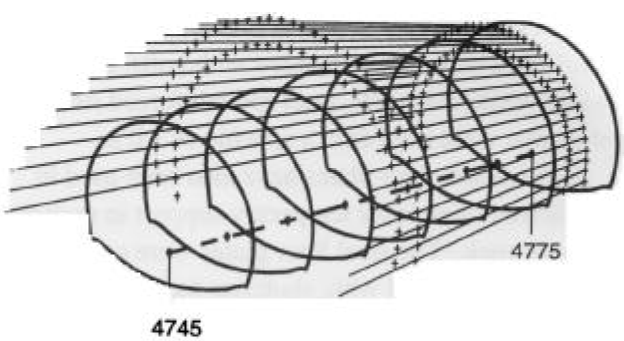

Fig. 12.7 Perspective drawing of the freezing tubes.

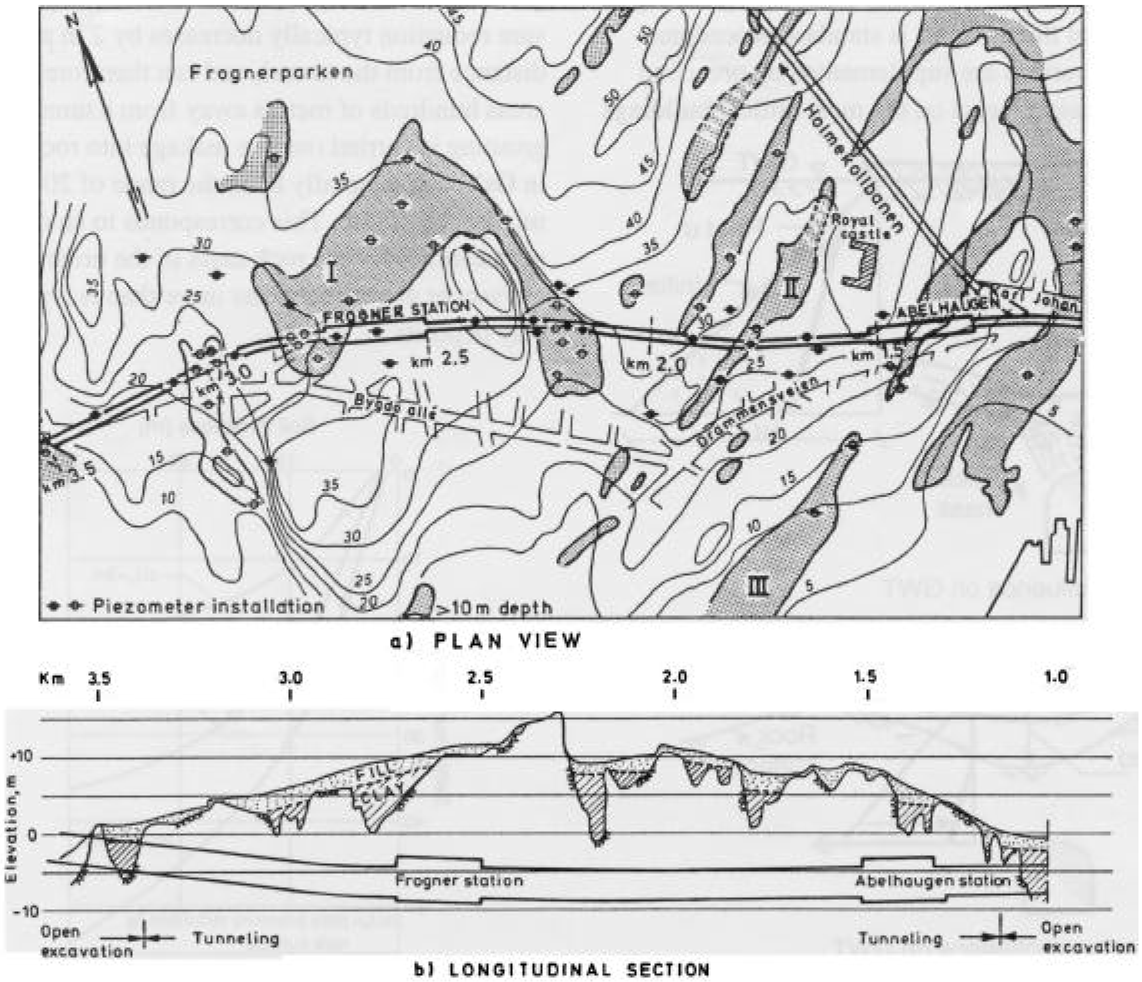

Fig. 12. 8 Layout of a railway tunnel.

Welded steel liner plates have also been used in TBM driven tunnels for temporary support in sections with poor rock. The steel liner plates were combined with, or interchanged with, shotcrete based support to provide permanent support.

3 GROUNDWATER CONTROL

The Holmenkollen subway tunnel, which was completed in 1916, caused surface settlements of up to 35 cm in clay-filled depressions within 200-400 m from the tunnel due to groundwater leakage. Severe damage was caused to many buildings in the area, especially those which were founded on the steep edges of clay-filled depressions. A tunnel driven under clay-filled depressions is illustrated in Figure 12.8. Observe the piezometer installations along the tunnel. A comprehensive programme for surveillance of pore water pressure, starting early enough to register the natural fluctuations, is standard procedure. These observations are supplemented by precision levelling measurements on the most critical buildings.

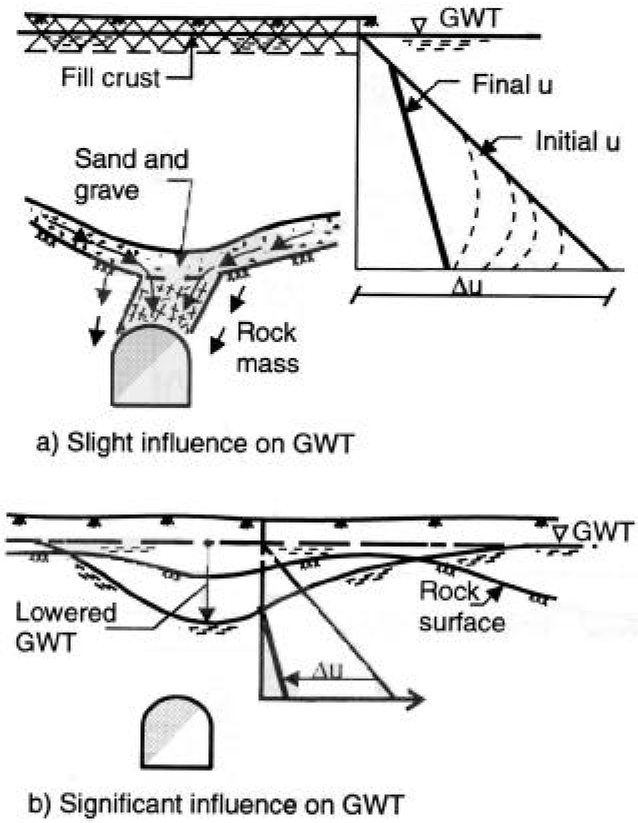

Fig. 12. 9 Impact on pore pressure and groundwater table (GWT) caused by tunneling.

Experience from rock tunnels constructed in Oslo over the past 30 years or so, shows that it takes a very small leakage into a rock tunnel to decrease the pore pressure at the clay-rock interface and initiate a consolidation process in the clay. The mechanism is illustrated in Figure 12.9.

Leakage, q (l/min/100 m)	Δu_R *(m)*
≤ 1	0
1 - 3	0 - 2
3 - 5	2 - 4
5 - 10	4 - 6
10 - 20	6 - 8
20 - 40	8 - 10
> 40	> 10

Table 12.2 Correlation between leakage and pore pressure reduction.

The pore pressure reduction, ?U_R, at the clay-rock interface straight above a rock tunnel may typically be as shown in Table 12.2 in relation to the groundwater leakage into the tunnel. The pore pressure reduction typically decreases by 2 m per 100 m distance from the tunnel, and can therefore influence areas hundreds of meters away from a tunnel. If no grouting is carried out, the leakage into rock tunnels in Oslo will generally be in the range of 20-40 liters/ minute per 100 m. This corresponds to an overall permeability for the rock mass of the order 10-5 cm/sec, or about 100 times more than is typical for the clay deposits.

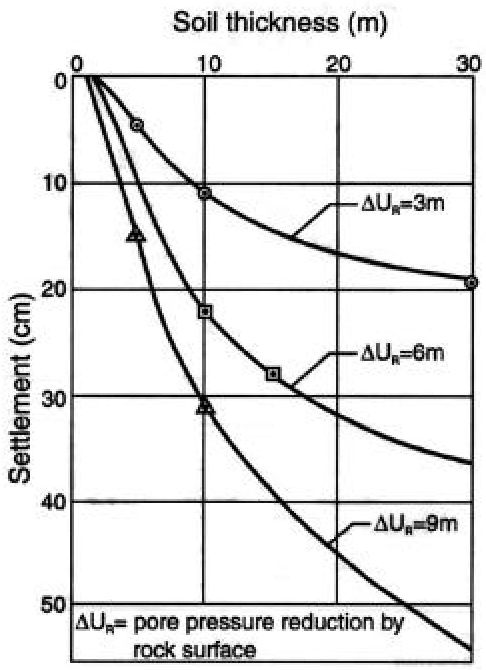

Fig. 12.10 Correlation between settlement, soil thickness and ?U_R for a given soil.

Figure 12.10 shows an example of total consolidation settlement as a function of soil thickness for a given pore pressure reduction at bedrock level. An example of how settlement develops with time is shown in Figure 12.11.

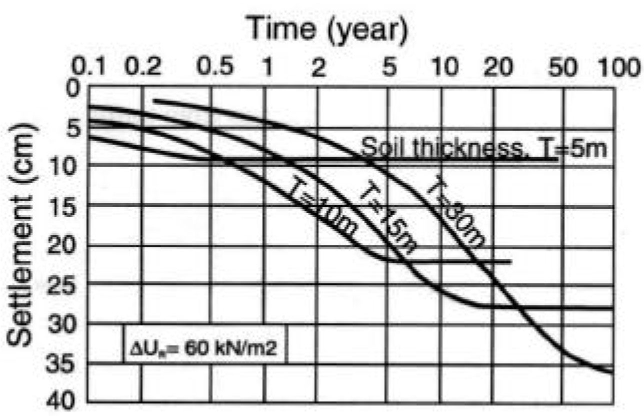

Fig. 12.11 Correlation between settlement, time and soil thickness for a given ?U_R.

As only a few centimeters of settlement are acceptable in order to avoid damage to buildings, this means that the acceptable leakage can be as low as 1-2 litres/minute per 100 m tunnel in the most sensitive areas.

The basic method for controlling the leakage of groundwater into tunnels and caverns is pre-grouting ahead of the face as shown schematically in Figure 12.12. Pre-grouting serves as temporary waterproofing and often also as permanent waterproofing. Pre-grouting is sometimes supplemented by post-grouting if the maximum allowed water leakage is exceeded. Experience shows that fairly good results from post-grouting can only be achieved in pre-grouted areas. Post-grouting is no alternative to pre-grouting. Generally results from grouting have gradually improved over the past years. The most significant improvement came when one started using a relatively high pump pressure, up to 3.5 MPa. The use of high pressure grouting has also made it possible for an extended use of both standard cement and fine grained micro- cements. These are far more economical than chemical grouts which were more in use earlier. Recent experience from road tunnels shows that, based on pre-grouting with cement, leakages can be reduced to about 2-5 liters/minute per 100 m on average for a tunnel with a cross section of 60-100 m^2. As discussed earlier, this will not be satisfactory in all cases in terms of the potential for settlements.

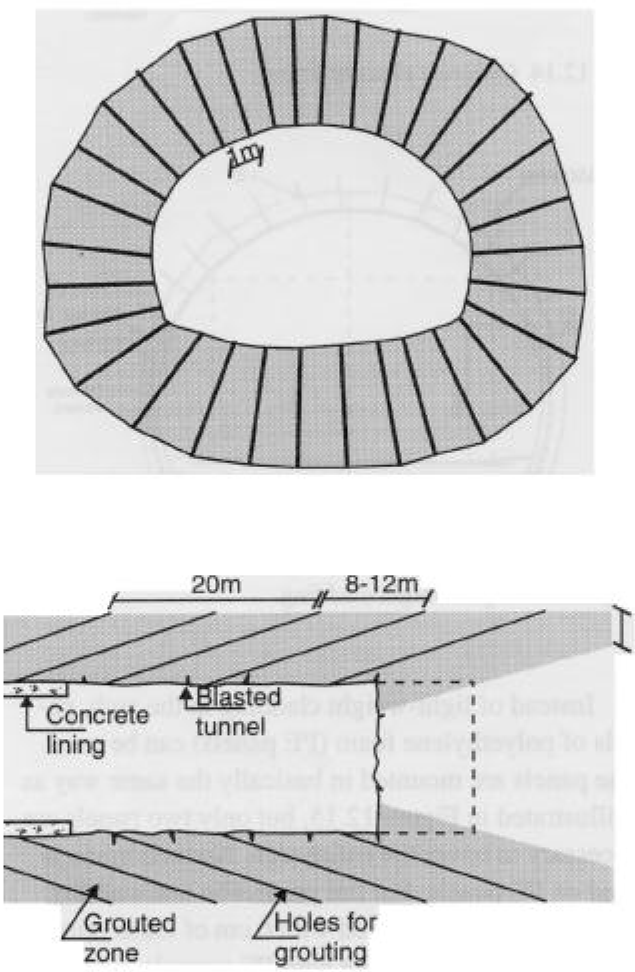

Fig. 12.12 Scheme for pre-grouting and lining.

Artificial infiltration of water, as illustrated in Figure 12.13, is frequently used to temporarily control pore pressure during the construction phase. The groundwater infiltration holes are generally drilled deep into the bedrock to intercept the most significant water bearing zones under clay-filled depressions. One tries to take advantage of the natural distribution system in the bedrock, represented by permeable faults, igneous intrusions, etc., to re-saturate larger areas. This has been quite successful, and in recent years, artificial infiltration has also been used as a permanent installation to stop subsidence around older underground structures where grouting has not given the desired result.

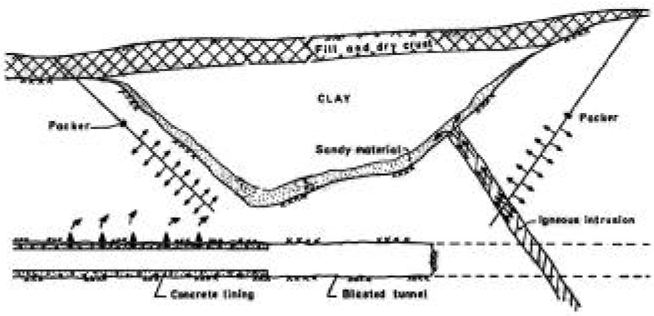

Fig. 12. 13 Artificial infiltration of water.

Cast concrete lining has been necessary in the most sensitive areas in order to achieve an acceptable level of leakage. The methods used for waterproofing a cast concrete lining are:

- Use of reinforced lining which is systematically grouted at the interface rock-concrete. Reinforcement of the lining allows the use of high pressure grouting, which is required to achieve good results. This type of lining worked very well in a railway tunnel with a 22 m span station, reducing leakage to about liter/minute per 100 m of tunnel.
- Use of unreinforced lining which is waterproofed by panels of bentonite, notch band and injection of notches and fissures. This has been fairly successful, however, since concrete shrinks during cold periods in the winter, there are problems with dilatation of existing cracks which causes new leakages.
- Use of a PVC membrane and a drainage sheet installed on a smooth surface of shotcrete before the lining is moulded. This method was used with success in a road tunnel which was completed a few years ago. In the late 1960s, a similar method using an asphalt membrane in an underground railway station was a complete failure. The failure was probably caused by a combination of complex geometry, difficult working conditions, inadequate planning, poor workmanship and insufficient quality control.

4 TUNNEL CLADDING

In road tunnels it is important to protect traffic against dripping water and, more important, from the formation of ice that can fall from the roof or build up on the road surface. To do this and simultaneously create a pleasant environment for users, different types of element tunnel cladding is used. Common to all types of cladding is that they are insulated. Depending on the dimensioning of the insulation, a cladding might have to be designed to bear some ice-load.

Figure 12.14 shows a sandwich element lining with the insulation (extruded polystyrene) placed between two layers of concrete. The waterproofing consists of a membrane. Experience with concrete element cladding is generally good and more of it will probably be seen in the years to come.

Figure 12.15 shows a so-called light weight cladding in the arch, i.e., cladding built up of panels of 1 mm thick sheets of steel and insulation to the required thickness. Each panel is 3 m x 1 m (width x height) and is held in place by fibreglass arches placed each third meter. The waterproofing is a polyethylene foil and packing at the back of the panels. The panels are easy to replace in the case of damage. Experience with light-weight cladding indicates that dimensioning for pressure and suction forces from traffic must be taken very seriously.

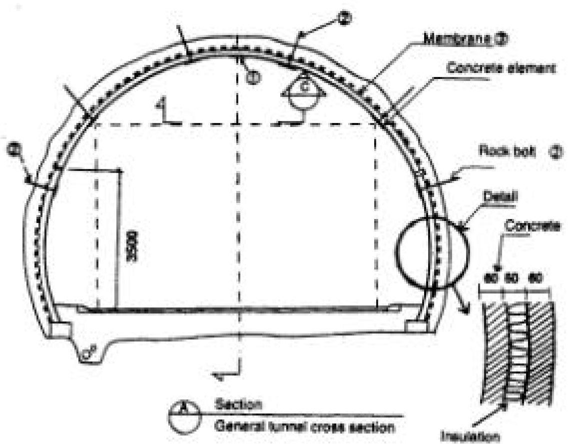

Fig. 12.14 Concrete element lining.

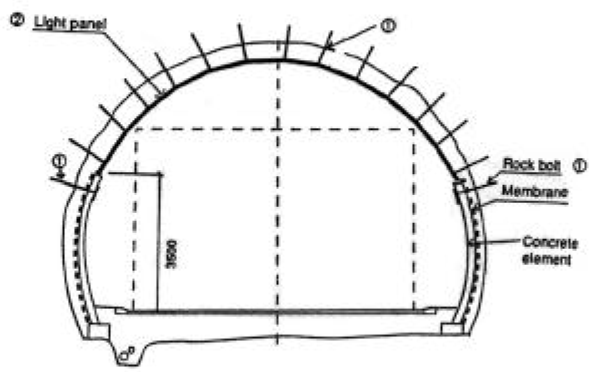

Fig. 12.15 Light-weight cladding.

Instead of light-weight cladding in the arch, panels of polyethylene foam (PE panels) can be used. The panels are mounted in basically the same way as is illustrated in Figure 12.15, but only two panels are necessary to cover the entire arch. No membrane is used on PE panels. For fire protection and stability, the PE panels are covered with 6 cm of mesh reinforced shotcrete. Experience with PE panels is generally good, however, it is difficult to achieve complete waterproofing.

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13 BLAST-INDUCED VIBRATIONS IN BUILT-UP AREAS -NORWEGIAN STANDARD NS 8141

Knut Tanbergmoen
Dyno Nobel

ABSTRACT: Norway has considerable experience of blasting in mountainous terrain. Previously, most blasting was required in conjunction with the development of hydroelectric power. Today, however, most major building projects are seen in conjunction with improvements to the national infrastructure, and, in consequence, blasting is almost always carried out in and around the major towns and cities. Economic considerations have led to considerable improvements in blasting techniques. Today, blasting operations require better follow-up routines than ever before, not only because of the relative sophistication of modem techniques but also because much more blasting is now being carried out in densely populated and built-up areas. The adoption of larger diameter shot holes and longer rounds has also led to the need for increased control of blasting procedures.

1 INTRODUCTION

Given the large numbers of building projects going on in built-up areas in Norway at anyone time, many of which require large volumes of rock blasted from the mountains, damage directly attributable to the vibrations set up by an explosion are very rarely seen. In almost all cases, blasting is subject to a set of threshold values, and the explosive weight is thus set in proportion to measurable vibrations. This leads us to the immediate conclusion that vibrations are not a problem and that modem blasting practice is of sufficiently high standard.

2 CAUTIOUS BLASTING, CONTROLLED BLASTING

2.1 Special considerations

In all cautious blasting, particular attention must be paid to the effect of the blast on the surroundings. In other words, to preserve the local environment, it is essential to maintain control of ground vibrations, spatter and airborne shockwaves. Drilling and charging must always be carefully planned, calculated and followed up. Individual blasting operations must also be documented to enable them to be reconstructed.

However, those working with blasting on a day-to-day basis know that this is not always as easy as it seems - especially those who have been called in after the event to check whether the blasting operation has caused any damage. Where is the blasting record? What has happened to the vibration measurements? Where can we find the inspector's report. Quite often all of these are missing. Whose fault is it? Often it's all too easy to put the blame on the subcontractor or the head of the blasting crew, but just as often it appears that the future proprietor and, more particularly, his consultant should accept a large portion of the blame for failing to ensure that all was in order in the first place.

3 DETERMINATION OF THRESHOLD VALUES

Norwegian Standard (NS) no.8141 is applied to the determination of threshold values for vibrations in buildings.

NS 8141 deals with the calculation of threshold values for blast-induced vibrations in buildings and structures, and these are used for the determination of the threshold values to be applied to any particular blasting operation. They are set such as to obviate damage to nearby buildings/constructions. The standard is applicable to all types of blasting operations in a wide variety of environments, tunnels and mines to crushing plants and building sites. The threshold values are set with a view to building technology rather than human beings or any vibration-sensitive plant or machinery which may be housed inside the building.

The threshold values are set as the vertical peak values of the vibration velocity.

3.1 What is the peak value ?

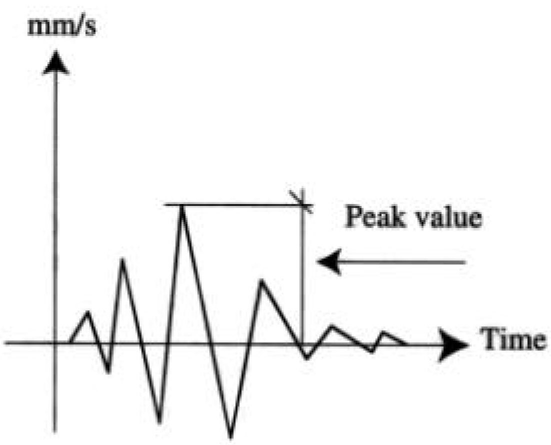


Fig. 13.1 What is the peak value?

3.2 Threshold values

The threshold values of NS 8141 are based on extensive experience of the correlation between vertical vibration velocity and the damage occasioned to buildings erected on a variety of foundation types.

The threshold value (v) indicates the vibrations set up by the blast at the level of the foundation

$$v = v_o \times F_c \times F_d \times F_t$$

where

v_o is the uncorrected peak value of the vertical vibration velocity expressed in millimeters per second. The value of vo depends on local ground conditions.

F_c is the construction factor.

F_d is the distance factor indicating the distance between the point of the explosion and the point of measurement.

F_t is the time factor indicating the length of the period during which the building is exposed to the vibrations.

If blasting is to be carried out such that the vibrations may give rise to resonance in a building, as, for example, by the use of electronic firing systems, the threshold values to be applied to the operation should be determined with special care.

Local ground conditions	**Uncorrected peak value of vertical vibration velocity, V_O**
Very wet ground/wet clay	Established separately
Loose stratified moraine, sand, gravel, clay (seismic velocity less than 2,000 m/s)	18
Solid stratified moraine, shale, soft limestone and equivalent (seismic velocity 2,000-4,000 m/s)	35
Granite, gneiss, hard limestone, dolerite (diabase) and equivalent (seismic velocity over 4,000 m/s)	70

3.3 Uncorrected peak value of vertical vibration velocity, Vo

The uncorrected peak value of vertical vibration velocity V_O depends on local ground conditions and is set on the basis of the following table.

3.4 Construction factor; Fc

The construction factor depends on the type and design of the building and of the material of which it is built. This factor is given by the formula

$$F_C = F_b \ \text{x} \ Fm$$

where

F_b is the building factor, and

F_m is the factor dependent on the type of material used in the construction of the building

3.5 Building factor; F_b

For the present purposes, buildings are divided into the five categories indicated in the following table:

Category	Type of building	Building factor, F_b
1	Heavy-duty edifices, e.g. bridges, harbor quays, defences	1,70
2	Industrial and office buildings	1,20
3	Ordinary dwelling houses	1,00
4	Especially vulnerable buildings, e.g. museums, buildings with high or of particularly wide span	0,65
5	Historical monuments and ruins in fragile condition	0,50

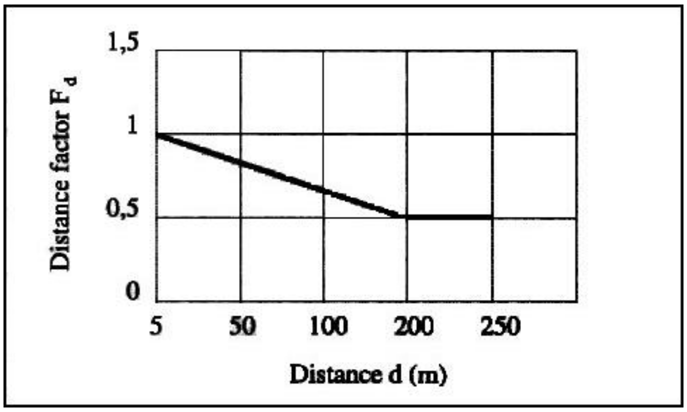
3.6 Material facto1; F_m

The materials of which a building may be comprised are divided into three categories indicated in the table below:

Category	Principal materials	Material factor, F_m
1	Reinforced concrete, steel and wood	1,20
2	Non-reinforced concrete, brick, cavity blocks, masonry, lightweight concrete and the like	1,00
3	Porous concrete	0,75

3.7 Distancefacto1; F_d

The distance factor F_d is the shortest distance d between the point of the explosion and the building. F_d is rectilinear between 5 and 200 meters. At distances of over 200 meters the distance factor is 0.5. Distances of less than 5 meters should be assessed separately.



3.8 Timefacto1; F_t

The time factor F_t depends on the length of the period during which blasting is to continue. The chief purpose of this factor is to distinguish between permanent and temporary operation.

Duration of construction	**Time factor F_t**
Less than 12 months	1,00
Over 12 months	0,75

3.9 Other requirements of the Standard

3.9.1 Requirements for measuring equipment

This is one of the more tricky aspects of the Standard, for it specifies both how the instrument is to monitor the vibrations and the frequencies which are to be measured. Here much can go wrong unless everything is carefully check throughout the operation.

The discrepancy is greatest at short distances, i.e. from 0 to 20 meters. At distances greater than 20 meters , the discrepancy becomes rapidly smaller.

To be meaningful in the present context, a discrepancy must be large, e.g. between 100% and 150% off the measurement that would be given by an instrument calibrated in accordance with NS 8141.

3.9.2 Measurement and assessment

Perhaps as important as accuracy of measurement is the correct assembly and siting of the instrument itself. Specifications are provided as to how this should be done.

Under special circumstances it may sometimes be desirable to extend the range of the measurements carried out. Provisions for extended measurement are contained in the Standard.

3.9.3 Record of measurements

The last section of the Standard lists the information which must be included in the records kept of all measurements carried out. (Note that this information must be provided!)

The report must:

- Specify the precise location of the instruments, the date, and the name of the person who carried out the measurements.
- Specify the type of instruments used, including the date when the instruments were last calibrated.
- Specify the sensor assembly, the background noise and the trigger level selected. Time checked.
- Indicate the distance between point of measurement and blasting site.
- Describe the local foundation conditions, design and construction of the building, geological condition, and any other conditions that may be of significance to measurement.
- Indicate the values measured at specific points of measurement.
- Give an assessment of the results obtained as compared with the threshold values calculated for the job.

3.9.4 Calibration of instruments

NS 8141 also specifies the calibration routines to be applied to both instruments and sensors. Calibration is, of course, a sensitive issue among those who own or rent out vibration measuring equipment. For those with access to UVS equipment the instrument itself is usually enough, since it can easily be checked out in the field. With geophones, on the other hand, practically an entire laboratory must be called in. Both types must be checked at least once every other year.

4 THE MEASURING EQUIPMENT OF TOMORROW

Until today, most changes made in this field took place in the actual instruments. However, with information technology developing apace, it is natural that representatives of the industry keep alert to any advances that are made. In our own particular field, for example, we have seen how a UVS 1500 instrument can be remote-controlled and programmed to transmit data directly to the PC by which it is controlled. We have recently developed software for the remote-control of a UVS 1500 for use in Romeriksporten, the new railway tunnel to Oslo's Gardermoen airport. Here, all readings will be made via a PC and the instruments will be linked to mobile phones out in the field. The illustration below shows the first version of the program.

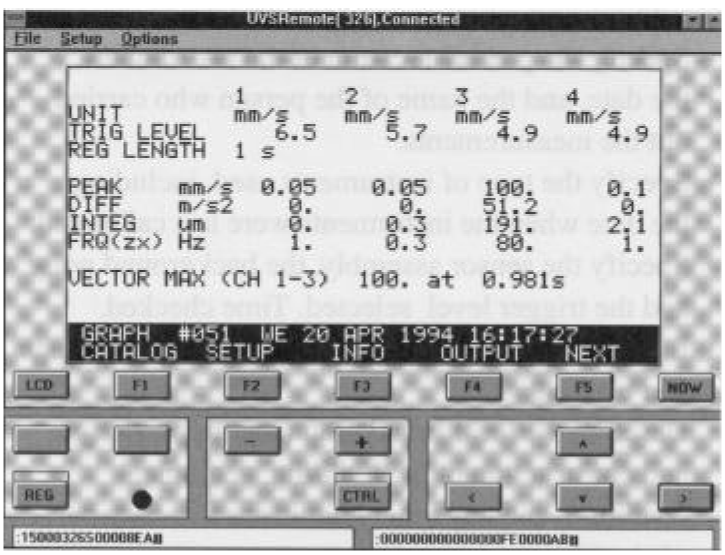


Fig. 13.2 The instrument has been rung up via a mobile telephone. Displayed is the UVS 1500 directory.

We can also see how an instrument could call up a user and provide information on the progress of events: the party renting the instrument would also be provided with a text-based pager, or else data could be transmitted to a GSM mobile phone via the SMS (Short Message System) service.

It will also be possible to transmit data to our local network at the Dyno office at Gullaug. Here, data will be stored for later retrieval by customers from their personal electronic mailboxes.

Out in the field, wireless transmission will enable signals to be sent from the sensors direct to the instrument. This technique has already proved itself in practice and is likely to become more common during the years to come. The present range is about 200 meters but will surely grow larger.

Until today it has been common to measure vibration velocity and a number of supplementary parameters as well. However, the trend in instruments has tended to reflect that of electronics in general, which, in our case, has manifested itself in the utilization of more advanced instruments by ordinary users. In an effort to control the blast and adjust the round as accurately as possible, the tendency today is to choose instruments that are becoming increasingly sophisticated.

In this context, it should not be forgotten that the switch to the Nonel firing system has in some degree contributed to the need for accurate measurement.

Vibration measurement should enable reconstruction of a chain of events after blasting. However, conventional measuring instruments are far from being able to provide us with a clear picture of what has happened; reality is more complex than that. What we can record is a gross oversimplification of an exceedingly complex process - and this should be remembered whenever we determine a set of threshold values and, above all, set about assessing and interpreting the measurements after the blast.

14 PERCEIVED DANGER AND DESIGN OF UNDERGROUND FACILITIES FOR PUBLIC USE [(1)]

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ABSTRACT: A common problem with underground facilities for public use is the feeling of danger and entrapment they generate. This article discusses these reactions in light of modern behavioral theory explaining the development of phobic reactions. According to this view the majority of phobic reactions are a result of two factors: A biological preparedness to avoid certain stimuli and a learning episode strengthening the avoidance behavior. A third factor, degree of perceived control, has important modulating effects on phobic reactions. A biological preparedness to avoid situations characterized by a limited possibility for escape can explain some aspects of the seemingly universal negative associations that underground space evokes. Some of the consequences this theory have for design of underground facilities for public use are discussed.

1 INTRODUCTION

The utilization of underground space for public use is increasing both in terms of excavated space and in terms of the number of activities performed in the buildings. Car parks, sporting and recreational amenities, libraries, shopping centers, restaurants, congregation centers, theatres and concert halls are some examples of public facilities now being built underground.

A public facility is differentiated from a work place by: (I) the short time each person on average will spend in the building, and by (2) the irregular and relatively infrequent visits each person will pay the facility. This means that psychological aspects of being underground will differ from those found in an underground workplace. While isolation and monotony are two common problems reported by employees in underground workplaces (Sommer 1974; Hollon, Kendall, Norsted & Watson 1980; Wada & Sagukawa 1990), long term effects of being underground will be of little relevance for people using public facilities.

The limited possibility for familiarization and habituation in public facilities means that people's experience of being underground more likely will be related to their preconceptions of underground space. An understanding of people' s image of underground space will therefore be useful in predicting the most likely psychological responses to underground facilities for public use.

The utilization of underground space for public use generally evokes negative associations. Although the content of the associations varies historically and across cultures, a negative image of underground space seems to be quite widespread and consistent (Carmody 1992). This negative image has been described and explained in different ways (Carmody 1992; Fairhurst 1976; Lesser 1987; Paulus 1976; Williams 1990), but a feeling of confinement and danger seem to be a general and important facet of the imagery evoked by underground space.

Attempts to explain the negative associations connected to underground space has so far lacked a clear theoretical foundation, and the explanations have not been very useful in determining what specific aspects of underground space produce the negative associations. This article discusses the negative image of the underground in light of modem behavioral theory explaining the development of phobic reactions. It will be argued that this theory can aid a more detailed understanding of what it is about underground space that evokes negative associations, and that this understanding may be helpful in developing design solutions that ameliorate some of the negative thoughts and emotions generated by underground space.

2 FEARS, PHOBIAS AND UNDERGROUND SPACE

A phobic reaction is commonly defined as a strong experience of fear in a situation where there is no obvious external danger. The phobic reaction is accompanied with a strong urge to avoid the non-dangerous feared situation even though the person realizes that such behavior is foolish or irrational. Phobic reactions are among the most common forms of maladaptive behaviors in people, and may in many cases cripple the person's social and professional life (Morris 1991).

According to modern behavioral theory the development of phobias is a result of two factors: (1) A biological preparedness to fear and avoid certain stimuli, and (2) a learning episode strengthening the avoidance behavior. A third factor, degree of perceived control, is important in modulating the likelihood and intensity of phobic reactions.

The notion of biological preparedness was first proposed by Seligman (1971) to explain that only some stimuli generate phobic reactions. According to Seligman people are prepared to develop certain fears based on the biological and evolutionary significance of certain stimuli and situations that are tied to their struggle for survival. Stimuli that frequently provoke phobic reactions have once in humankind's history indicated real danger, and natural selection has favored those among our ancestors who have avoided them. Although the behavior is no longer adaptive, these stimuli can still elicit fear and avoidance in people because of physiological predispositions. The biological preparedness notion has later attained experimental support (Öhman, Erixon & Løfberg 1975; Hugdahl, Fredrikson & Öhman 1977).

Biological preparedness is not, however, a sufficient etiological factor in the development of phobias. If it were we would all suffer from a wide range of phobias. The second necessary factor -a learning episode -is illustrated in Figure 14.1.

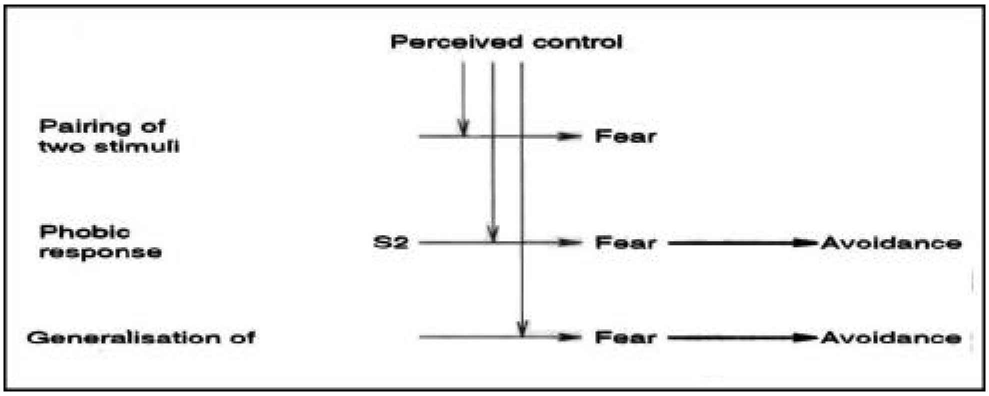

Fig. 14.1 The learning of phobic reactions.
(S 1 = Fear-provoking stimulus, S2 = Initially neutral stimulus, s = stimulus with some feature in common with S2.)

A fear provoking situation of some sort (for instance being trapped between two floors) is paired with an initially neutral stimulus ,(for instance being in an elevator). This leads to a fear response. After the learning episode the initially neutral stimulus will, when presented alone, generate a response of the same type as the one emitted during learning. This results in an "irrational" fear of the initially neutral stimulus. The fear response will easily generalize to stimuli that have some feature in common with the new fear provoking stimulus (i.e. the fear of riding elevators will become a fear of enclosed spaces).

Approaching the new fear provoking stimulus will result in intense anxiety, and this anxiety can be reduced by avoidance behavior. Avoiding the fear provoking stimulus will thereby be rewarded and reinforced by a reduction in anxiety and discomfort. This has two important consequences: First, the strengthening of the avoidance behavior means that the person will invest progressively more resources in avoiding the feared stimulus. In extreme cases the avoidance behavior can become so intense that it dominates the person's life. Second, the avoidance behavior will prevent the person from experiencing that the fear arousing stimulus is not really dangerous, and this will in turn reduce the likelihood of recovery.

An important modifying factor in the development and sustainment of phobic reactions is the person's experience of control. Both the learning of phobic reactions and the degree to which the feared stimulus gives rise to anxiety, seem to depend on the level of perceived control in a situation (Beck, Emery & Greenberg 1985), and the degree of perceived control is in turn closely related to environmental cues (Brehm & Brehm 1981). For example, being trapped need not provoke fear if it is possible to call for help by pushing a button. Being in enclosed spaces does not necessarily generate anxiety in a claustrophobic (a person afraid of enclosed spaces) if the person has knowledge of possible escape routes. This means that in the face of stimuli people are biologically prepared to fear and avoid, the intensity of discomfort or fear will at least partly depend on other situational characteristics or stimuli that influence people's perception of control.

The following three aspects of the behavioral theory explaining the development of phobic reactions are particularly salient to an understanding of people's negative image of underground space:

1. The possibility for fully developed phobic reactions.
2. The notion of biological preparedness.
3. The level of perceived control.

Relatively few people will experience intense anxiety while being in an underground facility. People with developed phobic reactions will most likely avoid situations they are afraid of. Problems can arise, however, if a person suffering from a phobia is forced to expose himself to the fear provoking stimuli. If a claustrophobic has to drive through a tunnel on the way to work, this can be an excruciating experience. Phobic reactions, though limited in number, should therefore not be underestimated in maintaining a negative image of the underground. Indeed, fear and panic reactions in underground facilities may be an important part of the folklore connected to underground space.

The notion of biological preparedness gives some indications of what specific aspect of underground space people may try to avoid or be afraid of. By considering the most common phobic stimuli, in order to evaluate whether they are aspects of underground facilities, it may be possible to detect specific characteristics of underground space that people are predisposed to fear arid avoid. Enclosed spaces, entrapment, darkness, crowds and getting lost are some common phobic stimuli that can easily be associated with underground space. Therefore, the prevailing negative image of the underground can, at least partly, be understood as a response to certain aspects of underground space that people are biologically prepared to avoid and fear.

It is not difficult to imagine that perceived control can be reduced in an underground facility for public use. The number of escape routes is limited and the escape routes may be difficult to detect, wayfinding in general can be difficult, technological aspects of underground space may be unfamiliar, and people unaccustomed to underground space may experience a lack of confidence in their traditional coping behaviors because of the new and unfamiliar surroundings. Thus, a reduced level of perceived control can also contribute to the popular image of underground space as dangerous.

3 SOME CONSEQUENCES FOR DESIGN

The possibility for systematizing existing knowledge regarding underground design is perhaps the major advantage to be gained by using the behavioral theory of phobic reactions as a background for discussing design of underground facilities for public use. Two broad categories of design considerations emerge as particularly important.

First, stimuli that are known as common elicitors of phobic reactions should be given special attention when designing underground facilities. Which stimuli should be considered most important will depend on the type of facility being designed. For instance; crowding and wayfinding may be of relevance in designing a concert hall, while being alone may evoke fear in an underground car park.

Second, underground facilities should be designed in a way that enhances people's perception of control. This can be done via a design that conveys relevant information to the users, and gives the users a chance to act upon this information.

By applying results from environmental psychology in the design of underground facilities, it is possible to reduce the impact of potential fear arousing stimuli and enhance people's experience of control. People's perception of crowding, for example, has been related to the following architectural factors: Shape of room, ceiling height, placement of activities in the room, partitioning of room(s), seating arrangements, brightness, presence of visual distracters (e.g. pictures on walls) and visual access to escapes (e.g. doors) (Bell, Fisher, Braun & Green 1990, pp. 309-310).

Wayfinding can serve as another example. Ease of wayfinding is related to the degree of environmental differentiation (e.g. variations in form, size, colors, architectural style), visual access (the degree to which different parts of the environment can be seen from other parts), complexity of spatial layout (e.g. number of possible destinations and routes) and sign-posting systems (Gärling, Book & Lindberg 1986).

Detailed guidelines for design of underground facilities that seek to enhance people's well-being and comfort can be found elsewhere (Carmody & Sterling 1987,1991; Wise & Wise 1984; Ylinen 1988), and therefore will not be repeated here. It is noteworthy, however, that many of the design guidelines, in addition to factors of relevance to people working in underground installations, seem to be related to common phobic stimuli or level of perceived control. Lightning arrangements, wayfinding, fear of confinement, perceived spaciousness and crowding are some important issues that are addressed in the guidelines, and that are related to common phobic stimuli.

It should be noted that attempts to influence people's experience of danger and control raises important ethical questions. It is possible to give people a false sense of security by manipulating design. This can be achieved, for example, by simply placing an emergency exit sign on a door leading to a dead end. Design of underground facilities should therefore not increase people' s sense of security at all costs. The aim should be to give people a chance to assess realistically possible dangers and to act in accordance with their assessment.

4 CONCLUSION

The behavioral theory of phobias seems to explain some important aspects of the negative image people have of underground space. Future research should address whether other aspects of this image

-such as underground space as a second best or lower status place -are related to the potential fear-arousing features of underground space.

The stimuli that may provoke fear in people in an underground facility exist in many above ground buildings as well, but the reduced level of perceived control in an underground facility will reinforce the effect of these stimuli. Based on the behavioral theory of phobias it is possible to draw some conclusions regarding design of underground facilities for public use. Many of the specific design principles that can be deduced from the theory are already well known, but can be systematized and made theoretically understandable by applying modem behavioral theory.

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