UNDERGROUND CONSTRUCTIONS FOR THE NORWEGIAN OIL AND GAS INDUSTRY

NORWEGIAN TUNNELLING SOCIETY

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DISCLAIMER

UNDERGROUND CONSTRUCTIONS FOR THE NORWEGIAN OIL AND GAS INDUSTRY

The present publication, number 16 in the English language series from the Norwegian Tunnelling Society NFF, has – as always – the intention of sharing with our colleagues and friends internationally the latest news and experience gained in the use of the underground; this time with focus on Underground Constructions for the Norwegian Oil and Gas Industry.

The publication coincides with the celebration of the 40th anniversary of the Norwegian oil and gas industry. In 1958, a well recognised national institute declared that Norway could disregard any possibility of finding coal, oil or sulphur along the coast or in the North Sea. One year later the Netherlands discovered its vast Groningen gas field.

The first exploration on the Norwegian shelf took place 1966. Traces of hydrocarbons were observed. Then, the day before Christmas Eve 1969, the country became an oil and gas nation. "Ocean Viking" hit the Ekofisk field and the proud exploration masters declared The North Sea being an endless oil basin right up to the North Pole. That was an exaggeration, however approximately 50 fields are now in a production stage in the Norwegian sector. Today some 35 % of the national income derives from the oil and gas industry and large quantities of oil and gas are exported

The consequences have been manifold, giving NFF and its members new opportunities.

NFF expresses thanks to the authors and contributors of this publication. Without their efforts the distribution of Norwegian tunnelling experience would not have been possible.

Oslo, April 2007

Norwegian Tunnelling Society
International Committee

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Contents

Introduction

Eivind Grøv

During the last 40-50 years or so the concept of underground hydrocarbon storage has been implemented in Norway with great success. The utilisation of the underground has not been limited to the use for storage but also for such purposes as pipeline tunnels, shore approaches and other purposes too. In this publication a variety of different types of sub surface projects for the oil and gas industry will be presented. The readers are hopefully enjoying the details of the presentations picking up interesting aspects to be applied in your own projects.

No negative influence on the environment has been recorded during these years of operation. As will be described, governmental requirements are governing the design. This is now a proven concept and new storage caverns are being built in connection with Norwegian oil and gas terminals and processing plants. The concept evolved from the growing hydropower development in the years of industrial growth in the post war Norway. The tunnelling industry established robust and effective tunnelling techniques which are now being applied for underground hydrocarbon storage. The most specific aspects of this concept are related to unlined caverns and the implementation of artificial groundwater to confine the product, which both are well documented in this publication.

In modern societies there are growing concerns related to the safety and security of our infrastructure system. In addition surface space is becoming a scarce resource placing limitations on urban expansion. The environment needs to be protected and the aesthetics considered. Underground storage of oil and gas has showed an extremely good record in all these important aspects of the modern societies and is thus a popular method for such products.

We sincerely hope that this publication can be a useful tool for friends and colleagues in the tunnelling business in their endeavours towards an improved use of the underground. Norwegian engineers have, through half a century of application of the underground, gained solid experience in underground construction for the oil and gas industry. An experience basis which is also considered a valuable asset amongst the owners, the oil and gas companies and finally to the benefit of the consumers. Also in projects abroad this competence and experience have been utilised, in various continents and cultures around the world.

Enjoy the reading and contact the Norwegian Tunnelling Society for further information.

1. STORAGE OF OIL AND GAS IN ROCK CAVERNS: - HISTORY AND DEVELOPMENT

Svein Martin Haug
Einar Broch

LINED CAVERNS

First time petroleum products were stored underground in Norway was probably during World War II. On the west side of the harbour in Trondheim several caverns were excavated in the granitic rocks. In these caverns steel tanks of similar shape and size as the normal "on-the-ground" tanks were constructed. The reason for this underground solution was to protect the important product against bombing or other war hazards. The storage is still in operation.

During the years 1960-62, an underground oil storage was built in a hill side at Muruvik east of Trondheim. The owner of the storage was the Swedish Ministry of Defence. The intention was to have a safe storage that could easily be reached by railway in case of heavy ice or war time activities in the Botnian Sea. Indirectly it gave Sweden an access to the Atlantic Ocean. In this storage the caverns are lined with steel plates. The storage is now commercially operated by an oil company.

CAVERNS BELOW THE GROUND WATER TABLE

The first storages for petroleum products in rock caverns below the ground water table are found in Sweden, where two old mines were converted to storage of heavy fuel oil in the period 1947-1950. During the 50s and 60s a range of underground storages were built in Sweden, mostly for fuel oil, but gradually also for crude oil, lighter products and LPG (Liquefied Petroleum Gas). During the 60s, this method for oil storage also became popular in Finland. Today, more than 5 million m3 crude oil and oil products are stored in caverns at Neste's Porvoo refinery. Construction of caverns for storage of LPG under pressure started in France during the 60s. Most of these projects are placed in cretaceous or limestone and are excavated mechanically without drilling and blasting.

From the 70s, several countries in Europe and other parts of the world started using caverns for storage of oil and gas. The oil crisis in 1973 caused increased construction of oil reserves, and large projects were started in many countries. Storages of millions of m3 crude oil were built and filled in for example Korea, Japan and USA. The construction of large caverns for refined petroleum products was also started in Saudi Arabia shortly after. These caverns, however, were of the steel lined type due to extremely low ground water level in most places.

In Norway there is one reference to a fuel oil storage for LKAB in Narvik in the period 1954-55, but it is not known if this storage was ever completed. In the beginning of the 70's, Norway was at the start of the "oil age", and only a few oil storage caverns had been built. Norwegian planners and contractors did not have the necessary experience yet, and several of the first caverns in Norway were planned and constructed by Swedish companies. The first unlined cavern for oil storage planned by Norwegian consultants was Esso's fuel storage at Høvringen near Trondheim in 1975.

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CONTRACTING, ENGINEERING, PROJECT DEVELOPMENT & FINANCING

Veidekke is one of Scandinavias leading construction and property development companies. It is the largest in its field in Norway. Veidekke has an annual turnover of approximately USD 1.5 billion and 7,000 employees.

Veidekke's vision is to build a better future for people in all aspects of life. Its business concept is to create value through designing, building and managing projects in partnership with customers who inspire growth and development.

The Skarnsundet Bridge is a 1,010 m long cable stayed bridge with 12 spans. The main span is 530 m long and the A-towers are 152 m high. Veidekke was honoured by the National Association of Architects for this project.

International Experience

Since 1982 more than 60 companies have merged into Veidekke, giving it a wide range of expertise and resources. Veidekke has participated in the development and construction of a large number of infrastructure projects in Norway and abroad. In addition to the other Scandinavian countries, the company's international experience includes countries such as Iceland, Greenland, Germany, Italy, East-Africa, Thailand and China.

Extensive Expertise

Veidekke offers specialised know-how for the construction of

- Residential and non-residential buildings
- Roads and highways
- Conventional and subsea tunnels
- Bridges, railways and airports
- Harbours
- Industrial and oil & gas facilities
- Underground facilities

Excavation work in the Gjøvik Olympic Mountain Hall, which was ice-hockey arena during the 1994 Olympic Winter Games in Lillehammer, Norway. This is the world's largest public mountain hall.

A shotcrete robot working its way through the Norwegian mountains.

Veidekke had the contract for 4,000 m of the 11,000 m long road tunnel through Folgefonna in western Norway.

2. GOVERNING REGULATIONS VS DESIGN

Levi Karlsen

There are few requirements in the regulations regarding underground constructions. It is always assumed that best practice and experience are used during the design and the construction of the installations.

One important requirement is, however, set in ”Regulations Concerning Flammable Goods”, laid down by Directorate for Civil Protection and Emergency Planning June 26th 2002,

”§ 3-2. Storage in rock caverns”, states :

”Installations in rock caverns shall be secured in a safe way to avoid leakage from the installation.

Where the groundwater level forms the barrier against leakage of the stored material, the groundwater level must correspond to the vapour pressure of the stored material, plus an extra 20 meters water column as safeguard against irregularities in the rock ”
(Unauthorised translation)

This requirement has a major impact on the design, as it determines the depth to which the cavern has to be excavated.

The requirement also means that the groundwater has to be kept at the its original level, which again requires that the cavern must be equipped with water curtains with reliable water supply.

Normally, the water curtains are established before the actual excavation of the cavern itself.

The groundwater level must be closely monitored before and during the construction period and also during the entire storage lifetime. This has to be considered in the design phase.

For pressure equipment with a pressure greater than 0,5 bar overpressure such as piping, vessels, safety accessories and pressure accessories used in connection with the storage, “Directive 97/23/EC concerning pressure equipment” and /or “Regulations concerning flammable or pressurised goods” must be applied.

Also “Directive 94/9/EC concerning equipment and protective systems in potential explosive atmospheres (ATEX)” must be applied.

The same requirement also applies for refrigerated oil products like liquid propane etc.where an ice cap are formed in the rock around the cavern storage.

For pipeline tunnels, “Regulation concerning transport of petroleum in onshore pipelines” applies. No special requirements for pipeline tunnels have been established.

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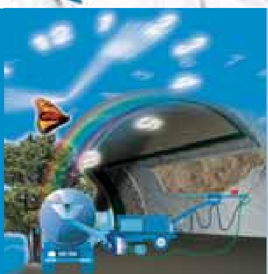

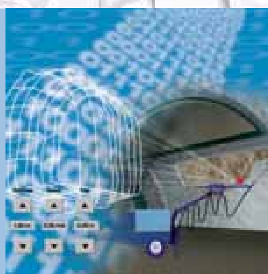

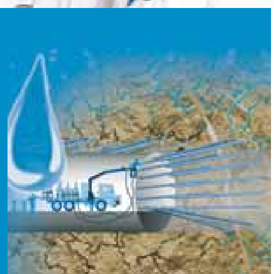

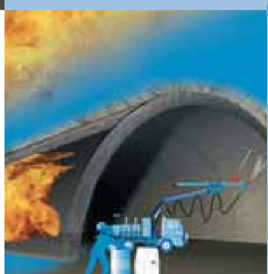

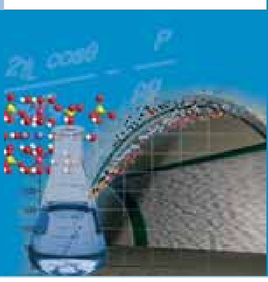

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3. THE CLIENTS DESIGN REQUIREMENTS

Per Arne Dahl

INTRODUCTION

The Norwegian oil industry has always used the underground for storage caverns and pipeline tunnels. The first crude oil cavern was built by Shell at Sola near Stavanger in 1965, and the first pipeline tunnels were the three Statpipe tunnels that were blasted under the three sounds between Kårstø and Karmøy in 1984.

From the owner's point of view, underground facilities have various advantages versus aboveground facilities, e.g.:

- Avoid use of valuable or vulnerable ground areas
- Reduced maintenance compared with similar above ground facility
- Improved safety concerning fire, sabotage, collision, oil spill and discharge of VOC (where actual)
- In many cases underground facilities have lower investment and/or running cost than above ground facilities. Underground facilities can in addition be located below the process plant
- Winter maintenance is minimised.

THE OWNER'S BASIC ASSESSMENTS BEFORE CONSTRUCTION START

When the owner makes his assessments whether to build in the underground or not, various subjects to be considered are as follows:

- Geology and hydrogeology in the actual area
- Object design, schedule and cost. The national and the company's design regulations are to be followed
- Risk for damage to other objects due to the construction activity
- The Norwegian and European Standard NS-EN 1918 to be followed , as well as the Regulation No. 744 from the Norwegian Directorate for Civil Protection and Emergency Planning.
- Availability of capable and experienced contractors and suppliers needed for the construction task. The nominated contractors must also document that they comply with HSE standards and records set by the client.

Matrix of risk evaluation Statoil Mongstad

Consequenses					
Personal injury	**Work environment**	***Business**	**External environment**	**Reputation**	
First aid accident	Minor impact	Minor < NOK 20k	Minor effect	Minor impact	1
Medical treatement injury	Limited impact	Limited NOK 20 - 200k	Limited effect	Limited impact (simple client compl.)	2
Serious personal injury (work absence)	Major impact	Major NOK 200 - 2M	Major effect Consession breakage	Major impact (group of client/ local environm.)	3
Serious personal injury with risk of perm. Effect	Occupational disease	Big effect NOK 2M - 20M	Big effect (Damage to external environment)	Big national impact	4
Serious injury and/or accident/ death	Work disabled	Very big >NOK 20M	Very big effect	Very big and/or international impact	5

* Impact on business includes both cost of repair and reduced income due to damage on equipment

Probability				
A Small probability (<1%)	B Possible (1 - 10%)	C During a 10 year period (11 - 30%)	D Within 1 - 2 years (31 - 60%)	E Several times a year (61 - 100%)
1	2	4	7	21
3	6	9	23	32
5	10	24	34	91
8	22	33	92	128
20	31	90	127	256

Risk = consequence x propapility

Fig. Matrix of risk.

RISK ASSESSMENT EXAMPLE - STATOIL

Low risk level, green area, 1 – 10: Acceptable risk, no following up activities demanded

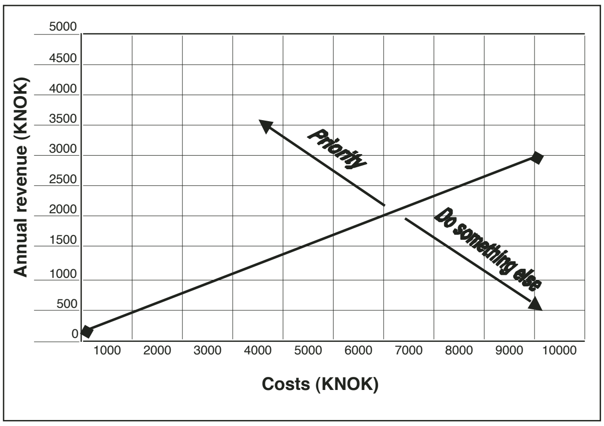
Medium risk level, blue area, 20 – 34: Explain whether corrective actions are necessary in order to reduce risk. Actions considered to be necessary shall be documented. Should the risk be assessed as acceptable and no following up actions take place, the management has to document this in a revisable way.

High risk level, red area, 90 or higher: Unacceptable risk that always demand actions in order to reduce the risk to a lower and acceptable level. Such actions shall be documented.

The categories Medium and High will always demand consultations with the project management.

Risk = (Consequence) x (Probability) x (Operating time, expressed with factor 1.0 for full time/continuous operation).

Rate of revenue / Minimum rate
Simple repayment time on investment projects has been set at 3 years. Minimum rate for annual earnings/repayment on such investments = NOK 200,000



Investment Period ≈ 1 year
Simple Repayment Time ≈ 3 years
Figure: Annual revenue (KNOK) versus Costs (KNOK)

UNDERGROUND FACILITIES SPECIALLY DESIGNED FOR OIL AND GAS

There are different design requirements for the various types of underground oil and gas facilities. The below mentioned groups of installations will be covered separately.

- Tunnels for pipeline installation
- Caverns for crude oil
- Caverns for refined oil products
- Caverns for liquid gas under high pressure
- Caverns for liquid gas stored at temperature below 0° C

TUNNELS AND SHAFTS FOR PIPELINE INSTALLATION

Pipeline tunnels give normally an optimal protection to the pipeline, and may be routed to avoid interference with other or future facilities. Pipeline tunnels below the ground water level are normally water filled.

Rock fall should be avoided, either by concreted tunnel arch, concrete cover slabs or sand fill over the pipeline. The owner requires that the integrity of the pipeline should never be questioned.

Pipeline tunnels have been widely used in Norway for pipeline landfalls. It might be very expensive, but gives and optimal protection to the pipeline through the rough shoreline. Pipeline tunnel or shaft is the clients' first choice for a landfall.

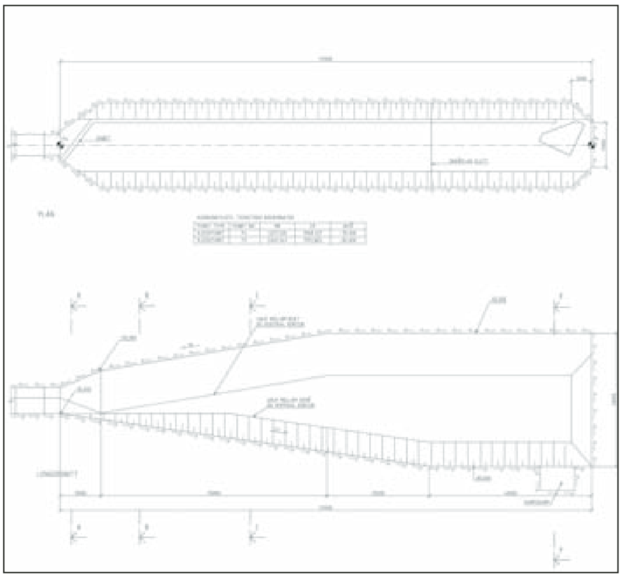


Fig. Crude oil cavern - plan drawing and longitudinal section.

CAVERNS, GENERAL DESIGN REQUIREMENTS

In not gas impervious underground a stable water level, naturally or artificially introduced by a water curtain above cavern top, is mandatory for caverns without gas tight lining. The water pressure at the cavern top should never be less than the sum of the vapour pressure of the stored liquid or gas + 20 m water column (Directorate for Civil Protection and Emergency Planning, Regulation No. 744).

The service shafts have to be built gas tight.

The cavern tops where the product fill and export lines and water pumping lines, level meters, sampling equipment, monitoring equipment and purging equipment are located will always be classified as Ex area. Adequate fencing is required.

All seepage water has to be pumped to a purification plant before it flows into the recipient.

The lay out of the cavern top must allow necessary space for maintenance of pump installations and modifications of piping.

Space for snow clearance during wintertime must not be forgotten.

The linings for the in- and export pipelines in the cavern must be designed in a manner that makes creating of a water loch during pump installation possible. An escape of hydrocarbon gas from the cavern during maintenance work is not acceptable.
It is mandatory that no solid item, rock or metal parts ever can fall down in the pump sump area and possibly damage the pumps, level switches, level gauges or instrument cables. It might be wise to install a slab above the pump sump.
A rock store will normally have one or more access tunnels during the construction
These have to be closed by concrete plug(s) before infill of hydrocarbons or similar chemicals. The plug (bulkhead) has to be located according to Regulation 744, as previously mentioned. The pre grouting of the tunnel in the plug area and a successful grouting of the joint between rock and concrete after shrinking of the concrete in the plug, is very important, especially where the stored liquid is an other than crude oil.

A minimum biological growth due to water seepage is preferred.

The outlet of the import pipeline and the inlet of the export pipeline of the store should be located in a way that provides an optimal mix of the stored liquid during filling and emptying.

CRUDE OIL CAVERNS

The crude oil caverns have normally a fixed water bed. In these caverns biological growth is not a problem. The crude oil water mix is normally not exposed to biological growth. A water bed will avoid transport of sand and heavy impurities to the oil export pumps.

In the crude caverns wax removal will be necessary from time to time dependant on type of crude. The oil has to be heated before exported. The heating is carried out by running the oil in the store in closed circuit through a steamer located at the cavern top until the oil temperature is well above the wax point temperature. The piping in the cavern must be installed in a way that makes this circulation of oil possible.

There will always be oil vapour in the cavern, VOC (volatile oil components). Venting of VOC from a store to the air is not accepted, only in case of emergency. A crude store plant has normally more caverns. Normal procedure is to shift the VOC between the caverns during filling and emptying. The VOC follows approximately the Gas Law.

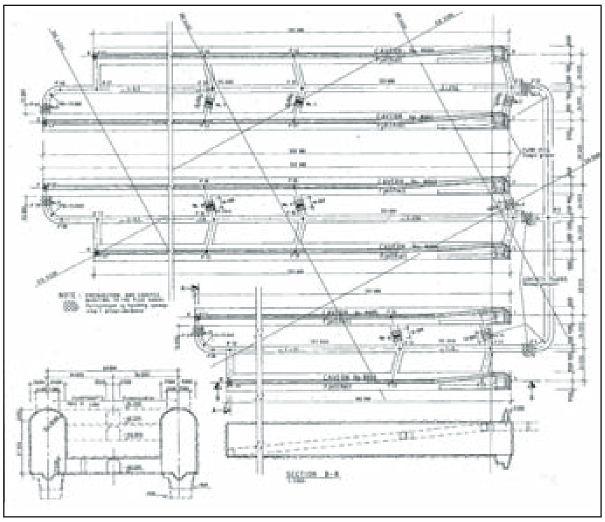


Fig. Drawing West prosess

CAVERNS FOR REFINED OIL PRODUCTS

The recent caverns designed for refined oil products as diesel oil, kerosene and naphtha do not have any water bed. To avoid biological growth, the contact between product and water should be minimised. The ingress of water into the cavern should be minimised through comprehensive grouting of the rock close to the store.
Leak of grout into the water curtain must be avoided.

CAVERNS FOR LIQUID GAS, PRESSURISED

To follow the design criteria top of cavern lower than vapour pressure of the liquid gas + 20 m water column, the location of the store must be rather deep.

This is the main difference to what applies for the cavern for refined oil products.

CAVERNS FOR LIQUID GAS, COOLED

Until now there have been built caverns for cooled ammonia, cooled LPG mix, cooled propane and cooled propene. The cooling of the cavern down to operation temperature, liquefaction temperature for the various fluids, is a critical operation.

All water in and close to the cavern will freeze and the rock surrounding the cavern will crack. To minimise the cracking, the cavern shall have a shape like a ball. This is not very practical during the construction, but well rounded corners shall be aimed at.
Ingress of water in the cavern during operation is not accepted, neither through the walls, through the cavern bottom, nor through the plug (bulkhead).

The volume of ice in the cavern is brought to a minimum by comprehensive grouting of the first 3.0 m of the caverns surrounding rock. Special care has to be taken to the plug and the plug area. The joint between concrete and rock will be gradually widened up during the cooling phase, and a separate grouting and a separate cooling may be necessary. No grout based on cement sets at temperatures below 0° C

To install several temperature gauges in a distance from 0.5 m to 6.0 m from the cavern rock surface, are recommended. Readings from these gauges gives valuable information to the operator during cool down of the cavern and later. An unexpected cracking of the rock followed be water ingress will cause an immediate temperature rise in the actual area.

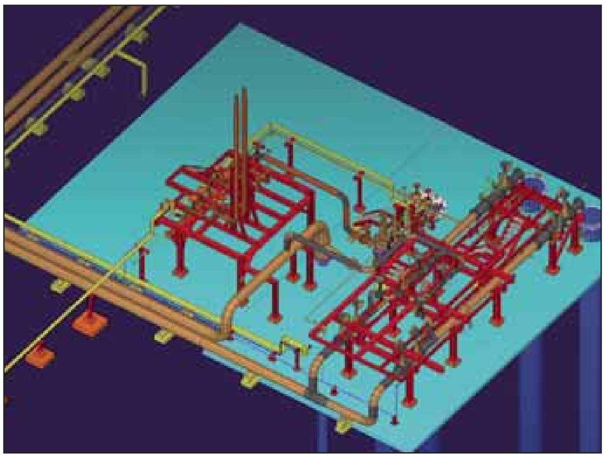


Fig. Cavern top.

4.1 STORAGE OF OIL AND GAS IN ROCK CAVERNS BELOW THE GROUND WATER TABLE - GENERAL DESIGN DEVELOPMENT

Svein Martin Haug

ABSTRACT:
The development and refinement of the underground storage technology is a result of the experience gained through many years of operation. The flexible and efficient utilisation of the underground space for rock cavern storage has enabled later expansion of several Norwegian storage facilities. The concept of underground storage in rock caverns has proved superior to surface storage and the method is recognised as "Proven Technology". Today there are more than 70 rock caverns for storage of hydrocarbons in operation in Norway.

INTRODUCTION
Experience from several years of operation has resulted in development of the storage method. The regulations controlling storage of explosive products have also been developed and have affected the storage methods. This article focuses on the development of underground storage methods.

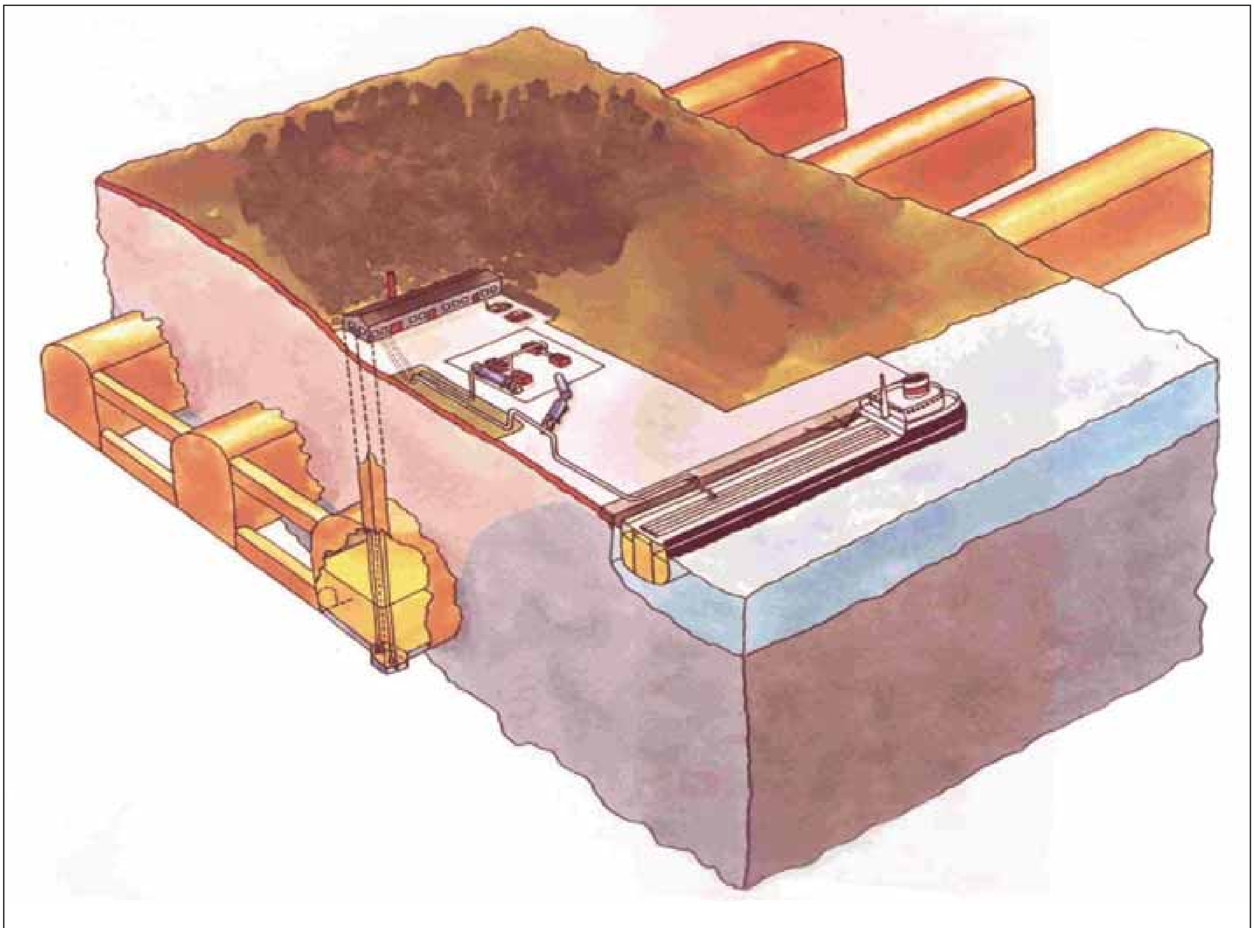


Figure 1: Typical underground storage facility with tanker unloading facilities and truck loading. (Illustration: Sentab / Skanska)

EXTERNAL CONDITIONS AND PRINCIPLES FOR STORAGE OF OIL AND GAS IN UNLINED ROCK CAVERNS

The following requirements describe necessary conditions for underground storage of petroleum products:

- The product must be lighter than water (i.e. specific gravity below 1 g/cm3)
- The product must be insoluble in water
- The ground water level must be stable throughout the area
- The quality and permeability of the rock mass must be suited for caverns with a certain span.

The three first requirements are absolute and emphasize the importance of the ground water. The last requirement is relative and will only have consequences for total cost of the project.

It is important that the level of product in the cavern is always lower than the ground water level. A slow flow of water through the rock mass towards the cavern prevents leakage of products to the ground water and also prevents gas from reaching the surface (Figure 2).

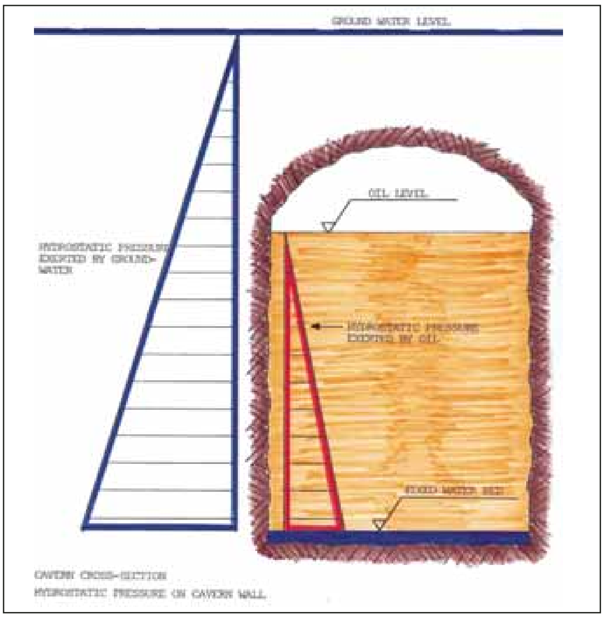


Figure 2: Principle - ground water pressure is higher than the pressure exerted by the product.
(Illustration: Norconsult)

The water leaking into the cavern will not mix with the products, but accumulate at the bottom of the cavern. The water is pumped out and cleaned before it is released (Figure 3).

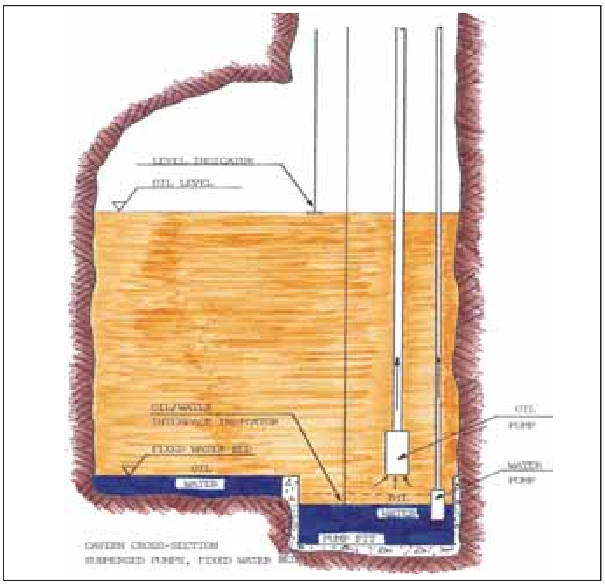


Figure 3: Installations in an underground storage. The water accumulates at the bottom of the cavern.
(Illustration: Norconsult)

STORAGE METHODS - BACKGROUND AND REQUIREMENTS

Different methods are used for different products, depending on storage temperature and pressure. Storage in underground caverns may be designed to meet different demands, just like ordinary steel tanks on the ground surface. Gases like propane may be stored under pressure or at low temperatures to keep them liquefied. Heavy crude oils and fuel oils may have to be heated to make it possible to move them through the pipelines. Today nearly all oil products are stored in closed caverns to avoid discharge of hydrocarbon vapours to the atmosphere.

The first attempts to store oil in unlined rock caverns were made with fuel oil in old mines. These mines were quite shallow compared to the ground water level. The Swedish regulations at that time said that the top of the cavern had to be at least 5 m below the ground water table. The caverns were made with an open vent line, i.e. when oil was filled into the cavern the oil vapours inside the cavern were forced out. When the oil was pumped out, air was sucked into the cavern to compensate for the lower level in the cavern. This method was working as long as there were no requirements for gas discharge, and as long as no explosive mixture of air and oil vapours was formed inside the cavern.

The Norwegian regulations are maintained by DSB (Directorate for Civil Protection and Emergency Planning), earlier called DBE (Directorate for Fire and Explosion Protection). The requirement today is location of the cavern at a depth below the ground water level which is at least equal to the vapour pressure of the product measured in metres of water column, in addition to a 20 m safety margin. For products with vapour pressure of 1.5 bar (15 m water column) or less it is common today to locate the cavern (15 m + 20 m) below the ground water table.

The Flammable Materials Regulations operate with three classes:

- Class A: Fluid with flame point ≤ +23 ºC (e.g. petrol)
- Class B: Fluid with flame point between +23 ºC and 55 ºC (e.g. Jet A-1)
- Class C: Engine fuel and fuel oil with flame point > 55 ºC (e.g. fuel oil)

Danger of Explosion (combustion) in an Underground Storage Cavern

Three elements are necessary to get an explosion (combustion):
- Flammable fluid (vapour)
- Air (oxygen)
- Ignition source (spark)

Air supply inside the cavern is the easiest factor to control in connection with underground storage. It is impossible to eliminate an ignition source because of electrostatic discharges between the rock mass and the product, and between the rock mass and installations inside the cavern. Mineral content in the rock decide how vulnerable the rock mass is.

DEVELOPMENT OF STORAGE METHODS

Variable Water-Bed

As mentioned above, the first caverns were used for fuel oil, i.e. a class C fluid which does not produce flammable vapours. When class A fluids (crude oil, petrol, naphtha) began to be stored in caverns, it was necessary to prevent air from entering into the cavern when the product was pumped out. A method with a variable water-bed was developed (Figure 5). The level of product in the cavern is kept constant at the top of the cavern, while water is pumped into the cavern at equivalent rate as the product is pumped out.

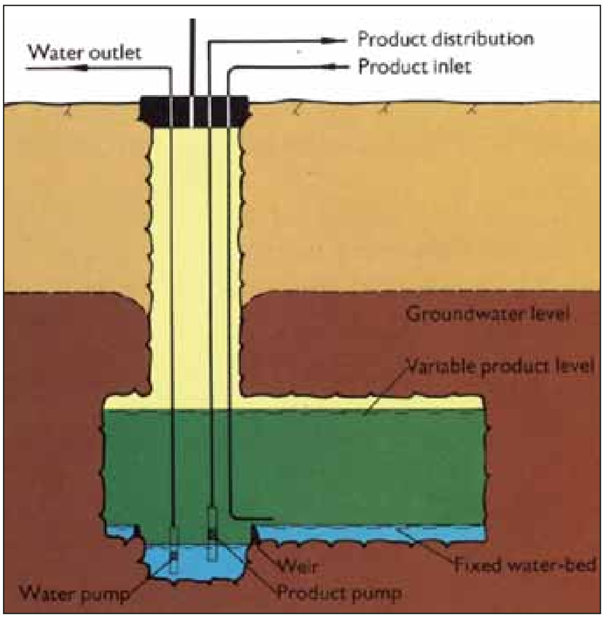


Figure 4: Cavern with fixed water-bed. (Illustration: Finncavern Ltd Oy / Neste Oy)

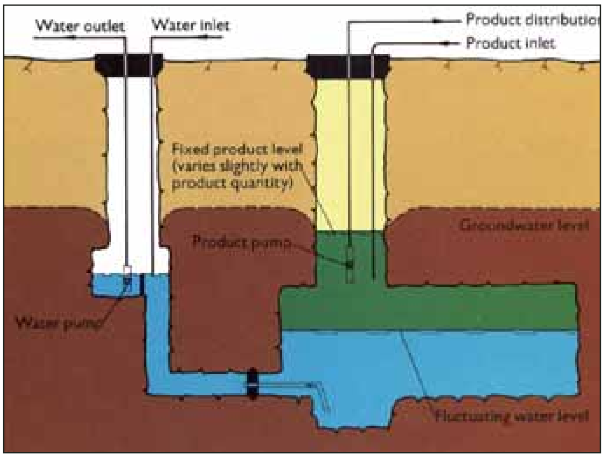


Figure 5: Cavern with a fluctuating water-bed. (Illustration: Finncavern Ltd Oy / Neste Oy)

Advantages and disadvantages with variable water-bed are summarized in Table 1. The disadvantages clearly outweigh the advantages so that the method is not used in new storage facilities.

ADVANTAGES	DISADVANTAGES
No gas which needs cleaning is taken out of the cavern and no air is let into the cavern.	Large energy consumption for pumping water in and out.
Operation with atmospheric pressure inside the cavern; not necessary with a cavern deep below the ground water level.	Management and maintenance of a large cleaning plant for the water is expensive.
	Large contact area between product and water.

Table 1: The table summarizes advantages and disadvantages with a fluctuating water-bed.

STORAGE UNDER PRESSURE

To avoid the disadvantages mentioned in Table 1 and to avoid leaky storages, it became necessary to store the products under pressure. The cavern will then act like a closed pressure tank which operates with variable pressure. It is therefore necessary with a deep location so that the ground water pressure balance the pressure inside the cavern. Today the caverns are normally designed for operation between +1.5 bars and -0.5 bar, a level that has been optimum in most cases. With a maximum operation pressure at +1.5 bars, the top of the cavern needs to be located at least 35 m (15 + 20) below the ground water table (Figure 6).

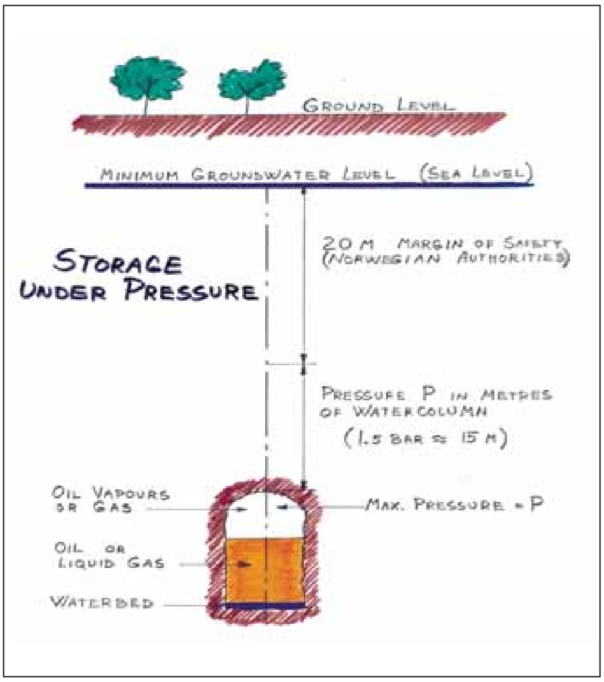


Figure 6: Storage under pressure. Principles with storage under pressure together with the margin of safety. (Illustration: Norconsult)

With a closed cavern and a maximum operating pressure it becomes necessary to have a gas cushion which can expand and compress with a varying level of product in the cavern. The law of ideal gases with constant temperature says pressure * volume = constant. With variations in pressure between +1.5 bar and -0.5 bar, there is a need for 25 % extra gas volume in the cavern (i.e. if the storage volume needed is 100 000 m3, it is necessary to make a cavern of 125 000 m3). This is an extra cost, but compensates for the disadvantages with variable water-bed.

Vapours from oil products are not ideal gases. Evaporation and condensation will influence the pressure inside the cavern. This will stabilise the pressure around the vapour pressure of the product over a period and make the calculations conservative. Vapours from products with low vapour pressure will follow the ideal gas law most closely. Products with a high vapour pressure, like propane doesn't follow the ideal gas law at all. When there are small changes in pressure or temperature, evaporation and condensation will occur almost instantaneously. Storage for propane under pressure does therefore not need 25% extra gas volume, but can be filled up to the ceiling.

Vapour pressure for propane is at about 6 barg at 10 ºC. The top of a propane cavern must therefore be located at least 80 m below the ground water table. In a warmer climate, the vapour pressure is higher (about 8 barg at 20 ºC gives a location of at least 100 m below the ground water table).

UNDERGROUND PUMP ROOM

Only submersible pumps are used in modern underground storage facilities. These pumps are submerged in the product and are directly connected to the end of the discharge pipe. The discharge pipe is hanging freely inside a pipe sleeve, and is suspended from the ground surface. (Figure 7).These pumps are usually electrically driven with the motor directly connected to the pump, and the motor must be pressure sealed to avoid contact with the product. Hydraulically driven pumps have also been used in some special cases.

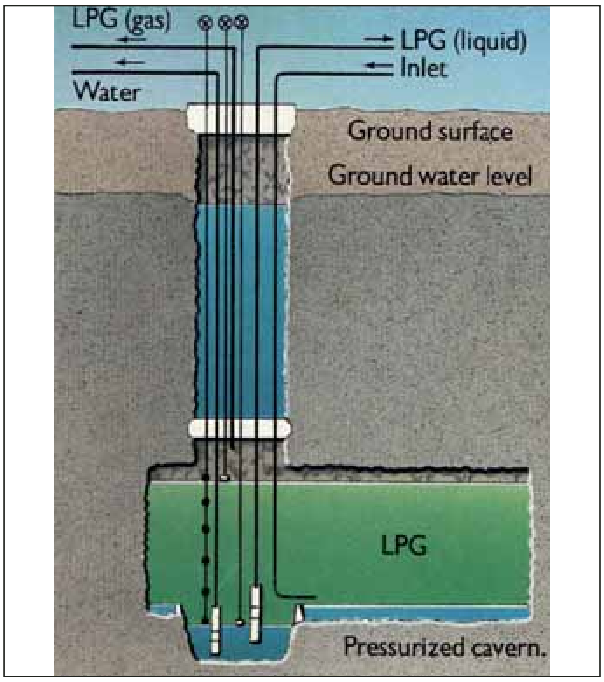


Figure 7: Submersible pumps in a LPG storage. (Illustration: Neste Oy)

During the 1970's many storage facilities with an underground pump room were constructed (Figure 8). The capacity of submerged pumps was too low at that time, only about 1000 m3/hour. For an oil terminal a tanker loading capacity of about 18 000 m3/hour would be required, which made it impractical and too expensive to use submerged pumps. Using 3 conventional centrifugal pumps with 6000 m3/hour capacity each the job could easily be accomplished.

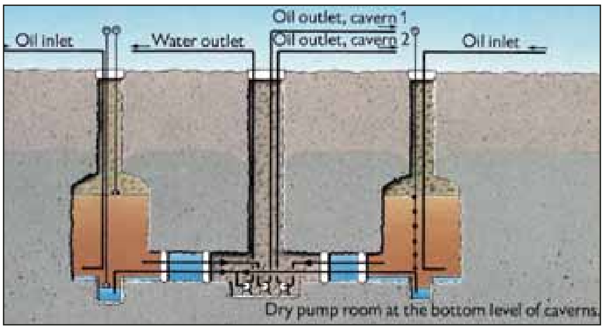


Figure 8: Underground pump room. Dry pump rooms were built in the 70's when submerged pumps did not have enough capacity. (Illustration: Neste Oy)

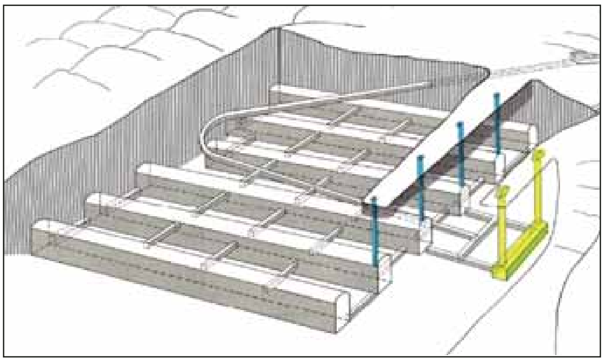


Figure 9: Overview of underground storage caverns and pump room. Pump rooms marked yellow. (Illustration: Norconsult)

Underground pump room is an expensive solution and has a number of safety problems:

- Fire- and explosion danger in connection with maintenance (especially class A fluids).
- Need for extensive fire detection- and extinguishing systems.
- Need for powerful ventilation.
- Emergency exits from the pump room 80-100 m below the surface are necessary.
- Water curtains are necessary to avoid leakage from the caverns into the pump room.

Due to the higher cost and the safety problems described above, the pump room solution is no longer used for new facilities.

WATER CURTAINS

For a cavern located a few meters below the ground water level and operating at atmospheric pressure, the water above the cavern will normally drain into the cavern. When storage under pressure was planned, it became important to maintain the ground water level above the cavern at a proper level to avoid gas blowouts. To solve this problem, horizontal water curtains was drilled from small tunnels above the caverns or from the ground surface (Figure 10). The pressure in these water curtains are maintained at a few metre above the ground water level.

Water curtains between the caverns are also necessary to avoid product leakage to neighbouring caverns (Figure 10). Cross-leakage of product must be prevented between caverns with different products, while cross-leakage between caverns with the same product will usually be allowed. In the illustration below there are three different products with two caverns for each product. The water curtains for cross-leakage prevention are drilled from the same small tunnels as mentioned above or from the surface. The distance between the drill holes is dependant on the permeability of the rock, and the experience of the designer.

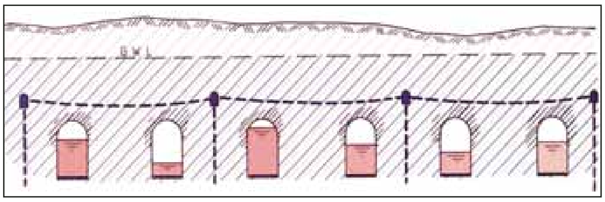


Figure 10: Water curtains drilled from small tunnels above the caverns. (Illustration: Norconsult)

Bacterial growth in water curtains
Water curtains to prevent leakage between caverns must be established before the product is introduced to the caverns. There are examples showing that water curtains established later have been plugged by bacterial growth. Oil consuming bacteria living in the interface between oil and water make parts of the oil into an organic polymer. This is a reddish brown, slimy material which plugs the drill holes and destroys the efficiency of the water curtain system. The material is observed in most underground oil storage plants, and commonly in the water-bed (the interface between water and product at the bottom of the cavern).

Analyses have shown that optimum conditions for these oil consuming bacteria are when the water has a pH at about 9. There is no bacterial growth when pH < 4 or pH > 11. Cleaning of the drill holes has been efficient when a 15 % hypochlorite solution is used (15 ml/m3 water). Chlorination must probably be repeated at regular intervals to avoid new bacterial growth. Anti-icing additives also restrain the bacterial growth, but the solubility in water is 200 times better than in the product. The additives will quickly disappear into the ground water and be a pollutant. The additives must therefore be added after the product is pumped out of the cavern.

"Dry" caverns to avoid microbiological growth
Microbiological growth inside the caverns has been

observed in connection with most oil products. This is a problem for both underground storage and storage in steel tanks. Micro-organisms do not live only from oil products. Free water containing sulphates and oxygen, or total lack of oxygen is vital necessities for the different oil reducing bacteria and fungi which may develop. The growth depends on the temperature; below 10 ºC the growth is very low for most bacteria. Caverns in a cold climate therefore have favourable conditions regarding microbiological growth.

Micro-organisms depend on water to be able to live and multiply, and it is natural to look at methods which can reduce the interface area between product and water. Concrete floor with a slope towards narrow channels along the side walls will reduce the water contact extensively at the bottom of the cavern. The channels slope in the longitudinal direction of the cavern and drain to the pump sump where the water is pumped out. An “umbrella” below the ceiling in the cavern may be installed to collect drops of water and lead it through gutters to the bottom channels.

REFRIGERATED STORAGE OF GAS

Propane (C_3) may be stored in caverns either under full pressure or cooled down to about -42 ºC at atmospherically pressure. It may also be stored at any point on the pressure/temperature curve between these two extremes.

Figure 11 illustrates the results of a computer simulation for cool-down of a propane cavern. The curves around the caverns are the locations of the 0o C isotherm from 10 to 20 years after start-up of the cavern. It is clearly seen that the propagation of the isotherm towards the ground surface has been stopped due to heat influx from the surface. On the other hand the isotherm on the bottom and the sides will continue to extend outward for another 20 years before a steady-state is reached.

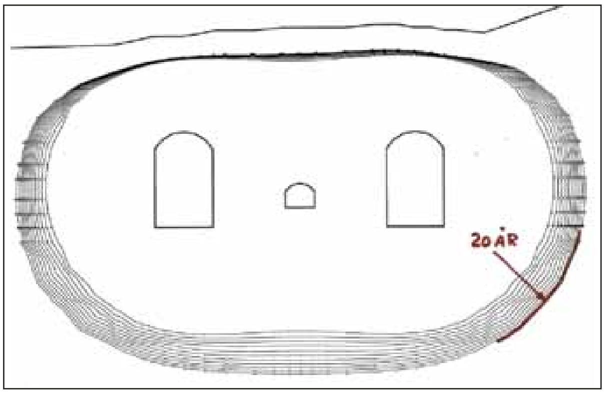


Figure 11: Refrigerated storage of propane. The 0 ºC isotherms from 10 to 20 years after start-up are shown. (Illustration: Norconsult)

Storage of ethylene (C_2) at -100 ºC and a pressure of 0.5 barg has been attempted. Start-up of the cavern had to be aborted due to fracturing of the rock mass at the bottom corners of the cavern causing extensive water ingress and consequently uncontrollable boil-off. The cavern was, however, later successfully converted to storage of propane at -35 ºC and pressure of about 0.7 barg.

STORAGE OF LNG IN UNLINED CAVERNS

LNG (Liquefied Natural Gas) is normally stored in nickel steel tanks with single or double containment to prevent catastrophic failure of the tank. The tanks are heavily insulated to maintain the temperature of the methane at minus 162o C at atmospheric pressure. This solution is very expensive mainly due to the costly nickel steel that must be utilised due to the low temperatures.

Underground storage of LNG in caverns without insulation has been tried in USA, England and Finland, but all attempts have failed due to high boil-off rates and rock stability problems.

Norconsult AS has developed a method for storage of LNG in caverns. To avoid fracturing in the rock mass and to control the boil-off as quickly as possible, insulation has be used between the inner concrete tank and the rock. Bentonite clay is used as membrane instead of expensive nickel steel between the insulation and the rock. The bentonite does not crack at extremely low temperatures and it is impervious the LNG. Successful scale testing has been performed by SINTEF and patents on the method have been granted in Norway and several other countries (Figures 12 and 13).

The method has several advantages.
· Expensive nickel steel is replaced by cheap bentonite which is available in most countries.
· It is constructed from materials readily available, and there is no complicated welding of special steel.
· When constructed in caverns it is very well protected from catastrophic failures.
· The method also covers construction in the ground or on the ground. An outside concrete tank then replaces the rock cavern.

STORAGE METHODS AND PRODUCTS

Table 2 gives an overview of products that may be stored in caverns together with storage method.

ADVANTAGES WITH UNDERGROUND STORAGE COMPARED TO SURFACE STEEL TANKS

• The costs for construction, management and maintenance are lower for underground storage.

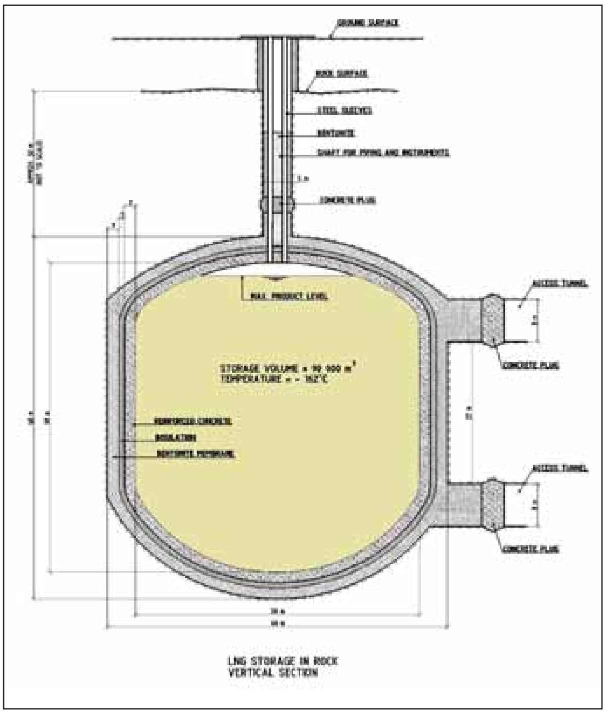


Figure 12: Vertical cross-section of underground storage of LNG developed by Norconsult AS. (Illustration: Norconsult)

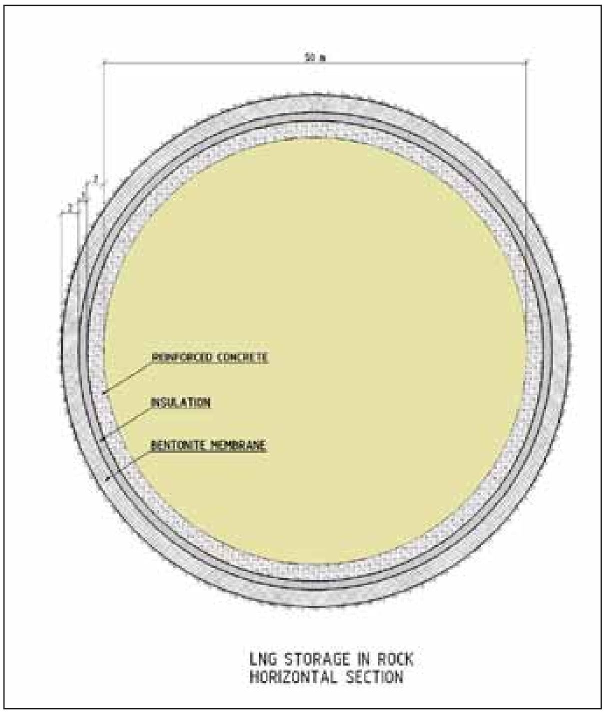


Figure 13: Underground storage of LNG developed by Norconsult AS, horizontal section. (Illustration: Norconsult)

- It is possible to build below existing facilities, even process areas, and thereby getting double use of the property.
- An underground storage is better protected against oil spills and fire disasters, and is environmentally better because of lower discharges to air and water.
- The underground facilities are also much better protected against sabotage.
- The product quality is better maintained for long time storage because of stable temperatures inside the caverns.

GASES	REFRIGERATED OR UNDER PRESSURE
LNG ((C1) with insulation)	-162 ºC, atmospheric pressure
Ethylene (C2)	-100 ºC (may be possible even if unsuccessful the first time)
Propane (C3)	-42 ºC, atmospheric pressure
Propane (C3)	+10 ºC at about 6 barg
Butane (C4)	+10 ºC at about 1,2 barg (refrigerated butane is not possible because storage temperature is too close to the freezing point for water)
FLUIDS (C5 - C8)	**UNDER PRESSURE (-0.5 TO +0.5 BARG)**
-Naphtha	Dry concrete bottom
-Motor petrol	Dry concrete bottom
-Diesel oil	Dry concrete bottom
-Jet A-1	Dry concrete bottom, ”umbrella”
-Heavy oil	Water-bed / with heating
-Crude oil	Water-bed /with heating of heavy crudes

Table 2: Overview of products that may be stored in unlined caverns

REFERENCES

1. Neste Oy Brochure (December 1976): “Underground Caverns - Safe, Economical, Non-Polluting”, 16 pages

2. Finncavern Ltd Oy Brochure (1980): “The Idea of Underground Oil Storing”, 16 pages

3. SENTAB / Skanska Cementgjuteriet Brochures on Underground Cavern Storage

4.2 GEOLOGICAL REQUIREMENTS AND CHALLENGES FOR UNDERGROUND HYDROCARBON STORAGE

Eivind Grøv

INTRODUCTION

Hydrocarbon storage may take place underground in a number of different ways and the most typical might be such as: aquifer storage, salt dome storage, abandoned mines storage, depleted oil and gas fields and finally in mined rock caverns. This article is limited to deal with the last storage concept, mined rock caverns being greatly the dominating method applied in Norway. Rock caverns for hydrocarbon storage has been described in several articles and papers reporting that such storage has taken place in a wide range of geological conditions.

A few basic principles apply for the suitability of the ground conditions to host an underground, unlined hydrocarbon storage, according to Ref. 1. These can shortly be characterised as:

- the rock types must not contain minerals which in contact with oxygen or stored products can create aggressive or reactive chemical products,
- the rock mass must be of such quality that it enables conventional tunnelling and excavation methods without requiring:
  - comprehensive and extraordinary measures to support the caverns or tunnels, and
  - that ground water control can be done by rock mass impermeabilisation using pre-grouting.

These principles are of course based on the fact that the great majority of Norwegian hydrocarbon storage facilities are constructed according to an unlined concept, that is no steel lining or other types of lining of cast-in-place concrete or PVC or similar are required, neither for the containment nor for stability reasons. The rock mass is the main construction material both for a) hosting the storage facility and b) keeping the stored product from evacuating.

Suitable ground conditions and rock mass for cavern construction exist throughout the world. A simplified overview of suitable rock supplies for underground storage is provided in a paper presented in 1987, Ref. 2.

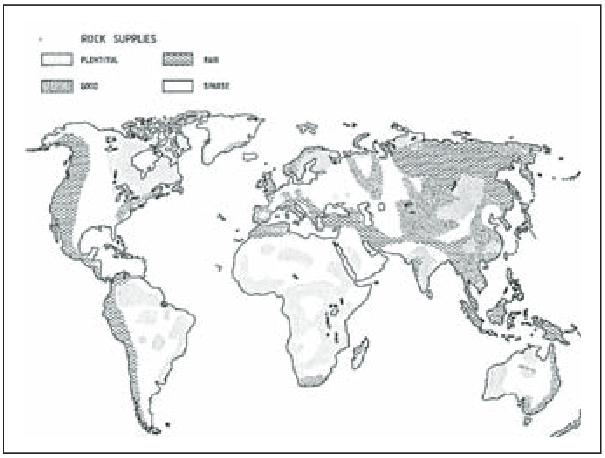


Figure 1: Provinces with suitable rock for underground hydrocarbon storage

A GENERALISED DESCRIPTION OF THE GEOLOGICAL BASIS IN NORWAY

Norway forms part of a Precambrian shield. Two thirds of the country is covered by Precambrian rocks (older than 600 million years), with different types of gneiss dominating. Other rock types from this era are granites, gabbros and quartzite. Approximately one third of the country is covered by rocks of Cambrian - Silurian age. The greater part of these rocks are metamorphosed, but to a varying degree. Rocks such as gneisses, mica-schists and greenstones as well as sandstones, shales, limestones and other un-metamorphosed rocks form a mountain range, which runs through the central parts of the country. In the geologically unique Oslo region, the rocks are partly made up of un-metamorphic Cambrian-Silurian shales and limestones and partly of Permian intrusive and extrusive rocks. These are the youngest rocks.

From an engineering geological point of view, Norway may be described as a typical hard rock province. The rocks have been subjected to folding and faulting, which may have a great influence on the stability in tunnels and underground openings. Another complicating factor is the irregular stresses in the rock masses, caused by tectonic events and further resulting from the steep and

irregular topography. Also high tectonic and residual stresses are encountered.
Due to the geological and tectonic events that have formed the landscape, the rock mass is also severely cut by various types and generations of discontinuities, from cracks and joints to zones containing totally disintegrated material. Tectonically, at present the province is tectonically stable for all practical aspects related to tunnelling work.
In Norway, the hydrogeological situation is dominated by a high, groundwater level, also in the rock mass resulting from a generous amount of precipitation. This situation is both favourable and unfavourable for rock tunnelling. One advantage of a groundwater regime surrounding an underground structure is that it provides a natural gradient acting towards the opening allowing the utilisation of unlined storage facilities. On the other hand, one disadvantage of such saturated conditions is the risk that the tunnelling activity may disturb the groundwater situation, thus imposing the potential of adverse impact on surface structures and biotypes.

The rock itself is in practical terms impervious, and the porosity is negligible. This means that the permeability (k) of a sound rock specimen is likely in the range of 10-11 or 10-12 m/sec. Individual joints may have a permeability (k) in the range of 10-5 to 10-6 m/sec. The rock mass is consequently a very typical jointed aquifer where water occurs along the most permeable discontinuities. The permeability of the rock mass consisting of competent rock and joints may typically be in the range of 10-7 to 10-9 m/sec.

A LISTING OF EXISTING HYDROCARBON STORAGE FACILITIES

In Tables 1 through 3 below the various types of rock storages are listed and the local geological conditions in each of these, according to Ref.3.

Project	Year of Completion	Main rock type	Width x height, m	Temp. oC	PressureMPa	Experience
Kristiansand, Skålevik	1951	Gneis-granite	Ø=32 H=15	40	0,1	No problems reported
Høvringen, Trondheim	1955	Quartsdiorite	Ø=32 H=15	40	0,1	As above
Sola, Stavanger	1960	Mica schist	Ø=15		0,1	Corrosion, decommissoned
Ekeberg I	1969	Granitic gneiss	12x10		0,1	No problems reported
Mongstad	1975	Meta-anorthosite	22x30	7	0,1	Some water leaks
Høvringen, Trondheim	1976	Quartzdiorite	12x15		0,1	Water curtain has been added
Herøya	1977	Limestone	10x15	8	0,1	Leak between caverns
Ekeberg II	1978	Granitic gneiss	15x10	60	0,1	Some blockfalls
Harstad	1981	Mica schist	12x14	7	0,1	No problems reported
Sture	1987/1995	Gneiss	19x33 ~1.000.000m3			No information
Mongstad	1987	Gneiss	18x33 1.800.000m3			No problems reported

Table 1: Norwegian crude oil storage facilities and refinery caverns for hydrocarbon products

Project	Year of Completion	Main rock type	Storage volume, m^3	Width× height, m	Temp. , ^oC	Pressure, MPa	Experience
Herøya	1968	Schistose limestone	50,000 excavated	10×12	6-8	0.8	No leakage, decom-missioned
Glomfjord	1986	Gneissic granite	60,000	16×20	- 28 to -33	0.1-0.13, max. 0.2	No leakage

Table 2: Overview of main data for ammonia (NH3) storage [Ref. 4]

Project	Commis-sioned	Main rock type	Storage volume, m3	Width× height× length, m	Temp. ,°C	Pressure, MPa	Experience
Rafnes	1977	Granite	100,000	19×22×256	~ 9	0.65, tested at 0.79	No leakage
Mongstad	1989	Gneiss	3 caverns, total 30,000	13×16×64	6-7	Up to 0.6	No leakage
Mongstad	1999	Gneiss	60,000	21×33×134	- 42	0.15	Reduced capacity
Sture	1999	Gneiss	60,000	21×30×118	- 35	0.1	No information available
Kårstø	2000	Phyllite	2 caverns, total 250,000	Approx. 20×33×190	- 42	0.15	No leakage
Mongstad	2003	Gneiss	60,000	21×33×134	- 42 (propane) +8 (butane)	0.15	No information
Mongstad	2005	Gneiss	90.000	22x33x140	6-7		Recently put in operation
Aukra	2007	Gneiss	63.000/ 180.000	21x33x95 21x33x270	6-7	0,2	Not yet commissioned

*Table 3: Overview of main data for petroleum gas storage *) [Ref.4]*
**) All with propane; Mongstad 1989 also stores butane and Sture 1999 stores a propane/butane mixture. Mongstad 2005 will be naphthalene, Aukra 2007 will be condensate*

As has been shown in other articles in this publication a number of compressed air storage facilities including such as air cushion surge chambers have been constructed in Norway and these are listed below in table 4 below.

The design of such compressed air storages rely very much on the same technical capacities of the rock mass as are relevant for the hydrocarbon storages listed above.

Project	Commis-sioned	Main rock type	Excavated volume, m3	Cross section, m2	Storage pressure, MPa	Head/ cover*)	Experience
Compressed air buffer reservoirs							
Fosdalen	1939	Schistose greenstone	4,000		1.3		Minor leakage
Rausand	1948	Gabbro	2,500		0.8		No initial leakage
Air cushion surge chambers							
Driva	1973	Banded gneiss	6,600	111	4.2	0.5	No leakage
Jukla	1974	Granitic gneiss	6,200	129	2.4	0.7	No leakage
Oksla	1980	Granitic gneiss	18,100	235	4.4	1.0	<5Nm3/h
Sima	1980	Granitic gneiss	10,500	173	4.8	1.1	<2Nm3/h
Osa	1981	Gneissic granite	12,000	176	1.9	1.3	Extensive grouting
Kvilldal	1981	Migmatitic gneiss	120,000	260-370	4.1	0.8	Water infiltr. Necessary
Tafjord	1981	Banded gneiss	2,000	130	7.8	1.8	Water infiltr. Necessary
Brattset	1982	Phyllite	9,000	89	2.5	1.6	11Nm3/h
Ulset	1985	Mica gneiss	4,800	92	2.8	1.1	No leakage
Torpa	1989	Meta siltstone	14,000	95	4.4	2.0	Water infiltr. Necessary

Table 4: Overview of main data for compressed air storage, including air cushion surge chambers [Ref. 4]
**) Ratio between maximum air cushion pressure expressed as head of water and minimum rock cover*

As can be seen in tables 1 through 4 above the geological host rock in these various facilities vary a lot, thus indicating that a complex list of parameters and evaluations need to be taken into account prior to completing and deciding upon a final location and design/layout of a project.

DEVELOPMENT OF UNDERGROUND HYDROCARBON STORAGE FACILITIES

In Norway, the first underground hydrocarbon storages were excavated during the Second World War, designed for conventional, self-standing oil tanks. Later, being located underground was basically for protective purposes during the cold war era. One project of such kind is located at Høvringen, near the city of Trondheim in central Norway, where ESSO is operating underground steel tanks, whilst one other storage is located at Skålevik, and is operated by BP. Following on from these first projects was underground hydrocarbon storage in steel lined rock caverns, designed and built in accordance with for example Swedish fortification standards. This concept implies in brief a steel lining with concrete backfill of the void space between the steel lining and the rock contour. One such project is located in Hommelvik outside Trondheim and is operated by Fina. This project provides the supply of gasoline to the nearby airport. The above described projects were commissioned almost a half a century ago, and are still being in operation. However, they represent an era and a concept which did not take into account the significant capabilities of the rock mass.

The hydroelectric power development in the sixties realised that the rock mass capabilities could be further utilised; large underground caverns, introduction of wet-mix sprayed concrete, unlined head race tunnels and air charge chambers were all contributions to an extended use of underground space. Thus the confidence in unlined tunnels and caverns grew, and the first unlined hydrocarbon storage project was initiated. Concept developments took place in other Scandinavian countries at the same time, however, in Norway unlined pressure shafts had been in use for some time in the hydroelectric power development, up to 1000m water head, and the importance of sufficient in-situ rock stress to prevent hydraulic splitting of the rock mass was recognised as an important success criteria.

As mentioned above, in the Norwegian concept lining as a barrier had been abandoned due to the significant costs associated with such solutions. Also the techniques of pre-grouting of the rock mass to stem or reduce water leakage started to be developed during this period. Adding to this, caverns with large cross-sections and complicated lay-out geometry were already in use as hydropower stations. Thus, the Norwegian tunnelling industry was prepared and technically ready for the new challenge of unlined hydrocarbon storage in rock caverns.

In the 1970's Norway grew to be a major oil and gas producing nation with the corresponding need for larger storage facilities. It also became evident that the use of surface structures needed to be reconsidered. The solution in Norway was to excavate large rock caverns, utilising the availability of suitable rock mass conditions and the tunnelling experience obtained through the hydropower development.

Underground oil and gas storages mainly utilise the following capabilities of the rock mass:

- Its impermeable nature, i.e. the actual permeability of the rock mass and associated discontinuities may vary from 10-5 m/sec to 10-11 m/sec.
- Its stress induced confinement, the in-situ stress situation varying from stress released rock bodies through a pure gravitational stress situation to stresses originated by long tectonic history of the rock mass.
- Its thermal capacity, i.e. the capacity to store energy over significant amount of time.
- Its self-standing capacity, i.e. the ability of the rock mass to maintain stability even after being subject to cavities being made, man made or natural.

Taking into account that "mother nature" is not a perfect material, and that the rock mass may have a set of imperfections, it is most common that the construction process involves various techniques and methods to assist the design of a construction material that suits its purpose. In the following a short description of these capabilities will follow.

Permeability control and hydraulic containment

For permeability control and hydraulic containment the impermeable nature of the rock mass is utilised with or without the assistance of construction techniques. The methods for controlling leakage from an unlined underground storage consist mainly of 1) permeability control and 2) hydrodynamic control (or containment). In figure 6 it is schematically shown according to Kjørholt [Ref.2].

By permeability control it is meant that leakage control is achieved by maintaining a specified low permeability of the rock mass.

Permeability control may of course by obtained without any particular measure as listed above. This can be achieved by locating the rock caverns in a rock mass that has natural tightness sufficient to satisfy the specified permeability.

However, the rock mass is a discontinuous media and

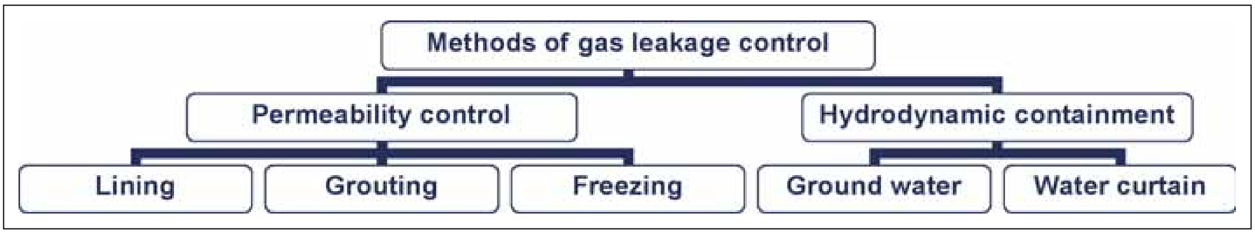


Figure 2: Methods for controlling gas leakage from a pressurised underground storage

the presence of joints etc. governs its permeability. Permeability control can be preserved by artificially creating an impermeable zone or barrier surrounding the rock caverns by; a) sealing the most permeable discontinuities in the rock mass by grouting; or b) introducing a temperature in the rock mass which freezes free water and filling material in the rock mass; or c) a combination of both methods. For crude oil or other products that require operation at ambient rock temperature or above solution a) is the only option, whilst for cooled storage the solutions b) and c) are applicable.

By hydrodynamic control it is meant that there is groundwater present in discontinuities (joints and cracks) in the rock mass and that this groundwater has a static head that exceeds the internal storage pressure. In practical terms it means that there is a positive groundwater gradient towards the storage, or the rock cavern. In general, sufficient groundwater pressure is obtained by a) a deep seated storage location which provides the sufficient natural groundwater pressure, or b) by an additional artificial groundwater such as provided by ´water curtains´ and similar arrangements.

The criteria set forth by the relevant Norwegian directorate indicate that the minimum groundwater pressure shall be 20 m higher than the internal storage pressure. A rock mass with a low permeability would reduce the quantity of water required for the ´water curtains´ arrangement. Investing in finding a rock mass with low permeability and treating the rock mass with pre-grouting to further reduce the permeability would pay off in the operation phase by reduced costs for the operation of the ´water curtain´.

Stress induced confinement
A condition for a successful operation of chilled gas storage can be expressed in the following equation, according to [Ref. 10]:

In-situ stress + tensile strength > thermal stress

In situations with a significant internal storage pressure this will contribute on the right hand side of the equation, however in the case of chilled storages the contribution from the internal gas pressure (0.1 – 0.3 bar) is negligible. Another effect to be considered is the ccapillarity which has a positive effect, thus it is taken as a safety reserve.

In the same way, water pressure caused by water curtains or by natural high ground water will act as a reducing factor on the in-situ stress situation, in other words destabilising the equilibrium.

In a system with a pressurised storage cavern, for example such as for LNG storage taking place at ambient temperature a high internal storage pressure would be required. To be able to withstand the internal pressure the in-situ rock stresses must be larger by a factor of safety than the storage pressure. A high in-situ rock stress must be considered as an important part of the containment system. If this condition is not present the internal storage pressure may accidentally lead to hydraulic jacking of the rock mass, resulting in cracking of the rock mass and opening of pathways that enable the stored product to escape from the storage and migrate into the surrounding rock mass, eventually reaching neighbouring tunnels/caverns or the surface. From the hydropower development the Norwegian tunnelling industry experienced the use of unlined pressurised tunnels with almost a 1,000 m water head. The basis of this design is a minimum stress component that is greater than the water pressure. The analogy goes for pressurised gas storage, namely that the following must be fulfilled:

$\sigma_3 > \sigma_{ip} \times F$ where:

σ_3 is the minimum stress component.
σ_{ip} is the inner storage pressure in the cavern and F is the factor of safety.

Thus, the importance of in-situ stress in the rock mass to balance the storage pressure is obvious. Fortunately, the in-situ stress in many cases is quite different from what can be calculated based on a pure theoretical approach, based on the gravitational component. Consequently, the in-situ stress situation need to be carefully measured by adequate stress measurements in case of designing an sub surface storage of hydrocarbons.

Horizontal stresses of geological origin (tectonic stresses) are quite common in Norway, and in many cases the horizontal stresses are higher than the vertical stresses, even at depths greater than 1,000 m. The majority of rock stress related problems in Norway actually originates from high horizontal stresses, rather than vertical stress due to the rock overburden. This has been the case in a number of road tunnels and tunnels connected to hydropower development, and high stresses have also caused considerable stability problems in power house caverns. This has again called for rock stress and displacement measurements.

However, the pure existence of high, or sufficient in-situ rock stresses, particularly horizontal stresses is an important condition that enable large underground caverns to maintain stability, particularly when the width of the caverns exceed 12-15 m which is the size of an oversized road tunnel. The in-situ stresses are normally exposing a nature which reflects the gravitational component in the vertical direction, whilst in the upper 500 m typically the horizontal component is much higher than the could be derived at by a pure theoretical approach. Weaknesses occur as discontinuities in the rock mass and they can exhibit rather varying characteristics and capabilities as far as being a construction material and may transfer stresses in different ways.

Thermal capacity of the rock mass
In Norway a number of cold storages were actually excavated and in operation before the chilled gas concept was developed. The first of these underground cold storages in unlined rock caverns was commissioned in 1956, with an approximate number of 10 projects being currently in operation. They were constructed with storage capacity in the range of 10-20,000 m3. Typically, the temperature in these storages varies between –25 ºC to –30 ºC. These cold storages have mainly been built for the purpose of storage of food and consumer products. Ice cream storage is one such utilisation.

From years of experience from the maintenance and repair of these facilities the operators have gained important experience regarding the behaviour of the rock mass in frozen state as well as how the ground reacted upon changes in cooling capacity.

For example, on occasions the freezing element was turned off and the temperature sensors in the rock mass were followed up to examine the temperature development in the storage caverns and the surrounding rock mass. A normal response to such changing circumstances was a rather slowly increase of the temperature in the rock mass. The 0-isotherme moved in a rather slow speed towards the tunnel periphery, in the same way as it moves slowly outwards whilst freezing takes place. The thermal capacity of rock in general implies that the material has a significant capability of maintaining its frozen state, once it has been reached, a factor that influences positively also to the cost aspects of those hydrocarbon facilities.

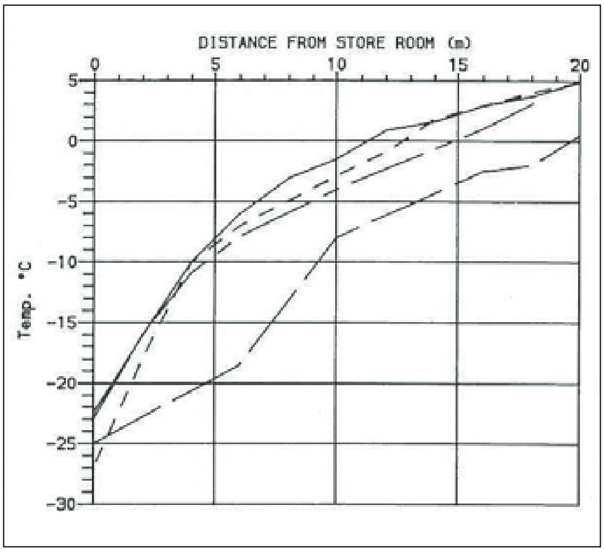


Figure 3: Temperature gradient in rock around a cold storage [Ref. 7]

Self-standing capacity
Most rock mass have a certain self-supporting capacity, although this capacity may vary within a wide range (Bienawski 1984). An appropriate engineering approach is to take this capacity into account when designing permanent support.

As for any type of underground structures the selection of the site location, orientation and shape of the caverns are important steps preceding the dimensioning and the laying out of the underground site.

Rock strengthening may, however be needed to secure certain properties/specified capacities, the same way as is the case for any other construction material. The fact that, the rock mass is not a homogenous material should not disqualify the utilization of its self-standing and load bearing capacity. Typically, rock support application in Norwegian oil and gas storage facilities consists mainly of rock bolting and sprayed concrete. The application of cast-in-place concrete lining in such facilities has been limited to concrete plugs and similar structures and is normally not applied for rock support purpose. The rock support measures are typically not considered as contributing to the containment, other than indirectly by securing the rock contour and thus preventing it from loosening.

Furthermore, the Norwegian tunnelling concept applies widely a drained concept, meaning that the rock support structure is drained and the water is collected and lead to the drainage system. Thus the rock support is not designed to withstand the full hydrostatic pressure in the rock mass. The experience with large underground caverns was obtained in Norway during the development of hydroelectric power schemes for which purpose a total of 200 underground plants were constructed. Commonly the caverns for power-houses and hydrocarbon storage were all typically seized to some 15-20 m width, 20-30 m high and tens-hundreds meter long.

Various types of monitoring to follow-up the behaviour of the rock mass and the support structures are available and used to document the stability and behaviour of the rock mass.

Identification of design parameters

The location of the rock caverns are normally fixed in the design concept and being based on information gathered during a comprehensive pre-investigation phase, however, pending on the actual rock mass conditions as encountered during tunnelling in the approach to the designed and planned location, relocation of the underground structure may of course take place. Several underground projects in Norway have experienced changed locations and local optimisation to better adapt to the actual rock mass conditions. It is common to take into account such information as related to the following:

- Rock types and mechanical properties.
- Characteristics and frequency, spacing of rock mass discontinuities.
- In-situ rock stresses.
- Groundwater conditions.

During the approach to the planned location of the cavern(s) the rock mass is thoroughly mapped, joint systems are observed and characterised, weakness zones are interpreted, in-situ rock stresses are measured, ground water is monitored. If these conditions are not in accordance with the expected and required quality of the rock mass, it may be conclusively decided to shift the location of the storage caverns, and other adjacent caverns and tunnels, or make some layout adjustments. Typically, the final layout of the caverns, their location, geometry, alignment, lay-out of the tunnel system and rock support design may not be finally decided upon until the above information is obtained from the excavation of the approaches of access tunnels. Numerical analyses as well as analytical calculations are useful tools for the design and planning of the caverns. These must of course be verified during the construction phase by adequate monitoring and follow-up of the stability of the under ground caverns.

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4.3 THE WATER CURTAIN – A SUCCESSFUL MEANS OF PREVENTING GAS LEAKAGE FROM HIGH-PRESSURE, UNLINED ROCK CAVERNS

Halvor Kjørholt
Einar Broch

ABSTRACT

In Norway, high-pressure air is stored in ten unlined rock caverns, called air cushion surge chambers. These surge chambers are characterized by pressures up to 7.7 MPa and volumes up to 110,000 m3. This paper describes the successful use of water curtains to prevent air leakage from three such caverns, even when the storage pressure is twice the thickness of the overburden.

INTRODUCTION

Thus far, no hard rock storage has been developed specifically for storage of natural gas or as a CAES (Compressed Air Energy Storage), although much work has been carried out to establish the economic and technical basis for such storage. The main concerns with regard to the acceptance of the hard rock concept are prevention of gas leakage through the fractured rock mass and storage economy.

At present, hydrocarbon gases such as propane and butane are routinely stored in hard rock caverns, but at a much lower pressure than will be required for natural gas storages and CAES. The only large-scale, high-pressure storage experience in hard rock caverns described in the literature comes from ten Norwegian air cushion surge chambers.

An air cushion surge chamber is a pressurized air-filled cavern, the function of which is to dampen transients in the headrace tunnel of hydro power plants (Goodall et al. 1988). Figure 1 shows the design principle for a hydro power plant equipped with an air cushion surge chamber. The surge chambers are hydraulically connected to the headrace tunnel by a short
(< 100-m-long) tunnel. The pressure in an air cushion is consequently dictated by the reservoir elevation. The surge chamber has a water bed below the air cushion. Compressors are used to fill and maintain the air cushion.

Figure 2 provides an overview of pressure and volumes for the surge chambers in chronological order. The first air cushion surge chamber was constructed at the Driva power plant, and commissioned in 1973 (Rathe 1975); the last chamber began operating at Torpa power plant in 1989. Figure 2 also shows that as many as six of the air cushions have pressures that exceed 4 MPa. The highest pressure is reached at Tafjord, where the maximum operating pressure is 7.7 MPa. The cavern volumes are generally less than 20,000 m3; an exception is the Kvilldal surge chamber, which has a volume of 110,000 m3.

Three of the surge chambers (at Kvilldal, Tafjord and Torpa) are equipped with so-called water curtains to restrict the air leakage through the rock. These water curtains are arrays of boreholes, with typical hole spacing of 5 to 20 m, drilled above the rock chamber. Water at a pressure slightly higher than the air pressure in the cavern is fed into the holes. Thus, an artificially high groundwater pressure is established around the cavern. This high pressure prevents air from leaking through the surrounding rock mass.

This paper describes the design and construction of such water curtains, and also discusses experience from ordinary operation and special tests performed at the three surge chamber sites.

Methods to Limit or Eliminate Gas Leakage from a Gas Storage

Figure 3 suggests different methods for limiting or eliminating leakage from an
underground gas storage. These methods are based on two main principles, – permeability control and groundwater control.

Permeability control means that the leakage is eliminated or kept at an acceptable level by ensuring that the rock mass around the storage has a sufficiently low permeability. No general permeability-controlling technique for non-leaking storages is currently available for full-scale commercial use. The most developed alternative is the steel-lined storage. The authors believe that the frozen storage concept will prove to be a realistic and favourable alternative. However, so far this concept has suffered from low research activity. It is further believed that cold storages equipped with a water curtain outside the frozen zone could be considered, in order to provide a double barrier against leakage. Permeability control is, however, beyond the scope of this paper.

The principle of groundwater control is based on the fact that the presence of groundwater reduces gas leakage. The leakage reduction, or degree of groundwater control, depends on the magnitude of the groundwater pressure as compared to the storage pressure. Leakage prevention by groundwater control offers two possibilities: control based on either (1) the natural groundwater pressure; or (2) groundwater pressure that is artificially enhanced by use of a water curtain. The sealing effect of the curtain is conditional, depending on a somewhat higher water pressure in the boreholes than in the storage. In this way, an inward hydraulic gradient, high enough to prevent outward gas migration is established. The water curtain should cover at least the crown of the storage. Under extreme conditions, a water curtain that completely surrounds the storage may be necessary.

To completely avoid leakage by groundwater control, the groundwater pressure in all potential leakage paths, directed upward from the storage, must exceed the storage pressure over at least a small (infinitesimal) distance.

Complete gas tightness based on natural groundwater is, in general, not an economical alternative for high-pressure storages because of the requirement that the allowable storage pressure must be low in relation to the thickness of the overburden. Therefore, a water curtain should be used to increase the groundwater pressure artificially. This type of arrangement will allow a higher ratio between storage pressure and depth, and will increase the operational flexibility. Experience shows that water curtains have been used successfully to avoid gas leakage at storages with pressure up to twice the hydrostatic groundwater head.

EXPERIENCE FROM THREE AIR CUSHION SURGE CHAMBERS WITH WATER CURTAINS

Water curtains have been installed at three air cushion surge chambers: Kvilldal, Torpa and Tafjord. Only at Torpa was the water curtain included in the original design. The two other water curtains were constructed as a consequence of unacceptable air leakages. The geometry of the three caverns and water curtains is provided in Figures 4, 7 and 10.

Kvilldal

The Kvilldal air cushion operates at a pressure around 4 MPa, with a minimum rock overburden of 520m in a steeply sloping terrain. The cavern was originally constructed without a water curtain, but experienced an air leakage of 240 Nm3/h after commissioning in 1981. In an attempt to reduce (but not necessarily eliminate) this leakage, an overlying water curtain, consisting of 47 percussion-drilled boreholes (diameter of 51 mm), was installed in 1983. The geometry of the water curtain is shown in Figure 4. As can be seen, the water curtain is very irregular, with borehole spacing of up to more than 20 m in certain areas.

This water curtain has completely eliminated the air leakage through the rock. Tests have resulted in the relationship between water curtain over-pressure and air leakage shown in Figure 5. As indicated in the figure, any leakage through the rock is eliminated at Kvilldal if the potential in the water curtain exceeds the potential in the air cushion (measured at cavern roof level) by 90 m of water head.

In 1986, a water curtain supply pipe broke and put the water curtain at Kvilldal out of commission. As can be seen in Figure 6, this action resulted in an increasing air leakage, approaching the level experienced before the water curtain was installed (240 Nm3/h). However, the leakage developed quite slowly; in two months, only 50% of this initial value had been reached. After the water curtain was repaired in 1987, the leakage through the rock mass was eliminated again.

Tafjord

The air cushion surge chamber at Tafjord (Figure 7) was constructed in 1982. The air cushion operates at a pressure between 6.5 and 7.7 MPa, while the minimum rock overburden is only 440 m (steeply sloping terrain).

Like the Kvilldal facility, the Tafjord surge chamber was originally constructed without a water curtain. Although the leakage at this site was somewhat less than that at Kvilldal, the compressors installed to maintain the air cushion did not have sufficient capacity. The surge chamber at Tafjord was therefore out of operation from 1982 to 1990 (i.e., the cavern was completely filled with water). Attempts to grout the surrounding rock did not improve the leakage condition.

In 1990, a water curtain was installed at Tafjord, partly as a research project. The curtain consists of 16 core drilled holes (diameter of 56 mm), which cover both the roof of the cavern and the upper part of the cavern walls. Results from a water curtain test at the Tafjord air cushion are shown in Figure 8. The upper curve represents the water curtain overpressure (difference between the water curtain potential and the potential in the air cushion at cavern roof level). The lower curve represents the air mass in the air cushion. The air cushion pressure was 7.6 MPa during the test.

At the start of the test, on February 15, the potential difference between the water curtain and the air cushion was 32 m of water head. Under this condition, no change in air mass was recorded, which means that no air was leaking from the air cushion. On February 18, the water curtain pump was stopped and remained shut off for two days. The air leakage (reduction in air mass) started immediately and reached a constant value (125 Nm3/h) within a few hours. When the water curtain was restarted, the air leakage simultaneously disappeared. A subsequent reduction in the potential difference to 8 m reduced the leakage to less than 5 Nm3/h.

Based on a number of such tests, it has been possible to draw the two leakage curves shown in Figure 9. The upper curve shows the air leakage at Tafjord as a function of the potential difference for an air cushion pressure of 7.6 MPa. The lower curve shows the same data for a pressure of 6.5 MPa. Two effects should be noted:

1. The necessary potential difference to avoid leakage increases with increasing storage pressure.

2. The leakage is significantly higher for a storage pressure of 7-6 MPa than for 6.5 MPa (as expected).

Torpa

At Torpa, the maximum air cushion pressure is 4.4 MPa, while the cavern overburden is only 220 m (see Figure 10). The pressure situation for the water curtain is even more extreme.

The routine operational pressure in the water curtain is 4.6 MPa, with a minimum overburden thickness of only 207 m. Torpa is the only air cushion surge chamber where a water curtain was included in the original design. The water curtain consists of 36 percussion-drilled boreholes (diameter of 64 mm), drilled from an excavated gallery 10 m above the cavern roof. Access to the gallery is through a vertical shaft extending from the cavern roof. The gallery is hydraulically separated from the surge chamber by a concrete plug in the shaft.

Tests carried out at Torpa show that the leakage without the water curtain in operation becomes 400 Nm3/h, and that no leakage is registered as soon as the potential in water curtain exceeds the potential in the upper part of the air cushion by 20 m of water head.

In all three water curtains discussed herein, the water consumption is somewhat less than 1.0 l/s. The water is untreated, and is of drinking quality.

WATER CURTAIN DESIGN

A water curtain is characterized by:
- The spacing of the boreholes.
- The distance between the boreholes and the storage cavern.
- The extent of the water curtain.
- The pressure (or potential) in the water curtain relative to the storage pressure (potential).

The main factors governing the design of a water curtain are:
- The storage pressure versus groundwater pressure and overburden.
- The storage geometry.
- The tightness requirement.

Other factors that should be considered are:
- Rock jointing.
- Access for drilling the water curtain boreholes.
- The upper pressure limit for the water curtain due to the risk of hydraulic jacking.
- Rock stress situation near the storage.
- Economic considerations regarding construction and operation.
- Any restrictions on water consumption or inflow to the storage.
- Maximum borehole length.
- Expected borehole deviation as function of borehole length.

Water curtain design is not an exact discipline. Even though the criterion of complete tightness is a simple one, its practical application to a fractured rock mass involves difficulties related to the irregular nature of the rock fractures. Therefore, practical design should be based on a combination of experience from existing storages, theoretical calculations, and hydraulic testing at the site.

The following general statements give an idea of the typical magnitude of the key design parameters:
- The distance between the storage and the water curtain should not be less than 10 m for small caverns, increasing to 30 m for large caverns.
- The practical borehole spacing should be in the range of 5 to 20 m.
- The water curtain should at least cover the roof of the cavern. As the ratio between cavern pressure (in m of water head) and overburden (in m) approaches 2.0, it is also necessary to cover the sides of the storage, as is the case at Tafjord and Torpa (see Figure 7 and Figure 10).
- The necessary pressure in a properly designed water curtain normally need not exceed the storage pressure by more than 0.5 MPa.

More detailed guidelines for design and construction of water curtains are presented in a doctoral thesis by Kjørholt (1991), and are also presented in Kjørholt and Broch (1992).

SAFETY ASPECTS

The typical safety concern for a gas storage facility is related to leakages that can cause financial losses, fire, explosion; or which may be harmful to people and the environment in other ways.

The possible ways that leakage may occur at a gas storage facility isolated by a water curtain can be divided into three categories:
1. Improper water curtain design or construction.
2. Long-term effects.
3. Operational problems.

Improper water curtain design or construction may cause minor leakages between water curtain holes, or outside the extension of the boreholes. Thorough hydraulic testing of the water curtain during construction, in addition to theoretical analyses, will minimize this risk. Still, if a leakage is experienced after commissioning, an increase in the water curtain pressure or a reduction in the storage pressure may be used to eliminate the leakage.

Long-term effects are related primarily to the possibility that the boreholes may gradually become clogged. Clogging will result in an increased head loss near the borehole walls and, thereby, reduced groundwater pressure between the boreholes. If this is the case, the groundwater pressure will approach its critical value over time, and eventually the storage will start to leak. A clogging phenomenon can be revealed by a reduction in water curtain consumption and reduced inflow to the storage.

At least three effects may cause clogging:
1. Particles in the water supply.
2. Chemical precipitation.
3. Bacterial growth.

By treating the water, it should be possible to reduce or eliminate these effects; Andersson et al. (1989) discusses possible actions that can be taken. If a critical reduction in water curtain efficiency occurs, immediate action should be taken to increase the water curtain pressure or to restrict the maximum storage pressure. In the case of bacterial clogging, it has been found possible to re-establish the water curtain by high-pressure flushing of the individual holes (Barbo and Danielsen 1980).

Operational problems, the third possibility for a water curtain failure include all possible problems in keeping the water curtain pressure at the desired level. Typical problems in this category are insufficient supply of water or power, and failure in pumps, pipelines and monitoring systems. It is believed that the desired level of safety against this kind of failure can be obtained through the use of black-up systems.

CONCLUSIONS

Experience from the use of water curtains at the three Norwegian air storages discussed herein, at pressures from 4 to 8 MPa, is encouraging. It has been found that a properly designed water curtain totally eliminates any gas leakage from the storage, even for a storage pressure head that is only twice the thickness of the rock overburden.

A water curtain may provide not only a cost-effective method to restrict gas leakage from unlined hard rock caverns; currently it also appears to be the only practical way of totally preventing gas leakage from a high-pressure storage.

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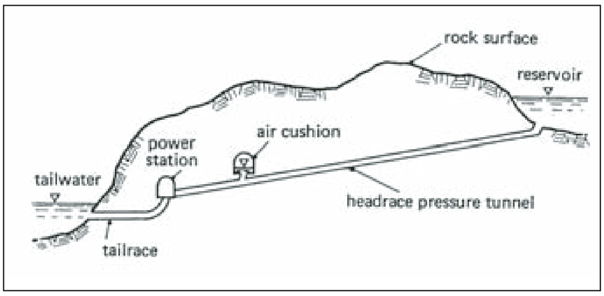

Figure 1. Concept of a power plant with an air cushion surge chamber.

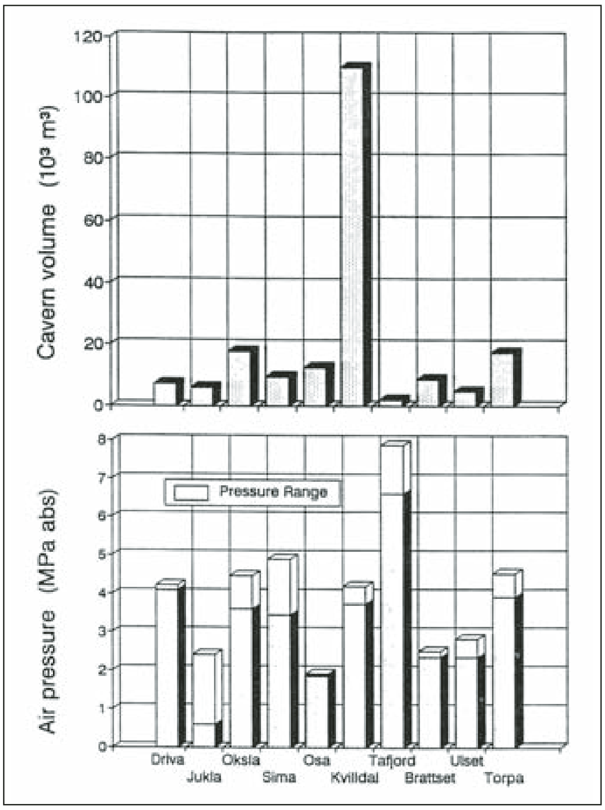

Figure 2. Volume and pressure of the air cushion surge chambers.

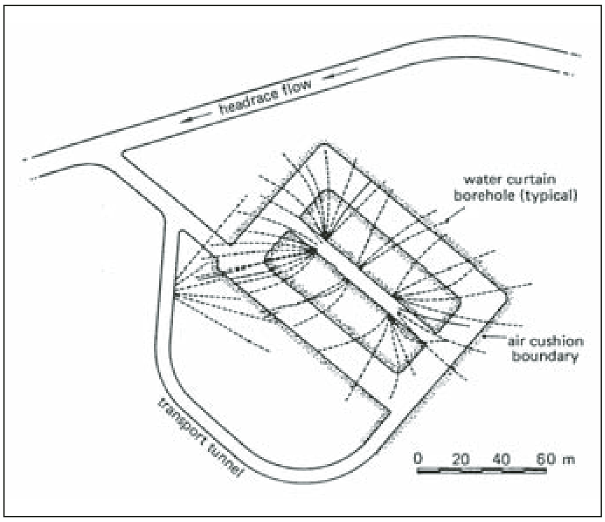

Figure 4. Plan of Kvilldal air cushion surge chamber with water curtain.

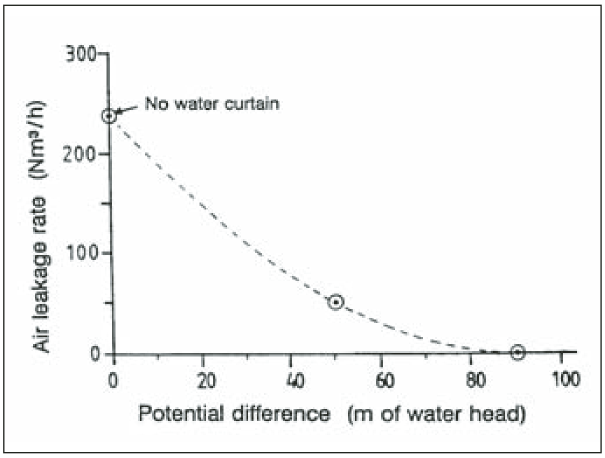

Figure 5. Air leakage at Kvilldal air cushion surge chamber as a function of the difference between the water curtain potential and the potential at the cavern roof.

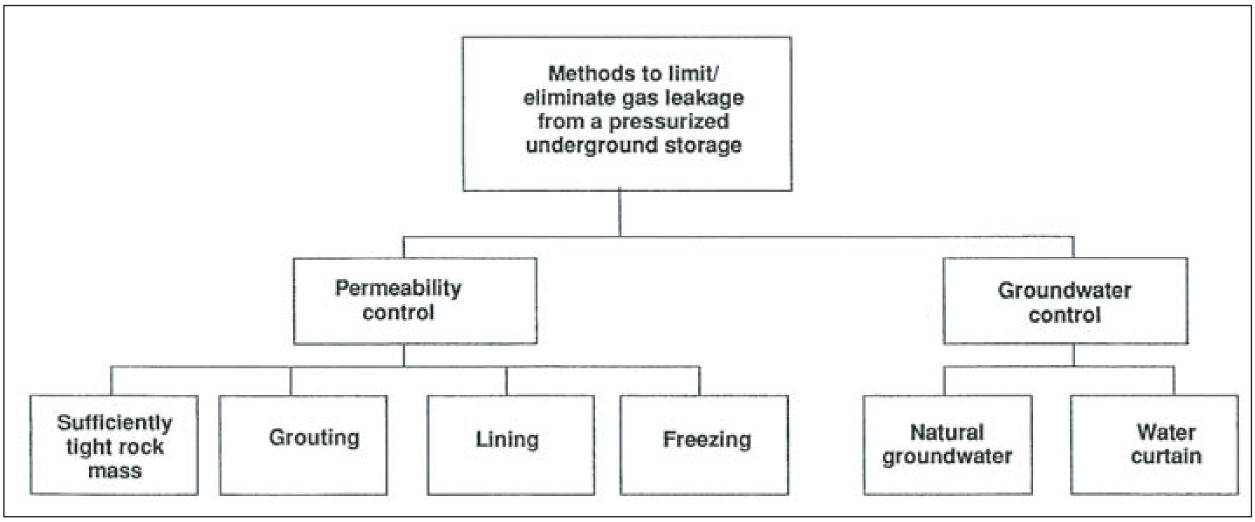

Figure 3. Methods to limit or eliminate gas leakage from a pressurized underground storage.

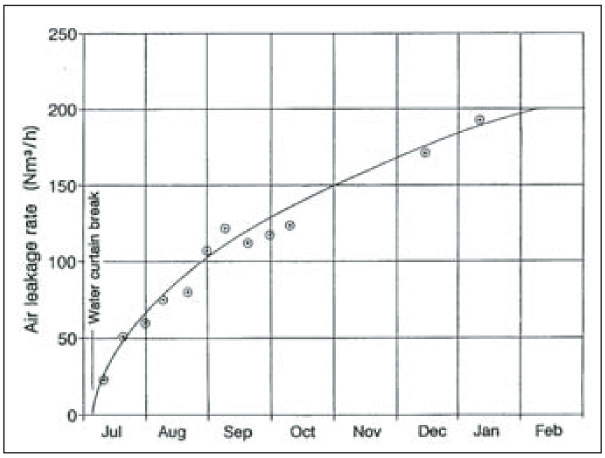

Figure 6. Air leakage development at Kvilldal air cushion surge chamber after water curtain break.

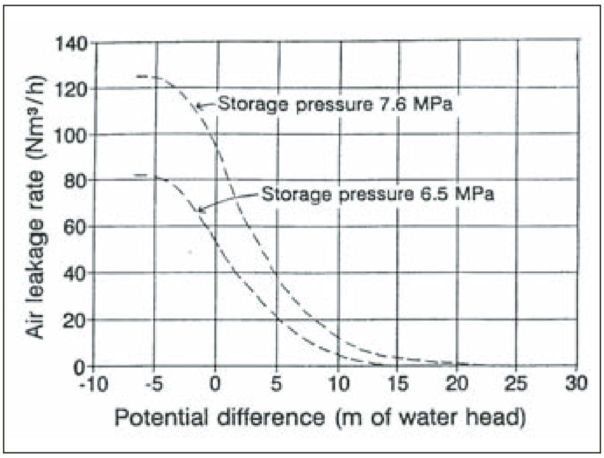

Figure 9. Air leakage at Tafjord air cushion surge chamber as a function of the potential difference between the water curtain and the air cushion.

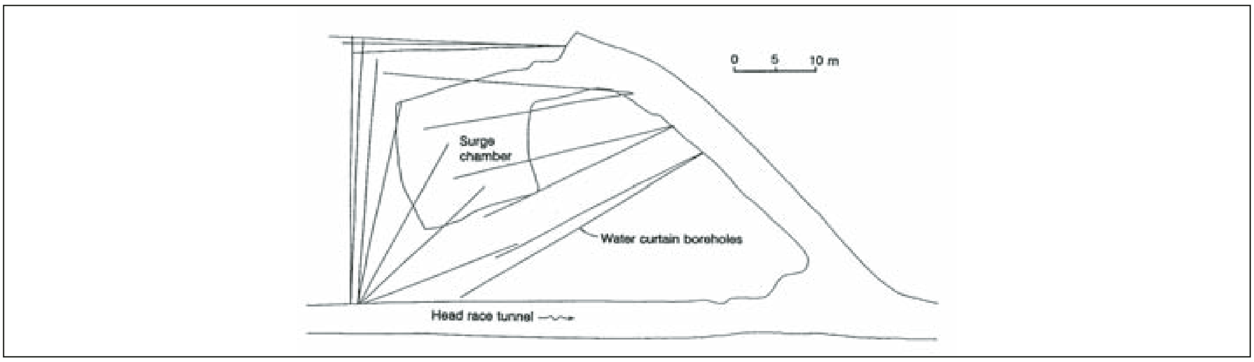

Figure 7. Plan of the Tafjord air cushion surge chamber with water curtain.

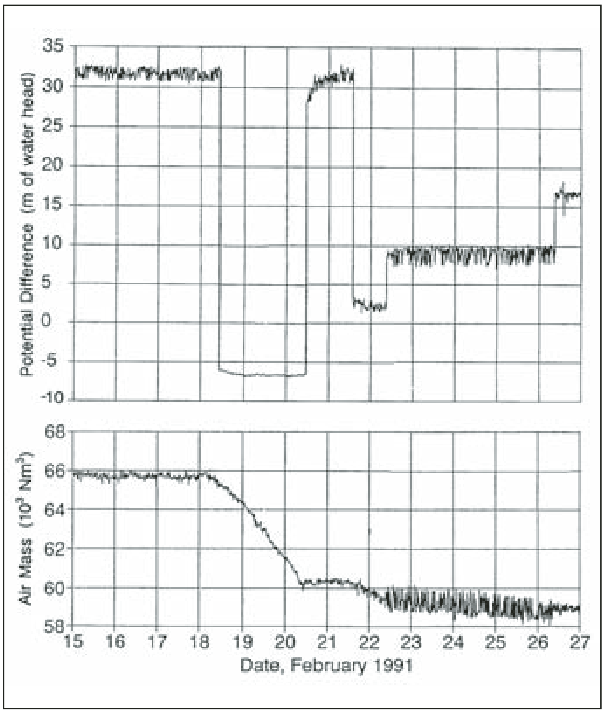

Figure 8. Air leakage at the Tafjord air cushion surge chamber in response to changes in potential difference between water curtain and air cushion.

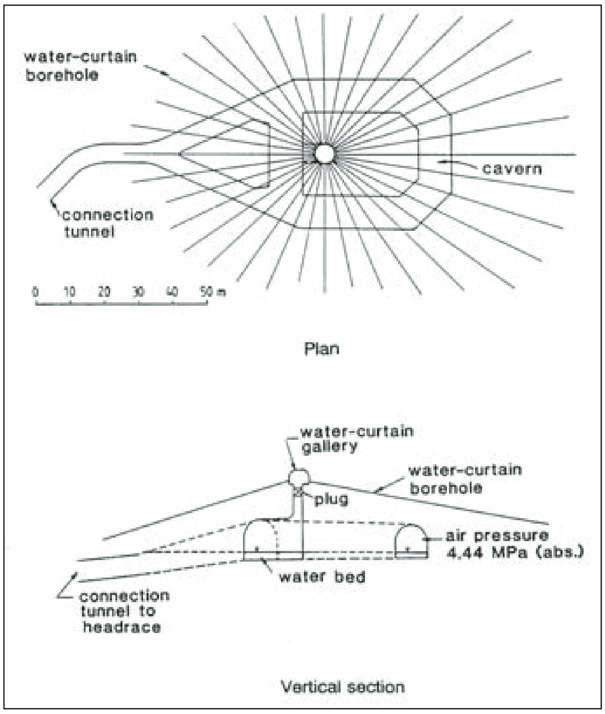

Figure 10. Geometry of the Torpa air cushion surge chamber with water curtain.

4.4 THERMAL BEHAVIOUR OF ROCK IN RELATION TO UNDERGROUND GAS STORAGE

Ming Lu

1. GENERAL DESCRIPTION

Three types of unlined rock caverns are used for gas storage: in porous formations such as depleted or abandoned oil or gas field and aquifers, in abandoned mines and in excavated unlined hard rock caverns. The storage temperature and pressure vary based on the product stored. Listed in Table 1 are the boiling temperatures of some LNG (Liquefied Natural Gas) and LPG (Liquefied Petroleum Gas) at the atmospheric pressure. This paper will concentrate on the unlined LPG storage caverns.

For most existing LPG projects the storage pressure is not high, usually no more than a few bars. The major rock mechanics problems are associated with the thermal stress resulting from the low storage temperature. The shock thermal stress can be calculated theoretically from $\sigma=\alpha E\Delta T/(1-\nu)$, where α, E, and ν are the thermal expansion coefficient, the E-modulus and the Poisson's ratio of the rock mass and ΔT is the temperature change. Take propane storage in hard rock cavern as an example, if the storage and ambient temperatures are -41°C and 8°C, σ=7E-6/°C, E= 30GPa and ν=0.28, the shock thermal stress would be 14MPa. However, the actual thermal stress resulting from the cooling process depends, in addition to the material parameters, also on the cavern geometry, overburden, in-situ stress and the length of the cooling period. LPG caverns are usually located shallowly with overburden starting from about 50m. The cooling period varies from 60 to 150 days. The actual maximum tensile stress thermally induced during the cooling-down may range from 5 to 12MPa for a cavern of 600m2 in cross section area in hard rock. This tensile stress may or may not create thermal cracking in the intact rock, but it is definitely sufficient to open the joints that exist inevitably. Opening of joints and possibly thermal cracking will cause excessive boil-off which is one of the major reasons why some LPG storages have been decommissioned.

The distribution of the thermal stress features (1) decays rapidly from the cavern surface and (2) the resulting tensile stress is often larger on the cavern walls than in the roof. It should be mentioned that the mechanical and thermal properties of the rock mass, on which the thermal stress depends, are also temperature-dependent. For example, from ambient temperature to -160°C E-modulus doubles, Poisson's ratio increases by 30% and linear expansion coefficient reduces to half. Thermal conductivity also increases with decreasing temperature (Goodall 1989).

During normal storage operation, the joints are filled with ice. The tensile strength of ice is about 0.7MPa and temperature-independent. The permeability of frozen rock may be low enough for minimizing gas leakage. Another important matter, however, is the tensile strength of the rock mass with joints filled with ice and whether the strength can balance the thermal tensile stress or not. Most critical is the location of the zero-degree isothermal line. If the isothermal line is far away from the cavern where the tensile stress has decayed below the tensile strength, the potential for gas leakage will be blocked by the frozen rock. Figure 1 illustrates the predicted distribution of temperature and tangential stress along a line normal to the surface of a propane LPG storage at the end of 60 days cooling period.

Gas	Methane	Ethane	Propane	I-butane	n-butane
Boiling Temp.[°C]	-162	-89	-42	-12	-1

Table 1 Boiling temperature of selected gases

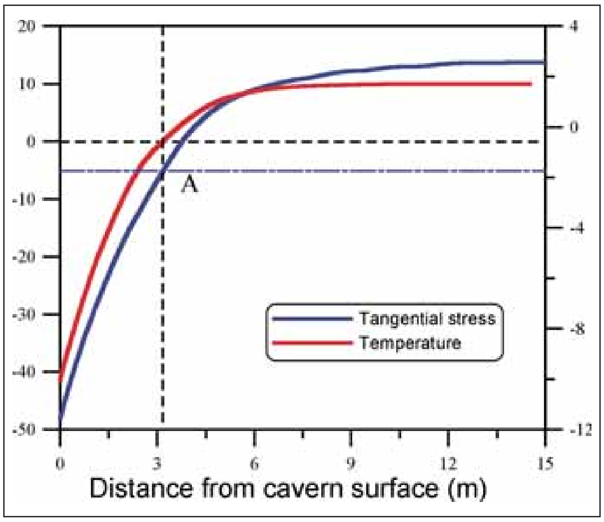

Figure 1. Distribution of temperature and tangential stress.

A typical example of the problem resulting from thermal stress is an ethylene storage cavern in the Stenungsund Petrochemical Centre, Sweden, (Jacobsson 1977). The 15m wide, 21m high and 45m long cavern is situated in gneissic granite of relatively low joint frequency. The overburden is only 12-15m which was considered sufficient for preventing gas leakage. The designed storage temperature is -100°C and the pressure is substantially atmospheric pressure, 0.3-0.7 bars. The cooling-down of the cavern was started with spraying propylene (-40°C) for six weeks and ethylene for one week, followed by pumping 500 tons of ethylene into the cavern. The intensive cooling procedure continued with spraying ethylene for another four months. When the cavern was again partially filled up an exceedingly high boil-off rate occurred. The cavern pressure continued to increase up to the maximum operating pressure, despite the refrigerators were running at the ultimate capacity. Some ethylene leaked out to the ground surface. Finally the liquefied ethylene was pumped out from the cavern. It was believed that the problem of high heat flux was caused by the opening and propagation of cracks in the rock mass surrounding the cavern. The ethylene leaked out through the open cracks which are either dry or containing ground water of pressure less than the cavern storage pressure. An extensive repair work was performed and the cavern was later converted to a propylene storage (-40°C) and has operated successfully ever since.

Another potential problem may be the water inflow during the cooling-down process. When the cooling starts the cavern temperature declines. Consequently the rock joints will open, leading to increased inflow of ground water. If some treatment for reducing rock permeability such as grouting has been done properly and the cooling process is sufficiently fast, the water will become ice within the rock mass before large amounts of water flow into the cavern. On the contrary, if significant water inflow has taken place through the open joints or other channels (e.g. plug) before the rock mass is frozen, a disaster will occur. The water which flows into the cavern will soon freeze inside the cavern. Also, deformation may lead to redistribution of leakage and concentration of streams. The concentration of water inflow may be larger than what the freezing capacity can close. Experience indicates grouting has little effect in this situation and it can hardly be predicted by numerical analysis.

2. NUMERICAL SIMULATION OF LOW TEMPERATURE GAS STORAGE

Numerical analysis is a useful tool for predicting time-dependent temperature distribution as well as the thermally induced rock stress during and after cooling-down of child gas storage caverns. In such simulations coupling of temperature calculation and stress calculation in transient state is necessary. There are two types of coupling: sequential coupling and full coupling. For sequential coupling in each time step temperature is computed first and then the stress is calculated based on the temperature changes. For the fully coupled simulations the temperature and stresses are computed simultaneously with mutual influence taken into account.

The thermal boundary conditions at the cavern boundary may be time history of temperature or applied heat flux. The far field thermal boundary condition can be set as constant temperature which can be taken as the annual mean temperature. The typical output of the analysis may include time-dependent distribution of temperature and stresses, which can then be used in evaluation of cavern stability and potential joint opening. The analysis can also be used for estimating the refrigeration capacity required for cooling-down the cavern. In more sophisticated analysis a coupled thermal-stress-fluid flow simulation can also be performed. For the discontinuous modelling such as using UDEC the water inflow to the cavern through joints can be calculated. However, all such simulations do not consider the phase change, i.e. the icing phenomenon.

Many commercial programs can be used for such simulations. Representative computer programs for continuous modelling and discontinuous modelling may be ABAQUS and UDEC, respectively. Given below in Figure 2 and 3 are illustrations of computed temperature distribution and major principal stress distribution at 150 days and 3 years respectively after commence of cooling –down process of a LPG storage cavern in hard rock. In this example a constant temperature is applied at the cavern surface since beginning of cooling-down. The temperature at the model boundaries is set to a constant value of 8°C.

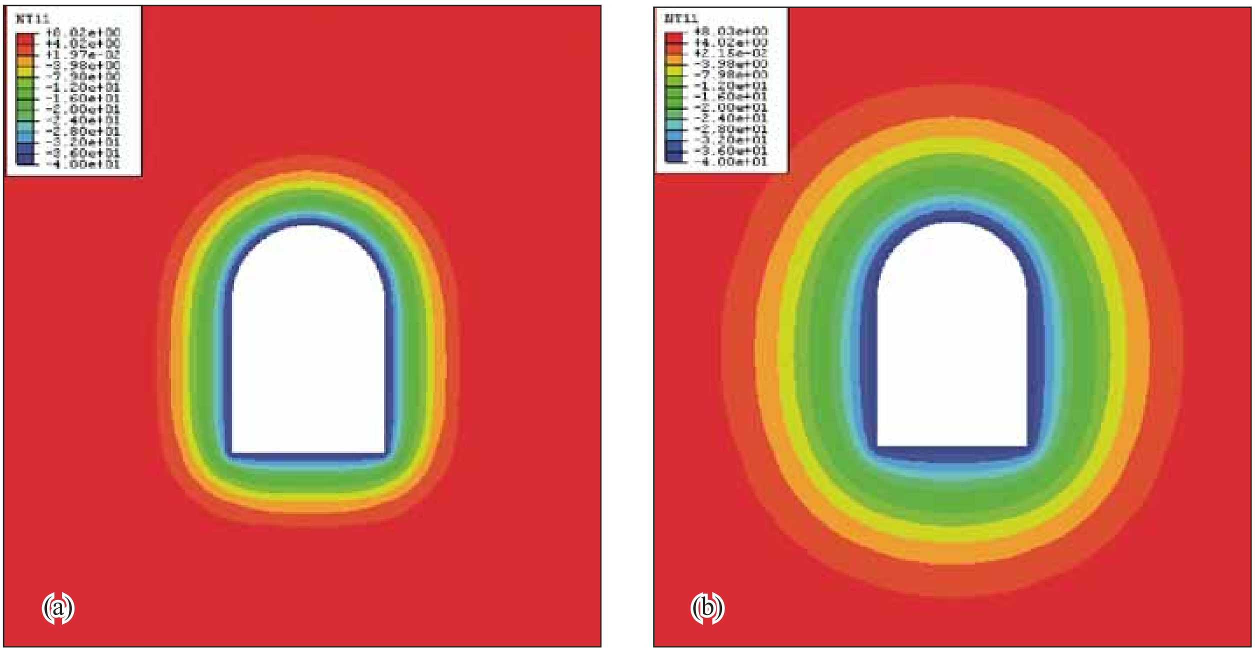

Figure 2. Temperature distribution around a LPG cavern (a) 150 days and (b) 3 years after commence of cooling-down.

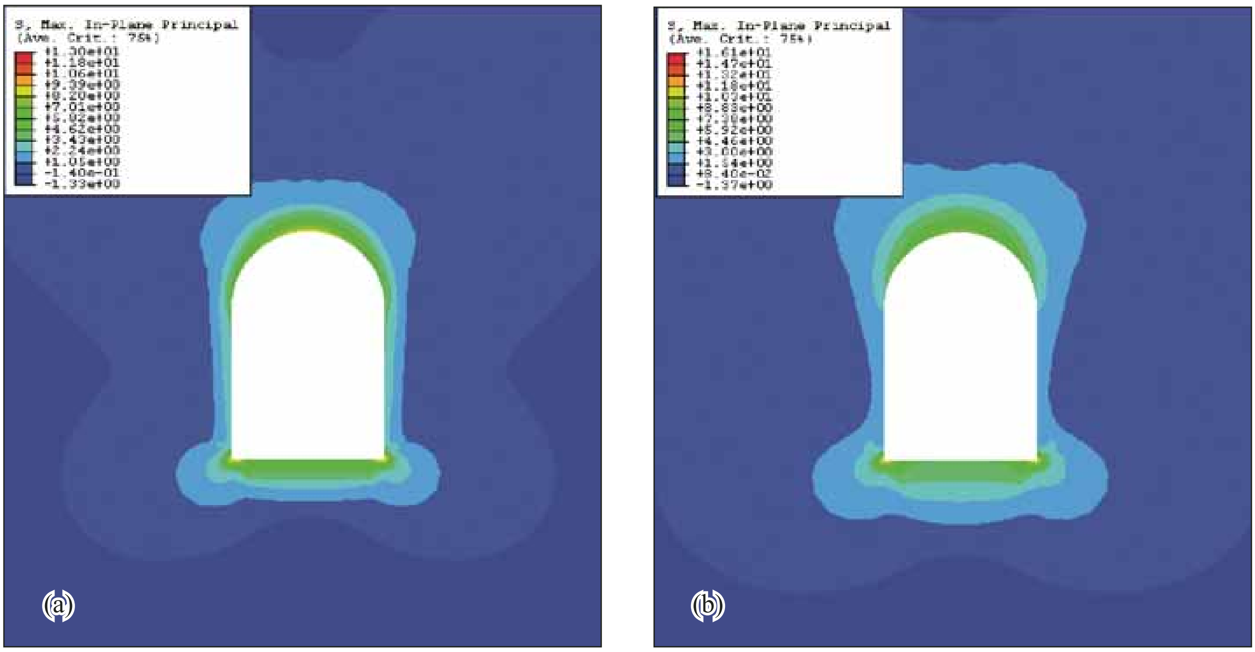

Figure 3. Distribution of major principal stress around a LPG cavern (a) 150 days and (b) 3 years after commence of cooling-down.

3. CASE STUDY – COOLING-DOWN ANALYSIS OF MONGSTAD LPG STORAGE

The Mongstad gas storage facility is located in west Norway and consists of two caverns: One is for liquefied propane and the other is for butane. The storage temperature in the propane cavern is -40.5°C. Two-dimensional numerical simulations have been performed to evaluate the thermally-induced stresses during cooling-down of the propane cavern. Three models are used in the analysis, i.e. a continuous model, a double joint set model and a single joint model. Two cooling schemes, namely 60 and 120 days, are analysed with the continuous model, in which both elastic and elasto-plastic material models are used in simulating the rock masses. The 126 m long cavern has varying cross sections of which the maximum is 21 m wide and 33 m high at the cavern end. The two dimensional numerical model is taken from here. This paper will focus on the continuous modelling.

The sequentially-coupled transient heat transfer and stress computation is performed by using the Distinct Element code, UDEC. The temperature distribution is computed first based on given heat fluxes on the cavern boundaries, which is then followed by the stress computation. Since this is a non-linear analysis, computations are carried out in a step-wise manner with 10 day as the time increment. In other words, the stress computation is performed following each heat transfer computation for 10 days. The Mohr-Coulomb model is adopted as the yielding criterion for the elasto-plastic analysis. Rock bolts and shotcrete are also included in the model. The 4 m long bolts are distributed along the cavern contour in spacing of 1.8 m. The shotcrete is 15 cm thick.

The material parameters used in the simulations are listed below.

- E-modulus [GPa]: 30
- Poisson's ratio: 0.28
- Friction angle [^{o}]: 38
- Dilation angle [^{o}]: 6
- Cohesion [MPa]: 0.5
- Tensile strength [MPa]: 0.7
- Density [kg/m^3]: 2764
- Heat conductivity [W/mK]: 1.64
- Specific heat [J/kgK]: 786
- Thermal expansion coefficient [m/mK]: 6.99E-6

The heat flux applied on the cavern boundaries are estimated by SINTEF Energy and is listed below in Table 2.

Table 2 Applied heat flux

Cooling period	Heat flux [W/m2]		
	walls	roof	floor
60	33.5	31.7	36.9
120	24.9	24.3	27.5

In-situ rock stress is estimated as

$$\begin{cases} \sigma'_v = 0.0171153h & [MPa] \\ \sigma'_h = 0.5 + 0.016056h & [MPa] \end{cases}$$

Figure 4 shows the UDEC model for the analysis. Figures 5 and 6 show the time-history of the temperature during the cooling period. Figures 7, 8 and 9 show the distribution of temperature and principal stress at the end of 60 day cooling-down period. Figures 10, 11 and 12 show the distribution of temperature and principal stress at the end of 120 day cooling-down period. Figure 13 gives the zero isothermal line after 60 and 120 day cooling period. A maximum tensile stress of about 10 MPa and a 5-6 m distressed zone are predicted from the elastic and elasto-plastic computation, respectively.

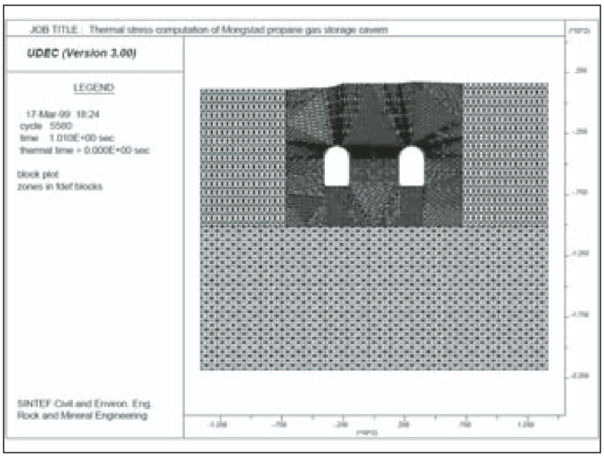

Figure 4. UDEC model.

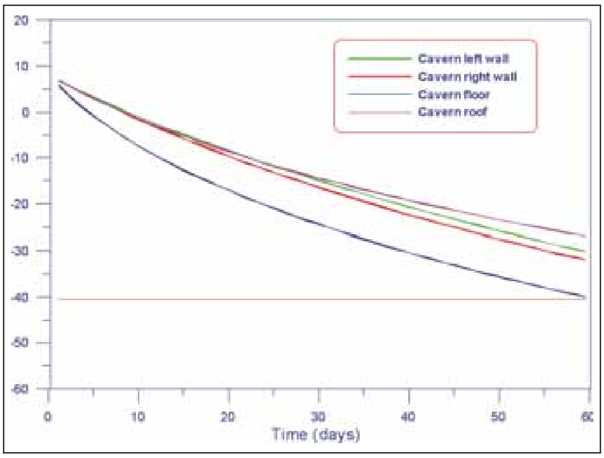

Figure 5. Time-history of temperature during the 60 day cooling period.

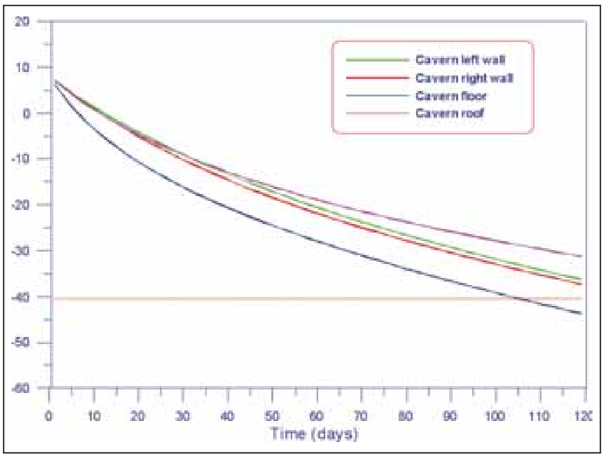

Figure 6. Time-history of temperature during the 120 day cooling period.

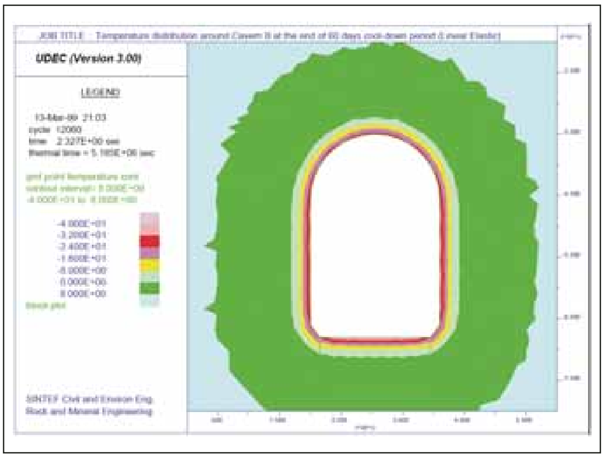

Figure 7. Temperature distribution around the propane cavern at the end of 60 day cooling period.

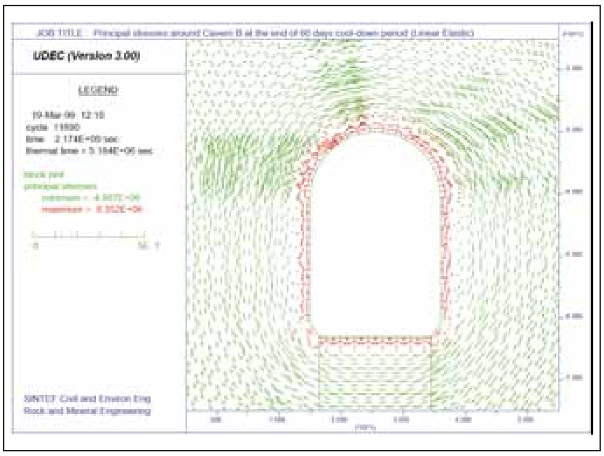

Figure 8. Distribution of principal stress around the propane cavern at the end of 60 day cooling from elastic computation.

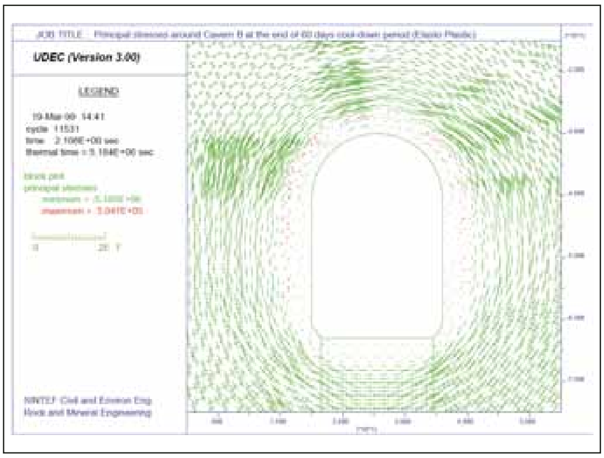

Figure 9. Distribution of principal stress around the propane cavern at the end of 60 day cooling from elasto-plastic computation.

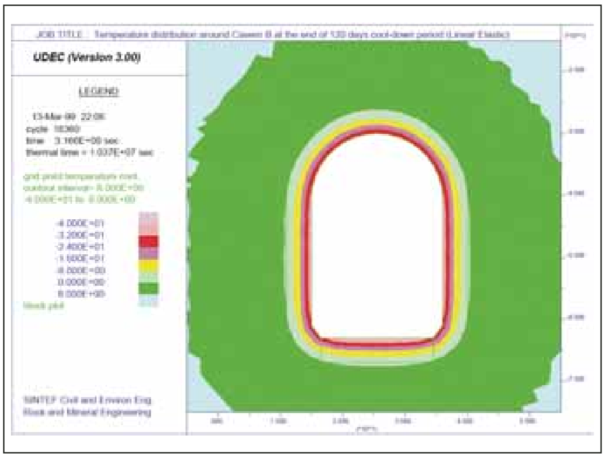

Figure 10. Temperature distribution around the propane cavern at the end of 120 day cooling period.

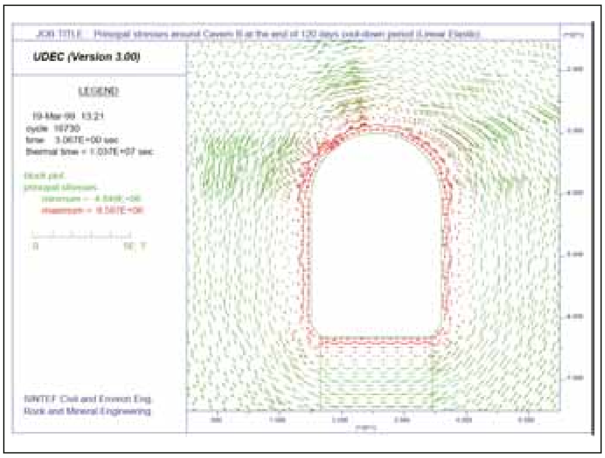

Figure 11. Distribution of principal stress around the propane cavern at the end of 120 day cooling from elastic computation.

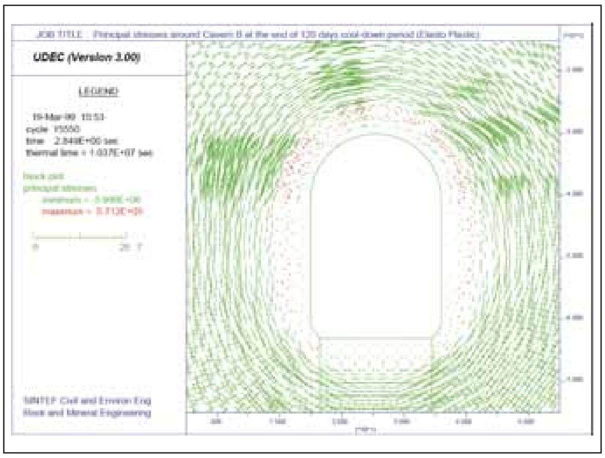

Figure 12. Distribution of principal stress around the propane cavern at the end of 120 day cooling from elasto-plastic computation.

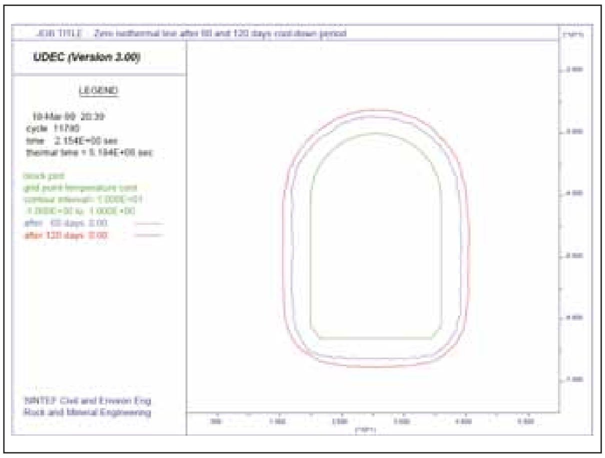

Figure 13. Zero isothermal line after 60 and 120 day cooling period.

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5.1 CAVERN STORAGE EXCAVATION - STURE

Nils Olav Midtlien

THE STURE TERMINAL

The Sture terminal in the municipality of Øygarden in County Hordaland in western Norway is an important port for shipping out crude oil. The terminal receives crude oil and condensate from the offshore fields of Oseberg and Grane.

The oil is received via the Oseberg Transport System (OTS) through a 115-kilometer-long pipeline from the Oseberg A platform and from the Grane field via the Grane Oil Pipeline (GOP) through a 212-kilometer-long pipeline.

The storage facilities at the Sture terminal comprise of a number of unlined rock caverns. Five crude oil caverns of 1 million cubic metres capacity in total, a LPG cavern of 60,000 cubic metres and a ballast water cavern of 200,000 cubic meters. The terminal also has a facility for recovering VOC (volatile organic compounds), which is environmentally important during loading of the oil and gas vessels.

The processing facility at the terminal recycles the lightest components from crude oil, with these being extracted as LPG mix (liquefied petroleum gases) and naphtha. Refined crude oil and LPG mix are stored in caverns and then shipped out.

The terminal has two export jetties. Both of them can accommodate oil tankers up to 300,000 dwt.

Photo: Norsk Hydro

Photo: Norsk Hydro

The Sture terminal also exports LPG mix and naphtha by the Vestprosess pipeline to the Mongstad terminal.

AN IMPORTANT TERMINAL FOR THE NORWEGIAN OIL AND GAS INDUSTRY

The terminal receives crude oil and condensate from a number of offshore oil fields:
Via the OTS pipeline crude oil and condensate are imported from:
- Oseberg Field Center
- Oseberg C
- Oseberg East
- Oseberg South
- Tune
- Brage
- Veslefrikk/Huldra

And via the GOP pipeline crude oil is coming from:
- Grane

The following products are exported from the Sture terminal:
- Oseberg Blend – crude oil and condensate from seven platforms.
- LPG mix – blend of propane and butane.
- Naphtha – consists of pentanes and hexanes, used with crude oil at refineries.
- Natural gas – methane and ethane, used for process heating at the terminal.
- Grane Blend – crude oil from the Grane platform.

Approximately 250 oil tankers and LPG tankers arrive at the terminal per year.

THE REASONS FOR SELECTING STURE AS LOCATION OF THE TERMINAL

The process for placing the terminal including the rock storage caverns at Sture was based on a careful evaluation of several alternatives and by optimisation of a number of factors like:
- Minimum of distance to the Oseberg oil field - to minimise import pipeline length.

Sture Terminal. Photo: Norsk Hydro

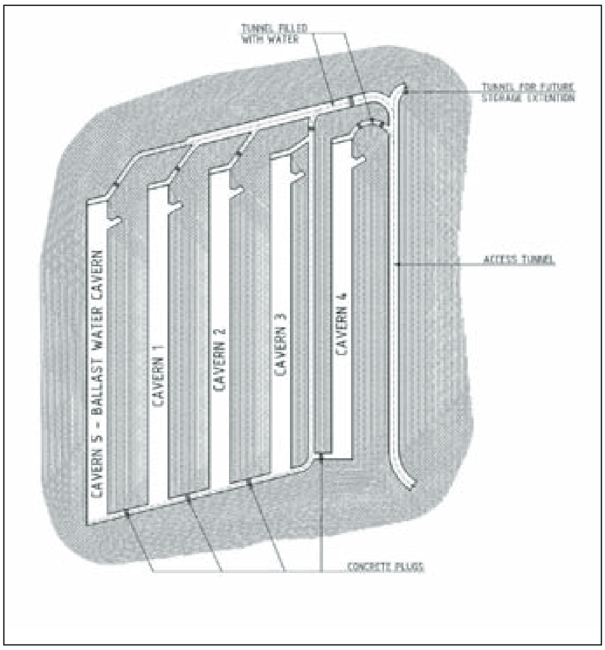
- Possibilities for shore approach of pipelines (via a subsea rock tunnel).
- Possibilities for building jetty for oil tankers up to 300,000 dwt.
- Suitable conditions for rock storage caverns.
- Suitable land area for the terminal.
- Possibilities for terminal expansions, more rock caverns, additional jetties, etc.

DEVELOPMENT STEPS OF THE STURE TERMINAL

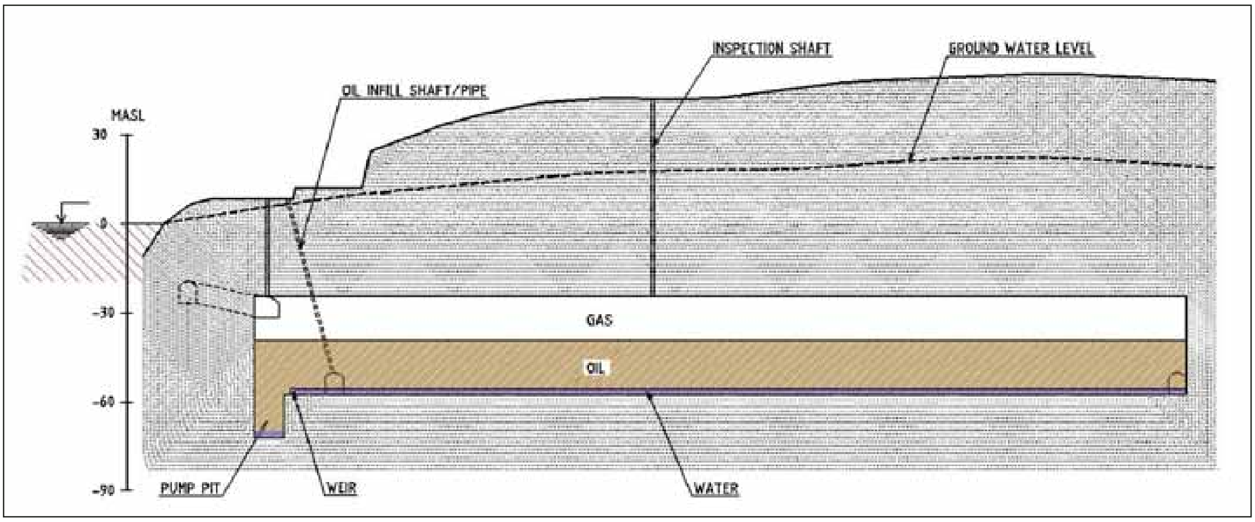
The terminal has been developed in several steps. The design and construction of the terminal started in 1985. The purpose was to establish a transit station for crude oil from the Oseberg oil field.

When the terminal opened in December 1988 it comprised of the following main elements: a 115 km long 30 inch diameter pipeline from Oseberg oil field with a capacity of 91 000 cubic meters daily and 5 rock caverns of 314 meters length, 33 meters height and 19 meters width. Four caverns were for crude oil and the fifth cavern was for storing ballast water from the oil tankers. A landfall tunnel with piercing at elevation –80 meters and a subsea strait crossing tunnel were constructed for bringing the pipeline onshore and towards the Sture terminal. The tunnels were also prepared for an additional future pipeline.

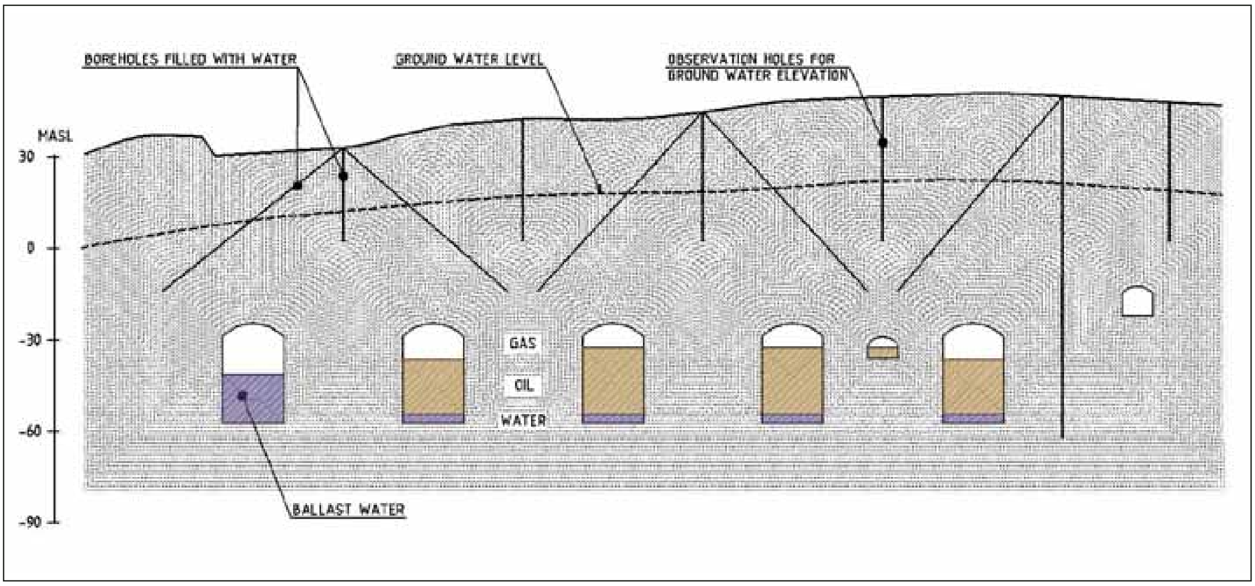
The total volume of the five caverns was approximately one million cubic meters. The terminal was built without any processing facilities and was only serving as a large temporary storage for export of crude oil.



Plan of Crude Oil Caverns



Longitudinal Section of Crude Oil Cavern



Cross Section of Crude Oil Caverns

In the year 1996 the storage facility was increased with one additional large rock cavern for condensate. The volume of the new U-shaped cavern was approximately 300 000 cubic meters. The design principles for the new cavern were similar to the existing storages.

In the year 2000 a process unit for stabilization of crude oil / condensate to ensure correct quality of the Oseberg Blend oil was finalized. The project named Sture Crude Upgrade Project also included a LPG storage facility of 60 000 cubic meters. One jetty was upgraded to handle export of LPG.

The import pipeline from the Grane oil field was finalized in year 2003. The spare capacity for an additional pipeline in the original shore approach tunnel system was utilized. At the terminal the large U-shaped cavern was converted to a crude oil cavern.

STORAGE PRINCIPLES FOR ROCK CAVERNS FOR OIL

The rock mass is not hundred percent watertight. For this reason there will always exists a ground water level. The principle for the oil storing in rock caverns is based on the simple physical law that for oil products lighter than water surrounded by ground water with higher pressure than the operating pressure inside the cavern, there will always be water seeping into the rock cavern. The oil cannot leak out into to rock mass due to the higher external hydrostatic pressure of the ground water.

In Norway most rock cavern storages for petroleum products are based on a principle with fixed waterbed. The storages are also closed, which means that the gases above the oil have no direct communication to the atmosphere. The gas pressure inside the caverns varies as a function of the oil level inside typically from 0.5 – 3 bara. Dependent on the type of crude oil, the pressure inside the cavern can in the long run gradually increase. If so, a portion of the gas has to be flared off.

At Sture three huge product pumps are installed in a pump pit in each cavern for ensuring a minimum of time for filling up the crude oil vessels. In the lower section of the product pump pit it is installed water pumps for pumping out the polluted leakage water. The leakage water has to be treated before it is discharged back into the nature. To minimize the volume and then also the cost for handling leakage water it is important to seal off leakages of water into the caverns during the excavation. The cost for handling of leakage water is related to the pumping cost and construction and operation of a water treatment plant. At Sture it is also a limited volume of fresh water available for artificially maintaining the ground water table.

The ground water table serves as a natural seal. According to regulations in Norway the ground water pressure shall always as a minimum balance the maximum operation pressure inside the rock cavern plus 20 metres (2 bars) as a safety margin.
To be able to ensure that the required elevation of the ground water table is maintained, an artificial system for supplying water is generally introduced. The artificial system can be made by injecting fresh water through 30 – 50 metres long holes drilled from the surface or from a tunnel system at a certain distance above the storage. The tunnel system can be combined with the access tunnel and the complete system can then be filled with water.

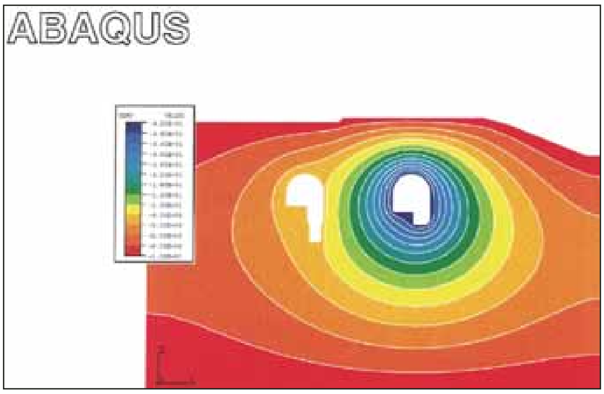
The elevation of the ground water table has to be checked regularly. A simple way can be to drill a number of vertical observation holes above and surrounding the storage area and manually measure the water level in each hole. By regular measurements the yearly variations and the long-term trend can be found.

If the ground water level drops too low, there is risk for the hydrocarbon filled atmosphere inside the cavern to start blowing out. This would be a critical situation for the storage and such a leakage can be very difficult to repair. In best case this will then give restrictions to activities on the area above the storage.

By changing the pressure in the water supply lines it is possible to control the quantity of water injected into the rock mass. This possibility can be important during a long and dry season.

STORAGE PRINCIPLES OF ROCK CAVERNS FOR LPG

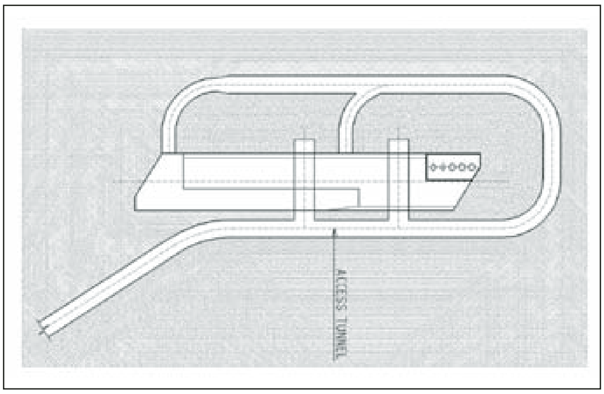
To reduce the volume of the LPG-storage, the gas has to be stored as a liquid. There are two methods used the make the gas to a liquid, by reducing the temperature or increasing the pressure. The most practical way for rock storage is to reduce the temperature to approximately -30˚C at which point the LPG is liquid. The LPG storage needs a special refrigeration unit for maintaining the temperature.



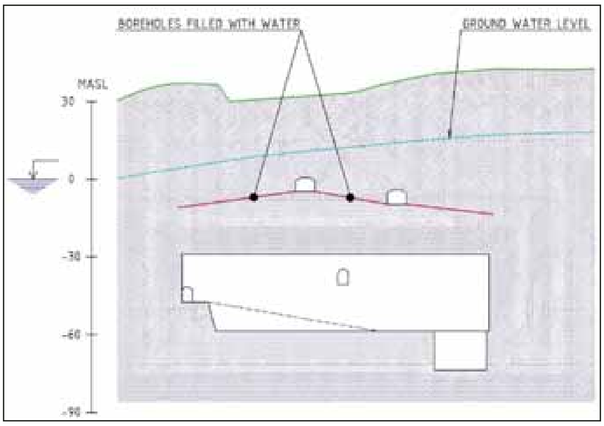
Typical temperature development in Rock Masses surrounding LPG caverns during cool-down of first Cavern. Figure: Norconsult AS

The storage of LPG is basically made on the same main principles as for oil products. The ground water is used as the seal for tightening the rock masses. The difference will be that due to the cold temperatures the water will freeze to ice.

A special cool-down activity is also required for preparing the storage for the first import of LPG. To seal off all water leakages in LPG storages during the cool-down period is fundamental for operation and lifetime. The leakages must be small as the heat energy in the water in a big leakage might prevent it from freezing during the cool-down period. If the leakages are not sealed off during the cool-down period, water continues to seep into the cavern the water will freeze inside the cavern and cannot be pumped out. Due to this build up of ice the LPG pumps can be blocked off and the available storage volume reduced. After some time, the storage can be considered as lost.



Plan of LPG cavern



Longitudinal Section of LPG cavern

GEOLOGICAL CONDITIONS

The rock mass at Sture consists of moderate to medium fissured gneiss. The information about the rock mass for the first caverns was based on field mapping, core drilling and special pump tests for defining the permeability.

The orientation of the caverns was decided based on foliation, fissures etc. to give the most favourable stability conditions and as a result a minimum of rock support.

The location of the first caverns was also selected taking any later extensions of the terminal and additional storage caverns into consideration.

OBSERVATION HOLES FOR GROUND WATER LEVEL

A system of vertical holes was drilled as soon as the location of the storage facilities was finally decided. The purpose was to collect detailed information about the ground water level. The data is important for the design of the Ground Water Control System. The observations show also the magnitude of the variation of the ground water level locally.

ARTIFICIAL GROUND WATER CONTROL SYSTEM

After a detailed review of the geological information, a system consisting of 54 mm diameter bore holes up to more than 50 meters in length was designed to ensure that the natural ground water level in the area could be maintained, despite the excavation of the caverns below, which could easily drain the area. The orientation of holes was selected with focus on crossing as many cracks and fissures in the rock mass as possible.

The ground water control system for the crude oil storage caverns was established from the surface. The advantage was the unlimited number of possible access points for drilling. A pipeline system between the holes and group of holes was located in trenches. The water supply system had to be protected from frost. Groups of holes were drilled almost horizontally above and vertically around the storage area. The storage caverns were located under the top of a hill and at the ends where the terrain dropped vertical holes were drilled to form a barrier to avoid the ground water table to drop.

The pressure in the ground water supply system was set to maintain a certain ground water table. The supply system was split into sections, which were controlled by a pressure reduction valve, manometer and a flow meter. To ensure the water supply and pressure a water tower of 1800 m3 was established.

There were two reasons for splitting up the water supply system into sections.
Firstly, to allow for smaller sections to be sealed off during the following grout mass injection work in the caverns. This to avoid any grout masses to spread out into larger parts of the system. As a precaution it was also set a limitation to the volume of grout masses to be injected in each round of grout injection/injection hole.

Secondly, if a drop of ground water level should occur at a later stage, it was decided to include the flexibility to be able to locally increase the water pressure in the supply system to feed in more water to try to increase the ground water level in that particular area.

To increase the possibilities to observe water leakages the ground water control system was put into operation at least 50 metres ahead of the excavation of the top heading. Without the artificial water supply the area could locally be drained before the geologists arrived at the tunnel face for making their observations.

The long small diameter boreholes are a challenge for the contractor due to deviation and must therefore be carefully drilled. A deviation of 18 meters in a less than 50 meters long hole was documented.

In future projects with ground water control holes established from a tunnel above the caverns the design could be further improved by also filling the tunnel with water, at least partly, prior to the cavern excavation. Due to constraints in the construction schedule, the filling of water might not be possible prior to the excavation of the top heading of the caverns but it should be filled with water prior to start-up of excavating the upper bench.

REGISTRATION OF GROUND WATER LEVEL

The registration of Ground Water Level should continue during the lifetime of the storage facility. After the construction period the information gained should be used as basis for how often the registrations should take place. Once per month could be a reasonable frequency. The requirement from the Authorities regarding the ground water level should be documented.

The regulations for oil and gas terminals give clear restrictions to the activities taking place within the area. This must be considered in the design of manholes for flow meters, manometers etc. to avoid that collecting that data become a task involving many people and requiring a lot of additional activities. An example is that prior to entering a manhole, gas measurements should be performed by a person with documented qualifications.

EXPLORATORY DRILLING DURING EXCAVATION

In the storage facility the rock masses 0 – 5 meters outside the storage volume functions as a sealing membrane. The scope will then be to make the "membrane" as impermeable as possible. The purpose of the extensive exploratory drilling program was to detect as much as possible of leakages and seal them off prior to excavation. The exploratory drilling was done systematically based on a pre-designed pattern and procedure.

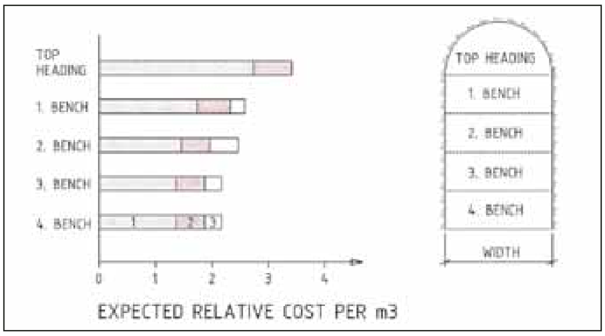
The acceptance criterion for leakages is for oil and gas projects stricter than for most other type of projects. The petroleum products will pollute the water seeping into the storage facility. The water has to be pumped out with a water head of typically 80 – 100 meters. A treatment facility must also be available for cleaning the water.

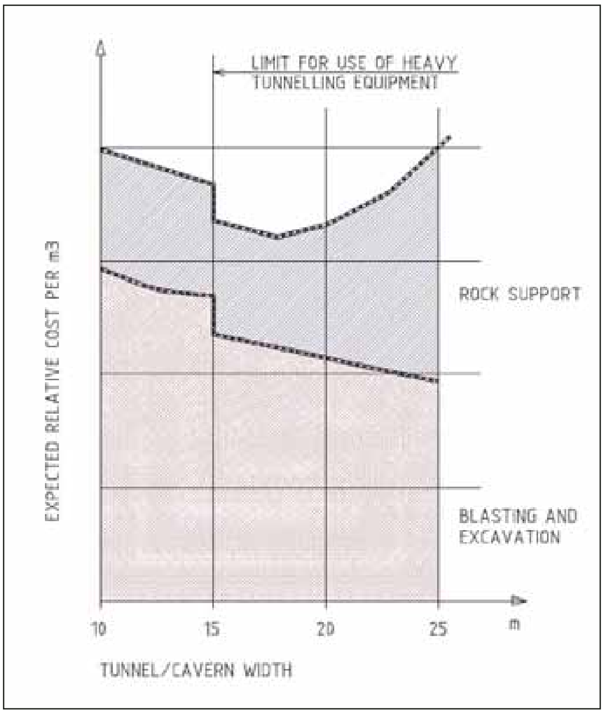
CAVERN DIMENSIONS

For the crude oil caverns the dimensions were optimised in the pre-design phase.
Based on the expected quantities and cost for:

- rock support.
- rock mass sealing (grouting)
- rock excavation
- capitalized cost for pumping out the product

The lowest total cost for the storage facility was calculated to be for a cavern width in the range of 18 to 20 meters and a cavern height in the range of 32 to 34 meters. The selected dimensions were 19 and 33 meters respectively.





Each of the caverns was 314 meters long. At one end a 15 meters deep pump pit was excavated. The pit in each cavern was made as an extension downwards of an end wall, which gave a total height of up to 48 meters in creating high tensions in the rock mass.

The pits should preferable have been moved at least 10 – 15 meters into the caverns to reduce the size of the highest end walls. This would have reduced the com-

Typical Cavern Cross Section
Photo: Norsk Hydro

Sleeves in Pump Pit, Weir in front of Pit
Photo: Norsk Hydro

plexity and extent of the rock support. When the simplification was realized it was too late for redesign due to restrictions set by the installations above ground and the ongoing drilling of shafts. Lack of rock cover was a hindrance for keeping the location of the pump pit and extending the cavern some 10 to 15 meters in length.

SHAFTS

To each of the 5 caverns it was drilled 9 shafts. The tolerances for the shafts were strict. In shafts all equipment like pumps and instrumentation should be free hanging from the top flange just above terrain level. The contractor, Selmer (Skanska) and their sub-contractor Entreprenørservice drilled all the 45 shafts in length of 35 to 68 meters and the largest deviation was 300 millimetres. Most shafts had a deviation less than 100 mm.

VERTICAL SHAFTS ABOVE THE PUMP PIT

Instrumentation:	1 pcs with diameter 2100 millimeters
	1 pcs with diameter 600 millimeters
Crude out:	4 pcs with diameter 2100 millimeters (one spare)
Water out:	1 pcs with diameter 2100 millimeters

In all shafts a steel sleeve was installed starting with a flange above ground level and down to the storage. The sleeves stopped approximately where the shafts entered into the caverns.

The contractor planned very well the installation of steel sleeves in the vertical shafts. The largest sleeves with a diameter of 1550 millimetres and length of 36 meters arrived at the site in full lengths. The sleeves were installed and ready for external concreting within 3 days.

SHAFT FOR INFILL PIPELINE

Infill pipe: 1 pcs with diameter 1000 millimetres
The shafts for the infill pipelines were drilled down to a niche at the bottom level of the caverns. From the niche the pipelines were buried in a trench to the opposite end of the caverns. The only exception was the cavern, which was combined with the transportation tunnel. Here the infill pipe ended up at the upper section of the tunnel.

SHAFT FOR INSPECTION

Inspection: 1 pcs with diameter 1000 millimetres (located at centre of the caverns)

At the centre of each cavern a shaft was designed for inspection purposes. The inspections should take place after the concrete plugs were concreted, the manholes finally closed and the external tunnel system filled with water. The shafts were used for two different purposes:

1. General inspection of the water tightness of concrete plugs.
2. For access to bring out a test plug from one cavern.

If a water leakage of unacceptable extent had been detected, access via the inspection shaft is the only way to identify the type of leakage and point out exact location. This is of utmost importance for being able to know how to plan and perform the improvement of the water tightening of plug or rock masses.

At Sture a test plug was lost in one of the caverns during the commissioning phase. The cavern was already filled with 15 meters of water. Divers, equipment and an inflatable rubber boat were lowered down the inspection shaft. The divers used some extra time to find the black coloured test plug and bring it out.

DESIGN CRITERIA FOR STEEL SLEEVES

A few meters at the upper end of all of the shafts the steel sleeves were designed as pressure tanks. The lower and main section of all shafts is considered as part of the cavern system. In the design it is considered that the surrounding concrete and the rock mass directly support the steel sleeves. The external pressure during concreting was the design parameter for the lower section of the sleeves.

ROCK EXCAVATION

The entrance tunnel and the transport tunnel between the caverns were both 10 meters wide to allow for two-way traffic. The entrance tunnel had also room for 3 large diameter ventilation ducts. The branch tunnels were designed for one-way traffic. See plan sketch of the 5 crude oil caverns.

The caverns were excavated with one top heading and 3 benches. The two upper benches were drilled and blasted using ordinary tunnelling jumbos. The lowest bench was drilled and blasted by means of vertical boring jumbos.

The pump pit had a horizontal cross section of approximately 100 square meters. Due to the depth of 15 meters, it was excavated in 3 steps. The excavation was performed without using a ramp. All equipment, rock mass etc. had to be lifted in/out.

ROCK SUPPORT

The caverns were systematically supported in the roof and pump pit area by means of shotcrete and rock bolts. Additional rock support was added where required due to the local rock conditions. The rock bolts used were of a type with mechanical anchor and could be later grouted. This was a very satisfactory solution.

VENTILATION – A MAJOR CHALLENGE IF SEVERAL CAVERNS

The main challenge during excavation was to control the airflow in the tunnel and cavern system. This was achieved by using a system blowing air into all 5 caverns. In addition a fan was located on the terrain and using the inspection shafts at the centre of each cavern. These fans extracted effectively the polluted air out from the caverns. Problems due to exhausts from equipment and gases from blasting operations did not spread out. Therefore, a lot of activities could continue in all other caverns and tunnels during heavy intensive excavations and transportations and with a minimum of delays due to blasting operations.

CONCRETE STRUCTURES IN CAVERNS

There are few concrete structures in the caverns.

- A small weir close to the pump pit
- Consoles 5 meters down in the pits supporting a 90 mm titan steam pipe
- Bend and end fixation of the infill pipelines
- Perforated slab at the bottom of pump pits

In total it was used less than 100 m3 of concrete in each cavern.

CLEANING OF CAVERNS AND PREPARATIONS FOR CLOSING

To avoid pollution of the first batch of crude oil to be shipped out, the caverns must be cleaned carefully by flushing of water just prior to closing them off by means of concrete plugs. The water bed was filled with water and several inspections performed to bring out all floating materials, which later could destroy any oil or water pump.

CONCRETE PLUGS

The caverns are divided into separate storage units by means of concrete plugs. The storage has eleven concrete plugs, which are designed for the following purposes:

- 5 plugs between water filled tunnels and storage caverns
- 2 plugs between water filled tunnels and the entrance
- 4 plugs between the different storage caverns

Concrete plugs separating the storage cavern from the water filled tunnels were designed to be as watertight as possible. The main design parameters of the plugs were the external water pressure and gas deflagration inside the caverns. The deflagration could give a maximum pressure of 8 bars.

The final location of each plug was decided based on the local geological conditions. The support of each plug was also improved by a carefully blasted notch around the perimeter of the centre of the plugs.

Two of the plugs had a cross section of approximately 100 m^2 and the others had a cross section of around 50 m^2. The largest plugs had a thickness of four meters and the others 3 meters. The plugs had all an access by means of a GRE pipe with diameter 800 mm and 600 mm respectively. Blind flanges, also made by GRE, were used to permanently close the access pipes through the plugs.

The pipes were utilized as access for the following purposes:

- To be able to bring out all formwork from the cavern side of all plugs
- Control of the injections work of the concrete plugs
- Manual removal of all floating materials from the water bed

Prior to concreting the plugs, perforated tubes were fixed to the rock. The tubes were placed in four to five rings spread out over the thickness of each plug. The tubes in each ring had a length of 4-5 meters and were placed with 20-30 centimetres overlap. Each tube was carefully marked and one end was connected to an air pump. Air was pumped through all tubes during the concreting works. By this clogging of the tubes was avoided. After the concrete was hardened and the temperature shrinkage almost stopped, the tubes were systematically grouted. First, the two outer tubes on each side were filled with polyurethane to set up a barrier towards the free surfaces. Later, the inner tubes were grouted with epoxy.

The temperature gradient through the plug and towards the surrounding rock mass should be kept as small as possible to minimize extent of cracks. The formwork was insulated to reduce the heat loss. Plastic sheeting was also utilized to avoid the surfaces of drying out. A number of temperature sensors were located at several places inside the plugs. The temperature development was recorded and when the preset criteria were met, the insulation was removed. If time had been a critical factor, an alternative to insulation could have been to design a cooling pipe system inserted in the plugs and used circulation water to bring down the core temperature.

WATER FILLING OF TUNNELS

When all formwork and construction materials were moved out the manholes were closed following to the procedure given by the GRE pipe supplier, and the tunnels were filled with fresh water.

WATER LEAKAGE INTO THE CAVERNS

The acceptance criteria for leakage of water into the 5 caverns were maximum 18 m3/hour in total.

The early indication based on loss of water from the artificial ground water control system and the volume of water pumped out of the caverns gave figures far below the acceptance criteria. Anyhow, to verify the functionality of the concrete plugs and confirm that there were no major leakage concentrations from the water filled tunnel system, the concrete plugs were inspected.

The plug areas in four of the caverns were located in branch tunnels at the elevation of the top heading of the caverns. The only way to reach the plugs was by climbing up the 25 meters high sidewalls to branch tunnels. The inspection was prepared for by installation of special bolts and special ropes for climbing. A special team performed the inspection.

The conclusion from the inspection was that the plugs were all working perfectly. If any leakages had been observed, the ability of having access to the inside off the plugs in the water filled tunnels is fundamental for defining the required repair work. If not, questions like: Where, how, how much etc would not have been possible to answer.

If the leakages are too close to or above the design limit the water in the tunnel system outside must be pumped out and repair work performed. The repair work requires mobilization of an almost complete set-up of tunnelling equipment. The most likely weak points with respect to leakages are the areas with the concrete plugs. If contractor is able to collect the required information about the leakage prior to emptying the tunnel system a delay of 2-3 months must be expected.

In future projects it is an advantage if the inspection shafts could be located ending up close to the inside of the concrete plugs. The complete surface of the plugs should also be possible to observe with a camera lowered down the inspection shaft. If the plug can be inspected by means of a camera the more high-risk activity of a manual inspection can be avoided.

At Sture after almost 20 years in operation the water consumption in the ground water control system and the quantity of water pumped out of the caverns are relatively stable and well within the original design parameters.

Inspection of Cavern
Photo: Norsk Hydro

Re-opening of Access Tunnel
Photo: Norsk Hydro

Re-grouting of Plug after too high leakages
Photo: Norsk Hydro

INERTING OF CAVERNS PRIOR TO START-UP

To obtain a oxygen free atmosphere before the caverns were filled with product, the first 5 crude oil caverns were inerted by diesel exhaust gas prior to put into operation. The procedure used was first to fill one of the caverns up with water. The water was pumped over to the neighbour cavern. This operation was repeated for all caverns. Diesel exhaust gases were successively filled into the caverns when the water was pumped out.

For the LPG storage the cavern and surrounding rock mass was cooled down with air. Inspections were performed twice a week to monitor the movements in the rock mass via mini extensometers. Observations of any ice formations were performed and noted for controlling when water leakages, which caused ice formations, were frozen. The time and energy needed was in advance carefully calculated to optimise the cooling equipments, both for the cool-down period and for the permanent operation of the storage.

EXPERIENCE FROM OPERATION

The Norsk Hydro operative organization at the Sture Terminal is very satisfied with the rock storage caverns. All caverns are after almost 20 years still operating without any problems. The water consumption for the ground water control system and the energy consumption for the LPG cavern have both always been within the design criteria.

Regarding maintenance, there has not been any cost at all related to the rock caverns. The maintenance performed regarding the caverns is only related to the mechanical equipment.

5.2 VPPC - VESTPROSESS PROPANE CAVERN PROJECT
Storage of liquid propane at atmospheric pressure in an unlined rock cavern

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John Jørgenvik Statoil/Vestprosess
Lars Murstam MIKA AS
Sven Oen MIKA AS

ABSTRACT:
In the autumn of 2001 MIKA AS was contracted by Statoil, main operator of the Vestprosess project, to construct a propane storage cavern. The cavern, located at Statoil's Mongstad refinery, has a total volume of 62,000 m3 and is intended to be a supplement to the existing propane cavern at the plant. The new cavern has been designed and constructed in much the same way as the earlier one, although with the application of different approaches for sealing off water inflows and for cooling the cavern. To minimise leakage of water into the cavern, stringent grouting standards were imposed. As for the cooling process, instead of using direct propane cooling, the cavern was first cooled using air and then cooled further with propane. The cavern has the shape of a "lying bottle", with the entrance to the cavern in the neck of the bottle. Access to the cavern is through a 600-metre long access tunnel at a grade of 1:7. The cavern is sealed with a concrete plug in the neck of the bottle.

1. INTRODUCTION
The Vestprosess project with its pipeline from Kollsnes and Sture in Øygarden to the process plant at Mongstad, north of Bergen, presented Mongstad with new challenges – the process plant became a major exporter of propane and butane. The "home market" (Northern Europe) was not large enough, and there was therefore a need for storage capacity close to the process plant that was of sufficient volume to fill the largest freighters. In 1999, a cavern with 60,000 m^3 of storage space for propane was constructed together with a similar cavern for butane.

The propane storage cavern was fully cooled, that is to say that the propane was stored in liquid form at atmospheric pressure and a temperature of -42 ˚C. In this propane cavern, the sealing and support work prior

Fig 1. Process plant at Mongstad

to cooling was inadequate, resulting in a rock fall from the roof and an inleakage of about 20,000 m3 of water with subsequent ice formation after the cavern had been filled with propane. Through the experience gained during the first year of operation, the leakage was stopped and the cavern was able to be kept in service – but with an available storage volume of only 40,000 m3. However, the ice in the rock cavern caused substantial uncertainty as regards equipment and installations in the cavern. Consequently, it was necessary to procure new storage space by constructing another cavern.
An investigation and review of the events in the first propane cavern resulted in there being every confidence in the concept, provided that the new cavern was constructed to satisfactory standards.

From its opening in 1975, the Mongstad refinery has concentrated on the storage of oil in rock caverns. Today, 26 caverns containing crude oil and products ranging from heavy fuel oil to propane are in use at Mongstad. The temperatures vary from 70 ˚C to -42 ˚C, and some storages caverns are constructed for pressures of up to 6 bar.

For 28 years this storage method has proven to be a safe and reliable method. With high quality pumps and equipment in the caverns, the maintenance of a cavern solution is much less costly than the maintenance of a combination of tanks and pumps above ground. The general level of safety is higher when the product is 25-100 metres below ground than when it is stored in huge tank farms above ground.

Good experience combined with low-cost construction and operation and an excellent safety record are the main reasons why propane cavern CA-6106 has been constructed underground. It is the 27th rock cavern at Mongstad, which highlights just how satisfied the user is with the rock cavern product.

2. DESCRIPTION

The concept and project design

An impermeable zone of ice around the cavern seals off water inflows into a fully cooled propane cavern. The frozen zone is established by maintaining a water pressure in the rock around the zone, ie, a water curtain with a supply of water from the surface. This network of water supply over and around the cavern fills all new cracks and freezes to ice nearest the cavern surface. Regular checks are made to ensure that the water column in the control wells around the cavern has a sufficient level to prevent gas leakage into the ground.

In the case of the new propane cavern, it was decided that the cooling should be done in two stages: first with air until the 0-isotherm in the rock had reached three metres, and then further cooling with propane until the temperature in the cavern had reached an operating temperature of -42 ˚C. With this solution, it was hoped that any stability and inleakage problems would occur whilst there was still an atmosphere made out of air in the cavern, and it was thus possible to implement measures before it was too late (ie, before a propane atmosphere had been established in the cavern).

Strategy to minimise water ingress

To make the cooling as efficient and simple as possible, it was decided that steps should be taken to minimise water ingress into the cavern before the start of the cooling process. As a design criterion, inleakage was therefore set to be < 15 l/min for the whole cavern.

Statoil and MIKA chose a solution involving systematic pregrouting as the most certain way of reducing the inflow of water into the cavern. A grouting programme was drawn up on the basis of a philosophy of "the simpler, the better". The range of mixes was kept to a minimum, W/C=1 and W/C=0.8. All grout material was micro cement of the type Rheocem® 900 and the only additive was the superplastisizer Rheobuild® 2000PF. Stop pressure for the grouting was set at 80 bar. The decision to utilise this grouting material was made due to its fast setting properties and good penetrating capability.

The basic principle for the rock mass grouting works was that the rock should be as watertight as possible 5 metres outside the 0 isotherm, ie, 8 metres outside the contour. Maximum distance between boreholes in a grout fan at maximum look-out was not to exceed 2.2 metres. On the basis of these criteria, and highly varying cavern geometry, each fan of grouting holes had its own configuration. The cavern was blasted out at different levels (top heading and benches), and the pregrouting resulted in simultaneous pregrouting of the next level. See Figure 2.

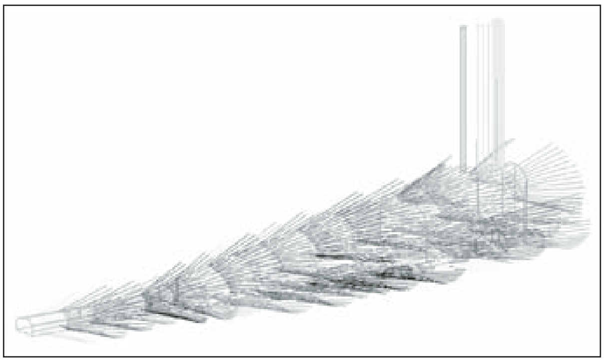


Figure 2. Grout fans

All told, 30,000 metres were drilled for grouting. The length of the fans varied slightly but in the main they were 24 metres long.

Rock support

The rock support application in caverns blasted out at different levels can be a challenge as regards finding the appropriate amount of support to provide stability. The rock in the area consists of light and dark anorthosic gneiss, amphibolite and gabbro. It was important to take the freezing factor into account. How would, eg, bolts, behave at -42 ˚C? MIKA was asked to draw up a proposal for rock support classified in support classes. In consultation with Geo Bergen, MIKA chose a classification solution on the basis of sequential Q-value calculations – so that a given Q-value indicated a particular support level. The solution involved four rock support classes:

A1 good quality rock
- Q>10. Bolt pattern 2.5m x 2.5m and fibre-reinforced shotcrete 8cm - 10cm

A2 fair quality rock
- 10>Q>4 Bolt pattern 2.0m x 2.5m and fibre-reinforced shotcrete 10cm - 12cm.

B1 Poor quality rock
- 4>Q>1 Bolt pattern 2.0m x 2.0m and fibre-reinforced shotcrete 10cm - 15cm.

B2 Very poor quality rock
- 1>Q>0.1 Bolt pattern 2.0m x 1.5m and fibre-reinforced shotcrete 15cm - 20cm.

The bolt lengths varied between 4 and 5 metres. Five-metre long bolts were used in the roof whilst 4-metre bolts were installed in the walls from the abutment and downwards. All bolts were fully grouted 25 mm diameter re-bar bolts. In addition, CT bolts were used to provide temporary support. In 85 % of the cavern, rock support was provided according to support class B1. Local variations were covered by the temporary support.

Creep and contraction of the steel bolts as a result of the cooling was not regarded as having any particular impact on the stability of the rock cavern because of the relatively high safety factor. Birgisson (2002).

Groundwater control
To maintain a groundwater pressure in the area around the propane cavern, about 2000 metres of drilling was done using down-the-hole drilling equipment to establish water curtains. Water-operated down-the-hole drilling equipment was described in the contract, but was found not to work satisfactorily in the rock mass at Mongstad. It was therefore decided to use pneumatic down-the-hole drilling equipment with water flushing to ensure safe drilling in rock which might contain gas pockets from existing adjacent caverns. The water curtains consist of a horizontal water curtain drilled from a side tunnel above or adjacent to the cavern and a vertical water curtain drilled from the surface. The boreholes have a diameter of 4". It was intended that the water curtains should be established and put under slight pressure before cooling commenced. This was to ensure that surrounding areas and solid rock would be filled with water, and become watertight when the water froze to ice.

3. EXECUTION OF THE PROJECT AND EXPERIENCE GAINED
The final size of the cavern was a height of 34 metres, a width of 21 metres and a total length of 134 metres. A pump pit is located immediately below shafts at the end of the cavern. The blasting was carried out by excavating the top heading first and then excavating the remaining volume with two horizontally drilled benches. Access to the cavern is through a 600-metre long tunnel excavated at a grade of 1:7.

The cavern was sealed off in the neck of the bottle by a concrete plug. The plug has to meet the same requirements as regards water-tightness as the rest of the cavern, and was a challenge in itself.

Infill of propane into the cavern was enabled through vertical pipes, fully grouted into raise bored shafts of about 70 metres in length. A total of six shafts were drilled to meet the need for infill and outpumping of product, instrumentation and the like. The infill process for cooling would take place via a spray system mounted in the roof.

Grouting
As it had been decided to carry out systematic pregrouting, no probe drilling was done as it would only be of academic interest.

The actual grouting was basically done using two separate pump units. In addition, a reserve pump was available at all times. The mixing process and registration of the grout takes was computer controlled. To withstand the high grouting pressures, special disposable packers were used that were reinforced with double back plates and extra locking rings. All valves and hoses were approved for the high grouting pressures. The expansion of packers placed in holes were performed hydraulically.

After each completed grout fan, a round of control holes was drilled. The number of control holes and their location was determined on the basis of the course of grouting of the main fan. In some instances where there were large grout takes in the round of control holes, a second round of control holes was drilled

Special grouting measures were implemented around the pump pit, end wall and shafts. The rock mass around the shafts was in addition pre-grouted from the surface.
Once blasting and grouting had been completed, water leakage measurements were made. The result obtained was 2 l/min, well within the requirement of 15 l/min.

Rock support
The poorest rock was encountered at the end of the cavern. The extent of CT bolt support was greatest here. The permanent rock support remained as planned without taking into account to any appreciable degree the

temporary support that had been installed. In addition, more shotcrete was applied in this area than planned.

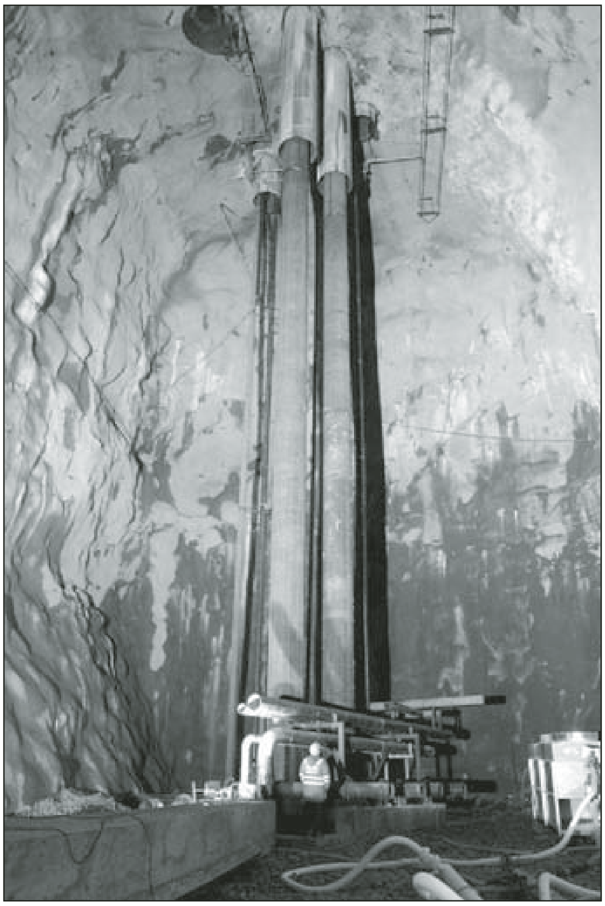


Figure 4. Installation in shafts

Shafts
Six shafts were raise bored for technical installations. The shafts have a diameter of up to 2.1 metres and each one is about 70 metres in length. The shafts are designed to be used for tasks such as the pumping in and out of propane, as well as instrumentation and various measurement readings.

Some grouting was carried out around the shafts, and casings were installed and cast in place. Some ingress of water could be observed in the rock/ concrete contact after the concreting. This was sealed by grouting from the inside of the cavern by first using polyurethane foam. Once the "barrier" had been established, 500 litres of Meyco MP320 silica gel was injected. The injection hoses in the steel/concrete contact were also injected with silica gel and epoxy.

The concrete plug
The concrete plug positioned in the entrance of the cavern was initially a simple 7-metre long concrete structure with a manhole. It was to be cast after the air cooling had commenced. MIKA wanted to have access to the cavern throughout the cooling process, and together with Statkraft Grøner they designed an access port which allowed access for small machines and equipment. This solution was also favourable from a safety point of view.

The plug was cast in three main parts and also included an extensive grouting programme. The last task to be done before the propane filling commenced was to cast the plug completely and fill the access tunnel with water. Cooling circuits were established for active cooling of the concrete plug so that water seeping into or around the plug would turn to ice and prevent leakage.

Figure 5. Cooling fans in the cavern

Cooling
The cavern was ready for the cooling process in April 2003. The cooling was initially an option for which MIKA had the best offer. Calculations were made in collaboration with Statkraft Grøner and Teknotherm, who also supplied the actual cooling plant. The cooling plant was installed just by the portal in a tent.

The plant is based on an ammonia coolant which, via a thin-film evaporator, cools down a CaCl2-brine. The cooler compressor has an output of about 700 kW. Pipelines for the brine circulation were installed from the portal to the cavern and eight evaporators (each having 3-4 fans) were installed in the cavern. The piping had to be adapted to low temperatures, large pressure loads, and relatively large fluid flows. The challenges were many when starting up the plant. Defrosting evaporators had to be adjusted so as to prevent the cavern from receiving excessive heating effect. In total, there was about 450 kW defrosting power. There had been plans to add methanol to the pump pit so that leakage water in the cavern would not turn to ice and could be pumped out of the cavern. However, MIKA solved this problem using other methods, and thereby avoided the use of methanol during the freezing process.

To be able to follow the progress of the 0-isotherm inwards through the rock, about 80 temperature sensors were installed at different levels in the rock: 0.5 m, 1.5 m, 3 m, and 6 m. The reading of the sensors was carried out automatically via Statoil's computer system at Mongstad. Readings were checked against computed values and showed almost the same trends as theoretically computed. The coefficient of thermal conductivity of the rock also corresponded well to the estimated value.

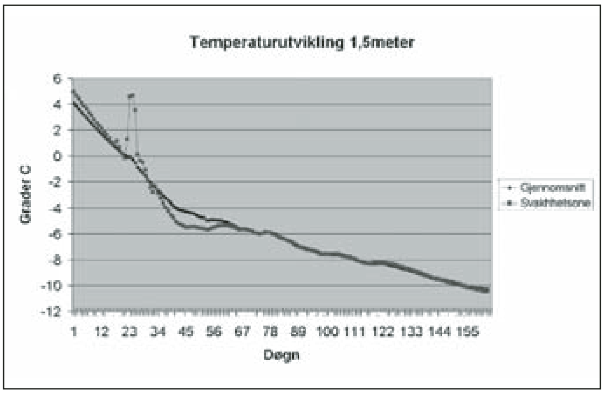


Figure 6. Development of temperature at 1.5m

After about one month of cooling, an area was discovered close to the pump pit where the temperature sensors did not show the same trend as the others. There was ingress of water and a section of the rock of about 100 m2 began to crack. The area was classified as B2 – very poor quality rock, and was well supported in accordance with the Q-classification.

Corrective measures involving cooling targeted towards the weakness zone were implemented.

Although targeted cooling of this section of the rock was carried out, the fracturing continued. Grouting and direct cooling in the rock were two other measures that were considered, but on the basis of factors relating to safety and performance, neither of the alternatives was implemented. It was decided to continue the cooling whilst observing conditions in the weakness zone. It turned out that there was a fracture zone in the water-bearing zone. The water froze to ice which pressed this section of rock inwards into the cavern. Temperature sensors in the area showed a rise in temperature when water entered the zone. See Figures 6 and 7. Measurements were taken regularly to have control over movements in this rock mass portion.

In October 2003, long after 0-isotherm had passed the prescribed three metres in the whole cavern, there was a rock fall of about 150 m3, including blocks as large as 25 m3. The area was inspected and then secured and the masses were removed. A gabion support wall was built and a safety fence was put up near the pump sump to protect installations from any new rock falls from the same zone. At the same time, some of the installations were modified and moved away from the area.

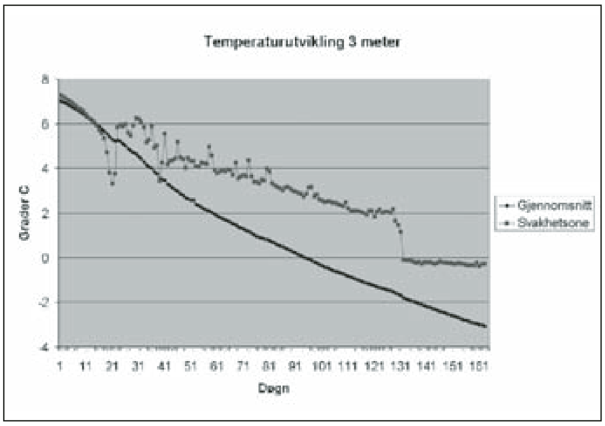


Figure 7. Temperature development at 3m

The use of air cooling prior to further cooling with propane meant that it was possible to gain access to the rock cavern in the first phase of the cooling. This allowed visual control and monitoring and the possibility of implementing measures to ensure an optimal end product.
This would not have been possible with direct cooling using propane, and shows that the choice of a concept involving air cooling was right for this project.

Figure 8. Mika built a jetty of rock masses from the rock cavern.

FACTS:

• Volume blasted	about 100,000m3, including access tunnel
• Shotcrete	about 2000m3
• Drilling for grouting	about 30,000 drilling metres
• Grouting mass	about 420 tonnes
• Drilled shafts	about 400 metres, diameter 2100mm.
• Construction concrete	1600m3
• Guided drilling/long-hole drilling for grouting and water curtains	about 4000 metres
• Number of working hours	about 70,000 -1 injury resulting in absence.

KEY PARTNERS:

• MBT Degussa	Grouting
• Entreprenørservice	Drilling of shafts
• Nor Betong	Concrete deliveries
• Vestnorsk Brønnboring	Drilling of groundwater system
• Norconsult	Project design
• Statkraft Grøner	Project planning for concrete plug and cooling
• Teknotherm	Delivery of cooling plant
• Rescon Mapei	Various grouting jobs
• Fjell Industrier	Casings for shafts
• Geo Bergen	Geological surveys, assessment of rock support

6. OWNERS CONTROL SYSTEMS FOR THE IMPLEMENTATION

Bjørn Helge Klûver
Ola Jegleim
Nils Borge Romslo

1. OWNERS CONTRACT PHILOSOPHY

Owners Contract Philosophy will be decisively for the arrangement of the control system at site, and the way the Site team is organized.
A "Hands on philosophy" has been a normal way for execution of large Norwegian underground civil works for the oil-and gas industry. Alternative organizing models have been tried, without the same success.

The "Hands on philosophy" approach to the Site management of a project will have as a prerequisite, a very close follow up from the Owner through all construction phases at site.
Unexpected situations can be envisaged, interpreted and decisions taken by a minimum loss of time. In the same way necessary design changes of lay out etc, can be handled with the best basis for a correct and favourable solution.
The philosophy is based on the fact that the actual building material, rock, with its frequently change in quality, makes it difficult to foresee the real situation in forehand, and thereby describe the correct reinforcement and rock support, at the Design office.
Due to this fact the Site construction team, holding the sufficient geological competence, may take decisions in matters of minor consequence, or, in matters of great importance, can report back on deviations to the Design office, often situated at a far distance from a remote underground project location.
In some cases the Owner may prefer to move an engineering "Follow on team" direct on the site, to be enabled to minimize the distance and time for correct decision taking.

Other models for the Owner's control system may also be preferred, like for example an EPCM-model (engineering, procurement, construction management) or similar arrangements.
In this article the "Hands on Philosophy" will be the basis for the description.

2. THE ENGINEERING PHASE

The engineering will comprise several phases for large underground constructions, from the Feasibility studies through the Concept phase up to the Basic engineering (Pre-engineering).
When construction starts the Detail engineering phase will cover the deliveries of drawings, specifications and necessary procedures to the site.
These phases are covered in other articles in this publication.

A figure demonstrating the project phases from engineering through construction, commissioning and start of operation, is shown in figure 1.

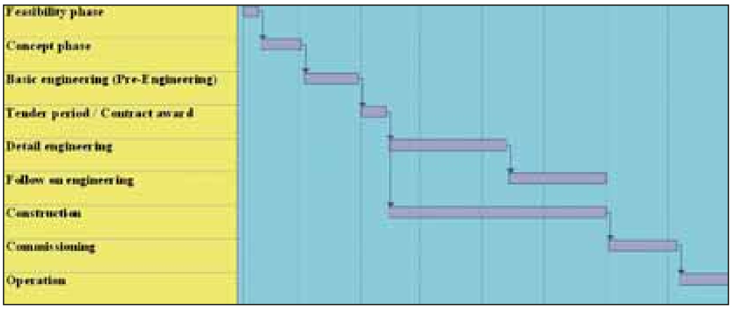


Figure 1: Project phases

3. SITE TEAM ORGANIZING /COMPETENCE IN THE CONSTRUCTION PHASE

The site team should be organized with managers holding competence in underground civil works and engineering geology with at least 10-15 years of experience. Younger engineers with minor site experience should assist the managers with competence in engineering geology. The site managers should have had the opportunity to participate in parts of the Basic engineering phase with Tender preparations and Award of contracts to the potential contractors.
Normally the Norwegian underground excavation contracts include description of all possible construction and support works to be executed at site with corresponding quantities expected.

This information will have to be based on map studies, site visits, seismic and probe drillings including ground water tests etc.
These investigations normally are of a high quality, but do not give detailed data for the specific tunnels and caverns location. The real updated information first becomes available for the Owner's Site supervision team, when the rock is exposed after blasting at the tunnel face.

Fig.2: Modern underground storage under construction

4. FOLLOW UP IN THE CONSTRUCTION PHASE.

4.1 Ground water control

The ground water level above the underground located construction is of crucial impact to maintain intact during the blasting period. Maintenance of this level may perhaps be a greater challenge than the removal of rock. The natural hydrostatic characteristics of the ground water should be disturbed as little as possible since it is extremely difficult to reinstate lost volume of pore pressure. Normally an intricate and finely balanced water injection system from the surface is established prior to any excavation. Consequently a programme for follow up the piezometer locations must be worked out, and will be one of the Site team's main responsibilities to supervise during the construction period.
As a lowering of the ground water table may be difficult to restore when first occurred, the establishment of the wells for water supply into the ground water, must continuously, during the blasting period, be brought in operation at a correct distance ahead of the tunnel front.
The cavern depth is just determined by the fact that ground water level, shall, at all times be at least at a distance equal to the maximum cavern overpressure, plus 20m above the cavern ceiling.

4.2 Control at the tunnel front

On the basis of earlier experience from rock conditions at site, the Site team is able to take immediate actions and give the contractor further instructions for the ongoing work.
This is particular valid for wall/ceiling and sole blasting, rock reinforcement work (bolting/shotcrete, cast concrete), for the adjusting of probe drillings ahead of the tunnel front and for the water sealing works.
Through a close follow up the immediate support works for the construction workers in some cases may be combined with the Owner's permanent support work.
It is of no doubt that operations like these mentioned above, are best handled and accommodated by the Owner's Site team, directly involved in the daily ongoing operations.

Fig.3: Impression of dimensions of underground constructions

For these operations it is of great importance that the contract includes descriptions of all actual work that may occur at site, and that adherent unit rates are included.
Anticipated quantities given in the contract, should on the other hand not differ very much from the quantities revealed during construction, still the tables for capacity for different support works, are included in all modern Norwegian contracts for underground works (equivalent hours).
Our experience from many years in Norwegian underground construction business, shows that an approach like the above described, with a clear "hands on" organisation of the follow up work, should secure the Owner a sufficient high quality of work to a fair price, and normally also within the milestones set in the Progress Schedules for the work.
There is a clear prerequisite for this, as mentioned earlier, no supervision team is able to handle such work in a satisfactory way without a proper knowledge and a thorough practical experience from underground construction execution.
Therefore the competence and composition of the Site team should be as we have pointed out in item 2 in this article.

In figure 4 below the crucial site activities are listed:

N0.	SITE ACTIVITIES	FOLLOW UP
1	Ground water level/pressure control	Executed by Owner. Contractor to perform the practical work (drilling of holes etc) according to Owner's decision.
2	Blasting control	Control executed by Owner. Blasting of contour holes shall not damage tunnel contour, concrete structures etc.
3	Decision on rock support (reinforcement)	Contractor is responsible for his workers safety and decides temporary rock support at the tunnel front. Owner to decide the permanent rock support and that all rock reinforcement has the project lifetime durability.
4	Ground water sealing (grouting) in tunnel and at the tunnel face during excavation	Owner to decide number and length of holes, grouting pressure, composition and amount of quantities.
5	Other civil works actual for Owner's control	Control as specified in the contract.

Fig.4: Important activities for the Site team

4.3 Concrete plugs-performance and tightening

After completion of the caverns the concrete plugs for shut off and tightening of each cavern are established. A proper filling up with concrete of the formwork for the plugs requires correct concrete mix and application of suitable equipment.
The injection hose installations and sequence of injection of the plugs towards the rock surface, require a good planning and completion of the work.

4.4 Control towards 3.party

The execution of the blasting with necessary warning and evacuation of other persons, stop in relevant neighbouring work at site, use of sufficient alarm sirens, may be an important activity in certain periods for the Site team, to see to that this under proper control and well accommodated by the actual contractor.
Further the observation of vibrations regarding impact on houses, constructions etc must be registered and evaluated. Blasting rounds should, if necessary, be adjusted to comply with acceptable vibration levels set in the contract.

The problem with drainage of ground water table from surrounding areas down into caverns or tunnels, may impose settlements in foundation of nearby buildings and constructions.
The Site team will have the task to survey and observe installed instruments to decide if this may introduce a problem for the project or not.

5. HSE AND QA SYSTEMS AND REQUIREMENTS DURING CONSTRUCTION.

5.1 HSE

All owners of underground construction in Norway today have a high profile on HSE. In additions to the Owner's internal requirements in this field, the Norwegian Authorities will have a sharp observation on the execution of work. Contractors with low score and rating in the HSE field, will be out of question for the execution of large underground works.
The site team will, of course, be brought into the HSE work with Qualitative risk analysis, Safe Job Analysis, Unplanned Incident Observations and Reporting, Unsafe Act Auditing, Safety Inspections Rounds at site etc.

5.2 Environmental control

The environmental execution should comply with the NS-ISO 14001 requirements.
The discharge permits from the Authorities must be followed up, and specially leakages and oil contaminated tunnel water, pumped out from a temporary treatment plant in the tunnels, and led to sedimentation basins in open air, prior to the discharge water is let out in sea, river or water, must be in proper operation through the construction period.

5.3 Quality Control

The Quality of the execution will be monitored through a QA system complying with NS-ISO 9001:2000 requirements. A comprehensive detection and documentation will be necessary for logging of the results and observations at the tunnel fronts at any time, support work, injection work, piezometer logging etc.

6. PUMP SHAFTS /MECHANICAL INSTALLATIONS

For crude oil and gas storage pump shafts must be established. Submerged pump installations are normal, but also "dry-installed" solutions may be designed.
The pump shafts normally are raise drilled, still shorter shafts may be established through long-hole drilling and blasting.
To maintain the ground water table around the shafts will require a thorough plan for injection work prior to the establishment of the shafts.

Several mechanical installations for warming up crude during unwaxing of the caverns in the Operation phase may be established. Further instruments for level control, ventilation to gas flare etc, may be brought in place.

Fig. 5: Pump shaft during construction

7. COMMISSIONING PHASE /PREPARE FOR INFILL OF PRODUCTS

A programme for commissioning and filling in of products into the caverns, must be settled in due time prior to the execution.
To avoid explosion in the gas zone above the crude, if falling rock should create ignition, when hitting the rock wall, the caverns may partly be water filled with fresh water, if available (sea water should be avoided due to the corrosion risk), before filling in crude, The rest of the cavern volume should be saturated by exhaust gas or nitrogen (expensive). In this way a none explosive atmosphere is achieved. The water should be pumped from one cavern to the next one, as the caverns are filled with crude during the commissioning.

During the filling up the site team should closely follow up instruments, and detect if any leakage to the free atmosphere of gas should occur.
In such a case the filling must be stopped and supplementary injection must be supplied.

After completion of the underground civil works, the Supervision team shall produce a report covering all civil works that have been carried out. This report will be an important document as a basis for the Operation personnel's supervision of civil works, water curtains etc. The report should also comprise guidelines for further work or repair if something should fail in the future. This part of the report shall have specific description if there should exist risks that the rock feature may change over time. This may for example be related to the water curtains. The conductivity in the holes may decrease after some years. Therefore a redrill or drilling of additional holes may be required. The report consequently should describe the possible change in rock features and how eventual repair work should be done.
Further the Site team should consider the possibility for the Operation management to understand and comply with the intention in the report. Hence the report should be presented in a way to make it possible for the Operation personnel to catch the purpose of the report, and implement the guidelines in their own control system.

8. START OF OPERATION PHASE/SURVEILLANCE

In the Operation phase the site team and also the engineering personnel have been demobilized. The Operation personnel normally don't have any civil or geological competence. Therefore a certain support for follow on assistance to the Operation management should be planned and catered for, to see that the crude storage plant behaves as planned

9. EXPERIENCE/LESSONS LEARNED

A Close out Report from a large oil-and gas project including the underground constructions, will be an Owner requirement, and constitutes substantial value for the design and construction of future projects.
Lessons learned in the project will help the Owner to continuously improvement and also to bring the best execution practice in use for new projects.

7.1 TROLL PHASE 1 – LANDFALLTUNNEL KOLLSNES

Jørund Gullikstad

BACKGROUND

In 1979, Norske Shell found Europe's biggest gas field in the North Sea, 80 km northwest of Bergen. The Troll field development was planned for about 10 years, before start of construction in 1990.
A 6 years construction period was scheduled for the total development, giving 10 % of the total need of gas in Europe when the gas production started in 1996.

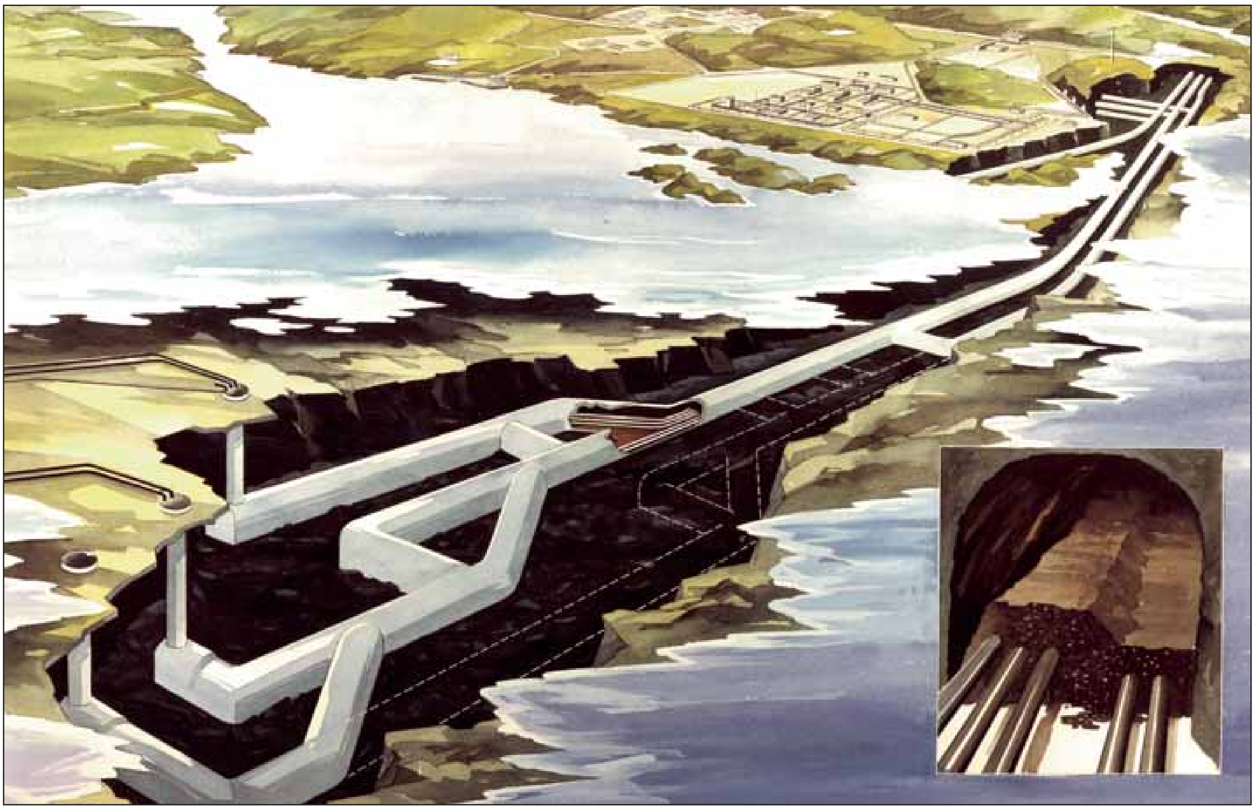
The yearly produced energy volume from the Troll field is about three times the total produced energy of all hydro power plants in Norway.

The Troll-field will produce gas in 50-70 years, where wet gas is pumped from the sea bottom, through gas pipelines ashore to a gas treatment plant at Kollsnes. At Kollsnes the gas is being processed and thereafter exported to Emden and Zeebrügge, for further distribution into Europe.

SHORE APPROACH SOLUTION

Between the gas field offshore and the gas treatment plant at Kollsnes, the sea bottom is very uneven, especially the last distance towards land.

It was therefore chosen a landfall solution with shore approach tunnels going 4 km out in the North Sea, where 3 import pipelines and 2 export pipelines are going out on the seabed in vertical shafts at approximately 170 meter water depth.



Project illustration - Norske Shell A/S

An extreme challenge, giving both the Client and contractor challenges and limits not reached so far in the tunnelling history.

SHORE APPROACH TUNNELS

8 km of tunnels with cross-section between 50 and 110 m2, was excavated in 21 months. The first 2 km was excavated with downhill 1:7 gradient. The rock was composed of amphibolitic gneiss, including several and difficult weakness zones containing active swelling clay. Many of these zones necessitated voluminous grouting and rock support work.

Open zones was sealed with normal cement in combination with mortars. Research work was performed to find the optimum combination to seal off the most open fracture zones.
Rock support was performed with systematic rock bolting, sprayed concrete and in a couple of zones full concrete lining was necessary.

The main tunnel has a low point approximately 250 meter below sea level and from there the tunnels continue upwards 1:100 ending in the piercing area 4 km out in the North Sea. The last part of the tunnels was excavated in more migmatitic gneiss, with few weakness zones and less need for rock support work. The tunnel system ends in three vertical shafts breaking the seabed at 157.5, 161 and 168.5 meter water depth.

PREPARATION WORK BEFORE SHAFT EXCAVATION

Prior to excavation of the vertical shafts, necessary preparation work was performed.
A safety barrier of concrete was constructed in each shaft tunnel to stop a potential uncontrolled in-leakage of water during shaft driving.

Further, seismic examination, systematic core drilling and grouting works were done in the shaft area to ensure that the optimal location was found, to identify the exact level and shape of the seabed and to avoid any uncontrolled in-leak of water during drilling and blasting of shafts.

Finally, examination of existing overburden on the sea bottom was performed. It was observed that the thickness of soil sediments above rock head was up to 4 meters. These sediments had to be removed to increase the probability for successful shaft breakthrough and to avoid huge volume of sediments/clay to be stuck in the shafts after blasting of the tunnel piercings.

The removal of sediments was performed after grouting work with a submersible vehicle called SEMI-2 having 12.000 HP propels. The vehicle removed rock up to 3 tonnes from the sea bottom.

SHAFT EXCAVATION

3 shafts with 35 m2 diameter and 25-35 meter length were excavated with specially designed Alimak-equipment, until a piercing plug of 6-7 meters was remaining.

Piercing area - shaft excavation completed

PREPARATION WORK AFTER SHAFT EXCAVATION

Following rock support in the shafts, a steel cone was installed 20-30 meters above the tunnel floor. This steel cone was machined which chould be installed to match the riser bundle containing the gas pipelines, after the tunnel piercings were successfully completed. Each steel cone weighed approximately 17 tonnes, and was installed with 2.5 mm accuracy using a specially constructed winch and sheave system enabling the steel cone to be installed without any persons in the shafts or below on the tunnel floor.

In addition, two steering constructions in steel were installed above the steel cone to ensure the correct rotation of the riser bundles during installation.

Finally several concreting lines with special built concrete locks were installed above the steel cone to resist the forces from the final blast, and to enable concreting between the rock walls and riser bundles after installation. The constructions in the shaft were designed to stay undamaged and resist the forces from the final blasts containing approximately 1500 kg of explosives, thereafter followed by the rock masses going passed the constructions and finally 20 bar of water pressure on the concreting lines with concrete locks/valves.

FINAL BLASTS

The requirements for the final blasts were completely different from historical experiences from the hydro power industry where no requirements to surrounding rock or installations nearby had to be considered.

The planning and engineering of the final blasts at Troll were started more than one year before execution, where the contractor and client in cooperation found the optimum way of designing and performing this 'world record'.

Three special requirements where especially challenging to solve:

1. A riser bundle containing the gas pipelines and weighing approximately 450 tonnes should be installed in the shafts after the final blasts. The final blasts should therefore be designed as careful blasting where the following had to be ensured:
   - no rock is left inside the contour of the blast
   - the steel and concrete constructions in the shafts could not be damaged
2. The final blasts had to be designed and performed to ensure that the total volume of the masses (2 x theoretical volume) was safely transported to the steel cone which was only 46 % of the shaft diameter. The final blasts were therefore designed in a delayed sequence to ensure that:
   - the rock masses were not stuck inside the steel cone
   - no remaining rock above the steel cone should hamper the riser bundle installation
3. After final blasts and riser bundle installation, the riser bundles had to be concreted in the shafts and the gas pipe installation should be performed in dry conditions in the tunnel. The final blasts therefore had to be designed and performed to ensure that:
   - the surrounding rock was tight and still sealed after the blasting was performed
   - the concreting lines including valves/locks was undamaged with no leaks

The final blasts were drilled with a special built Nemek drilling rig, installed on a steel construction above the steel cone. About 230 boreholes with extreme tolerances, inclusive 8 nos. of 6'' cut holes, were drilled per blast.

The explosives were specially designed by Dyno, and non-electric detonators were used for safety reasons, for the first time used for underwater piercing.

After drilling and charging of the final blasts, the shaft was partly water-filled, and the air volume between the water and the final blasts were pressurised up to 13 bars.

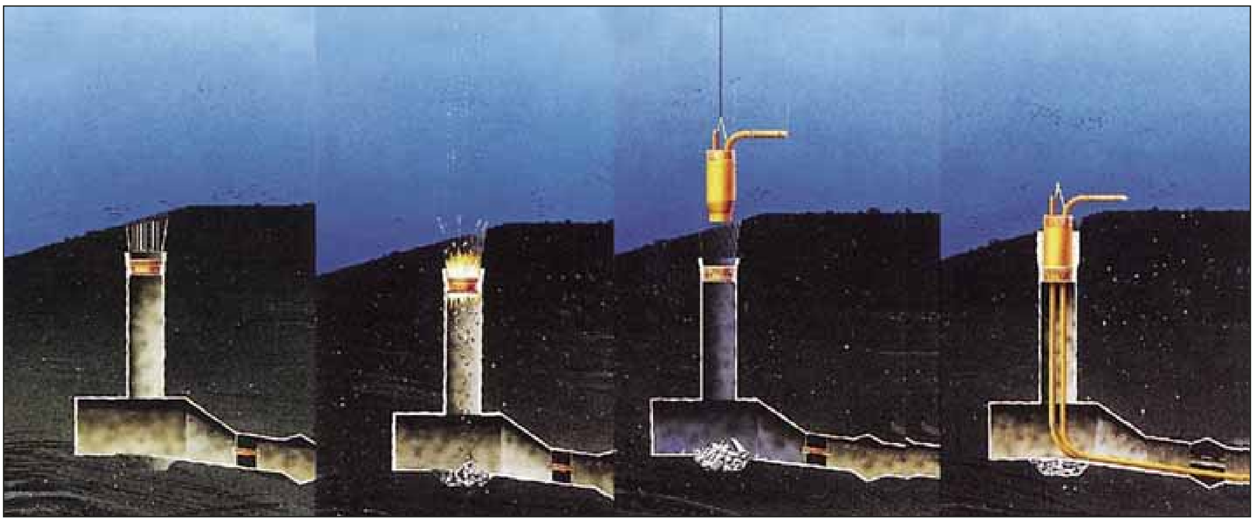
All the three final blasts were successfully completed in February 1994 with the following result:

- No rock within the contour of the final blasts
- No damage to any of the steel and concrete constructions in the shafts
- No rock from the final blasts remaining above or within the steel cones
- No cracks or leaks observed in any of the shafts

RISER BUNDLE INSTALLATION

The 450 tonnes riser bundles were thereafter installed with the multi-vessel Regalia. The installation was performed using guide lines to the steering construction in the shaft, and the landing speed was recorded to 0.05 m/s, well within the requirement of 0.11 m/s.

Finally the riser bundles were concreted in the shafts using underwater concrete especially designed for



Work sequence - Piercing and pipeline installation

200 meter water depth pumped through long concreting lines from the dry part of the tunnel behind the concrete plugs.

After concreting, the piercing area was emptied for water and the concrete plugs were removed. Then the gas pipe installation could continue in the tunnel without a drop of water coming into the tunnel system in the piercing area.

Raiser bundle incl. Gas pipelines

7.2 ÅSGARD TRANSPORTATION PROJECT - KALSTØ LANDFALL - COMBINED TUNNEL AND BORED SOLUTION

Jørund Gullikstad
Arild Palmstrøm

ABSTRACT:
The gas from the Åsgard-field, 150 km northwest of Trondheim out in the North Sea, is transported to the terminal at Kårstø. From the Åsgard B platform, 700 km of 42" gas pipeline is bringing the gas to the landfall point at Kalstø. From Kalstø landfall the gas is following the 1.5 km existing Sleipner landfall tunnel, through Kalstø valve station, and 21 km further over land and fjords to Kårstø terminal. The paper describes two tunnelling milestones in sub-sea tunnelling that were achieved when the Kalstø landfall was constructed in 1998:
1. ***Excavation of a large sub-sea chamber with only 15 m to the sea bottom at 55 m water depth***
2. ***Dry piercing to the sea bottom and pull-in of a pipeline without use of divers***

INTRODUCTION

Through the oil & gas period in Norway, several different landfall solutions have been performed:
- For the Statpipe-lines to Kalstø, a prefabricated concrete culvert with huge amount of vessels and divers
- For the Oseberg-pipeline towards Sture, a landfall tunnel ending in a concreted pull-in chamber and final piercing to sea by blasting, the work in the chamber after blasting was perfomed by divers
- For the Sleipner-pipeline to Kalstø, a landfall tunnel with concreted pull-in chamber similar to Oseberg but with a drilled solution instead of blasting, but still with use of divers
- For the 5 Troll pipelines, vertical blasted piercings followed by riser bundle installation, but for the first time without divers
- For the Heidrun-pipeline into Tjeldbergodden, an underwater trench was the optimum solution all the way to the landfall area

Traditionally another landfall solution was selected on Åsgard. The method chosen was to utilise the existing the Sleipner landfall tunnel, and from that use a drilled solution to the flat sea bottom at about 60 meter water depth approximately 1 km out in the North Sea. To avoid use of divers, a seal tube was constructed to enable both drilling and pull-in in dry and safe conditions.

The offshore Åsgard oil and gas field is located Northwest of Trondheim. Gas from this field will be pumped through the 42" Åsgard Transport pipeline, which has a steel thickness of about 50 mm, to Statoil's gas treatment plant at Kårstø. Here, natural gas will be stripped from the lean gas to bring the latter to sales specification before it is sent to Emden in Germany through the Europipe II export line (Figure 1).

Figure 1: Overview (Palmstrøm, Skogheim 1999)

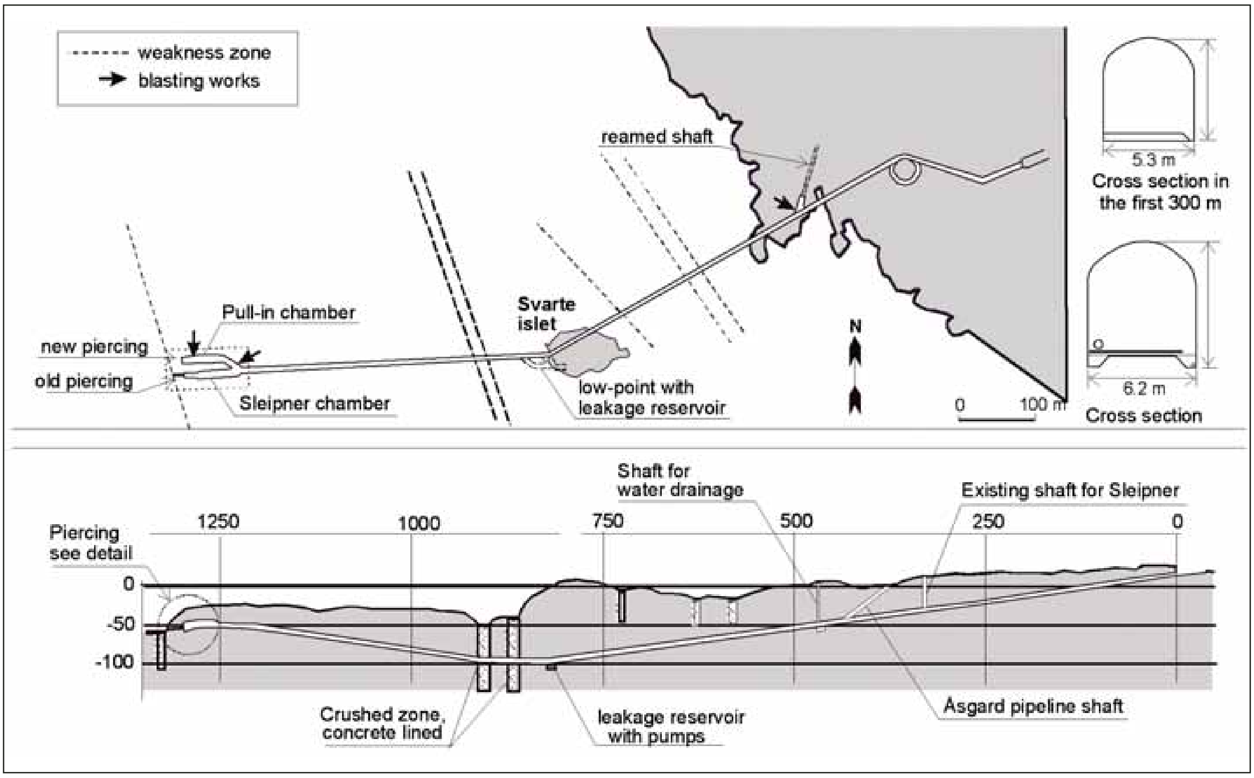


Figure 2: The conditions at the Kalstø landfall tunnel. (Palmstrøm, Skogheim 1999)

Before arriving onshore from the North Sea the pipeline enters into an existing landfall tunnel at 60 m water depth to be protected from sea wave damage. This landfall tunnel was constructed in 1990 - 92 for the Sleipner condensate pipeline. It is 1350 m long with the deepest point 100 m below sea level. In the first 300 m, the tunnel is 5.3 m wide; in the rest the span is 6.2 m, as shown on Figure 2. The rock cover (overburden) is 30 to 60 m.

The ground consists of gabbro, often metamorphosed to a gneissic rock. The rocks are generally moderately jointed with Q-value 4 – 25 (fair to good). A few large weakness zones were encountered, having a quality Q = 0.01 to 1 (extremely to very poor). In addition, many small shears and minor weakness zones occur.

The rock support in the tunnel was tailored to the rock mass conditions encountered. No support was performed where few joints occurred, else the support was shotcrete and fully grouted rock bolts. Concrete lining was only applied at of the large weakness zone near the low-point of the tunnel, making a total of 38 m, or 3 % of the tunnel length.

PREPARATION WORK BEFORE TUNNEL EXTENSION WORK

After completing the Sleipner condensate pipeline installation in 1992 the tunnel was flooded with sea water. Therefore, prior to commencing the work for Åsgard in 1997 the existing Sleipner condensate landfall tunnel had to be dewatered for approximately 50 million litres of water, and the necessary supplementary rock support performed.

Additional rock support, rock sealing and installation of ventilation, light, water and high voltage electricity was performed in a few months time from February 1997.

TUNNEL EXTENSION WORK

The tunnel extension work was especially challenging as the existing Sleipner condensate pipeline daily transports condensate worth about 20 million NOK through the tunnel. A longer stop of the condensate transport to Kårstø could in worst case stop the oil production at both Sleipner and Statfjord totally.

The extra piercing chamber had already been excavated in 1991, see Figure 3. Some modifications in the landfall tunnel and chamber had, however, to be made for the installation of the Åsgard gas pipeline. This consisted of the excavation of 3500 m3 by drilling and blasting, partly performed as close as 5 m from the existing Sleipner condensate pipeline, which was in operation.

The following preparation and protection work was therefore performed before tunnel extension:

- Mechanical impact from blasted rock was avoided by installing New Jersey road blocks backfilled with absorbable sorted fraction rock.
- In addition protective constructions using concrete and timber were used in especially sensitive areas, prior to the normal blasting mats and fibre mesh.

- Blasting plans were carefully designed for each blast to accommodate strong vibration requirements. The vibration velocity limit was set to 30 mm/s. During blasting, the vibrations on the condensate pipeline, surrounding rock and concrete foundations were closely monitored. See Figure 3
- In order to determine the drilling, charge and ignition plan a full-scale test-blasting program in the piercing chamber was carried out by using a similar protected Sleipner pipe, prior to start of extension work.
- Four alternative methods to take out the necessary rock volume were evaluated: sawing, expansive cement, hydraulic splitting and pigging by hydraulic hammer.
- Other existing installations like electrical cables and similar were protected using split plastic pipes covered with sprayed concrete.
- An experienced engineering geologist from Norconsult closely followed-up the tunnel works and the need for rock support and water sealing by grouting.

The existing Statpipe piercing chamber was enlarged to accommodate the pull-in of the Åsgard gas pipeline. Located at 60 m water depth with only 15 to 20 m rock cover, the chamber was widened from 8 m to 11 m span, and the height lifted from 7 to 9 m. The tunnel extension work using careful blasting was successfully completed without any damage to either permanent or temporary installations.

Each blast was planned with a unique drilling pattern and use of explosives. The drilling varied between 2 to 4.5 drilled metres per m3 hard rock. Traditional explosives as dynamite, Dynotex 1, 2 and 4 were used, with an explosive quantity between 0.5 and 1 kg/m3. It was also restrictions on charges per interval, dependent upon distance to the existing Sleipner condensate pipeline.

The large dimensions of the piercing chamber and the water depth caused extra challenges during the blasting, rock support and piercing works. The small rock cover of only 15 - 20 m resulted in low rock stresses, which imposed an extra risk for joint opening and development of water leakage.

Upon completion of the rock blasting and rock support works, approximately 17 meter of rock was remaining before the North Sea and the piercing operation could start.

PIERCING TO THE SEA BOTTOM

Piercing of tunnels to the sea bed is not a new concept in Norway. In connection with hydropower plants, some 600 - 700 of so-called “lake taps” or “bottom piercings” have been used [3 to 6]. For the landing of pipelines from the North Sea, this vast experience has been utilised.

A main goal for landing of the Åsgard gas pipeline was to pull in the pipeline without use of divers.

Preparations

The piercing was performed using a well planned drilling and reaming procedure. The client, Statoil, determined the specifications and the method to be applied, while the contractor, AF Spesialprosjekt, was responsible for the planning and performance of the works in compliance with the strict specifications, both to HSE and QA/QC. For this, AF Spesialprosjekt had experience from similar operations, among others for the Troll Phase I Project in 1991-1995, comprising 3 piercings at 160 – 170 m water depth [7, 8].

A special steel structure, the so-called seal tube system (ESD-valve, pipe receiver, stripper valves, drill string bearings, flushing system etc.) was developed and delivered by Statoil to provide a “dry” piercing and pull-in operation into the piercing chamber.

After piercing, the rock face had been reinforced with rock bolts and shotcrete, the following works were performed:

1. Drilling of several probe holes to check the distance

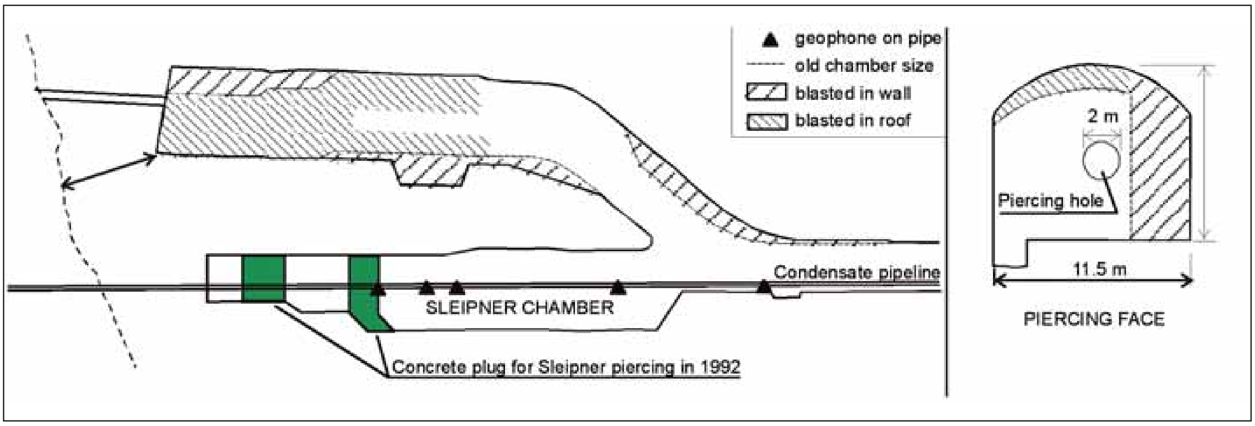


Figure 3: Left: Plan showing areas enlarged in the pull-in chamber from blasting. Right: Cross section of chamber. (Palmstrøm, Skogheim 1999)

to the sea, and to collect information about the rock quality and water leakage conditions.
2. Rock grouting/injection of the rock masses in the piercing area to prevent potential water leakage.
3. Rock mass reinforcement by fully grouted rock bolts in a pattern adjacent to the planned piercing hole.
4. Blasting of a 2.2 m diameter and 4 m deep “cylinder” along the piercing hole centreline for seal tube system anchoring purposes.
5. Drilling of grouting, casting and sea water holes for future casting around the Åsgard pipeline.
6. Installation of the seal tube system with:

- Anchoring systems (casting and rock bolts).
- Mechanical installation (steel structures, pumps, valves, computer systems, hydraulic systems, etc.).
- Testing and commissioning.

After extensive grouting works the water leakage into the piercing chamber was reduced to 30 l/min.

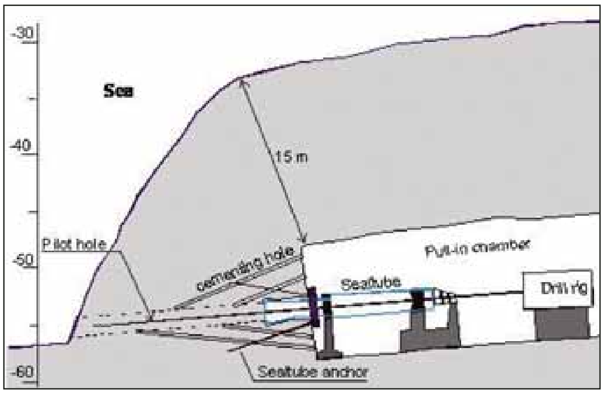


Figure 4: Layout of the piercing with the small pilot hole (made by directional drilling). The cementing holes were used for filling cement grout around the pipeline in the piercing hole after pull-in. (Palmstrøm, Skogheim 1999)

Drilling of the piercing hole and pull in of the 42” Åsgard gas pipeline

The piercing operation can be divided into the following steps, as shown in Figures 4 and 5:

- Directional core drilling of the first 56mm diameter pilot hole until 3 metres from the sea bed. The hole was then enlarged to 76 mm diameter using a standard core drilling rig.
- Installation of the seal tube system, which was anchored to the rock face.
- Installation of a drill rig behind the seal tube system for reaming of the pilot hole
- Reaming of the 76 mm hole to 308 mm (12¼”) diameter including drilling of the remaining 3 m to the seabed.
- Drill string was then disconnected from drilling rig and the messenger wire attached to the drill string. Marine vessel (DSV) pulled the drill string with messenger wire attached out of the piercing hole and up to on the vessels deck. A new drill string with the Ø1.6 m reamer head was then connected to the Ø30 mm messenger wire and lowered down to the sea bed.
- The drill string was then pulled into the 12¼” pilot hole and the reaming of the 1.6 m diameter borehole started from the sea towards the seal tube. Initially, the reaming was performed very carefully to minimise vibrations from the drill string/reamer head. Drilling debris/cutting ships were continuously removed by a water jet system installed behind the reamer head. See Figure 6.
- Upon completion of the bore hole, the drill string and messenger wire were pushed/pulled out and hoisted with air bags onboard to the DSV.
- The Ø90 mm pull-in wire was then attached to the messenger wire and pulled into piercing chamber via the seal tube system and finally connected to the 500 tonnes linear winching system, which was installed in the same position as the drilling rig was in the previous operation.

Final Pull-In

ROVs (remotely operated vehicles) equipped with video cameras were used for all sub sea works (connections, inspections, etc.). All sub sea activities was closely monitored in the observation/control centre via TV-links and UHF radio communication (land – sea – tunnel).

In May 1998, one of the world’s biggest pipeline installation vessels - LB200, arrived at Kalstø.

Pipeline production starts onboard the magnificent vessel immediately after arrival.

In close communication between the control room at Kalstø, the control room at LB200 and the operational resources in the tunnel, the 42’’ Åsgard pipeline is safely installed into the Kalstø seal tube, see Figure 6.

After the pull-in, the Åsgard pipeline was anchored to rock by grouting between the pipeline and the Ø1.6 m piercing borehole walls by using the pre-drilled concreting and injection holes.

After a few days curing, the seal tube was dismantled, and the 42’’ Åsgard pipeline showed to be safely anchored with no water leaks into the tunnel, ready for further pipe installation towards Kårstø.
All the challenges in this complex project were solved and accomplished according to schedule and given specifications, thanks to well-planned preparations and great achievements from all parties involved.

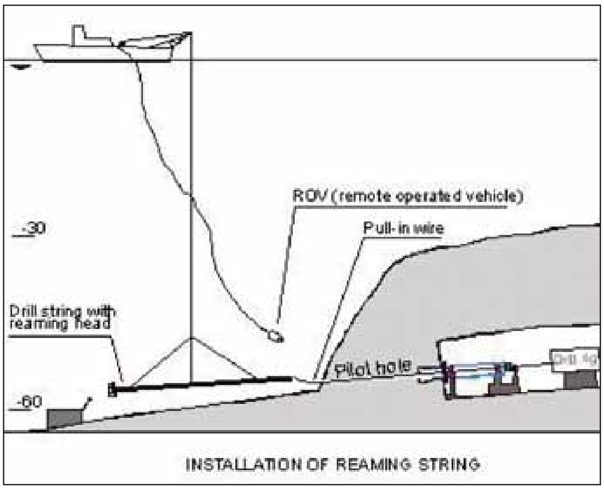


Figure 5: After the pilot hole had been drilled, the reamer was pulled into the hole. (Palmstrøm, Skogheim 1999)

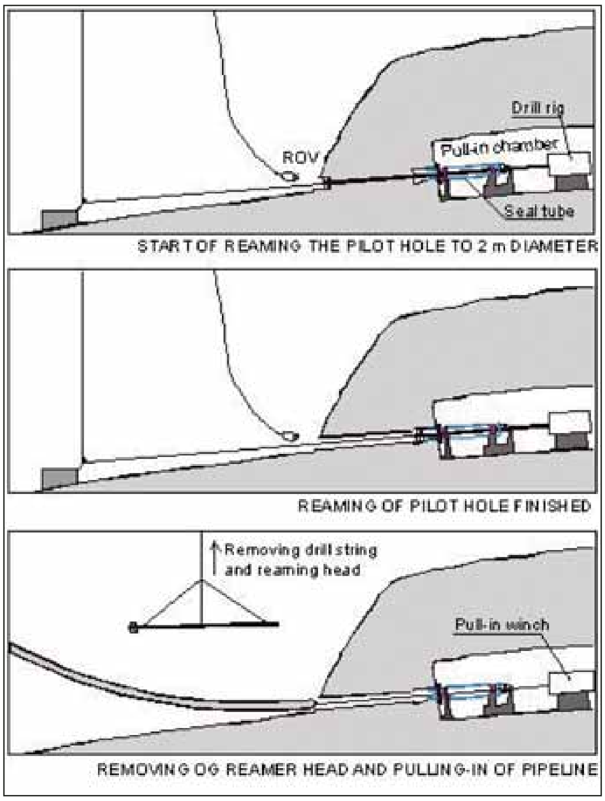


Figure 6: The reaming of the pilot hole and the later pull-in of the pipeline. (Palmstrøm, Skogheim 1999)

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7.3 KVITEBJØRN RICH GAS PIPELINE PROJECT KOLLSNES LANDFALL - TUNNEL SHORE APPROACH

Arild Neby
Thomas K. Mathiesen

ABSTRACT:
Pipelines from offshore oil and gas fields in the North Sea are serving several onshore process plants on the Norwegian west coast. The shore approach itself is normally a challenge as the subsea topography off the coast is rugged and seldom facilitates landfall sites with gently sloping sandy beaches coming up from the continental shelf and slope. Large diameter bore holes drilled from within the process plant area through the rocky barrier and out in the sea have been a common solution for landing pipelines where other methods have not been available. The bored solutions have however often proved to be expensive.

For the Kvitebjørn Rich Gas Pipeline the conceptual solution was a bored landfall with an optional tunnel solution alternative. During the detail design phase a feasibility study on a tunnelled solution combined with underwater tunnel piercing techniques, adapted from the hydropower civil works sector, revealed that for this project the tunnel solution was feasible. Bids were made to both solutions and after evaluation of bids it was recommended to go ahead with the tunnel alternative based on an evaluation of economy, technical aspects and HSE. The Kvitebjørn Landfall tunnel was constructed in 4 months during the summer of 2002.

INTRODUCTION

The Kvitebjørn gas and condensate field lies in block 34/11, east of Gullfaks in the Norwegian North Sea, operated by Statoil. Production from Kvitebjørn began in 2004. Rich gas and condensate (light oil) from Kvitebjørn are piped to Kollsnes near Bergen and Mongstad further north respectively.

Rich gas from Kvitebjørn is piped in the 147 km long OD 30" (Ø762 mm) Kvitebjørn Rich Gas Pipeline (KGR) to the process plant at Kollsnes (see Figure 1). After processing at Kollsnes, the dry gas is piped to continental Europe. The separated NGL is transported by pipeline to the Vestprosess plant at Mongstad for fractionation into propane, butanes and naphtha.

Condensate travels through the Kvitebjørn Oil Pipeline (KOR), which ties into the Troll Oil Pipeline II to Mongstad (see Figure 1). Based on current plans it is expected to recover roughly 55 billion cubic metres of gas and 22 million cubic metres of condensate.

The partners in the license are: Statoil ASA (43.55%), Petoro AS (30%), Norsk Hydro Produksjon a.s (15%), Total E&P Norge AS (5%) and Enterprise Oil Norge AS (6.45%)

The article covers the civil aspects of the detail design and construction of the landfall tunnel section at Kollsnes as well as going briefly into the conceptual design discussions.

The Kvitebjørn Landfall tunnel was constructed during the period May - September 2002. The drill and blast tunnelling works commenced on 21 May 2002. Final piercing at depth -66 m was executed 12 September 2002. The civil works part of the project was cost estimated to approximately NOK 25 million.

The pipeline was pulled in through the landfall tunnel to Kollsnes on 15 May 2003. Production from Kvitebjørn began on 26 September 2004. The field began delivering natural gas through the pipeline on 1 October 2004.

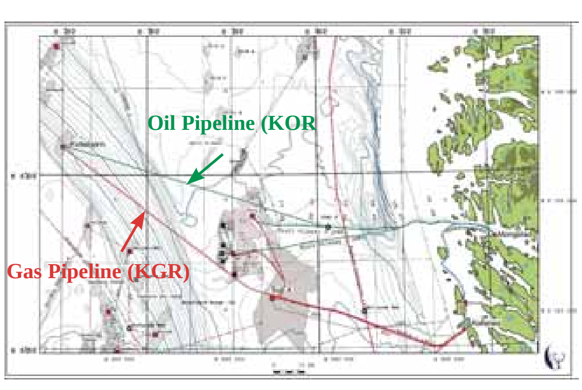

Figure 1: Map showing Kvitebjørn Pipelines (Illustration: Statoil)

CONCEPTUAL DESIGN ISSUES

At the conceptual design stage the different landfall solutions were not described in detail, only the principles of the landfall design. The chosen alternative consisted of a borehole from the sea to Storholmen and an onshore trench at Storholmen and crossing of the Njupselsundet strait (See Figure 1).

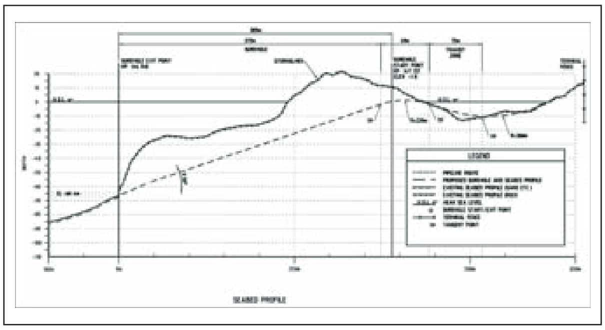

Figure 2: Pipeline route longitudinal section - bored solution (Illustration: Statoil / ABB)

The main challenge, as civil works were concerned, was the establishment of the landfall borehole or drill and blast tunnel. The other parts of the landfall involved only ordinary civil work tasks. By using well known technology, careful prequalification procedures of contractors for drilling of the borehole (or excavation of the tunnel) and focus on the design solutions, the construction of the borehole (or the tunnel) was at this stage considered also to be well within a safe execution.

The feasibility study on the tunnel solution resulted in a recommendation to go ahead with the tunnel alternative based on an evaluation of economy, technical aspects and HSE. The different evaluation aspects for the two alternatives are summarised in Table 1.

For the recommended alternative with a tunnel solution, a range of risk reducing measures was identified as shown in Table 2.

Evaluation Aspect	Alternative 1 - Borehole (Ø914 mm / Ø1200 mm)	Alternative 2 - Tunnel (A=12-14 m2)
Cost Elements	– Ø914 mm: 120 -140 % of tunnel cost – Ø1200 mm: 180 – 200% of tunnel cost	– Linear meter tunnel cost = 100 %
HSE - Risk Elements	– Personnel: 5-10 persons. – Few moving vehicles or mobile equipment involved in work execution. One bore machine (rotating), truck or other lifting equipment for handling of drill rods or other equipment. – Sea transportation for personnel only and for mobilisation/demobilisation.	– Personnel: 25 - 30 persons (total for two shift) – Min. one tunnel rig, one LHD loader and one scaling/rock support truck. Pick-up truck for transport of explosives and materials. Towboat and barge for muck transportation. – Personnel staying below sea level dependent on continuous power supply for pumps, ventilation and lighting. – Small cross-section tunnel gives limited access in case of accidents. – Transportation and storage of explosives.
Work Execution - Risk Elements	– Risks related to the ability of borehole completion within the contract time frame in case of unpredicted conditions. – The tunnel alternative serves as “Back-up” solution.	– Risks related to minimal rock cover and possibility of major water ingress due to lack of pre-grouting or lining – No planned “back-up” solution. – Risks related to failure of piercing blast and subsequent rectifying underwater works at - 65 m depth for preparation of the pull-in operation.
Mobilisation Area and Trench Works Elements	– Requires a relatively large rig and mobilisation area. – Possible pipeline alignment line-up or adaptation will cause additional landscape damage at Storholmen Island in order to optimise strait-crossing of Njupselsundet.	– Rig and mobilisation area can be reduced to the minimum required by the contractor. – The tunnel can easily be adapted to the pipeline alignment without additional cost.

Table 1: Evaluation of shore approach alternatives

Area of Risk Reducing Measures	**Alternative 2 - Tunnel (A=12-14 m2)**
HSE - Risk Reducing Measures	– Provide continuous manning of personnel skilled at HSE follow-up at the Kollsnes site. – Staff the Client's site team with necessary engineering geological competence to ensure safe tunnelling execution. – Execute "HAZOP" and "toolbox" meetings related to the various work operations. – Place a safety container for tunnel excavation works at Storholmen Island. – Utilisation of Statoil Incident Report Register, to identify accidents and risks from similar work operations/projects. – Prepare Emergency Action and Notification Plan in cooperation with the Process Plant – Quality assurance follow-up on machines and equipment, provide back-up power supply for pumps, lighting and ventilation. Work Execution -
Risk Reducing Measures	– Ensure correct execution and adequate extent of rock mass pre-grouting. – Planning and verification of underwater tunnel piercing blast. – Evaluate the need for computer model analysis of the piercing blast. – Prepare for possible submerged operations by establishing contact with sub-sea excavation contractors

Table 2: The Client's identified risk reducing measures for the tunnel alternative

DETAIL DESIGN

Design Basis

The basis for the landfall tunnel solution is as listed below:

- Tunnel data:
  Length approximately 400 m. Slope 1:6.
  Water filled during pull-in.
- Pipeline data:
  ID : 710 mm, wall thickness: 28,7 mm
  Coating : 6 mm asphalt enamel and 50 mm concrete weight coating
  OD incl. coating : 880 mm
- Level of piercing point shall be between 2.5 and 4.5 m above sea bottom which is situated at approximately -68 m depth.

Geological Conditions

The geological conditions for the project was based on surface mapping on the island "Storholmen" and on information gathered from core drilling. Although a significant part of the tunnel, as well as the piercing point, was under the seabed outside Storholmen, the geology of the area was generally expected to correspond well with that which was observed / mapped. However, the joint orientations were expected to vary to some extent, due to folding and faulting.

The project area is located in Middle Precambrian rocks, i.e. rocks with age 900 to 2500 million years. The rocks within the project area consist mainly of granitic gneisses with some minor layers of dark amphibolitic gneiss. These layers often display de-lamination and are sometimes accompanied by higher joint density and/or clay-filled joints. A few pegmatite lenses have been observed.

Figure 3: Long joints belonging to set 1 intersecting the rock masses. Some sub-horizontal foliation joints can also be seen. (Photo: Statoil / Norconsult)

Figure 4: Joints of set 2. The ruler shown is 30 cm long. (Photo: Statoil / Norconsult)

The foliation of the gneiss strikes generally NE-SW with a gentle dip (0° - 15°) towards SE. Joints along the foliation form the main joint set 1 with joint spacing mainly between 0.2 and 2 metres.

The two other joint sets are steep-dipping and occur approximately normal to joint set 1.

Set 2 with strike/dip = 140 - 170°/80 - 90º E, joint spacing 0.3 - 3 m, and
Set 3 having strike/dip = 30 - 50º /90º and joint spacing 0.2 - 3 m.

The joints occur unevenly distributed, in some areas only one joint set occurs, in others all three sets. In general, the joints divide the rocks into blocks, which vary between 0.1 m³ and 3 m³. Most joints have rough joint surface with slightly undulating joint planes.

Locally, the joints in sets 2 and 3 have only 0.02 - 0.2 m spacing forming joint zones. The width of such zones is most often 0.2 - 3 m. In some zones the spacing between the joints is so short that the zone may have the character of a crushed zone.

The identified weakness zones generally follow the direction of the two steep-dipping joint sets.

Generally the geological conditions were found to be relatively good, and hence, favourable for a sub-sea tunnel. Uncertainty regarding permeability of the rock mass, especially in the piercing area, was to be given special attention during construction phase. Thorough and continuous routines for exploratory drilling and grouting, throughout the entire excavation period of the landfall tunnel as well as at the piercing itself, were a prerequisite.

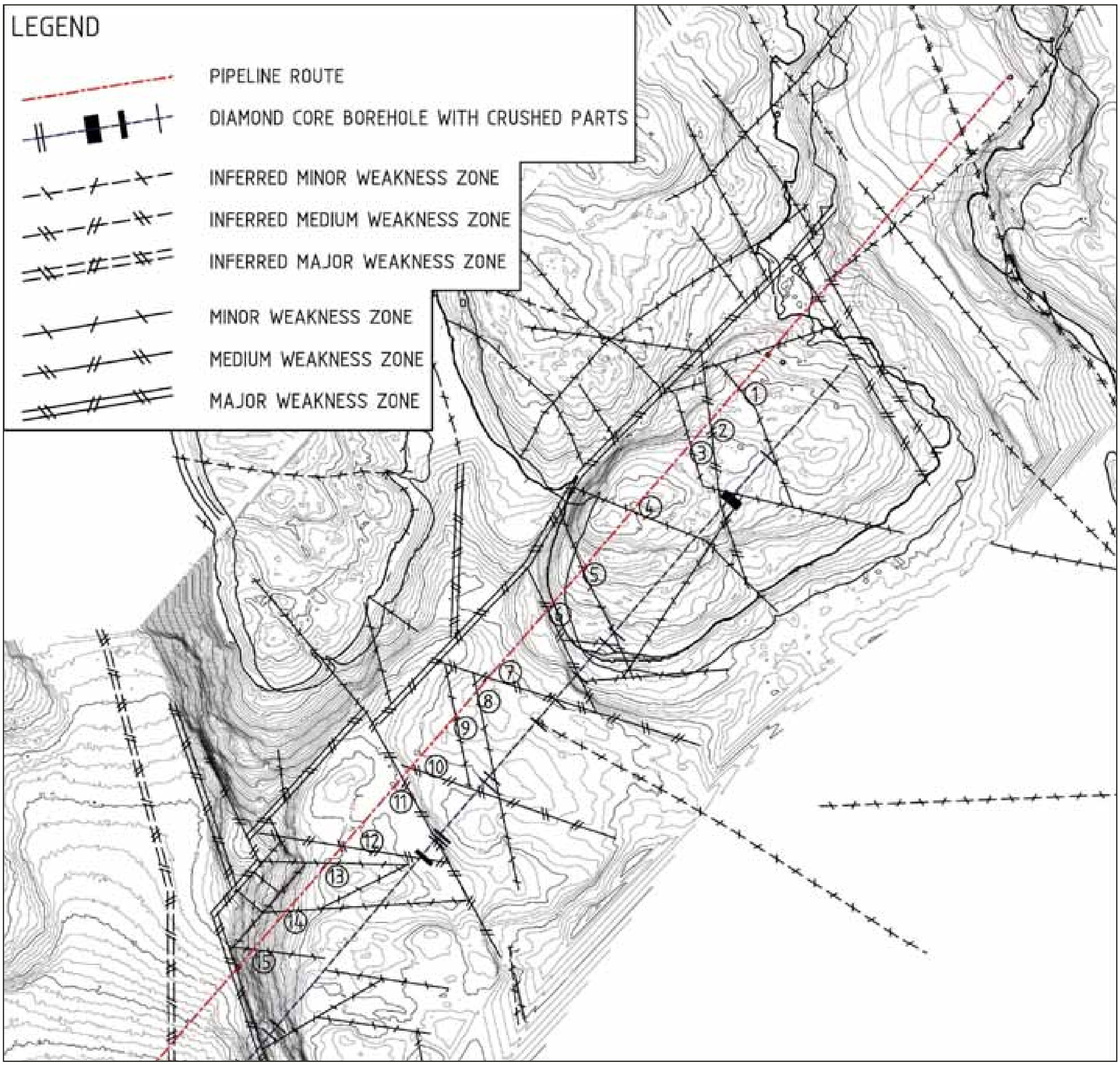

Figure 5: Engineering geological map showing pipeline route and location of core-drilled hole. (Illustration: Norconsult)

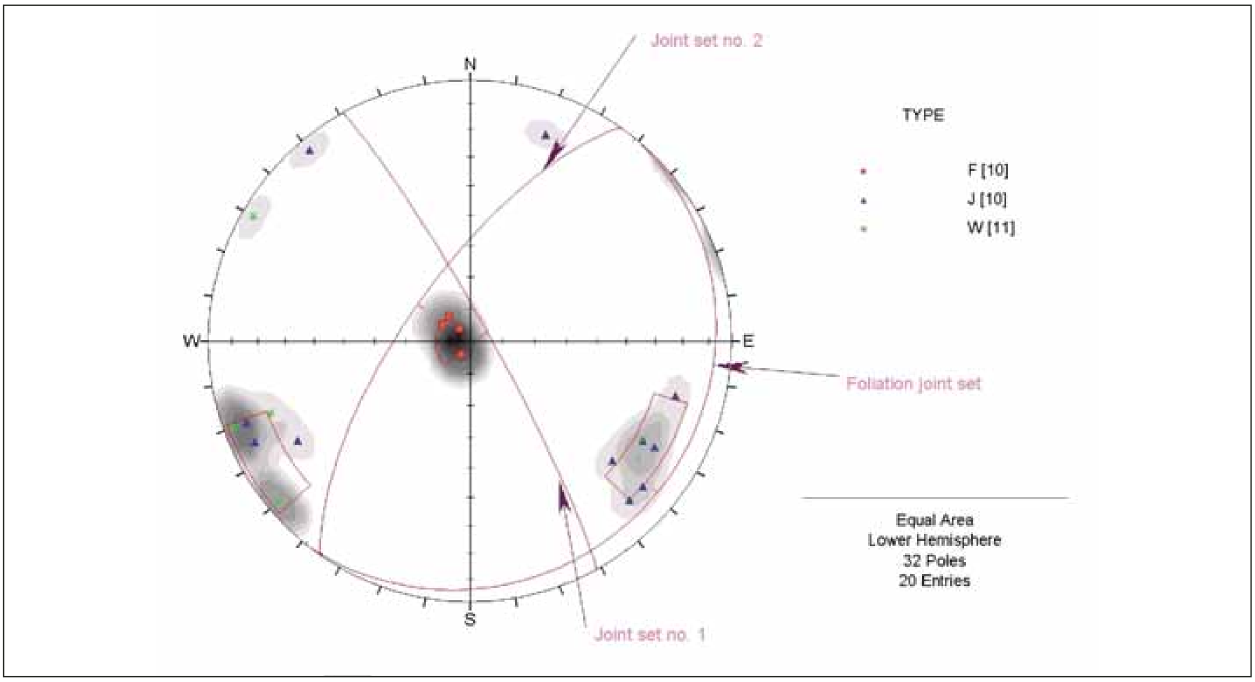

Figure 6: Stereographic pole-plot from joint mapping at Storholmen. The different poles are denoted as follows: F = foliation; J = joint sets; W = weakness zones/faults. The great circles illustrate the average strike and dip of the three main joint sets. (Illustration: Norconsult)

LANDFALL TUNNEL DESIGN

Tunnel longitudinal section

The topography dictates the geometrical constraints for tunnel alignment. As the location of the piercing point must be aligned with the pipeline in the tunnel and the piercing point has to be just above the sea bottom sediments, as well as the tunnel should start at an elevation safe from spring tide sea level, - the straight line between these geometrical "fix points" has a grade approximately 1:6. Excavating the tunnel at a steeper decline was possible but not found to be beneficial to the project.

According to refraction seismic surveys performed in 1990, the rock mass in the area of the sub-sea tunnel is generally good. However, the maximum rock overburden in the sub-sea part of the tunnel is about 26 m, and a significant length of the tunnel will be excavated with rock overburden less than 15 m, see Figure 7. This is less than a normally used criterion for sub-sea tunnelling.

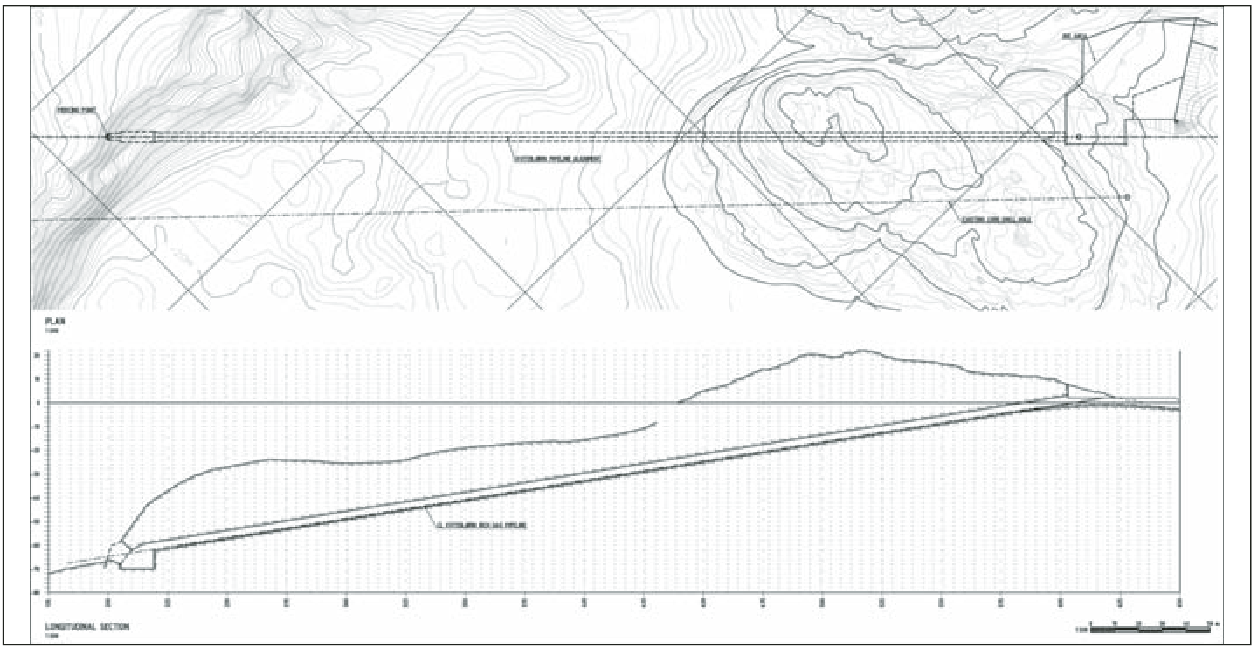

Figure 7: Longitudinal section and plan of landfall tunnel. (Illustration: Statoil / Norconsult)

Tunnel cross-section
To accommodate the pipeline in the operating phase, the cross section of the tunnel should be as small as possible. The pull-in operation was in principle the same as for the 1 m diameter borehole. The optimum size of the tunnel cross-section would therefore be determined by the space requirements for the tunnelling equipment and ventilation duct in the construction phase. Towards the piercing point, where the overburden was thinner, the cross section was to be reduced.

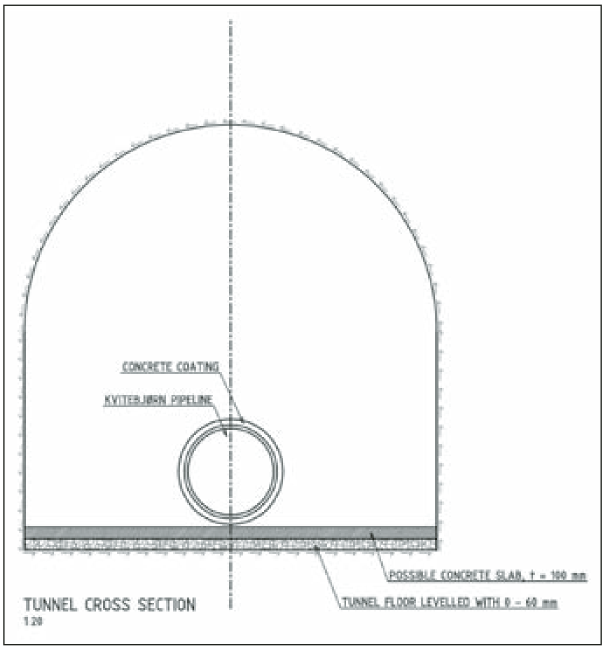

Figure 8: Cross-section of landfall tunnel.
(Illustration: Statoil / Norconsult)

Tunnel internals
The requirements for the internals of the tunnel had to take into consideration both the pull-in phase and the operating phase.

The tunnel was designed for a 25 years lifetime. For protection of the pipeline, necessary rock support was to be installed in the tunnel to avoid damage to the pipeline from rock down-fall in the pipeline life time.

The pipeline was to be pulled in from the lay barge through the piercing opening and through the tunnel by a winch, which was eventually located onshore at the Kollsnes side of Njupselsundet. The pipeline should be protected by a suitable method during the pull-in to avoid damage to the coating and the anodes. It was suggested that the pipeline should be pulled in on a concrete slab on the tunnel invert, see Figure 8.

CONSTRUCTION PHASE
The construction phase was divided in to stages, 1) trench and rig area excavation and 2) tunnel excavation.

Rig and Mobilisation Area
The rig and mobilisation area was established by excavating the tunnel entrance open cut large enough to facilitate the contractor's need for rig and mobilisation space. The rock masses from the submerged trench and the entrance cut excavation was used to temporary round off the adjacent terrain formations for later re-allocation to natural terrain.

Tunnel Excavation
The tunnel was constructed by the Norwegian contractor NCC Construction for the purpose of hosting one pipeline with an approximate outer diameter of 900 mm. The contractor chose an approximate 3 x 4 m cross section (width/height) as an optimal dimension based on the tunnelling equipment to be used, - resulting in an average cross-section of about 15-16 m2. The total length of the tunnel is 407 m, starting in a pit at elevation -0.8 m with an average decline of 1:6.2.

Due to the small rock cover thickness at certain portions of sub-sea tunnel and expected water bearing weakness zones, exploratory drilling to both sides of the tunnel as well as above the crown, was performed through-out the sub-sea tunnel length. At positions where exceptionally low rock cover was expected, holes were drilled out into the sea for verification purposes. Special packers were used to plug these holes after penetration of the sea bottom.

The tunnel was excavated by traditional drill and blast technique with a hydraulic 2-boom tunnel jumbo. The normal drill length in this tunnel was 4.5 m, resulting in approximately 4 m effective advance per round. Mucking out was performed with an LHD (Load-Haul-Dump) truck. Approximately half way down the tunnel, a niche capable of storing about one tunnel blast round of muck was established in order to reduce the time necessary to clear the face and to make an earlier restart of drilling for the next round possible.

Ground Water Control
During the whole construction period, grouting material and equipment was standby on the construction site. From the point where the tunnel passes the shoreline and continues sub-sea, 3 exploratory holes of 18 m length were drilled for every 3rd round. In case of water ingress, grouting would be performed as necessary. As a guideline criterion, grouting would commence if water ingress exceeded 5 l/min in one hole or 10 l/min in total for all the holes at one face.

The rock mass was found to be almost impermeable as the discontinuities were mostly filled with fine silt and clay. Some clay samples showed slightly swelling properties but these zones were small and confined in between solid rock mass. Throughout the whole tunnel, grouting was only performed at one location, in addition to the final 5-10 m before the piercing point.

Rock Support
Due to the small cross-section of the tunnel and the good geological conditions (rock mass quality, water ingress, stress etc.), the tunnel is mostly unsupported. Systematic manual scaling after each blast round took care of the safety aspect and reduced the need of temporary support to a minimum.

- Approximately 30 bolts are installed in the cut at the tunnel entrance. The bolts are 2.4 - 3.0 m long, Ø20 mm, galvanised and end-anchored with polyester resin. A total of 10 m of galvanised steel band has been installed between bolts to stabilise larger blocks.

Figure 9: Installed rock support in the tunnel entrance open cut area. (Photo illustration: Norconsult)

- 101 bolts are installed throughout the tunnel mostly in the roof and northern wall. The bolts are 2.4 - 3.0 m long, Ø20 mm, galvanised and end-anchored with polyester resin. 2 m of galvanised steel band has been applied. The Client's project engaged engineering geologist has marked out in detail all bolts installed on tunnel mapping sheets.

- No shotcrete was applied in the tunnel.

Rock Mass Conditions
The tunnel construction work was performed without any major difficulties. The only few halts were caused by breakdown of machines and pumps.

The most challenging area due to geological conditions was found over a 20 m long distance between chainage 275 m and 295 m, caused by 3-4 densely jointed distinct zones with clay, silt and crushed rock mass spaced some 4-5 m apart. The excavation in this area was performed by shorter rounds, 2 - 2.5 m long and thorough scaling of the roof and walls of the tunnel.

Figure 10: Tunnel portion with repeated parallel clay zones. (Photos: Statoil / Norconsult)

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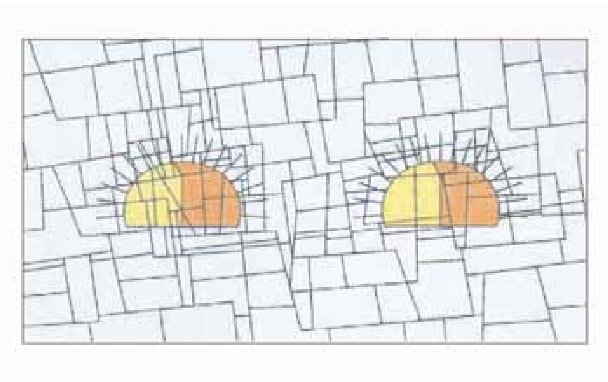

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8.1 UNDERWATER TUNNEL PIERCING A NORWEGIAN SPECIALITY DURING THE LAST 100 YEARS

Øyvind Solvik

Underwater piercing of tunnels was generally connected to development of hydroelectric power schemes with the intention of utilizing the potential of lake reservoirs beneath the natural outlet for power production. Such tunnel piercing has been carried out for 100 years, but not exclusively connected to hydroelectric power development. Nevertheless, the first underwater piercing in Norway was the lowering of the Lake Demmevatn located on the west side of Hardangerjøkelen in south west Norway. In this particular case a glacier dammed up the Lake Demmevatn serving as a particular unreliable weir that could break through any time and cause uncontrolled destruction to the valley Simadalen being located downstream the lake. A tunnel was excavated below the bottom of the lake and the piercing was performed at 20 meters water depth.

During the first half of the 20th century a great number of underwater tunnel piercings were carried out at moderate water depth in connection with hydroelectric power development, and in the years before the last world war some hundred underwater piercing were already completed. There is little information about the methods that were used at that time and how successful these piercing blasts were. It is assumed that the contractors had their own procedures of execution and that limited documentation existed, but some times a combination of good luck and good management prevailed. If something went wrong, the job was often completed in the best possible way without any particular publishing or documentation of something that might be considered a blunder. The worst case for a contractor was an unsuccessful final blast without achieving a successful break-through. It was considered a major risk to order people to enter the tunnel face as it could not be granted that the water would not break through, which would be a disaster.

There are good reasons to presume that the previous investigations regarding rock mechanical and geological aspects were less comprehensive than the requirement of today when designing an underwater piercing. Some times serious landslides took place especially in places where marine clay appeared. In one particular case including marine clay it was suggested that the compensation cost for damages caused by an underwater landslide amounted to the same cost as the constructional cost.

In this period the development of hydroelectric power schemes comprised mostly of medium to small power plants and although such damage cost could cause considerable financial deficit to anyone single hydroelectric power project, such incidents may not have an effect to the national economy. It was generally acknowledged that such underwater tunnel piercings provided a cost effective utilisation of the water reservoir in most cases. The method became internationally known as “The Norwegian Method”.

The rebuilding of Norway following the last world war involved a particular focus on hydroelectric power development, which also called for an optimum exploitation of draw down reservoirs. The submerged tunnel piercing became more difficult as the limits of experience were exceeded. Some unsuccessful cases made it clear that the physical processes involved in the piercing were not fully understood and this called for more research work. SINTEF-NHL had the capacity and competence to address the problems appropriately at the hydro-technical laboratory in Trondheim which was reputed for its problem solving ability associated with the development of hydroelectric power. By means of scaled model testing it was now possible to study the flow conditions inside the tunnel during the blasting process and if necessary introduce improvements to the design. The first physical model test was done in the beginning of the 1960`s and marked the commencement of a research programme that was carried out during several decades along with the most active years of the hydroelectric power development in Norway.

As a result of this comprehensive research, it is justified to say that today knowledge exists on how to work out the design for underwater tunnel piercing and execute the blast in a successful manner, also with water depth which earlier was classified as a hazard.

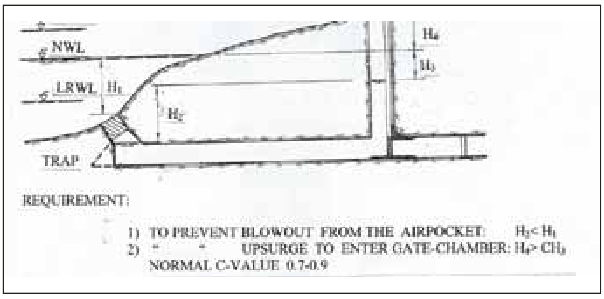
It should be noticed that the modern oil industry has taken advantage of the experience and knowledge gained through the comprehensive hydropower development concerning the shore approach solutions for the pipelines in the North Sea. The deepest underwater tunnel piercing ever done was in this connection at approximately 200 m. The technique was also used for a number of cool water tunnels at the land-based oil terminals.

Since few of these tunnel piercing are identical with regards to water depth, geological conditions, rock quality, location of closing gate etc. a variety of different design solutions for the underwater tunnel piercing were gradually executed. The most applied methods were found to be 1) a system open to the atmosphere and 2) a system with an air pocket isolated from the atmosphere. These two main systems are used considering that both systems may be used simultaneously in cases that comprise more than one single blast:

1) THE OPEN METHOD
2) THE CLOSED METHOD

1) THE OPEN METHOD

The main characteristic of this method is that the plug and the tunnel has an open connection to the atmosphere through the gate shaft or a cross-cut.



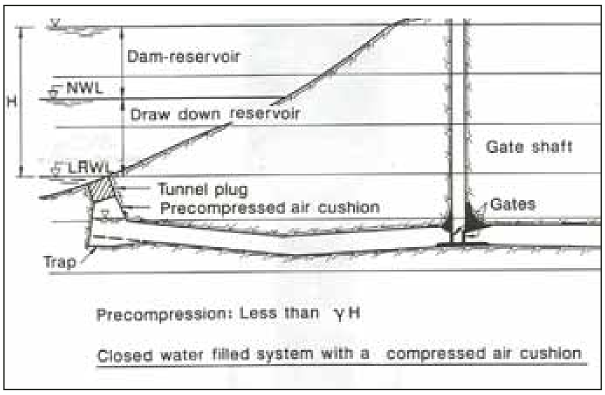
The open method. Fig. 1

The open method without water filling will set up high inflow velocities after the blast and complicate the design and construction of an effective debris trap. The upsurge in the gate shaft will usually be unacceptable unless particular mitigation measures are introduced. Water filling is therefore used in most cases. This must be done following particular criteria set up to avoid failure. It is important to make sure that the water level under no circumstances is covering the charges since explosives set off in water will cause destructive shock waves towards the gate. The filling level in the shaft must be sufficient to prevent the upsurge to enter the gate house, and at the same time not be too high so that the pressure in the air pocket, covering the plug, is higher than water pressure outside the plug.

These requirements are shown in Fig. 1 and since they may be contradicting each other, they call for instrumentation to check the water level in the pocket at the plug and in the shaft. If the blast by a mistake or ignorance is initiated in water which is coherently covering the gate, unacceptable damage may occur.

If the water is filled up in accordance with given guidelines the inflow velocities will be reduced and make the collection of the debris in the trap easier and comprehensive pre-calculation may not be necessary. On the other hand, if the situation for different reasons do not allow for a recommended water filling to reduce the water velocity, comprehensive pre-calculation or model tests are required. The open method is adequate for a physical model test and this has been used frequently.

2) THE CLOSED METHOD

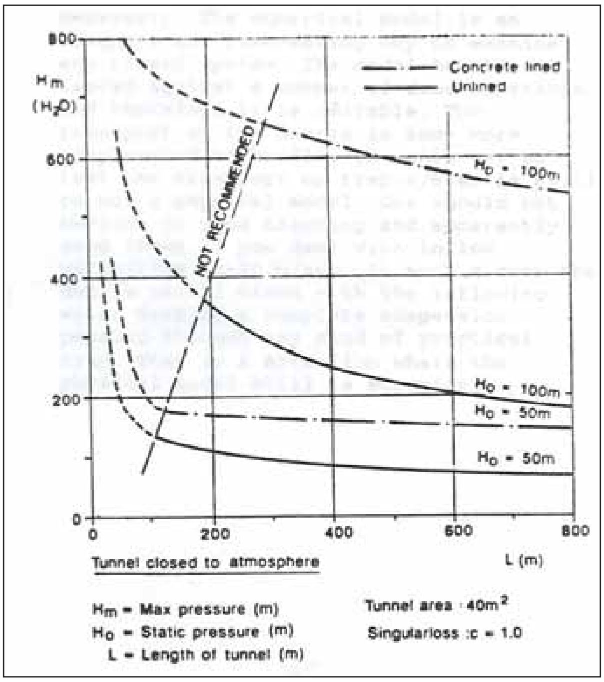


The closed method. Fig. 2

The closed method is characterized by the air pocket at the plug not being connected to the atmosphere and then compressed by the inflowing water after initiation of the final blast round. Such a situation is more demanding and calls for comprehensive pre-calculation and evaluation. If the planning and execution is done by experienced personnel, the closed method is more flexible and safer than the open method and preferable in cases where the open method is unsuitable.

Pre-compressing of the enclosed air pocket makes this method applicable and often it is the only recommended solution especially when dealing with small air volume and high pressure. Pre-installation of pressure transducers to check the pre-compression and the water level is necessary.

The closed method may also be used without pre-compression of the enclosed air pocket and little or no water filling at all, but only if the length of the closed tunnel is considerably larger than the length of the water column outside the plug. As an example, if the tunnel length is 10 times the water depth, the maximum pressure after the blasting will not exceed the static pressure by more than 10%.



The closed method. Fig. 3

Fig. 3 shows how the maximum pressure decreases with increasing length of the dry tunnel. It also shows the influence of another important parameter, namely the tunnel roughness. It should be noticed that the maximum pressure in a concrete lined tunnel will more than double the pressure compared to an unlined rock tunnel.

The closed method calls for advanced computerised calculations. In addition, the computer model has to be calibrated as many empirical factors are involved in the model. These factors must be determined by measurements during the execution of the piercing by the closed method.

There are many factors that affect the maximum pressure in a tunnel closed to the atmosphere such as the external water pressure on the plug, the area and volume of the plug, the amount of dynamite, area and length of the tunnel, the tunnel roughness and not to forget the pre-compression. The system receives energy from the inflowing water and the charge. The different losses are, hydro-mechanical losses, heat transmission between water, air and rock walls, etc. The difference between incoming energy and losses give information to calculate the maximum pressure.

The computer models which have been developed are calibrated based on full-scale measurements and have proved to be very reliable when provided with accurate information and input.

The closed method is less suited for physical model-test than the open method as such model-tests require special remedial action. The reason is that the atmospheric pressure is a dominating factor in the compression phase and should be scaled like the outside water pressure. This is very complicated. Other simulations have been tried for the purpose of carrying out physical model tests, but insofar they are not to be recommended compared to the computer models.

The closed method has been applied in a lot of cases at different water depths. The deepest ever done was in connection with the oil activity in the North Sea. To enable a shore approach for a pipeline from the North Sea an underwater piercing was performed at 200m water depth to connect the pipeline to a tunnel to reach the land based terminal. In this case the closed air volume was limited and consequently a very high pre-compression had to be used. Both the explosives and the detonators had to be adapted to the actual water pressure. In this case the pressure measured during the blast, closely corresponded to the pre-calculations. The uncertainty in such calculations is mainly connected to the estimation of gases from the detonation of explosives and expansion of the volume outside the plug. The uncertainty in pre-calculation is less when high pressure is used but is not considered to be determining factor. More important is that the plan of approach has been prepared thoroughly. If high pre-compression has to be used, it is important to make sure that blasting can be executed on short notice.

A rough description of a plan of approach regarding the closed method with a limited air volume is as follows: Water-filling to protect the valve against debris from the plug, calculation of the remaining air volume and then the choice of preliminary or final pre-compression. The detonation of the explosives will increase the pressure and if the probable pressure becomes less than the external water column, post-compression will take place caused by the inflowing water and result in the maximum pressure in the air pocket. The pressure on the gate may be corrected according to the location of the gate.

Such calculations must be done by skilled personnel preparing specified and dedicated procedures to design a successful underwater piercing. It has become common to carry out measurements of pressure build up during the execution of underwater tunnel piercing using both the open and the closed methods. The results have been used to correct the computer models and improve the other calculations that are necessary to further develop and modify the method which is still named the "The Norwegian Method".

8.2.1 DRILLED LANDFALL FROM A ROCK TUNNEL INTO THE NORTH SEA AT KALSTØ, NORWAY

Trond Øiseth

The Norwegian oil company, Statoil needed a safe solution for their new oil pipeline from the Åsgard off shore oil field to the Kalstø Refinery at the West Coast of Norway.

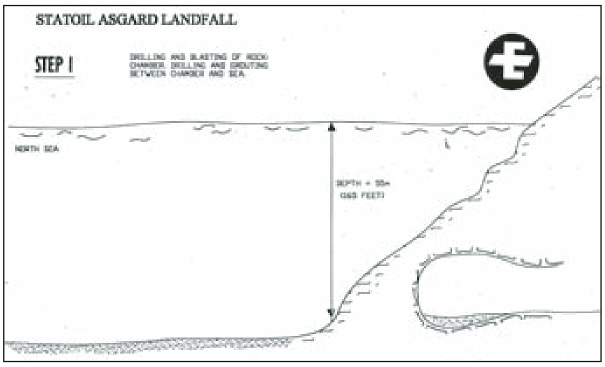
Statoil decided on a pipeline from the seabed, at a water depth of 55m, through a rock tunnel system. The connection from the sea into the tunnel was considered to be one of the most critical and difficult parts of the landfall project.

The project engineering in detail started in June 1997 and was completed in January 1998 and concluded with a drilled tunnel from a rock chamber through a seal tube system as the best solution. Entreprenørservice AS was awarded the contract for the horizontal drilling (pilot hole drilling and reaming) of the 1600m m diameter tunnel with their Raise Boring Machine, Indau R 90 H. The drilling operation took place in the period 22 February – 06 April 1998.

The following figures will show the procedure, step by step, for the drilling of the approx. 30m long landfall tunnel at Kalstø:

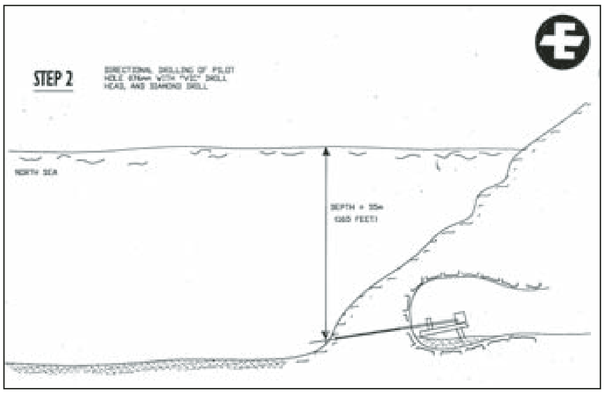
STEP1

Drilling & blasting, and preparation of the drilling chamber with a concrete platform for the seal tube and the raise boring machine. In order to prevent sea water from flowing uncontrolled into the tunnel system, the rock massive between the sea and the chamber was grouted through drill holes.



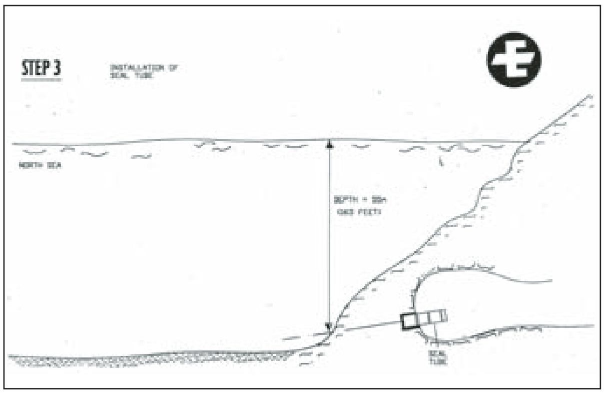
STEP 2

A 76mm diameter directional drilled diamond core hole in the centre of the micro tunnel was fulfilled to assure the accurate length of the micro tunnel.



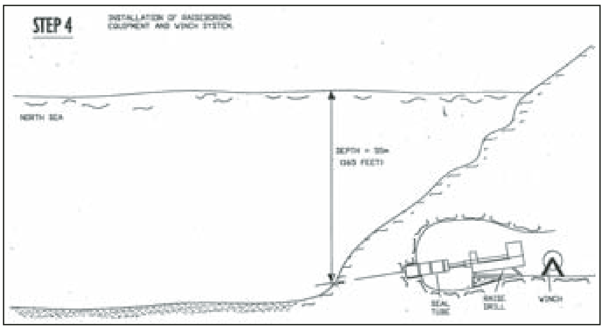
STEP 3

Installation of the seal tube system with different diameters to secure that sea water would not flow into the chamber. The seal tube had packers and valves for the 311mm diameter pilot hole and for the 1600mm reaming diameter of the drilled micro tunnel, and finally, a seal system for the oil pipeline.



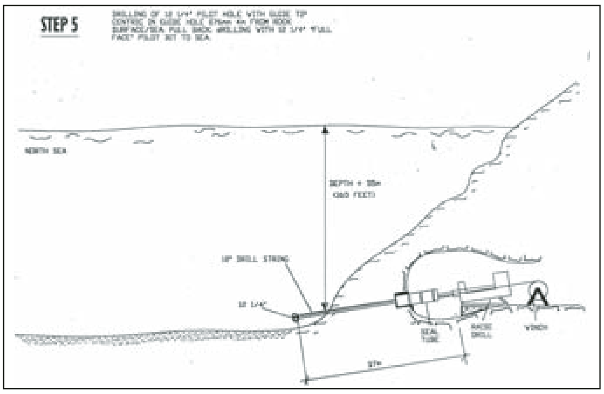
STEP 4

The raise boring equipment and a winch system were installed.



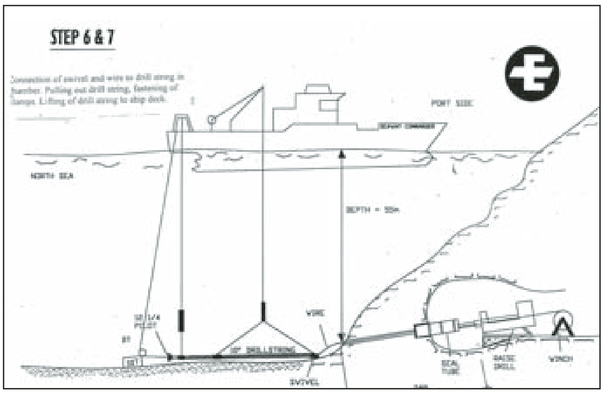
STEP 5

Reaming of the 76mm diameter diamond core drill hole to 12 ¼ inch (311mm) diameter by use of a roller reamer with a guide tip from the chamber, through the seal tube and out to the seabed.



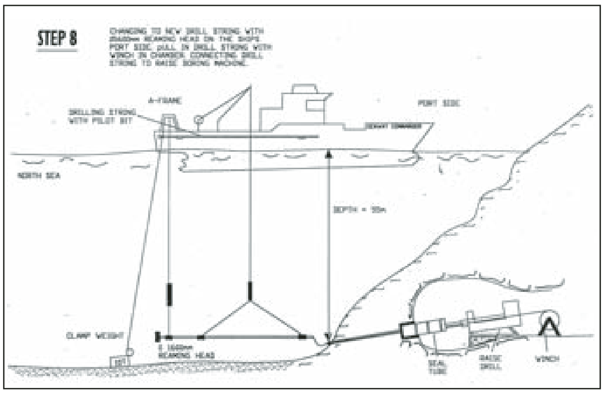
STEP 6 AND 7

Connection of swivel and wire from the winch to the inner end of the drill string in the chamber. The 37m long, 10 inch diameter drill string was then pulled out of the pilot hole by means of a winch onboard the vessel, "Seaway Commander" and a block wheel mounted on a 10 ton counter weight in the bottom of the fjord. Then clamps were fastened to the drill string and the drill string with pilot bit/roller reamer was then lifted up and placed on the deck of the vessel.



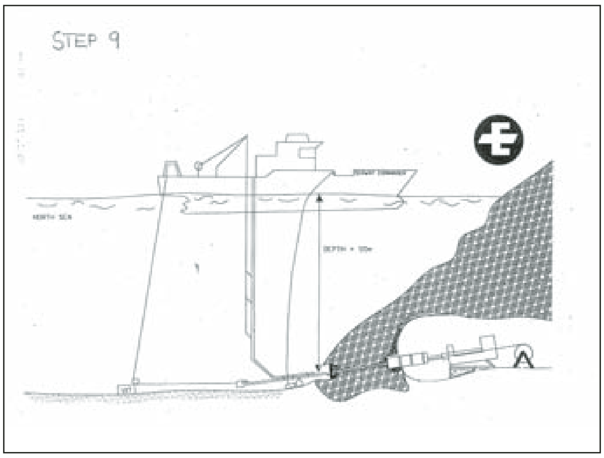
STEP 8

The swivel and wire from the winch in the chamber, attached to the pilot drill string were moved to a new drill string connected to a 1600mm diameter reamer head. The new drill string was then lowered into the sea by means of cranes onboard the supply vessel. Then the drill string was pulled through the 311mm diameter pilot hole by use of the winch in the chamber and connected to the raise boring machine.



STEP 9

Reaming of the 1600mm diameter tunnel from seabed to the seal tube. During the boring operation the supply vessel was operating a vacuum system to clean out the muck from the reaming process. When the reaming of the tunnel was completed, the reamer head and drill string were pushed back to the seabed by the raise boring machine with some pulling help from a winch onboard the supply vessel. All of the equipment on seabed was loaded onboard "Seaway Commander" by use of its own deck cranes.

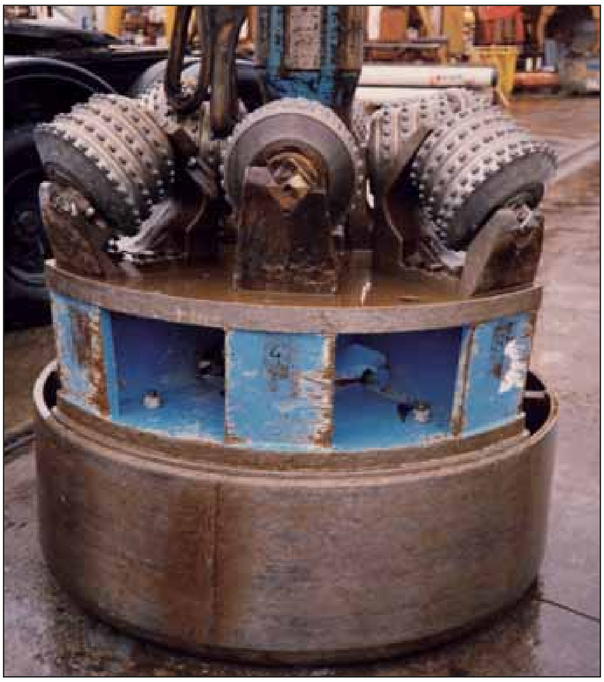


CONCLUSION

Thanks to experienced people and very detailed planning of the project the whole operation became a success. For a supply ship surging in heavy seas, it is very important that the ship has good and powerful engines, an accurate positioning system, as well as a trained crew. The whole operation was performed without any diver support at all. A WROV (Workclass Remotely Operated Vehicle) was used for inspection of the activities in the sea during the operation.

PICTURES:

1. Boring through the seal tube system in the background. In front the red painted raise boring machine.



2. The 1600mm diameter reamer with stabilizer ring.

3. The reamer head connected to the drill string, prepared with lifting clamps and ready to be loaded onboard the supply vessel.

4. The drill string is being pulled through the seal tube system by the winch, prior to the reaming operation.

CODAN

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8.2.2 KVITEBJØRN RICH GAS PIPELINE PROJECT - KOLLSNES LANDFALL - UNDERWATER TUNNEL PIERCING

Arild Neby
Thomas K. Mathiesen

ABSTRACT:

Pipelines from offshore oil and gas fields in the North Sea are serving several onshore process plants on the Norwegian west coast. The shore approach itself is normally a challenge as the sub-sea topography off the coast is rugged and seldom facilitates landfall sites with gently sloping sandy beaches coming up from the continental shelf and slope. Large diameter bore holes drilled from within the process plant area through the rocky barrier and out in the sea have been a common solution for landing pipelines where other methods have not been available. The bored solutions have however often proved to be expensive.

For the Kvitebjørn Rich Gas Pipeline the conceptual solution was a bored landfall with an optional tunnel solution alternative. During the detail design phase a feasibility study on a tunnelled solution combined with underwater tunnel piercing techniques, adapted from the hydropower civil works sector, revealed that for this project the tunnel solution was feasible.
A successful underwater piercing blast connected the Kvitebjørn Landfall tunnel to the sea 12 September 2002.

INTRODUCTION

The Kvitebjørn gas and condensate field lies in block 34/11, east of Gullfaks in the Norwegian North Sea, operated by Statoil. Production from Kvitebjørn began in 2004. Rich gas and condensate (light oil) from Kvitebjørn are piped to Kollsnes near Bergen and Mongstad further north respectively.

Rich gas from Kvitebjørn is piped in the 147 km long OD 30" (Ø762 mm) Kvitebjørn Rich Gas Pipeline (KGR) to the process plant at Kollsnes (see Figure 1). After processing at Kollsnes, the dry gas is piped to continental Europe. The separated NGL is transported by pipeline to the Vestprosess plant at Mongstad for fractionation into propane, butanes and naphtha.

Condensate travels through the Kvitebjørn Oil Pipeline (KOR), which ties into the Troll Oil Pipeline II to Mongstad (see Figure 1). Based on current plans it is expected to recover roughly 55 billion cubic metres of gas and 22 million cubic metres of condensate.

The partners in the license are: Statoil ASA (43.55%), Petoro AS (30%), Norsk Hydro Produksjon a.s (15%), Total E&P Norge AS (5%) and Enterprise Oil Norge AS (6.45%)

The article covers the detail design and execution of the underwater piercing for the landfall tunnel as well as well as going briefly into the pipeline pull-in operation through the tunnel to the Kollsnes process plant.

The Kvitebjørn Landfall tunnel was constructed during the period May - September 2002. The final piercing at depth -66 m was executed 12 September 2002. The civil works part of the whole shore approach project was cost estimated to approximately NOK 25 million.

The pipeline was pulled in through the landfall tunnel to Kollsnes on 15 May 2003. Production from Kvitebjørn began on 26 September 2004. The field began delivering natural gas through the pipeline on 1 October 2004.

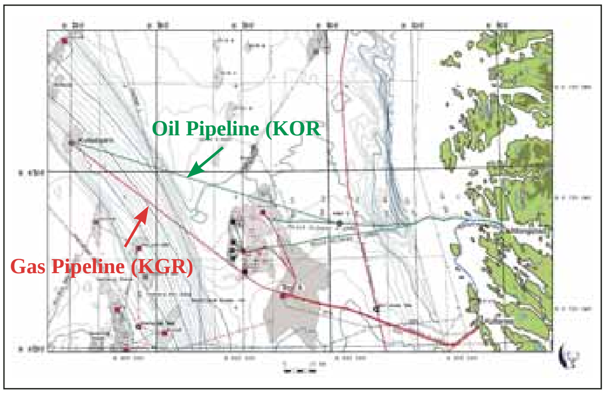

Figure 1: Map showing Kvitebjørn Pipelines (Illustration: Statoil)

DETAIL DESIGN PHASE

Design Basis

The basis for the solution of a landfall tunnel with a final underwater piercing blast to open the tunnel to the sea was as listed below:

- Tunnel data:
  Length approximately 400 m. Slope 1:6.
  Water filled during pull-in.
- Pipeline data:
  ID : 710 mm, wall thickness: 28,7 mm
  Coating : 6 mm asphalt enamel and 50 mm concrete weight coating
  OD incl. coating : 880 mm
- Level of piercing point should be between 2.5 and 4.5 m above sea bottom which was situated at approximately -68 m depth.
- Rock surface interpretation at piercing point were to be based on sea bed mapping and ROV view survey (videos), until verification data from exploratory drilling become available during construction.
- Offshore dredging work after piercing execution and before pipeline pull-in should be avoided. This criterion was probably not to be fulfilled for the conventional piercing method, but the design of the final blast should emphasize on controlling the debris inflow, in order to avoid or minimise such work.
- The breakthrough piercing blast was to be performed with a partially water-filled tunnel, ensuring that most of the blasted rock was flushed into the spoil trap. If any complementary work, such as levelling or smoothening, was needed, this was assumed performed by an ROV, by seabed based excavator or by dredging from a ship. Dredging carried out from a ship, was believed to probably be the fastest and easiest method.
- The pipeline should be protected by a suitable method during the pull-in to avoid damage to the coating and the anodes. It was suggested that the pipeline should be pulled in on a concrete slab on the tunnel invert,

Geological Conditions of the last 30 m of tunnel
The predicted geological conditions and the topography for the underwater piercing area was based on interpolation of surface mapping on the island "Storholmen", on information gathered from a core hole drilled parallel to the landfall tunnel some 40 m away and refraction seismic survey, carried out in 1990 for the sub-sea pipeline tunnels forming a part of the landfall for Troll Phase I Project.

Two seismic sections cover the area along the planned sub-sea tunnel from Storholmen to the seabed piercing point. The first section, which is following approximately the planned route of the tunnel, indicated a zone with low to medium seismic velocity located 100 - 110 m from Storholmen (11) as well as one major weakness zone creating the escarpment holding the piercing point. The second section, which is perpendicular to the planned tunnel route, is intersecting the first section 160 m from Storholmen. This section indicated a 10 m wide weakness zone located about 30 m to the north of and in parallel to the tunnel alignment. Apart from these zones the sections only yielded velocities that were high (5700 - 6000 m/s). Neither of the sections indicated any loose material above the rock seabed.

The rocks within the piercing point area were predicted to consist mainly of granitic gneisses with some minor layers of dark amphibolitic gneiss, as for the rest of the tunnel. These layers of amphibolites often display delamination and are sometimes accompanied by higher joint density and/or clay-filled joints. Pegmatite lenses had been observed from the core samples and at the island, and could be expected to occur also in the underwater piercing point area.

Joints along the foliation form the main joint set 1. The foliation of the gneiss strikes generally NE-SW with a gentle dip (0° - 15°) towards SE. The joint spacing was observed to be mainly between 0.2 and 2 metres. The

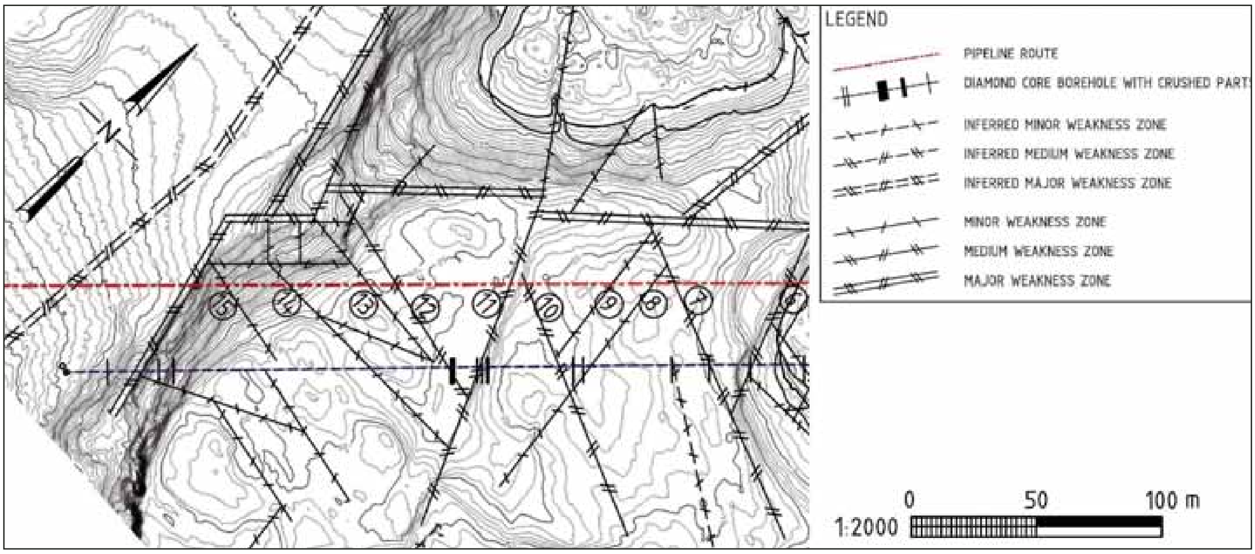

Figure 2: Engineering geological map showing the piercing point, pipeline route and location of core drilled hole. (Illustration: Norconsult)

two other joint sets are steep-dipping and occur approximately normal to joint set 1.

In general, the joints divide the rocks into blocks, which vary between 0.1 m³ and 3 m³. Most joints have rough joint surface with slightly undulating joint planes. In some zones the spacing between the joints is so short that the zone may have the character of a crushed zone. The identified weakness zones shown on Figure 2 generally follow the direction of the two steep-dipping joint sets.

Generally the geological conditions were found to be relatively good, and hence, favourable for a sub-sea tunnel. Uncertainty regarding permeability of the rock mass, especially in the piercing area, was to be given special attention during construction phase. Thorough and continuous routines for exploratory drilling and grouting, throughout the entire excavation period of the landfall tunnel as well as at the piercing itself, were a prerequisite.

UNDERWATER TUNNEL PIERCING - PRELIMINARY DESIGN

Principal method

For the final piercing of the remaining rock plug, a final blast round using the same principle as for submerged water intakes, often referred to as “Lake Taps”, was considered the most feasible solution for this project. This involved preparing the final breakthrough blast from the face of the tunnel, allowing the blasted rock material to settle in a pre-prepared spoil trap inside the tunnel. An ROV could then be used to fetch a wire located inside the tunnel to connect this to the pipeline coming from the lay barge.

The method comprised that normal excavation procedures were followed up to a point 30 m to 100 m from the final rock plug location, depending on the local rock mass conditions. From this point on special excavation procedures would commence, involving careful blasting with divided cross sections and extensive use of exploratory drilling and grouting as part of the excavation cycle up to the final rock plug.

Requirements for the tunnel piercing

Space requirements

The pipeline is approximately 900 mm in diameter. However, the minimum cross section at the piercing point was thought to be determined by the space requirements for the ROV needed to pick up and connect the pull-in system wire to the pipe. Considerations were also made to the possibility of facilitating a second future pipeline in the cross section of the final plug, as well as the blast geometry was governed by the consideration and the space needed to facilitate removal of debris.

Geometrical requirements

The pipeline axis through the piercing point should have the same incline of 1:6 as for the rest of the tunnel. Hence, the minimum vertical opening of the piercing should correspond to this. Two alternative solutions were proposed for the final plug:

- Alternative 1, which aims to remove a large part of the overhanging roof section at the piercing point.
- Alernative. 2, which is a horizontal piercing forming a tunnel.

For both solutions the rock mass from the final blast was intended to settle just outside the opening, and in the spoil trap inside the tunnel.

None of the solutions could guaranty that no debris would settle in the opening. The choice between alternatives was to be done based on a cost/benefit evaluation, considering risk for remaining debris and the procedures to remove such material from the path of the pipeline.
The rock mass in an underwater piercing blast would

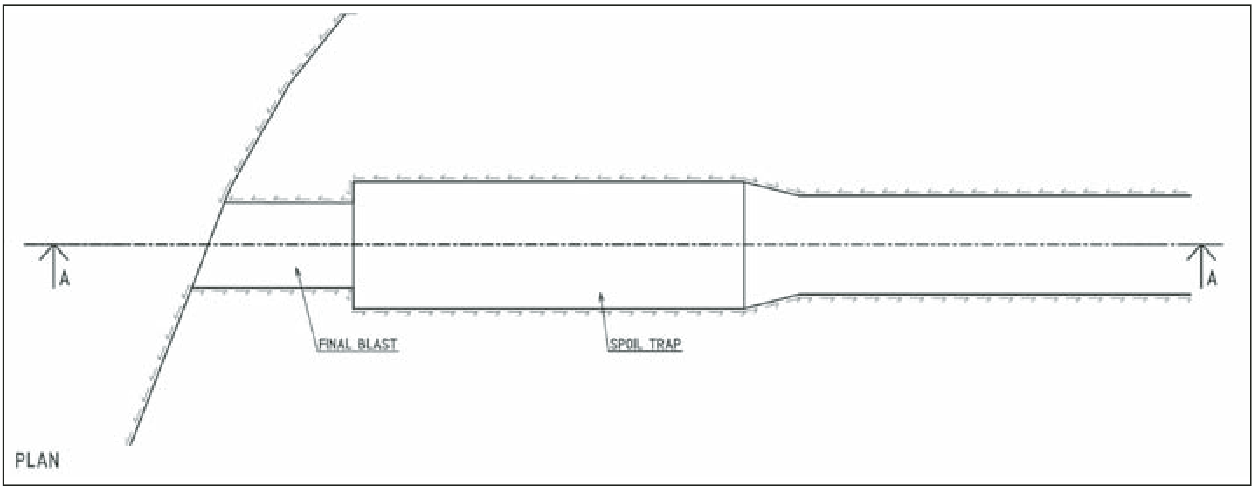

Figure 3: Preliminary design - plan view of underwater piercing blast round, piercing chamber and spoil trap. (Illustration: Norconsult)

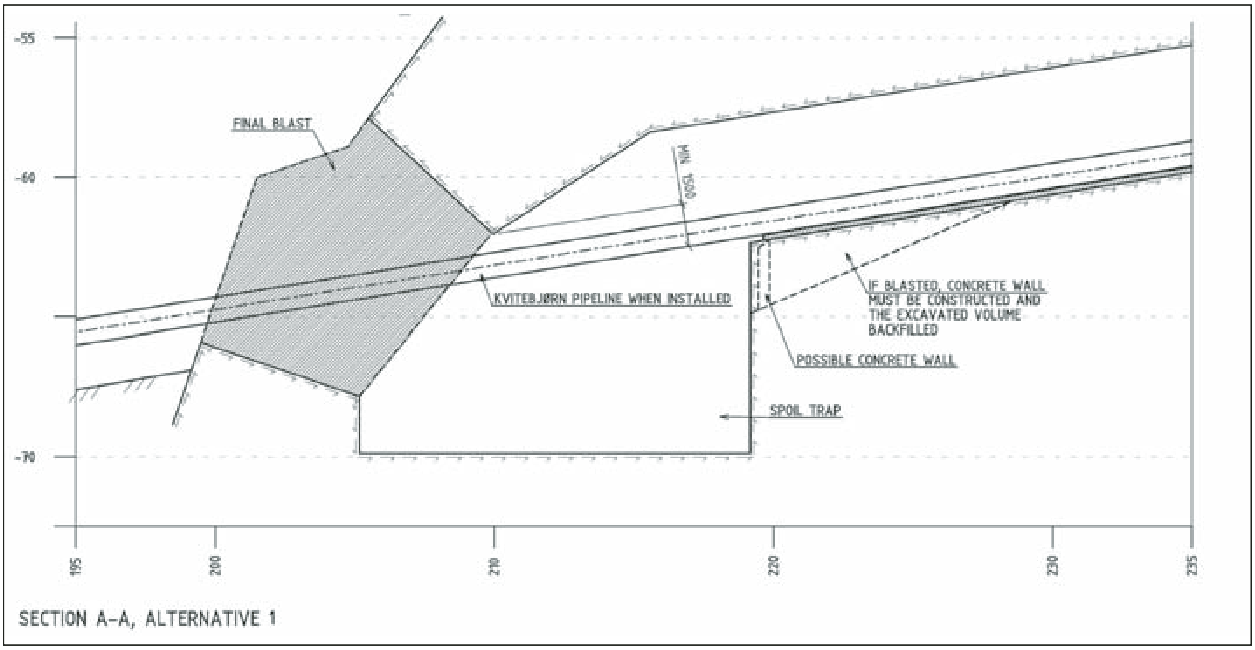

Figure 4: Alternative 1 - section view of preliminary underwater piercing blast round. (Illustration: Norconsult)

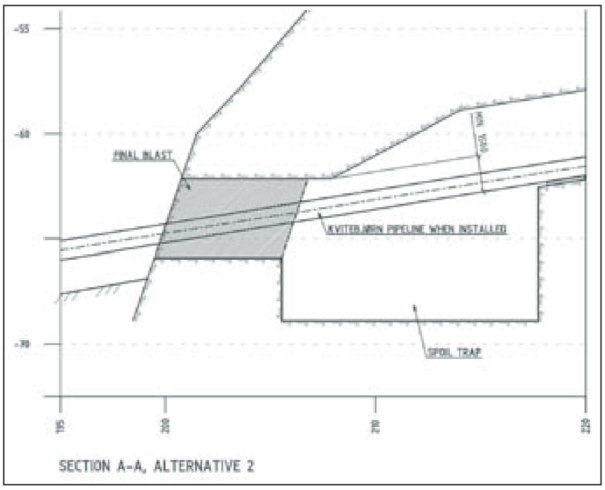

Figure 5: Alternative 2 - section view of preliminary underwater piercing blast round. (Illustration: Norconsult)

normally expand more than the usual 50 - 60 % without significant scattering. The drilling pattern and the blast round were to be designed to result in best possible fragmentation. Towards the seabed side the spoil material was expected to become coarser, possibly containing some block fragments.

Some break out and sliding outside the blast contour could be expected to occur, possibly resulting in rock blocks mixed with the blasted rock material. Bolting from inside the tunnel prior to the final blast was recommended to prevent such sliding to some extent, however loose blocks from the seabed could not be secured. Apart from material from the seabed and from outside the blast contour, the material was considered to be finely fragmented, from fist-sized fragments and smaller, due to the high specific charge.

Blast shock wave requirements
It was assumed that there were no blast sensitive installations at the piercing point or in the tunnel. Hence, no shock wave reduction measures would be implemented. Further, the tunnel was assumed partially or nearly completely water-filled, in order to control the flow of debris from the final blast. Hence no significant upsurge was expected.

An underwater detonation would, however, induce a shockwave to the surrounding seawater. The spherical propagation of this wave causes a fast dampening of the energy. Nevertheless, proximity to boats, swimmers and particularly aquaculture industry was to be further evaluated.

Procedures for exploration drilling and grouting
When approaching the final plug, the principle of at least three blast round overlaps between the exploratory drillholes was to be maintained. As the rounds become shorter towards the final plug, the exploratory holes were to be shortened in order not to break through to the sea.

When the excavation is close to the start point for the final rock plug, at least 8 holes were to be drilled through to the seabed in order to determine the exact thickness of the remaining rock mass at the tunnel piercing point. It was also considered important to investigate the minimum distance to the seabed, which possibly was not straight ahead. On basis of this, the design and trimming of the final blast round was to be performed. The holes drilled through to the surface were to be plugged. This could be done by long tapered wooden dowels or packers.

When approaching the final rock plug, the chances of performing undesired hydraulic jacking of fractures in the rock mass by grouting at too high pressures increase. It could therefore be required to carry out grouting adjacent to the final rock plug by means of chemical grouts like polyurethane. Compared to cement-based grouts, chemical grouts do not require high pressures to fulfil the sealing functions in the fractured rock mass.

The procedures to be implemented depend on the detected seepage pattern determined by the exploratory drilling and could therefore not be defined in detail until further information was obtained on site during construction.

Procedures for careful excavation - spoil trap and final blast

Depending on the overburden and the rock mass quality, at least the last 30 m of tunnel towards the final rock plug was to be excavated with caution making sure that as little as possible of the rock contour was damaged by the blasting. The tunnel was to be excavated with pilot and benching. Further, the round length should be gradually reduced as the tunnel approaches the plug. The final trimming and drilling of cut- and blast holes for the plug was to be done from the muck pile before the spoil trap was cleared of remaining muck.

Final rock plug thickness

Successfully performed underwater piercings have in general been carried out on rock plugs with thickness from 2 m in solid, good rock to 10 m at locations with adverse rock mass conditions in combination with large tunnel cross-sections. The soil overburden has usually varied from 0 to 6 m.

The final piercing blast drill plan should either be circular or rectangular with chamfered corners in order to ease setting out and drilling as well as ensuring less confined blasting and in theory the most stable geometry of the remaining opening.

Based on the information available at this preliminary stage, the minimum distance between the chamber and seabed was recommended at 6 m. Further reduction of this distance was to be evaluated during trimming of the chamber when the rock mass conditions were actually exposed.

Initial and final rock support

The core drilling performed along the alignment of the tunnel generally suggested favourable rock mass conditions. However, some crushed zones were encountered, and at least one zone with significant water leakage has been detected. Further, video images from an ROV survey at the piercing point indicated that there could be significant joint systems normal to the tunnel at the piercing point, which could cause stability problems and/or water leakage.

Based on this it was recommended to stabilise the rock mass over the tunnel piercing by installing rock bolts from the piercing chamber before the final blast. This and the need for grouting was to be determined when the exact geometry of the seabed and the rock mass conditions were revealed at the face.

With regards to rock mass stability after the final blast, Alternative 1 was considered the most flexible solution, as most of the potential unstable rock directly above the piercing was removed, and since this alternative also enabled easy access for potential remedial measures such as dredging in case that some of the rock debris remains in the path of the pipeline.

Alternative 2 would require possible removal of rock debris by ROV.

Drilling of the piercing blast round

The drill pattern design in the preliminary design was to be based on the use of “Extra Dynamit 35□600 mm” and reinforced special edition Gr. 1 millisecond electrical detonators, both tested for the actual water pressure and the time the explosives and detonators would be exposed to seawater.

Normally the breakthrough piercing blast would be drilled with the same equipment as the rest of the tunnel. However, in case of significant remaining water leakage, it could be necessary to drill Ø 2½” holes and use plastic casing with an outer and inner diameter of 59 mm and 52 mm respectively. The drill pattern was to be designed to compensate for potential blocked holes. It was stated that the piercing blast round was to be drilled before the spoil trap was cleared.

Required drilling accuracy was indicated as follows:
- Collaring ± 5 cm
- Drill deviation max 5 cm/m
- Length of holes -0.5 m to -1.0 m from piercing of the seabed.

Charging

The water pressure was anticipated to be approximately 65 m. It was assumed that the time between charging and the final blast would be maximum 5 days. The tunnel was assumed partially water filled, with some water pressure at the face. The explosives for the final blast should therefore be “Extra Dynamite” with a high content of blasting oil, or an equivalent type of explosive with the same water resistance quality.

The following procedures were recommended to be followed:

- Both “Extra Dynamit 35x600 mm” and millisecond detonators with protective sheet and 6 m lead wires, were to be designed to withstand 80 m water pressure for 7 days. Suppliers were to document that the delivered lot has been tested and were fulfilling the requirements.
- Before charging, all holes were to be controlled with a stemming rod.
- All holes were to be charged as determined on the charging plan.
- In all charged holes, 2 separate detonators with the same number were to be used, each with its own intact 6 m lead wire.

Stemming
The uncharged part of the holes was to be stemmed with stemming plugs of polystyrene. To keep the charge and the stemming in place a wooden dowel, with a groove for the detonator wires, was to be used.

Specific charge
The specific charge would be dependent on a function of the cross section, length of plug and the type of piercing. Typically the specific charge for the two alternatives would be 5 and 6 kg/m3 for Alt. 1 and 2 respectively.

Coupling of detonators
At the time of the piercing blast, the face of the tunnel would have a water pressure determined by the level of water filling in the tunnel. All couplings of the leading wires were to be exposed to conductive seawater. Hence, it was required that all couplings were carefully sealed and watertight.

The two detonators in each charged hole shall be coupled in their own individual series, which are then coupled in parallel. The circuits were recommended to be checked using an approved ohmmeter; with a maximum allowed deviation of ± 1 %.

Firing cable - Isolation
Requirements and routines for firing were recommended to be as follows:

- The firing cable should be new and of high quality, and designed for the actual piercing blast round and the firing battery to be used.
- The firing location was assumed to be by the tunnel entrance.
- 2 separate firing cables, each 2.5 mm2 for the whole length without splicing, were to be used. (If the firing cables had to be spliced, this was then to be done with care making sure that the splice was sealed and watertight, using shrinkable tubing, silicon and tape. The distance between splices should be minimum 2 m).

UNDERWATER TUNNEL PIERCING - ACTUAL DESIGN

The contractor, NCC Construction, was responsible for the final planning, design and execution of the underwater tunnel piercing. In cooperation with Dyno Nobel, the explosives supplier, the contractor issued a set of detailed procedures for all activities related to the piercing blast operation. The procedures went through a commenting round with Statoil’s project engaged consultants, Norconsult and Sweco Grøner, prior to the final revision and issue.

Changes from Preliminary Design Basis
Basically there was only one change in the actual design compared to the preliminary design basis, but this one change caused a whole range of new requirements for the underwater tunnel piercing:

- The commonly used and preferred underwater explosive “Extra Dynamit” was no longer available, not even in stock, after an accidental explosion at the explosives factory, which forced Dyno Nobel to close down the whole production facility.

Without this preferred explosive only dynamite with somewhat better water resistance abilities than common dynamite was easily available for the underwater blasting operation. This dynamite could however not be guaranteed for the strict requirements of withstanding 80 m water pressure for 7 days.

As a possible solution, since there were no gates or closing devices in the tunnel that could be damaged by the hydrodynamic pressures from the inflowing water, the contractor suggested to perform the underwater breakthrough blast as a dry tunnel piercing without water filling in order to make use of the available explosives. The consequences of this method change were, however, several and significant:

- Water leakages in the rock plug area had to be minimised in order to reduce the chances of explosive malfunction due to water pressure.
- Extra protection of explosives by performing charging in plastic pipes.
- Adjustment of borehole diameters and stemming to ensure proper draining of blast holes.
- The atmospheric air environment at the tunnel face made it possible to change the detonation system from Group 1 electrical system to the much safer NONEL system.
- The spoil trap was removed from the construction drawings as this trap would not any more serve its purpose due to the high water velocities resulting from the 66 m water column pressure difference between the sea and the tunnel. Water velocities of 30 m/s was expected in the piercing opening, though gradually reduced to roughly estimated 25 m/s in the piercing chamber due to singular head losses in contractions and friction losses

in the wet tunnel periphery, in bends and in the water front. Such water velocities would easily transport the well fragmented blast debris over any spoil trap situated just below the piercing blast round.

- Even though the water was expected to slow down considerably on its way up the tunnel due to the gradually reduced pressure difference, gravity and friction loss, the velocity would be high enough to make the planned not reinforced concrete slab on the invert, buckle and break into pieces that could jeopardise the pipeline pull-in operation. The concrete slab had either to be reinforced and anchored thoroughly to the rock invert or simply just removed from the design. The latter was eventually chosen.
- Thorough cleaning of rock invert for larger loose blocks that could cause problems for the pull-in operation if not removed.
- Great concern and uncertainty for the unpredictable and complex flow of gas, water and rock debris in suspension with regards to transportation and sedimentation in the tunnel.
- Uncertainties of the upsurge level in the rig area onshore at the tunnel end.

Actual Underwater Tunnel Piercing Layout

With the design changes mention above and the decision that the piercing opening should be large enough to facilitate a possible second future pipeline, gave a piercing layout as shown in Figure 6. Based on ROV investigations of the pipeline route outside the piercing point the invert elevation of -66.0 m was chosen as the final target of the piercing opening.

For the final piercing a rock plug of 3.5 - 4.0 m was left intact. A total of 5 probe holes were drilled from the tunnel face to the sea penetrating the seabed to explore the exact length of the final rock plug.

Actual Drill Plan

As shown in Figure 7, the final blast round consisted of 58 holes, which were charged with a total of 180 kg of explosives. A total of 7 holes of diameter 4” were left uncharged as empty holes in the cut area. The blast round geometry was as such slightly conical going from a cross-section in the piercing chamber of 4 m x 4 m, to the collaring at the face at 3.5 m x 3.5 m, ending up in a 3 m x 3 m opening at the seabed.

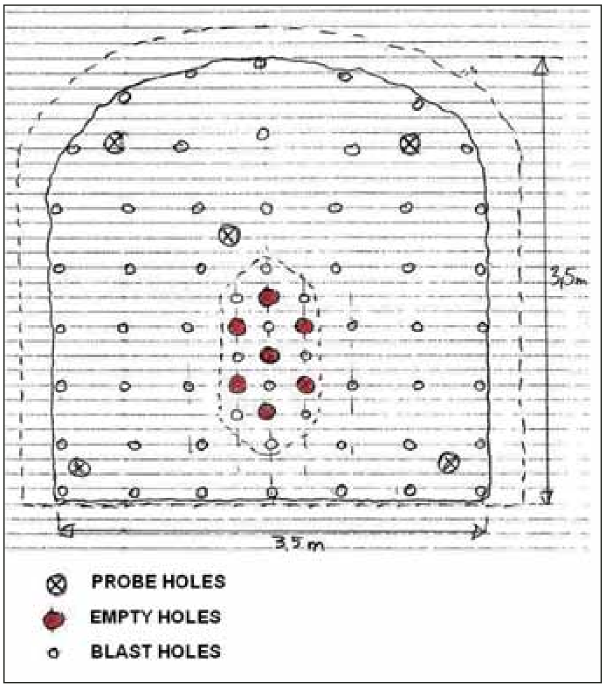

Figure 7: Final drill plan for underwater piercing blast round showing probe holes, empty holes and blast holes. (Illustration: Dyno Nobel)

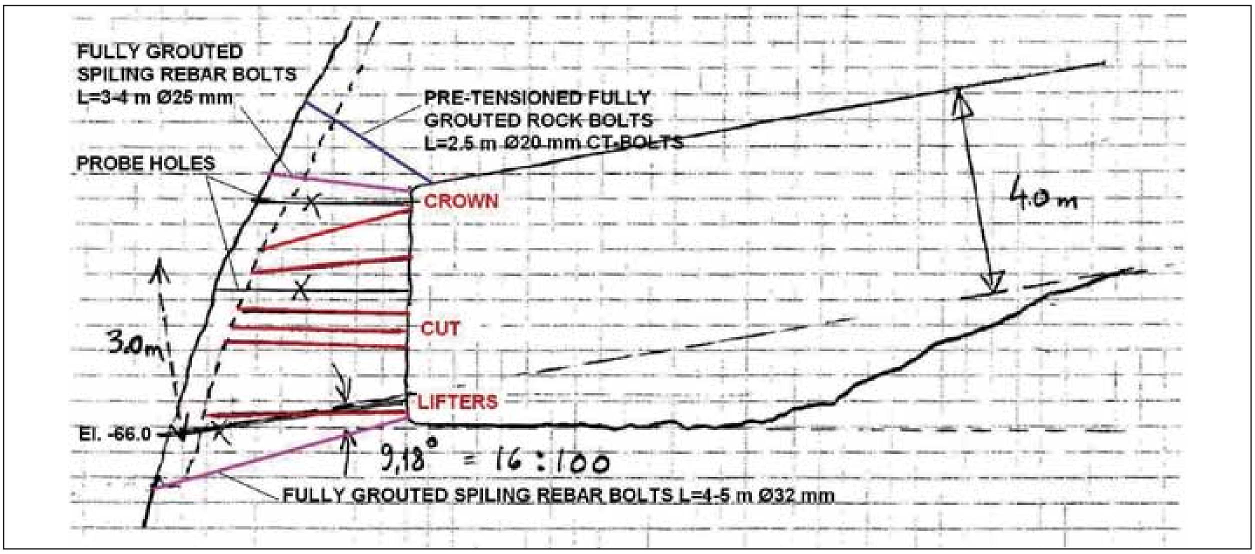

Figure 6: Final layout of underwater piercing blast round showing probe holes, category blast holes and final rock support. (Illustration: Dyno Nobel / Norconsult)

Actual Ignition Plan and Connection of Firing Cables

The eventual ignition plan consisted of a combination of millisecond and long period tunnel detonators as shown in Figure 8. NONEL MS detonators with intervals from 3 to 13 were used in the cut area. NONEL LP detonators with intervals 0 and 4 -11 were used for the rest of the blast round. The LP 0 detonator was utilised as the instantaneous blast opening detonator. The total delay in the blast round from opening of the cut to complete piercing opening should then amount to approximately 1.1 seconds.

Two detonators of the same interval, one in the bottom and one in the middle, were used in each hole. With NONEL hose lengths of 6.0 to 6.6 m the bottom detonators and the top detonators could be split in 4 separate bundles, which again was interconnected to constitute two separate circuits. Each circuit was to be ignited by a separate electrical detonator, which again was connected to a separate firing cable leading out of the tunnel. The two firing cables were finally coupled in parallel to the blasting machine.

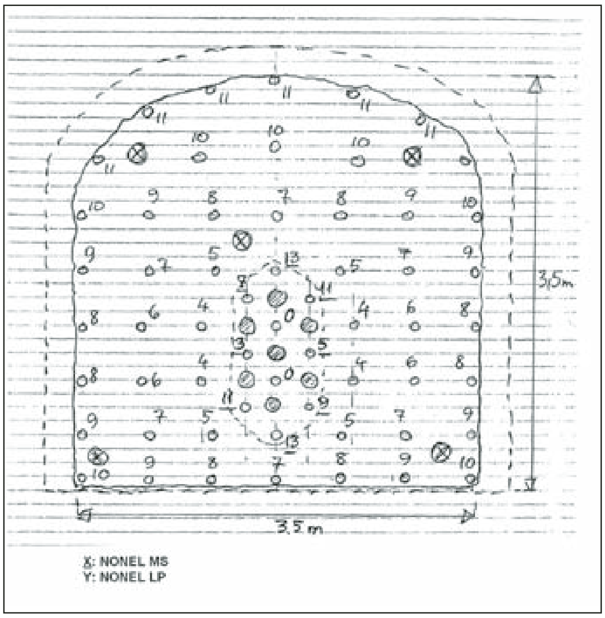

Figure 8: Final ignition plan for underwater piercing blast round. (Illustration: Dyno Nobel)

Rock Support

The amount of rock support installed at the underwater piercing blast round face is shown in Figure 6 and Figure 9 and consisted in the following:

- 16 ea. Ø32 mm, fully grouted rebar bolts were installed around the contour of the final piercing as spiling rock bolts
- 4 additional 2.4 m long Ø20 mm end-anchored and later grouted CT-bolts were installed in the roof above the piercing.

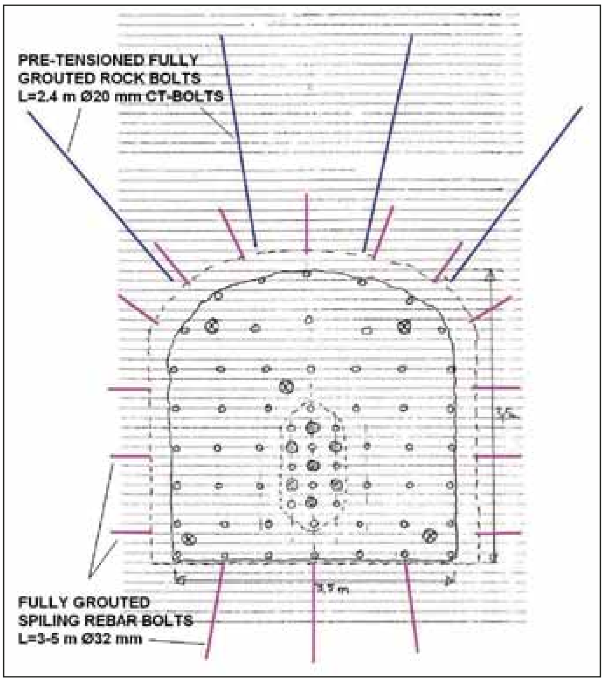

Figure 9: Installed permanent rock support for underwater piercing blast round. (Illustration: Dyno Nobel / Norconsult)

Final Preparation for Pull-In Operation

Before the final blast, a messenger wire was stretched through the tunnel and hooked on to a steel bolt in the tunnel roof, approximately 10 - 15 m from the exit point. This wire intended to be picked up by an ROV and attached to a thicker wire to be used in the pipeline pull-in operation.

UNDERWATER TUNNEL PIERCING - EXECUTION

On 12 September 2002, the breakthrough piercing blast was executed at depth -66 m with the whole tunnel left dry. The rock mass in the remaining rock plug was expected to be completely crushed by the blast, washed into the tunnel with the water flow, and deposited over a large area in the tunnel.

Figure 10: The actual ignition plan drawn on the entrance cut wall in scale 1:1. (Photo: Norconsult)

The final blast was generally successful:

- The upsurge was, however, somewhat higher than the contractor expected as can be seen on the photo series in Figure 11.
- A video inspection (performed by a ROV) revealed that the spoil was deposited along approximately 100 m length in the middle of the tunnel, with an estimated maximum thickness of about 0.5 to 1.0 m.
- The messenger wire was intact in the tunnel; however, partly buried where the maximum spoil deposits appear.
- At the tunnel piercing point, a small ledge was remaining from the middle of the tunnel and to the left (southern) side of the inlet.

The reason for this was most likely related to the geological conditions at the location. There is a system of vertical joint zones almost perpendicular to the tunnel, cutting off a ledge at the exit point. The joint zone would probably have prevented probe holes for the final blast to pass through without major water leakage. Thus, the blast holes in this area were probably drilled too short in order to break off this ledge.

Figure 11: The upsurge arriving at the island Storholmen (Photos: Norconsult)

AS-BUILT LAYOUT

Based on the contractor's changed blast design and the ROV survey made after the blast, as-built documentation as shown in Figure 12 was prepared.

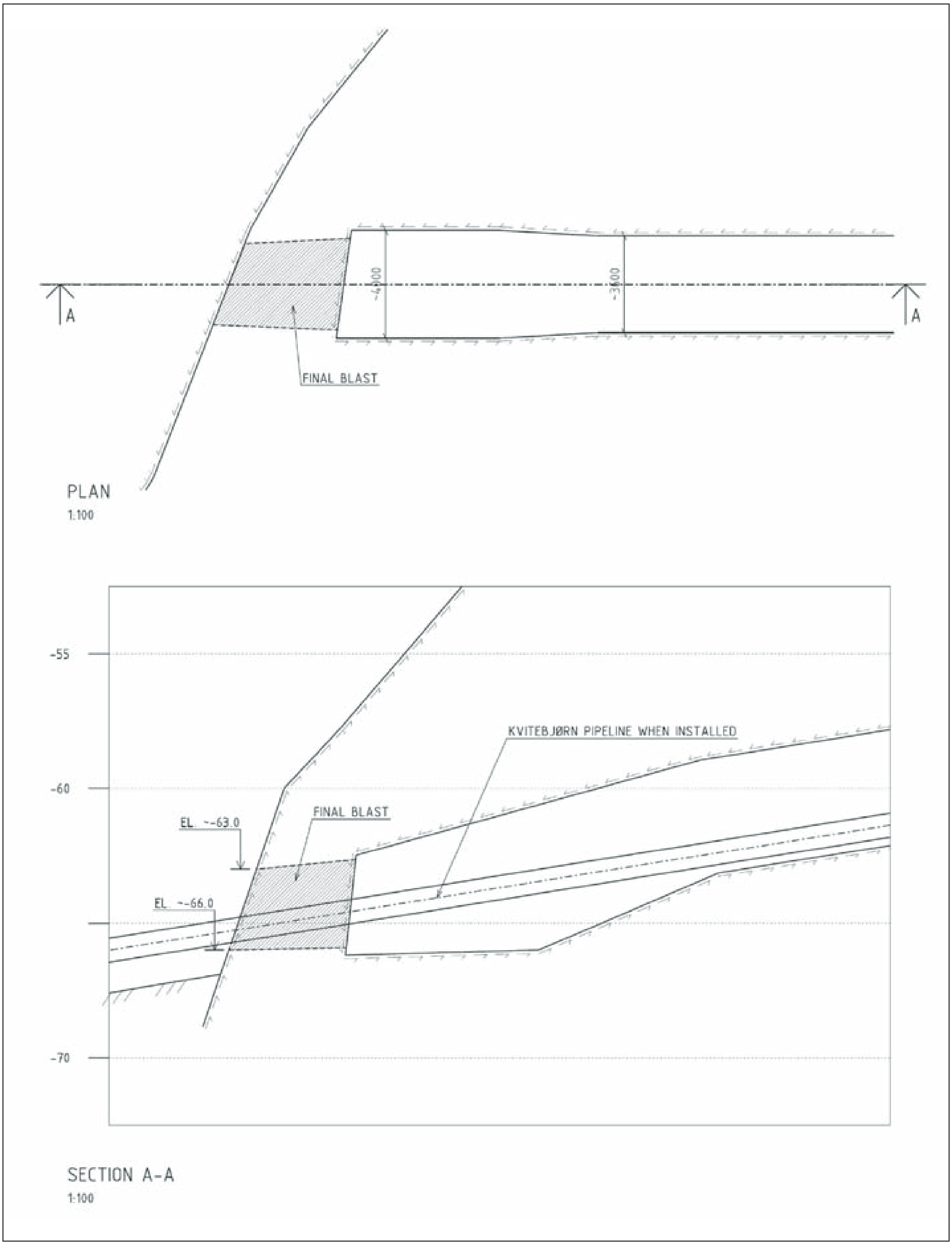

Figure 12: As-built lay-out of the Kvitebjørn Landfall. (Illustration: Statoil / Norconsult)

PIPELINE PULL-IN OPERATION

The Kvitebjørn Rich Gas Pipeline was pulled in through the landfall tunnel to Kollsnes on 15 May 2003.

As can be seen at Figure 13 the pull-in was carried out from a winch fundament constructed on the process plant side of the strait Njupselsundet between Storholmen Island and the Kollsnes process plant facility.

The pull-in operation was performed in an efficient way with continuous pull from lay-barge to shore without any modifications made to the pipeline design (concrete coated line pipe).

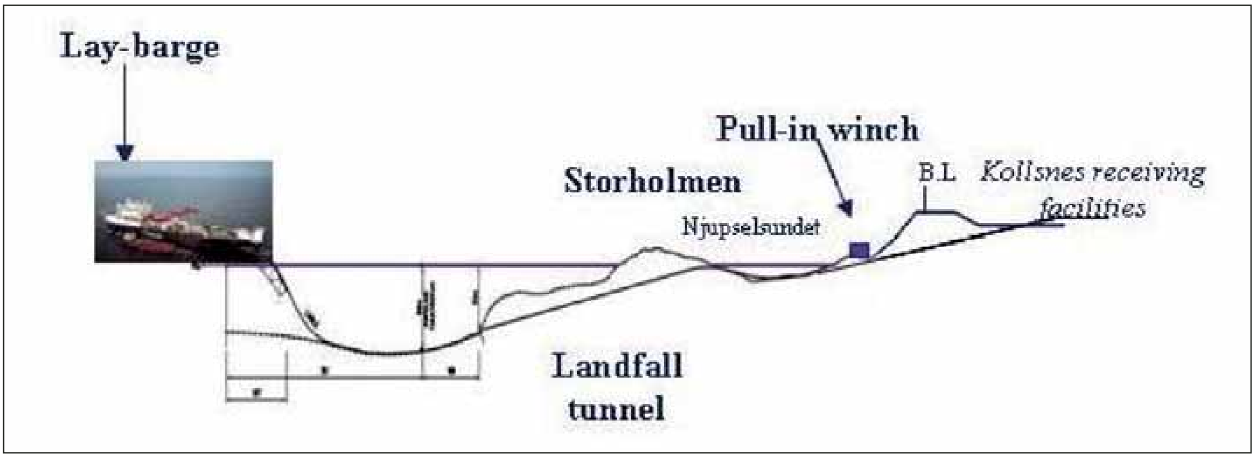

Figure 13: Principle sketch showing the Kvitebjørn Rich Gas Pipeline pull-in operation (Illustration: Statoil)

Figure 14: The lay-barge in position for pipeline pull-in (Photo: Norconsult)

Figure 15: View from inside the landfall tunnel prior to the pipeline pull-in (Photo: Statoil)

Figure 16: Overview picture from the Kollsnes landing site prior to the pipeline pull-in. Pull-in wire direction illustrated with dark line. (Photo: Statoil)

Figure 17: The pipeline appears at the Storholmen Island after a successfully pulled-in through the landfall tunnel. (Photo: Norconsult)

Figure 18: The pipeline being pulled over the strait. (Photo: Norconsult)

Figure 19:The pipeline at its final destination (Photo: Statoil)

Figure 20: The Landfall Island, Storholmen and the pull-in winch location after terrain restoration. With the land reinstatement in place the environmental impact were considered to be minor (Photo: Statoil)

FINAL COMMENTS

The Kvitebjørn Landfall was a major success. The solution with a landfall tunnel designed to facilitate a direct pull-in from the lay-barge, which was located almost at the beach, to the process plant vicinity proved to be a very cost effective alternative compared to the solution chosen for Troll Phase I landfall for the same process plant. In fact only 60 m of additional pipe was necessary to connect the pipeline to the Kollsnes process plant valves.

Conventional drill & blast tunnelling in Norway gives normally low risk for the progress schedule compared to a bored solution with large diameter in hard rock, which has often proved to be a challenge both technically and for the progress schedule.

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8.3.1 INTAKE / OUTLET TUNNELS - MELKØYA - COOLING WATER TUNNELS - PIERCING BLASTS

Arild Neby

ABSTRACT:

The Snøhvit natural gas field lies some 140 km north-west of Hammerfest in Northern Norway. The Snøhvit LNG Project is the first offshore development in the Barents Sea. Without surface installations, the Snøhvit project involves bringing huge volumes of natural gas to land at Melkøya for liquefaction and export from the first plant of its kind in Europe – and the world's northernmost liquefied natural gas facility. As operator, Statoil aims to produce the field and its land-based facilities at Melkøya outside Hammerfest without harmful discharges to the sea. The cooling water for the process plant is taken from the sea at depth -80 m through an underwater tunnel piercing and an approximately 1075 m long Intake tunnel. The cooling water is discharged to the sea at depth - 30 m through an approximately 365 m long Outlet tunnel connected to the sea by another underwater tunnel piercing.

INTRODUCTION

The Snøhvit natural gas field, which is operated by Statoil, is situated some 130 km north-west of Hammerfest in Northern Norway. The recoverable reserves of the field are 193 billion cubic metres of natural gas, 113 million barrels of condensate (light oil), corresponding to 17.9 million cubic metres and 5.1 million tonnes of natural gas liquids (NGL)

The partners in the license are: Statoil ASA (33.53 %), Petoro AS (30%), Total E&P Norge AS (18.40 %), Gaz de France Norge AS (12 %), Amerada Hess Norge AS (3.26 %) and RWE Dea Norge AS (2.81 %).

Figure 1: Location of Snøhvit field and Hammerfest LNG plant at Melkøya. (Illustration: Statoil / Google Earth)

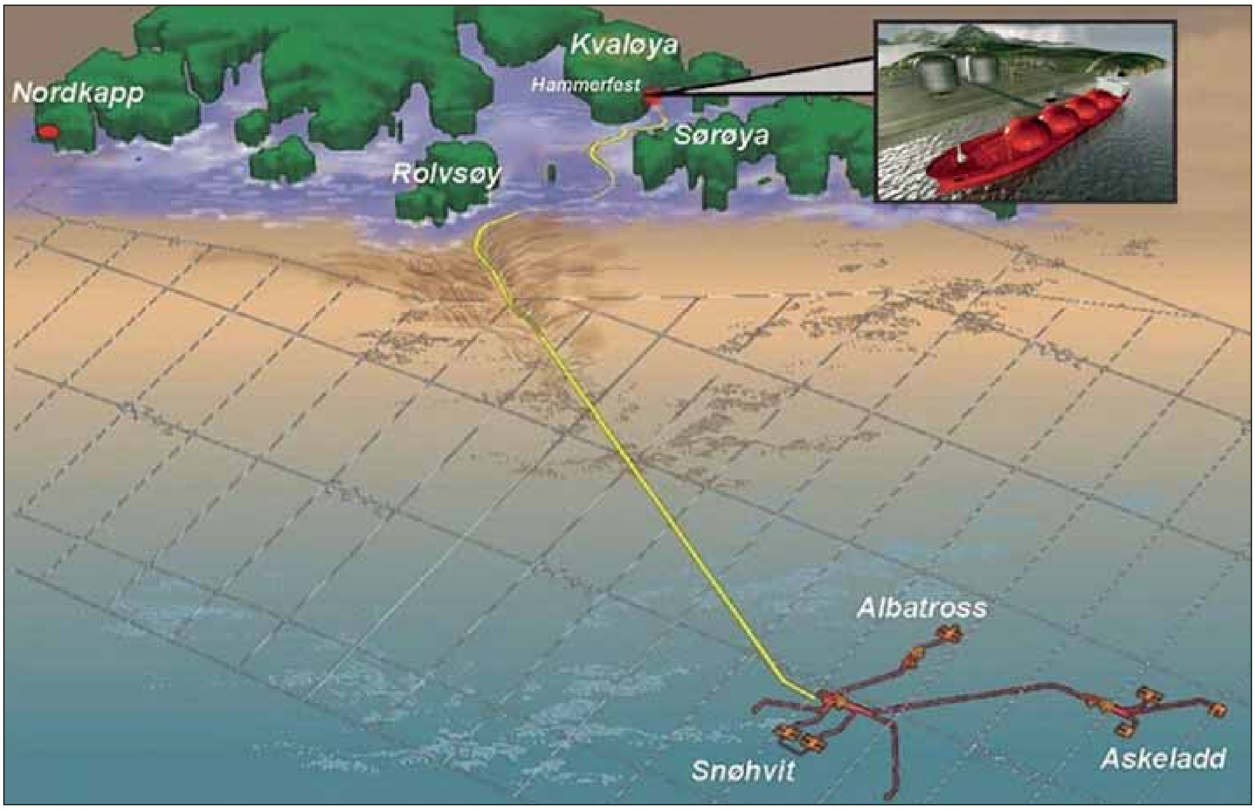


Figure 2: Perspective view of the subsea installations, from the gas field to Melkøya. (Illustration: Statoil)

This article summarises briefly the experiences from the piercing blasts performed in the Seawater Intake and Outlet Tunnels at Melkøya 2003-12-03 at 16h00 by the Site Preparation contractor AFS Pihl Group. Other observations like weather and tunnel conditions directly influencing on the blast and inflow performance, is evaluated.

Further the results of the water velocity verification carried out during the piercing are presented. The results are compared to computer model-based predictions, made prior to the blast.

The tunnels were originally designed to allow for blasting with an air cushion towards the separating rock plug at the sea bottom. The air pockets were to be created by filling the tunnels with water and entrapping air in the inclined tunnel portions towards the underwater piercing points (Figure 6 and 7).

GENERAL GEOLOGY

The rocks at Melkøya and presumably the area to the northwest belong to the Kalak nappe complex containing numerous individual thrust sheets. At Melkøya the rock mass consists of gneiss and the foliation strikes ENE dipping gently southwards. Mylonite benches and bands rich in biotite or amphibole occur in the gneiss on the island. These features, presumably thrust sheet boundaries, often create scarps where competent sub-horizontal benches overlay less competent layers.

Major trends of discontinuities and prominent faults and weakness zones are shown on Figures 3 and 4.

The landforms on the island reflect the WNW and ENE discontinuity directions; joint set 2 and 3, coupled with the sub-horizontal foliation, joint set 1, on Figure 4. The same trend may be discerned from the bathymetric maps covering the area of planned piercings to the NW of the island.

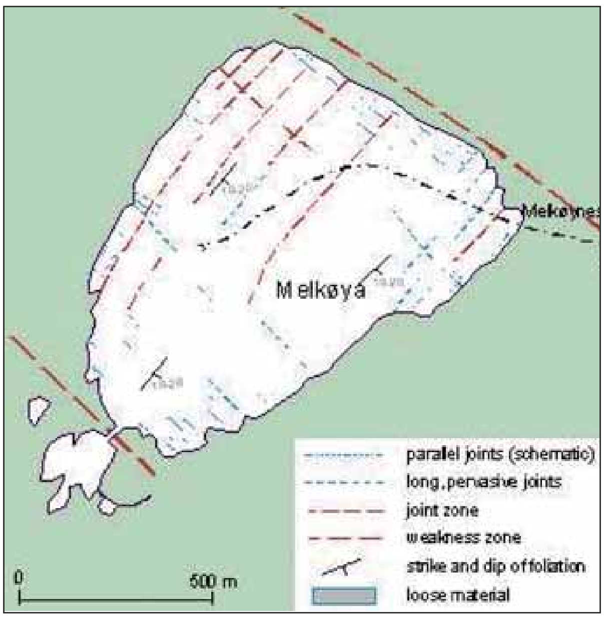


Figure 4: Joint rosette. (Illustration Norconsult)

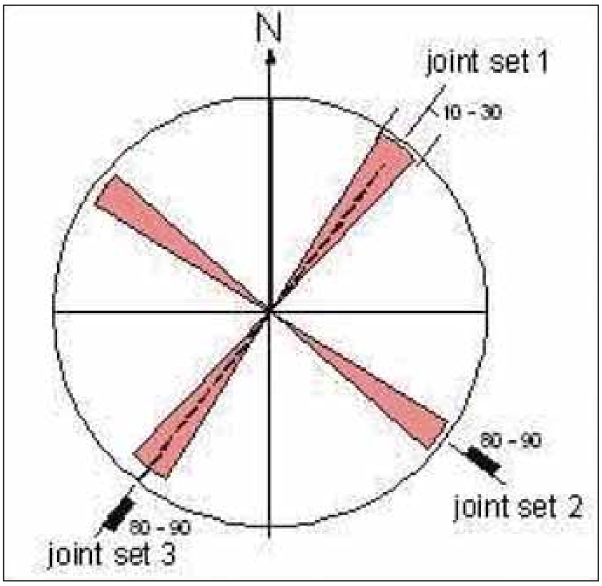


Figure 3: Observed weakness zones and major joints. (Illustration Norconsult)

UNDERWATER TUNNEL PIERCING - DESIGN IMPLICATIONS

The cooling water for the process plant is taken from the sea at depth -80 m through an underwater tunnel piercing and an approximately 1075 m long Intake Tunnel. The cooling water is discharged to the sea at depth - 30 m through an approximately 365 m long Outlet Tunnel connected to the sea by another underwater tunnel piercing.

The contractor, AFS Pihl Group, was responsible for the final planning, design and execution of the underwater tunnel piercing. In cooperation with Norconsult, the contractor issued a set of detailed procedures, design reviews and method statement reports for all activities related to the tunnel excavation and the underwater piercing blast operation. The reports and procedures went through a commenting round with the operator, Statoil, prior to the final revisions and issues.

Design Basis Requirements

Basic requirements were listed as follows:

- The tunnels shall provide sufficiently large and stable openings for the flow of cooling water.
- Loose deposits at the piercing locations must be prevented from accumulation in the tunnels in amounts that could impair the function.
- Flooding of the site located 5 metres above mean sea level is not acceptable
- The tunnels or material left in the tunnels must not produce erosive agents during operation.
- Roadbed shall be removed to top of rock knobs in the invert and this shall be the state also after completed piercing.

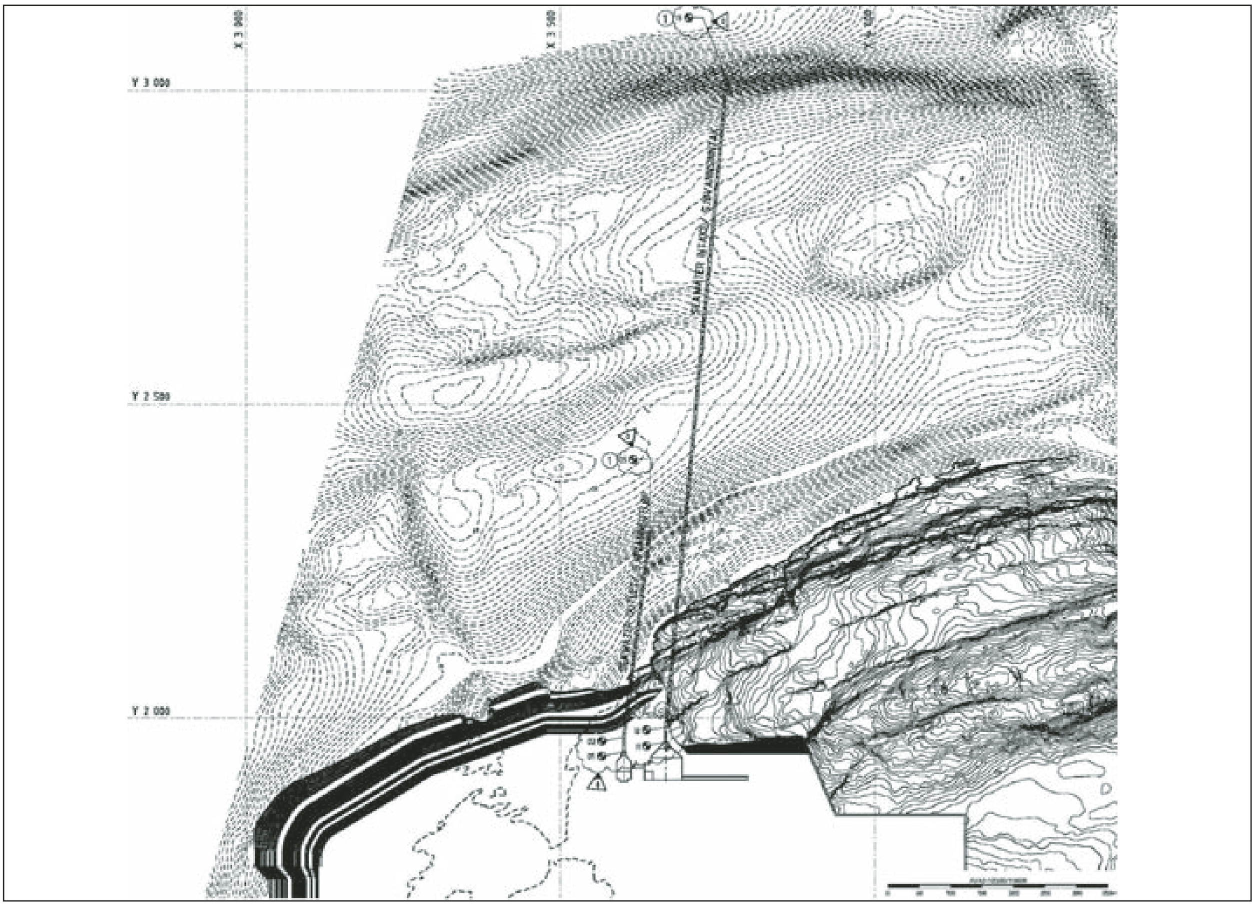


Figure 5: Plan view of the Intake and Outlet Tunnel. (Illustration: Multiconsult /Statoil)

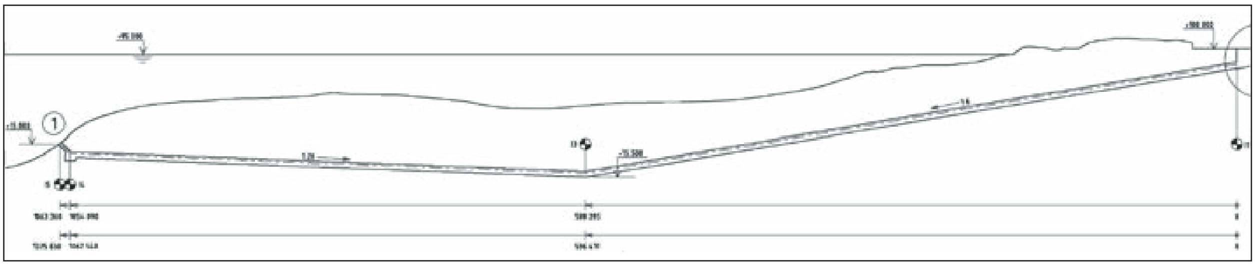


Figure 6: Longitudinal section of the Intake Tunnel with spoil trap and low point. (Illustration: Multiconsult /Statoil)

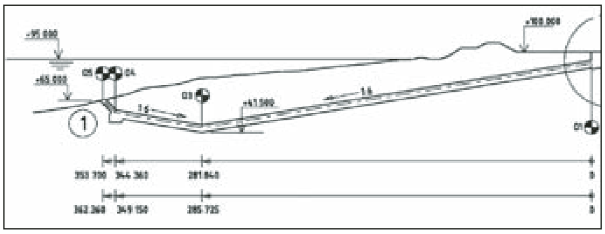


Figure 7: Longitudinal section of the Outlet Tunnel with spoil trap and low point. (Illustration: Multiconsult /Statoil)

EROSIVE AGENTS

The requirement regarding erosive agents together with a construction schedule that was based on the completion of concrete works in the receiver pits prior to the underwater piercing blasts against set stop logs gates was the background for the tentative piercing layout including spoil traps. Underwater piercings performed with pressurised air cushions towards the charged blast rounds would in theory give small water velocities for the inflowing seawater and most of the rock debris from the blast was expected to fall down in the spoil trap. Hence the debris from the blast itself would hardly contribute to additional loose material on the tunnel invert containing erosive agents.

The contractor and their consultant went thoroughly into the presumptions and requirements for the whole underwater tunnel piercing concept during the detail design phase for the final blasting operations.

Regarding erosive agents the rock mass itself could be disregarded as a source, since the tunnel and the rock mass adjacent to the piercing would be adequately supported. This was leaving the loose materials, which is left on the invert as a possible source. This material is susceptible to erosion by the permanent flow in the tunnel and may be transported towards the receiver pits. Figure 8 below shows a diagram for erosion, transport and sedimentation as a function of flow velocity, also known as the "Hjulström diagram". According to the diagram fine sand is the fraction most susceptible to erosion.

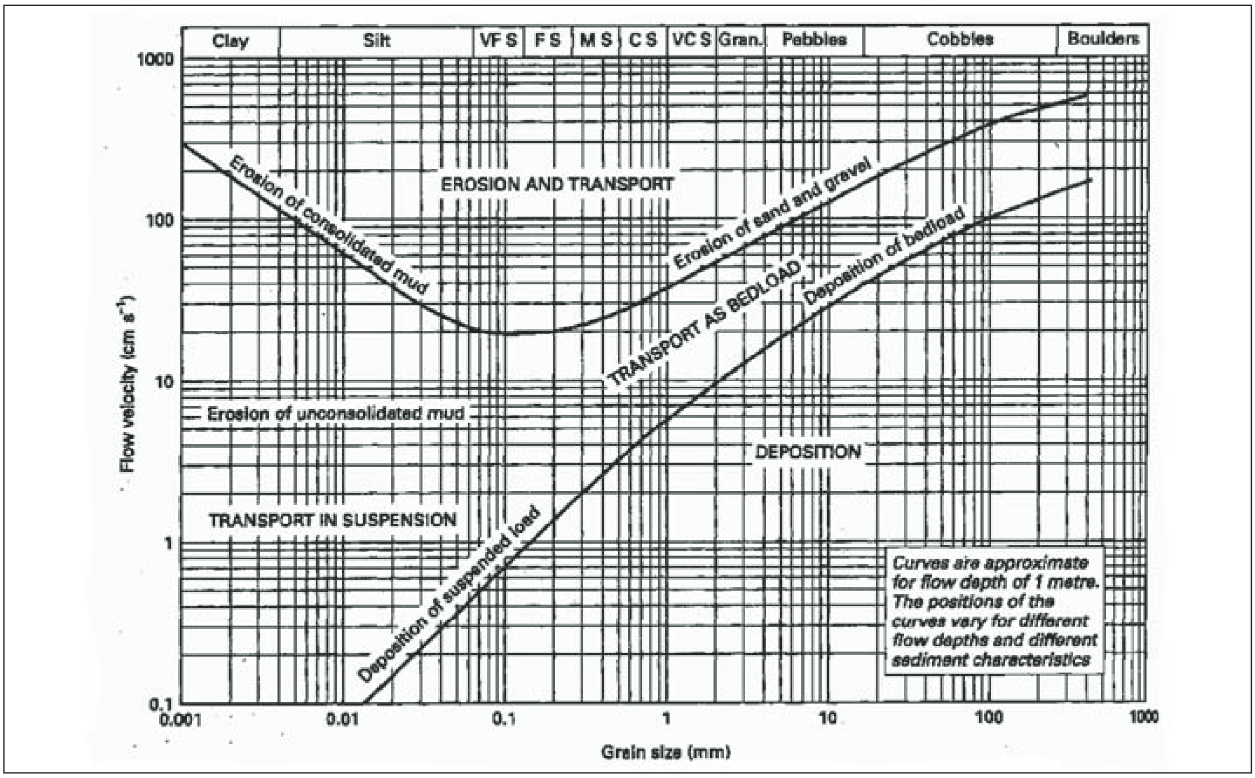


Figure 8: Hjulström diagram (Illustration: F. Hjulström, modified by J.C. Harms et al. 1982)

It should be noted that the volume of material, which may be eroded, is proportional to the exposed surface of spoil at the invert and not to the bulk of spoil left in the tunnel. The erosion stops as soon as a protective skin of coarser material is created. The formation of a stable protective skin is a natural process, also observed in hydro-tunnels with water-velocities in the range of up to 1.5 m/sec and where roadbeds that are left in place, have remained intact after years of operation. The design capacity of seawater flow in the intake tunnel is reported as 96 000 m3/hour, which corresponds to a maximum velocity of approximately 1 m/sec in the intake tunnel.

The above discussion indicates that there would be no practical difference regarding erosive potential whether the roadbeds were left in place on the invert, the invert was cleaned to top of rock knobs or the invert to some extent was covered by debris from the blasting of the final plug. The alternatives regarding tunnel cleaning was therefore considered to be either to clean the tunnel completely or to do nothing, and hence also consider letting spoil flow into and through the tunnel during piercing.

Underwater Tunnel Piercing Alternatives
Based on the conclusion regarding erosive agents, alternative methods of underwater tunnel piercings were evaluated together with the tentative method by use of computer model analysis.
As the amount of erosive agents was independent of the method chosen, only the type of explosives to be used was dependent on the selected alternative for piercing. All calculations were based on a cross section at the piercing of 9 m2 and that the blasts were performed at mean sea level at elevation 0 (project elevation +95.0 m).

The evaluated alternatives for piercing methods are listed as follows:

1. Dry piercing; dry tunnels:
a. Stop log gates closed
b. Stop log gates open

2. Water filled tunnel with air cushion to separate piercing plug and water:
a. Water filling between the piercing plug and closed the stop log gates
b. Water filling of the tunnels, including receiver pits, but with the gates open

3. Water filled tunnels without any air cushion:
a. Water filling between the piercing plug and the closed stop log gates
b. Water filling of the tunnels, including receiver pits, but with the gates open

All calculations were based on a cross section at the piercing of 9 m2 and that the blasts were performed at mean sea level at elevation 0.

Alternative 1b) with dry tunnel and open gates was recommended as piercing method for the intake and the outlet tunnels. This alternative was the simplest solution from a technical point of view. It gave a comfortable safety against flooding of the site and was also considered the most economic alternative. The potential inconvenience due to risk of erosive agents during operation, was recommended to be further evaluated, but not considered to constitute a risk of significance.

Alternative 1a) with dry piercing and closed gates were not recommended for further evaluation due to the high pressure generated at the gates. The same was the case for alternative 3a) with water filled tunnels without a controlled air cushion at the face and the gates closed.

The remaining alternatives; 2a) water filling to elevation above -9.0 m, controlled air cushion and gates closed, 2b) water filling to elevation above -9.0 m, controlled air cushion and gates open and 3b) water filling to elevation above sea level, no controlled air cushion and open gates, were all feasible and recommendable solutions. The simpler solution among these alternatives seemed, however, to be alternative 2b), without a controlled air cushion and with the gates open, even if this alternative demanded the use of more water resistant Ø64 mm primers with a cast body of TNT/RDX and a cap sensitive part of pressed PETN, as the preferred explosive. This solution required drilling of larger blast holes than usual and an increased volume of water for filling of the pits.

For detail design of the selected alternative, 2b) dry piercing with stop logs open, more detailed calculations on up-surge, pressure pulse and sediment transport were performed together with detailed procedures for the work towards and at the piercing.

ACTUAL TUNNEL EXCAVATION

Compared to the theoretical layout shown on Figure 5-7, the tunnels have actually been excavated as shown on the planar parts of sketches shown in Figure 9 and 12 with typical cross-sections as shown in Figure 10 and 11.

At approximately every 100 m in the Intake Tunnel, a turning and mucking niche of some 350 - 400 m3 have been excavated. For the Outlet Tunnel, similar sized niches have been excavated at two locations. In addition, for both tunnels, pumping and cable niches of some 40 - 50 m3 have been excavated at intervals of approximately 100 m on the opposite side of the tunnel compared to the large T&M niches, numbering 6 in the Intake Tunnel and 2 in the Outlet Tunnel.

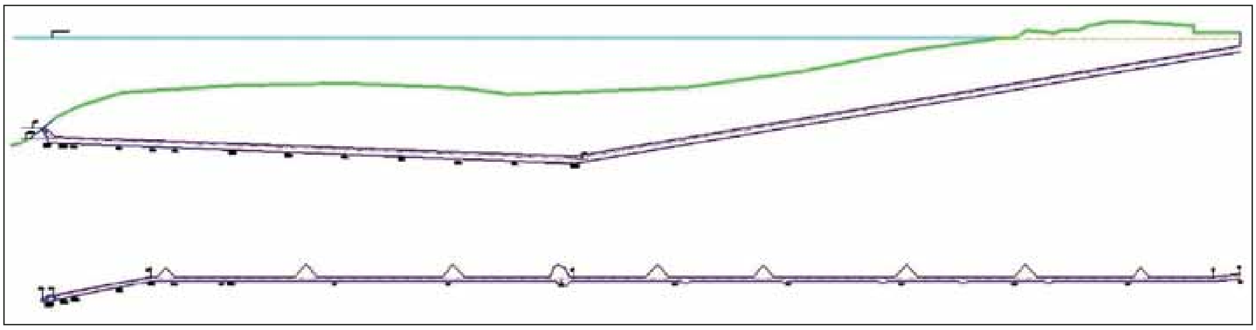


Figure 9: Intake Tunnel layout with niches (Illustration: Norconsult)

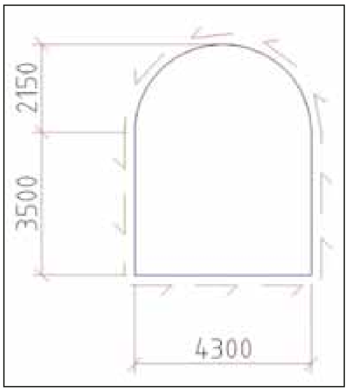


Figure 10: Typical tunnel cross-section

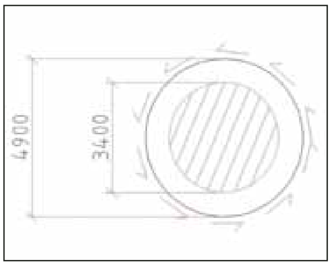


Figure 11: Piercing shaft cross-section

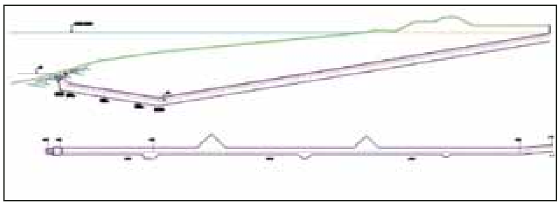


Figure 12: Outlet Tunnel layout with niches (Illustration: Norconsult)

Figure 13: Drilling of underwater piercing blast round for Intake Tunnel completed. (Photo: AFS Pihl)

EXPLORATORY DRILLING AND ROCK MASS GROUTING

A detailed exploratory drilling programme was elaborated for the different portions of the cooling water tunnels. The programme was divided into 4 stages following specific chainage numbers related to the actual rock cover.

Figure 14 and 15 shows the planned extent of exploratory drilling and rock mass grouting for both tunnels.

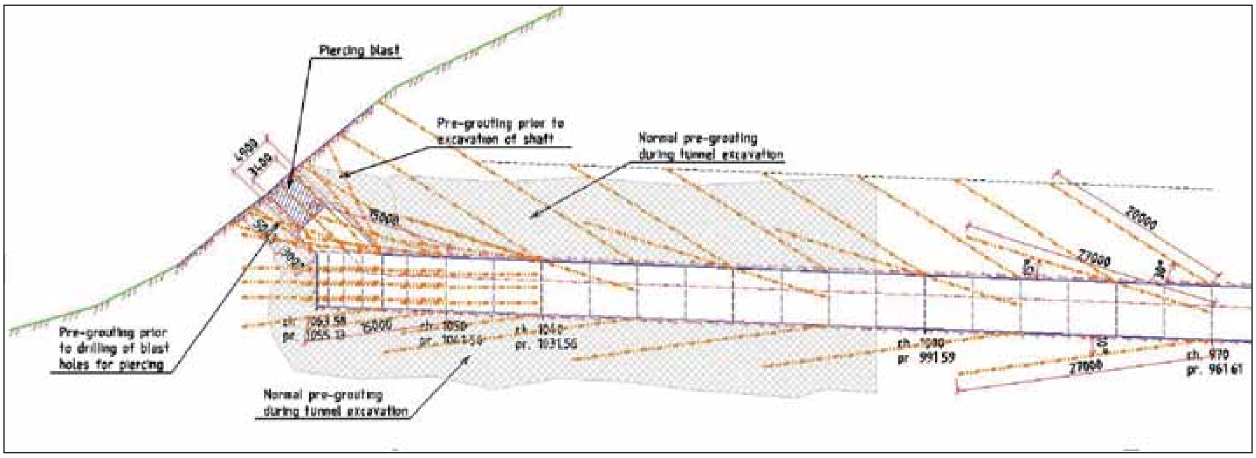


Figure 14: Intake Tunnel - exploratory drilling programme and rock mass grouting. (Illustration: Norconsult)

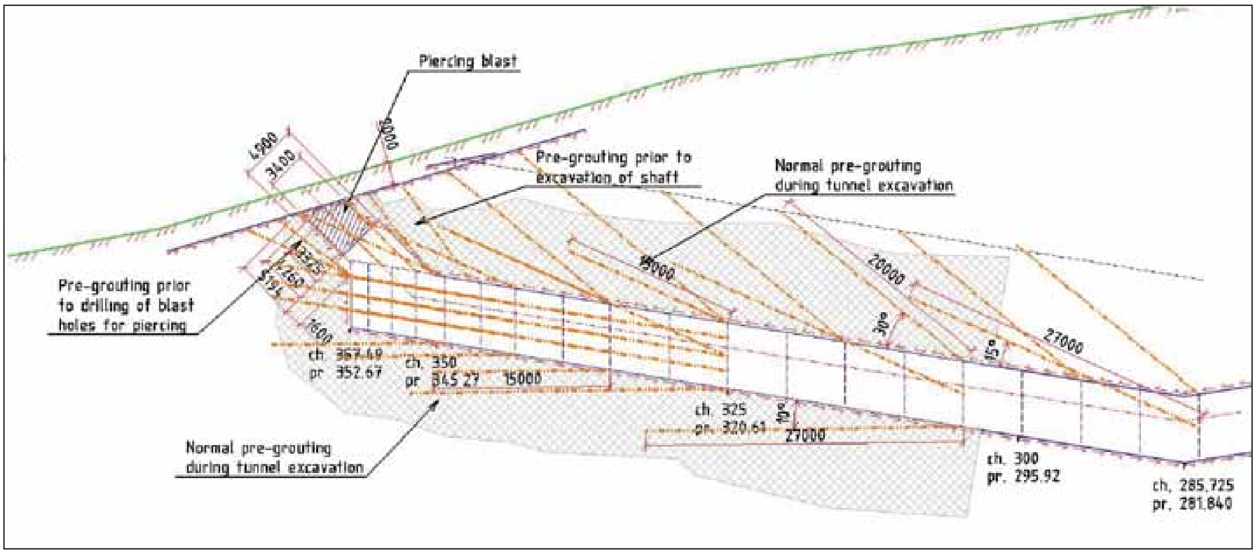


Figure 15: Outlet Tunnel - exploratory drilling programme and rock mass grouting. (Illustration: Norconsult)

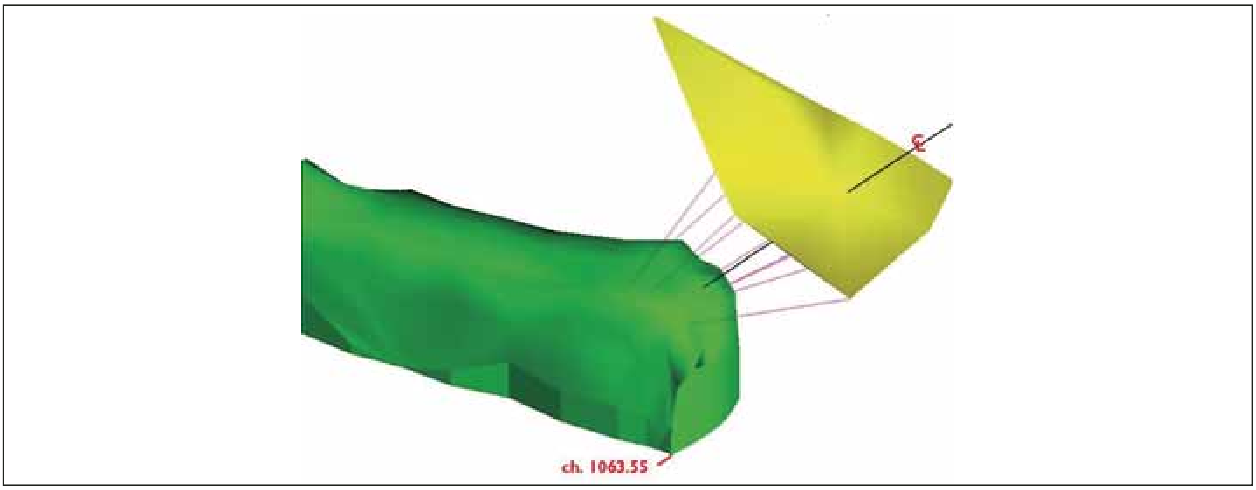


Figure 16: Intake Tunnel - 3D image of actual executed exploratory drilling from ch. 1063. (Illustration: AFS Pihl)

ACTUAL DRILLING, CHARGING AND IGNITION

Also for the circular piercing blast rounds, there were in the final stages made some adjustments to the initial excavation plan. Instead of diameter of 3.4 m, the final round was drilled with a 3.5 m diameter in order ensure the minimum required opening. The circular cross-section increased then from 9.1 m2 to 9.6 m2.

All blast holes were drilled with Ø48 mm bits. Large diameter holes were reamed up to 102 mm (4”) diameter.

Drill hole data were as follows:

Large (uncharged) holes:	**5 pcs**
Blast (charged) holes:	**79 pcs**
Cut holes:	16 pcs
Stoping holes, Ring 1:	14 pcs
Stoping holes, Ring 2:	21 pcs
Contour holes:	28 pcs

The final piercing blast rounds were charged with ordinary dynamite specially tested for water resistance. The dynamite cartridges were loaded into thin PVC-pipes closed in the hole bottom end.

Charge quantities and specific charges were as follows:

Blast (charged) holes:	**327.3 kg**	**→ 8.6 kg/sm3**
Cut holes:	72.8 kg	→ 14.6 kg/sm3
Stoping holes, Ring 1:	14 pcs	→ 5.9 kg/sm3
Stoping holes, Ring 2:	21 pcs	
Contour holes:	28 pcs	

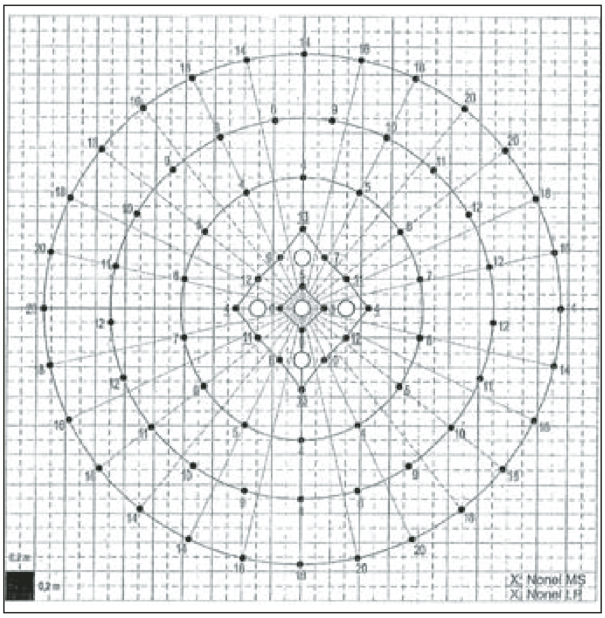


Figure 17: Ignition plan for a Ø3.5 m underwater tunnel piercing blast. (Illustration: AFS)

The ignition system consisted of a double set of detonators in each hole. The detonators used were a mix of Nonel MS and Nonel LP, coupled in two separate circuits. Except for the zero-detonator, millisecond (MS) detonators were used in the cut area. In the rest of the blast round Long Period (LP) detonators were used (see Figure 17). A double set of shotfiring cables of type Nonel Dynoline were cross-connected to both the ignition circuits.

ESTIMATED HEAD LOSS AND WATER VELOCITY

The extra niche volume actually excavated had not been accounted for, neither in the calculations of head loss during operation nor the water velocity model for the piercing performance.

For the operational stage of the plant, the volume of the niches adds positive area to the tunnel cross-section and gives as such a beneficial effect on the head loss. Also the niches will serve as sedimentation basins as the water velocity will be reduced at these locations due to the considerable increase in the tunnel cross-section.

The increased size of the piercing opening will also contribute to reduced water loss during the operational stage. For the predictions of water inflow and velocity during and just after the blast, a larger opening will give a marginal reduction in head loss and a somewhat higher water velocity below the opening.

For the calculations on timing of piercing blasts and water velocity in the tunnels during the piercing operation, the niches added some uncertainty to the water front movement in the dry tunnel, as the niches would serve as energy dissipaters as well as causing a piston effect on the advancing water column. On the other hand, the effect of higher intake velocity contributes somewhat in the other direction.

WATER ACCUMULATION AT LOW POINTS PRIOR TO BLASTING

After the pumps were stopped and the discharge system removed the water levels at the low point in both tunnels were measured twice prior to blasting. Based on these measurements the following water leakage rates have been calculated:

Intake Tunnel: 122 l/min
Outlet Tunnel: 77 l/min

The above water ingress rates give water accumulation pictures as shown on Figures 18 and 19 at the respective moments of blasting for the two tunnels.
These levels of accumulated water gave comfortable safety margins to the recommended maximum levels.

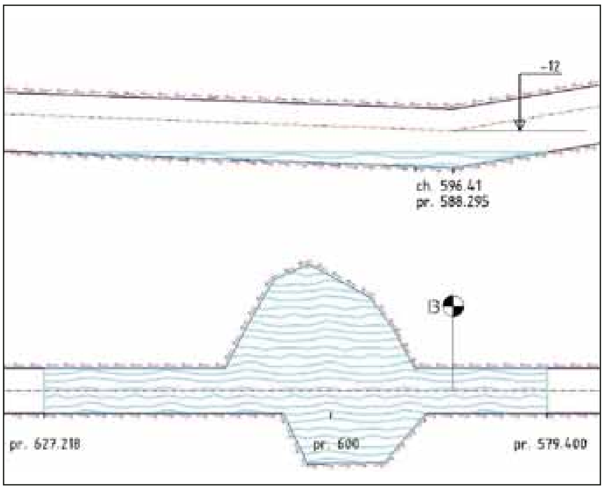


Figure 18: Water accumulation at the intake tunnel low point at the moment of blasting. (Illustration: Norconsult)

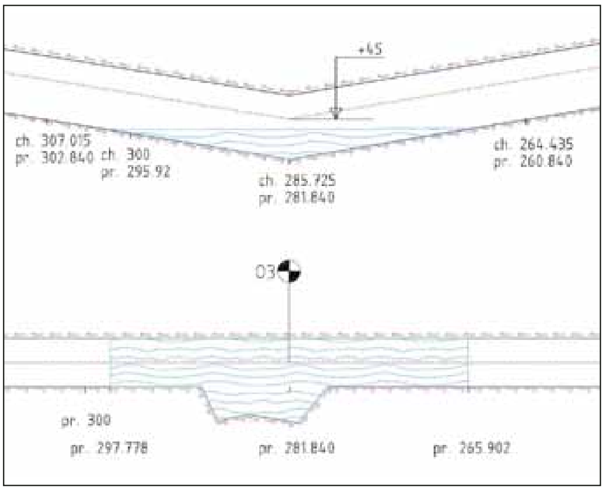
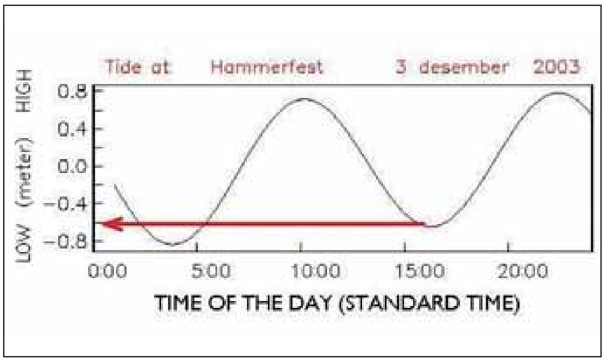


Figure 19: Water accumulation at the outlet tunnel low point at the moment of blasting. (Illustration: Norconsult)

TIDE AND WEATHER CONDITIONS

The blasting time at 16h00 had been chosen specifically to coincide with the afternoon low tide in order to maximize the possible safety margins for the upsurge in the pits. The tide fluctuations for 3 December 2003 are given in Figure 20. Meteorological records are given in Figure 21.



As can be seen from the data, the low tide is estimated to -0.64 m below normal zero, which gives a project elevation of 94.36 m for the sea level, without taking into consideration the weather conditions present.

The combined effect of the barometric high pressure before noon, the lack of precipitation and the increasing wind from WSW is calculated to have reduced the low tide level at 16h00 by roughly 20 mm. The predicted upsurge from the calculations, which was 0.6 m for the Intake pit and 0.9 m for Outlet pit, would then become 94.34 m + 0.6 m = 94.94 m. As the two pits were not separated by the initially planned concrete wall, at the time of piercing, the upsurge of the Intake pit would govern the maximum upsurge capability.

Figure 20: Tide fluctuations for Hammerfest without influence of weather 3 December 2003. (Illustration: www.math.uio.no)

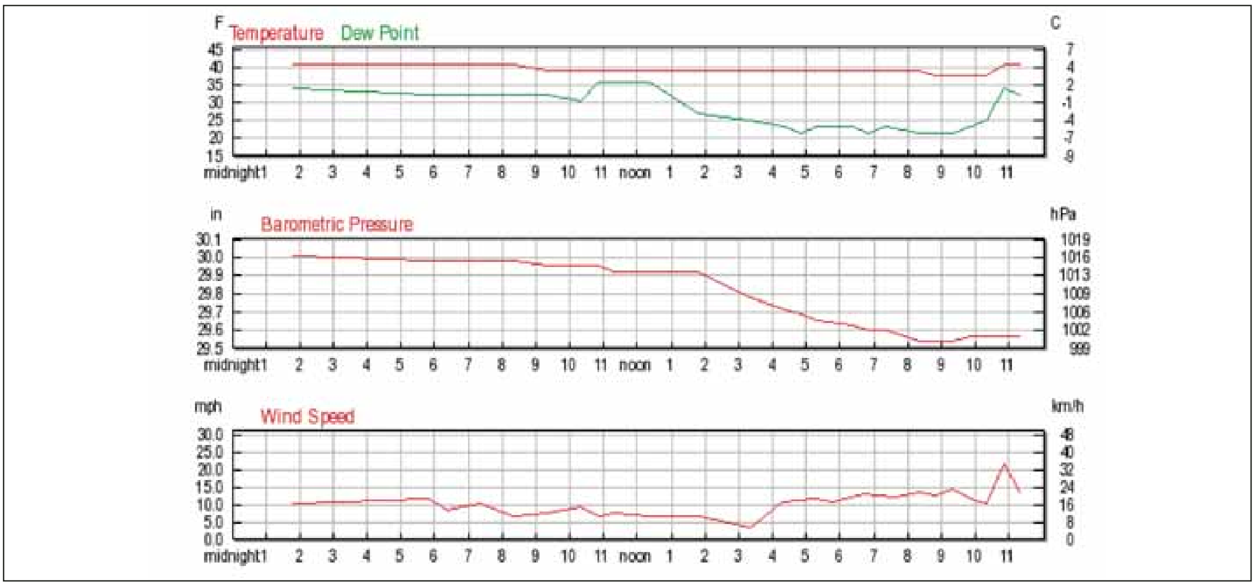


Figure 21: Meteorological data from Hammerfest Airport 3 December 2003. (Illustration: norwegian.wunderground.com)

ADJUSTED TIMING BETWEEN BLASTS

The timing of the two piercing blasts was proposed to take place with a 170 second interval between the Intake Tunnel and the Outlet Tunnel. This interval was based on a preliminary survey of the excavated pits, which located the high point in-between the two connected pits at project elevation +89.0 m (El. -6.0 m), as well as the desire to have the two waterfronts levelling simultaneously at this common high point.

After the preliminary cleaning of the pit inverts prior to blasting was completed, it became clear that the actual high point between the two pit areas was not located as indicated by the preliminary survey, but was coinciding with the stop log body sill of the Outlet Tunnel aperture at project elevation +88.47 m (El. -6.53 m).

If maintaining the 170-second delay for the Outlet Tunnel blast, there was an obvious chance that water from the Intake Tunnel could enter into the Outlet Tunnel before the waterfront of the Outlet piercing had reached the pit. This fact together with the larger piercing blast cross-sections and the significant increase in volumes contributed by the turning & mucking niches necessitated an adjusted timing of the blasting interval.

Based on new calculations the interval was reduced to 75 seconds between firing of the Intake piercing blast and the Outlet piercing blast. The predictions were then that the waterfront of the Intake Tunnel should arrive in the pit 124 seconds after the detonation of the piercing round in the Intake Tunnel and that the Outlet Tunnel waterfront should arrive in the pit 5 seconds later than the Intake waterfront. The predicted travel time for the Outlet Tunnel waterfront would then amount to 55 seconds.

This adjustment also gave the shot firer an additional 80 seconds safety margin in case of any malfunction or trouble occurrence for the blasting machine used for the Outlet piercing.

RESULTS FROM WATER VELOCITY VERIFICATION

As part of the verification process of successful piercings into the sea, as well as a calibration of the computer model used for the prediction of water velocities, water discharge, upsurge and arrival times, measurements of real water arrival times in the tunnels were initiated.

For measurement of real waterfront arrival times, a total number of 22 sensors were distributed along the two tunnel inverts at recorded locations. The distribution for the Outlet Tunnel is shown in Figure 24. One such sensor (shown on Figures 22 and 23 below) consists of an electronic clock, which is calibrated and synchronized with the other sensors and a computer, that will record a specific moment of time when the floating waterproof sensor housing is exposed to a trigging movement or a dislocation caused by the waterfront.

Figures 22 and 23: Exterior and interior of a floating waterproof sensor

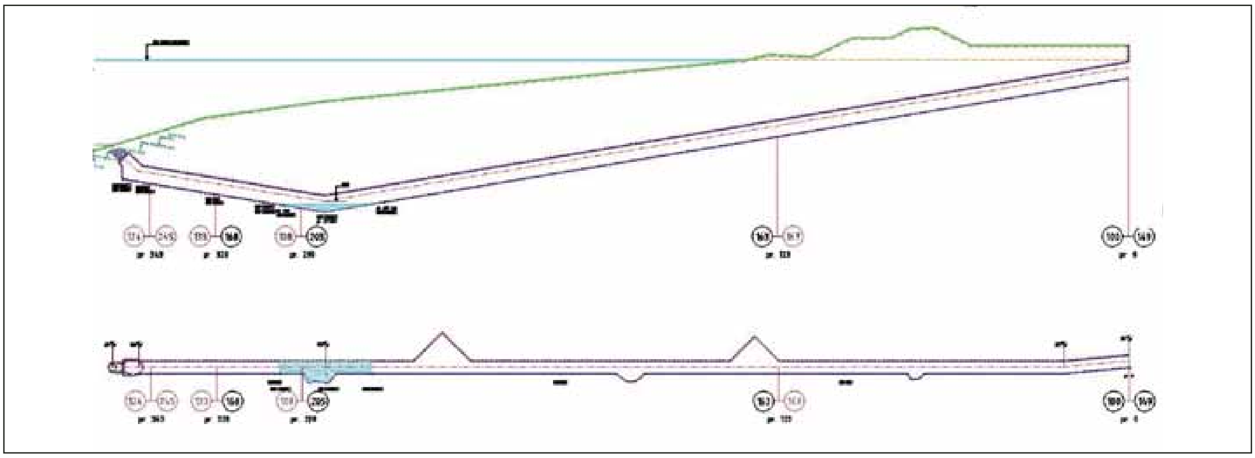


Figure 24: Location of sensors in Outlet Tunnel (sensors with black circles was recovered after the blast)

The piercing blasts were carried out at the following recorded times (computer synchronised time):

	Blasting	Water Arrival
Intake Tunnel:	**15:59:42**	**16:01:47**
Outlet Tunnel:	**16:00:57**	**16:01:52**

In total 13 sensors out of 22 were recovered after the blasting. As expected, none of the sensors placed in the inner part of the tunnels survived the joyride towards the pit. The results of the measurements are shown in Figure 25 below in comparison with the data from computer model analysis. As can be seen from the curves the real arrival times for the waterfront is quite near the computed values. Actual recorded travel time for the waterfront in the Intake Tunnel was 1 second longer (125 seconds) than computed and the travel time for the Outlet Tunnel was exactly as computed (55 seconds).

In Figure 26 and 27 below are real velocities compared to the computed velocities. As the real velocities are mean speeds between measured points, the curves will deviate somewhat from the computed curve as this curve shows mean flow rate calculated for every 0.2 seconds. The tendency is however that the real waterfront speed has been higher than computed in the beginning of the tunnel, but has been slowed down more rapidly towards the pit than computed.

The good correlation with the computed flow rates indicate, however, that the openings towards the sea have similar or better swallow capacity than what has been used as input for the model.

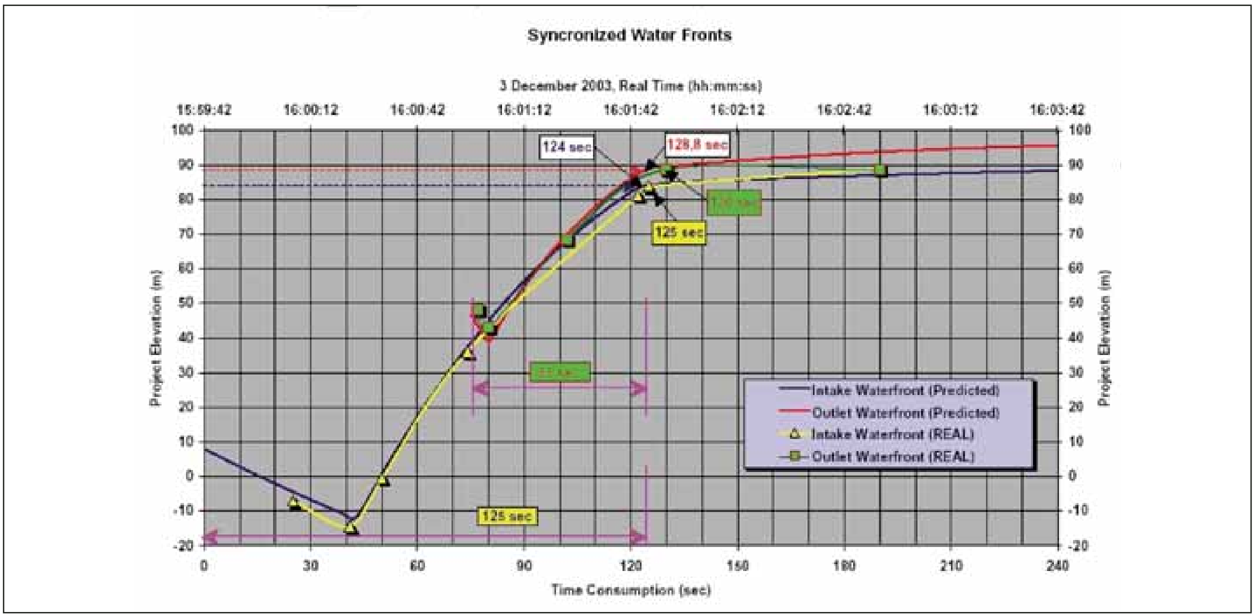


Figure 25: Comparison between computed arrival times and sensor recorded arrival times for the waterfronts

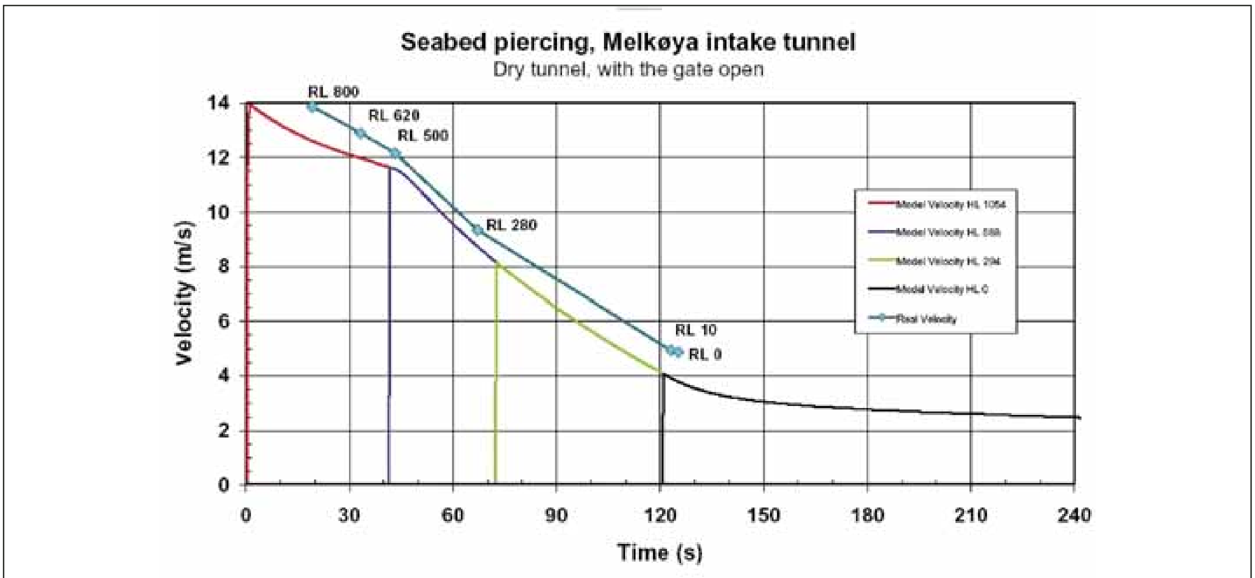


Figure 26: Comparison between computed flow rate and sensor recorded mean velocity for the Intake Tunnel

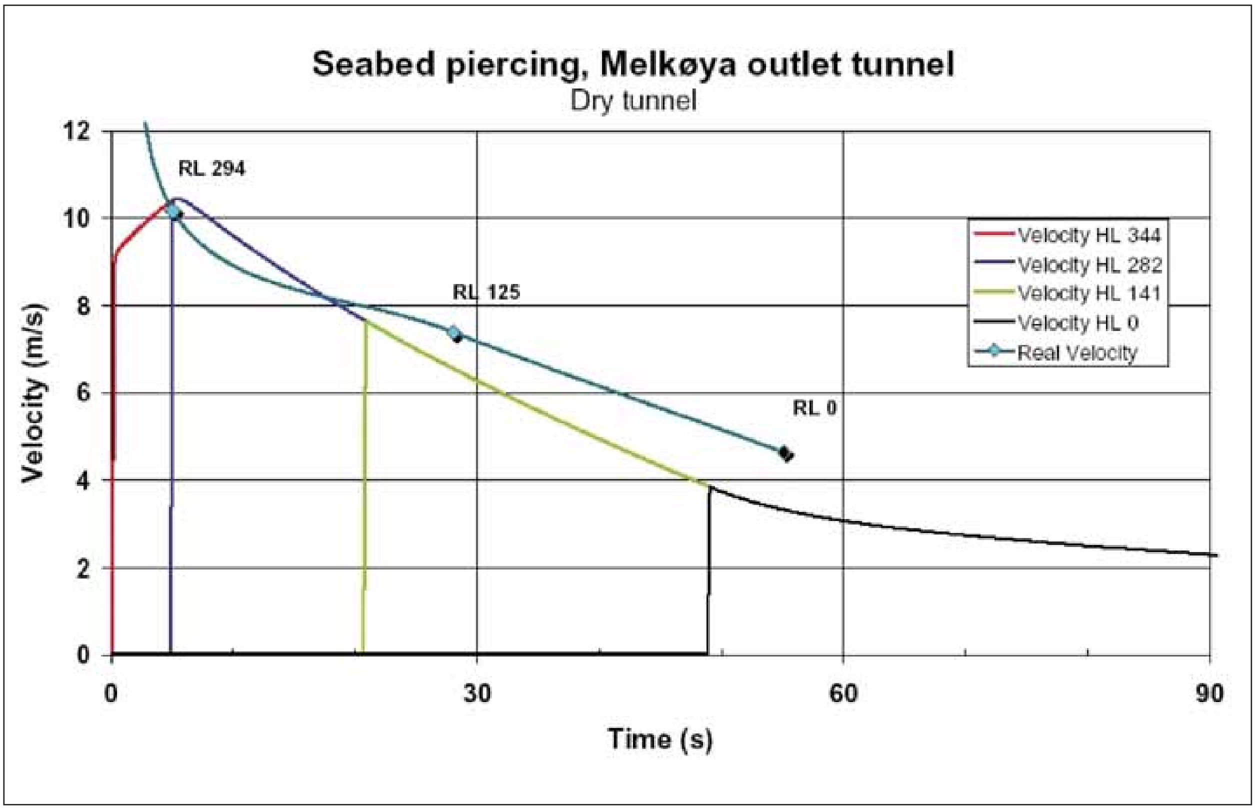


Figure 27: Comparison between computed flow rate and sensor recorded mean velocity for the Outlet Tunnel

MAXIMUM UPSURGE AND WATER FILLING RATES IN PITS

From Figure 28 below the theoretical surveyed pit volume at project elevation +95 (El. 0.0 m) is estimated to 36 968 m3.

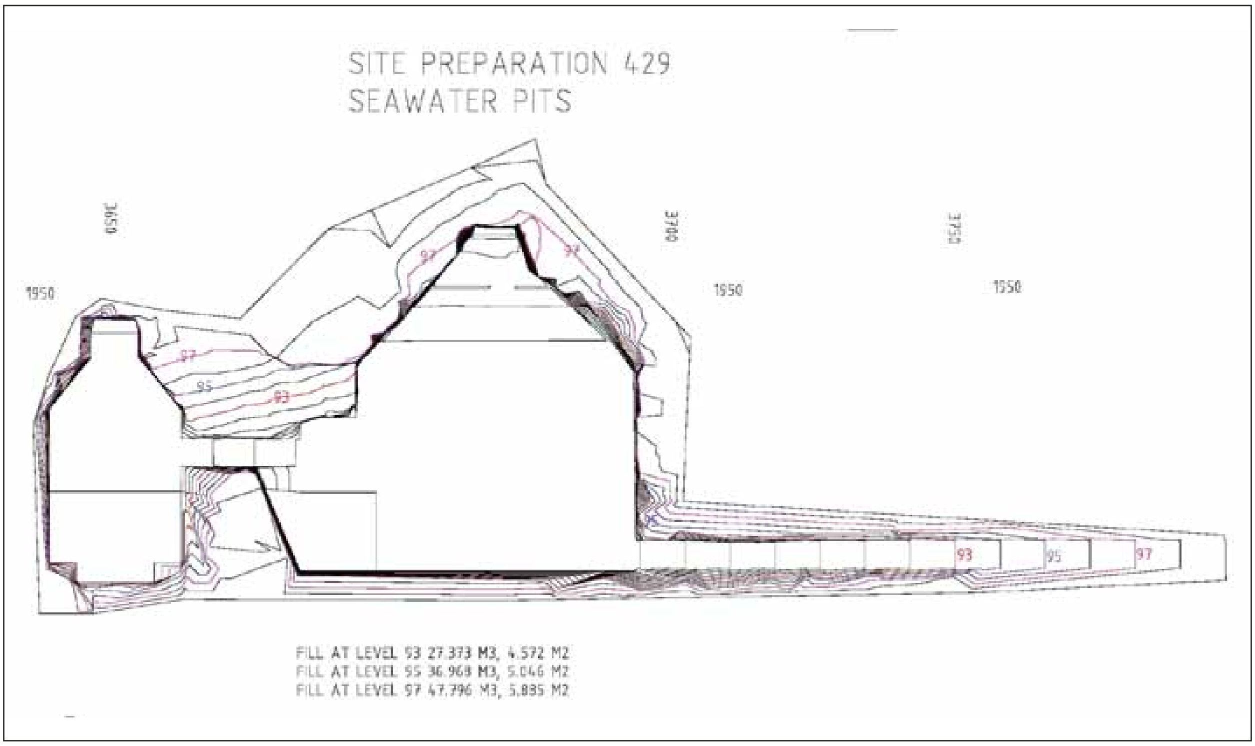


Figure 28: Pit volume estimates from surveying. (Illustration: AFS Pihl)

Figure 29: Arial view of receiver pits August 2003. (Photo: Statoil)

For the full picture of the swallow capacity of the two piercing openings, one will have to look upon the water filling capacity.

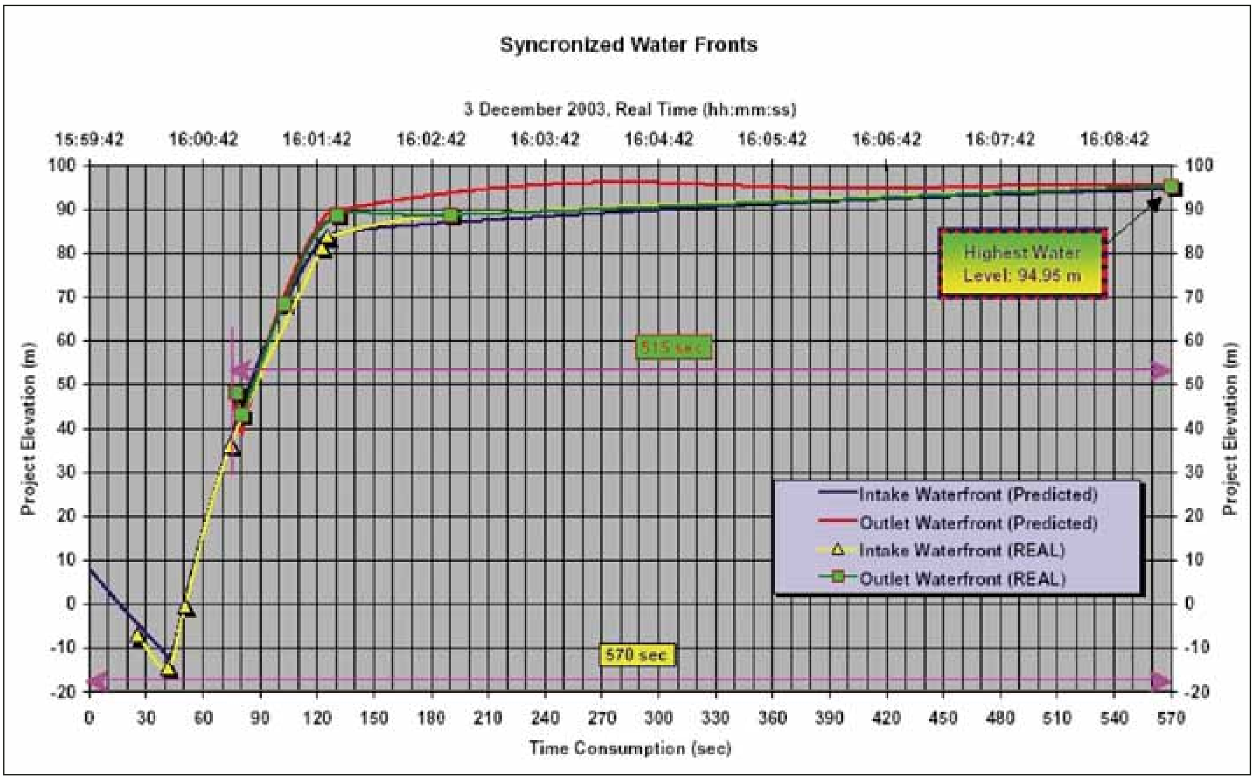


Figure 30: Comparison between computed filling times and recorded filling times for the two waterfronts

Figure 31: Maximum upsurge as wet markings on the stop log structure

From video recordings and time keeping during the pit filling process, the maximum upsurge was timed to 570 seconds (or 9 minutes and 30 seconds) after detonation of the Intake Tunnel piercing blast (see Figure 30 above). For the Outlet Tunnel this is quite much longer than what would have been the case if a concrete wall had separated the pits (red line in Figure 30).

The maximum upsurge was, as indicated on Figures 30 and 31, recorded at project elevation +94.95 m (El. -0.05 m). This is 10 mm higher than the theoretical calculations taking weather conditions into consideration.

The mean theoretical swallow capacity for the two tunnels together was then calculated to be approximately 84 m3/sec, which gave a mean capacity of 42 m3/sec for each tunnel. This figure was in line with the average discharge rates of the computer model.

CONCLUSIVE COMMENTS

The Seawater Intake and Outlet Tunnels piercings at Melkøya 3 December 2003 were considered successful. The blasts were carried out in good accordance with all elaborated procedures and plans. The travel times for the waterfronts and the recorded water filling and maximum upsurge indicate that the openings were sufficiently large and that the blasts have detonated as intended. The water-filling rate also indicated that the amount of debris from the blasts at the tunnel inverts was evenly distributed and that the likelihood of any major constrictions in the tunnel system was minimal. This assumption was later confirmed by ROV video images.

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8.3.2 CHALLENGING DRILLING AND BLASTING OPERATION IN STEEP TERRAIN ON SEABED, 82 METRES BELOW SEA LEVEL, ORMEN LANGE MAIN CIVIL CONTRACT

Bjørn R. Morseth

ABSTRACT

After a failed attempt to achieve a tunnel breakthrough on the Ormen Lange project at Nyhamna, in the municipality of Aukra, Mid-Norway, in October 2005, there was a need to find a safe method of obtaining the required opening to the sea.

The underwater tunnel piercing site is at a depth of 82 metres below sea level. The seafloor at this point is steep with an inclination of 60 - 70o, and there is a partial overhang.

A project team composed of engineers from both Skanska and external companies was established. The team evaluated several alternative methods, and in January/February 2006, one method emerged as probably the most suitable, and was investigated further. This method was based on mounting conventional drilling equipment on a remote-operated tool carrier. It was intended that the equipment should rest on the bottom just outside the breakthrough area and be raised and lowered to the bottom from a barge. Development of the method involved testing of conventional explosives (Dynoprime/ initiators) that worked at the depths in question, modification of drilling equipment, tool carrier and barge, and customisation and manufacture of special equipment.

As an alternative to this method, another method, which involved plugging the failed tunnel and excavating a new tunnel to breakthrough, was investigated in parallel.

After a thorough review of the entire operation, the work on site commenced on 13 March 2006. The drilling and blasting operation was completed on Saturday 1 April, and a successful breakthrough was established. The need for finishing operations in the opening was minimal and the operation was considered a success.

The total cost of the implemented method was NOK 18 - 20 million.

1. INTRODUCTION

There are two cooling water tunnels at the Ormen Lange plant. One of the tunnels with a cross-section of 25m2 is meant to function as an intake tunnel for cooling water and pierces the seabed at 82 metres below sea level. This tunnel is 1,345 metres in length from tunnel portal to piercing point and has a branch to an intake reservoir. The other tunnel with a theoretical cross-section of 20m2 is an outlet tunnel that is about 980 metres long and pierces the seabed at 40 metres below sea level. One of the purposes of this tunnel to is lead cooling water from the process plant back out to sea.

The excavation of the tunnels was started in June 2004. The outlet tunnel and intake tunnel were completed and secured in June 2005, as were the intake and outlet reservoirs with associated shafts.

In the contract with Hydro, Skanska was responsible for designing, planning and performing the tunnel breakthroughs to the sea. The approximate position of the piercing sites was given by Hydro, but Skanska was responsible for their exact positioning.

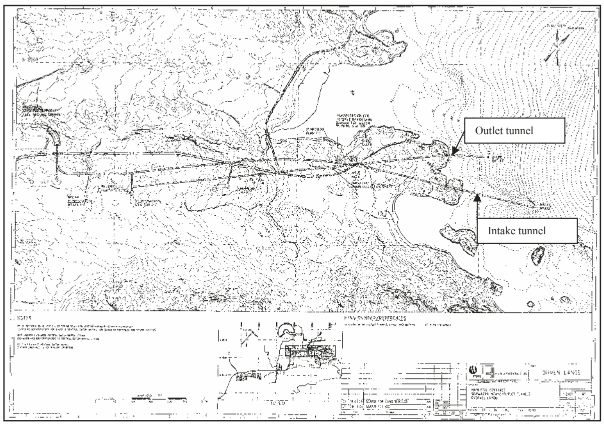

Figure 1. Plan of cooling water tunnels

As can be seen from Figure 1, the outlet tunnel pierces the seafloor in a relatively flat area. The intake tunnel, on the other hand, runs along a distinct ridge and ends in a steep rock face. In the light of available bathymetric maps and ROV investigations, the need to move the piercing sites was not considered at first. In the case of the intake tunnel, this was later reassessed, and it was decided to turn the tunnel the last 25 metres before breakthrough. This turn ensured a good rock cover over the tunnel and a more favourable intersection of fractures and weakness zones out towards the breakthrough. The rock surface in the piercing area is steep and ROV investigations identified a partial overhang.

In consultation with Hydro, it was decided that the firing of the breakthrough rounds should be done with a dry tunnel.

On Wednesday 15 September 2005 the first breakthrough round was fired at 40 metres below sea level. The breakthrough round was successful and the calculations made in advance for upsurge and filling time corresponded well to real observations.

The breakthrough of the intake tunnel at 82 metres below sea level was planned for Thursday 26 October 2005. The underwater tunnel piercings were initially planned to be approximately simultaneous, but it was decided to delay the last breakthrough to allow necessary clearing and concreting work in the bottom of the intake reservoir to be done.

This underwater piercing was not successful. The round was registered as having gone off off, but water did not enter the tunnel in the expected volume.

This meant that we were in a situation where we had a failed underwater tunnel piercing at 82 metres below sea level. There was no question of sending people into the tunnel to see what had gone wrong, and there was no easy access for divers. This was hardly an ideal situation to be in.

2. DRIVING THE INTAKE TUNNEL TOWARDS THE PIERCING SITE

The TP-28 procedure "Procedure for piercing rounds in sea tunnels" was followed from chainage 1340 to 1397 in the intake tunnel as regards round lengths and probe drillings. A change of direction for the tunnel was made from chainage 1397 towards piercing (chainage 1421), and a revised probing programme was drawn up (see Figure 2).

The rock in the breakthrough area is dark banded amphibolitic gneiss of excellent quality. The rock was unusually hard to drill through, and was marked by strong foliation oblique to the axis of the tunnel. The foliation had a steep varying fall of 50 - 78o to N or S. There was a marked clay-infected fracture perpendicular to the axis of the tunnel and with a fall of 60 - 70o to N of the working face. Five grout screens were drilled in the area of chainage 1397 – piercing round. The boreholes varied in length from 6 - 12 metres. Before the breakthrough round, we had one certain through-drilling above the tunnel (chainage 1405). This was plugged and grouted.

A more extensive probe drilling was carried out at chainage 1414.5 to check the rock cover. Holes of 4 - 6 metres in length and with varying orientation were drilled in front of the working face. Holes were also drilled at a downward angle to identify any overhangs. None of the holes were drilled right through. On the basis of these drillings, it was decided to take two smaller rounds of 2 metres in length. The tunnel cross-section was changed so that the height was reduced and adapted to the shape of the piercing round. After these rounds had been fired, we believed, on the basis of plotted longitudinal profiles and working face position, that 5 metres of rock remained before we were out in the open sea.

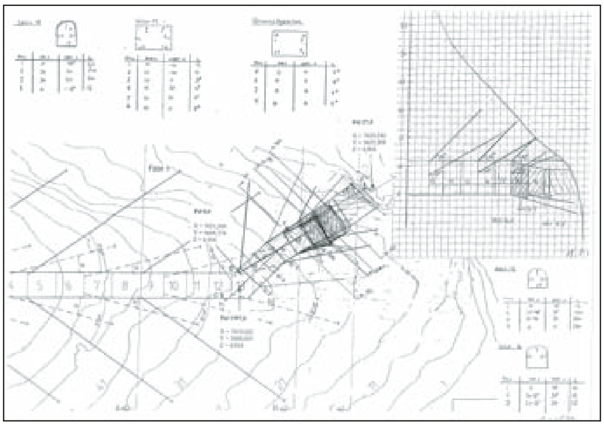

Figure 2 Plan for probe drilling and route towards the breakthrough point. Revised.

In connection with the drilling of the breakthrough round, there were plans to drill six probe holes to check the distance from the working face to the rock surface. The through-drillings were also to provide a basis on which to determine the length of the blast holes for the breakthrough round (see Figure 3).

The holes were located on the right, in the middle and on the left in the upper part of the working face. There was some reluctance to drill right through in view of the problems associated with sealing leakages.

The first holes were drilled on the left hand side in the upper part of the working face. After drilling for 3.69 metres in this area, water at high pressure was encountered. After the procedure, the drill string has to be passed out a minimum of 1 metre after the "through-drilling" has been registered. The drilling rig was used to pass a rod with a packer into the hole. However, it was difficult to get the packer into the hole because of the water pressure (difficult to centre or hit the hole,

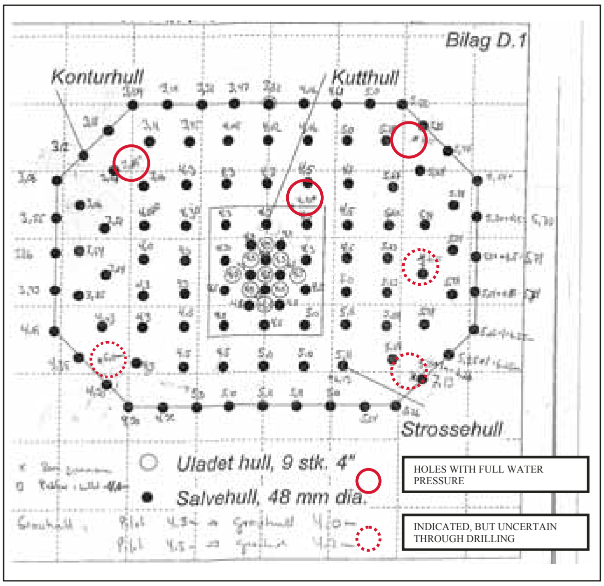

Figure 3. Drilling plan with hole lengths for breakthrough round

the packer was forced out of the hole, one packer became twisted). To secure the hole, two packers were inserted. The next hole was drilled in the upper right-hand half. Here it was necessary to use two extension rods to drill right through. Through drilling was indicated at 6.15 metres. Hole 3 was drilled in the middle of the working face, immediately above the cut. Here, through-drilling was indicated at 4.91 metres in that large amounts of water were encountered. Three holes were drilled in the lower part of the working face. On the left-hand side, "through-drilling" was indicated at 5 metres, whilst in two holes on the right-hand side it was indicated at 6.15 metres and 7.13 metres. The details around the last three holes in the lower half are a little unclear. "Through-drilling" was probably registered as a hollow sound in the drill string and increasing water seepage, not as physical penetration. The hollow sound was characteristic and well known from through-drilling in the upper half of the working face, and from through-drillings made in the outlet tunnel.

The through-drillings that were made corresponded extremely well with the profiles that had been drawn as regards the distance between the working face and the sea. There was therefore no reason not to trust the probe drillings.

Based on these drillings, a plan was drawn up for the drilling of the breakthrough round. All holes for the round were planned to be drilled 0.5 - 0.8 metres shorter than the indicated penetration. The probe drillings, and the profiles and bottom contours indicated that the length of the breakthrough round would vary from 3 - 4 metres on the left-hand side to 6 - 7 metres on the right-hand side. Dyno and other blasting experts were consulted with regard to the skew face. The difference in the borehole lengths was not considered a problem for securing breakthrough.

Drilling of the round went well and no major water leakages were encountered. However a steady seepage of water from most of the holes in the upper part of the round was registered. All told, 109 Ø48 mm blast holes and nine Ø102 mm empty holes were drilled. The drilling plan was drawn up by Dyno. Specific charge quantities were calculated to be 4.4 kg/m3.

Charging of the round was started on 25 October 2005. This task was carried out by the face team under the supervision of a representative from Dyno and our own foremen and engineers. Dynomitt 35 x 380 mm was used in cut and stope holes and Dynomitt 30x380mm in contour holes. The charges were prepared in plastic tubes at the face, and inserted into place in the holes. The round was initiated using a Nonel line which was passed from the face through shafts to a safe position for the blaster on the surface.

The round was fired on 26 October 2005 at 12.00 hours and it was quickly ascertained that it had failed.

3. WHAT WENT WRONG WITH THE TUNNEL BREAKTHROUGH ROUND?

Immediately after it had been ascertained that the round had failed to break through, measures were initiated to find out what had gone wrong.

A ROV (remote operated vehicle) was requisitioned to make recordings outside the opening area. These images show that there was no hole leading into the sea. Fissures and loose blocks of rock that were interpreted as fresh could be detected, ie, it could be seen that the rock had moved. Furthermore, a "cave" of about 2.5 m x 1 m x 2.5 m (l x h x d), with small blocks of rock on a ledge, was found in the face area. This was a new an unexpected piece of information. The rock in the area otherwise seemed to be competent and intact.

These investigations did not identify any probe holes open to the surface.

Afterwards and in connection with the last drilling of the breakthrough round, it was determined that the caves were under the floor of the tunnel and therefore had no effect the first breakthrough round.

The next question to be answered was whether the round had gone off properly. Skanska considered it unsafe to go into the face, and so the tunnel was closed to prevent all entry immediately after the "breakthrough round" had been fired. Several alternative solutions for inspecting the face were evaluated. It was eventually decided to fill the whole tunnel with water and drive a ROV in from the tunnel mouth to the working face. The tunnel was filled with water also out of consideration for contractual relationships. Other contractors were to start work in neighbouring areas and were dependent upon being able to work in safety. As long as the tunnel was not filled with water, a sudden collapse could result in the flooding of areas where crews were to work. Of course, this was not acceptable.

Whilst the tunnel was being filled with water, a more extensive ROV investigation was carried out to find probe boreholes in the piercing area. This equipment had positioning means, and the area around the piercing site was searched. Only one probe hole was found during this investigation. A measuring-in of the rock surface was carried out to compare these data with existing bottom contour maps. Small differences were found in areas where the rock surface was sloping. However, in the steep portion in front of the breakthrough area, a difference of an approximately 1-metre long defined profile was found.

The ROV investigation inside the tunnel went smoothly. About 1,300 metres of cable were rolled out before the working face and the round could be filmed. Sonar images were taken so that the length of the round that had been fired could be determined. Measurements of debris and the length of the round indicated that the round had gone as planned. The debris lay mainly on the left-hand side of the working face and 20 metres backwards (sighting direction towards the working face). At the right hand side, there was a smaller mass about 15 metres back from the face.

Based on these observations and the measurements made at this stage, it was concluded that there was probably still 1.5 - 3 metres of rock before breakthrough (see Figure 4).

4. CHOICE OF METHOD FOR ENSURING BREAKTHROUGH

We found ourselves in a new and unfamiliar situation. A number of methods to ensure breakthrough were evaluated:

- **Use of additional charges**
  This was considered unsafe, since the thickness of the remaining rock plug was not known with sufficient certainty.
- **Drilling through a casing with heavy drilling equipment mounted on a barge.**
  This method was considered the most appropriate for a long time. Project planning and the ordering of equipment were well underway. However, after a more detailed feasibility analysis, the method was abandoned.
- **Drilling with land-based drilling equipment mounted on a tool carrier on the seafloor**

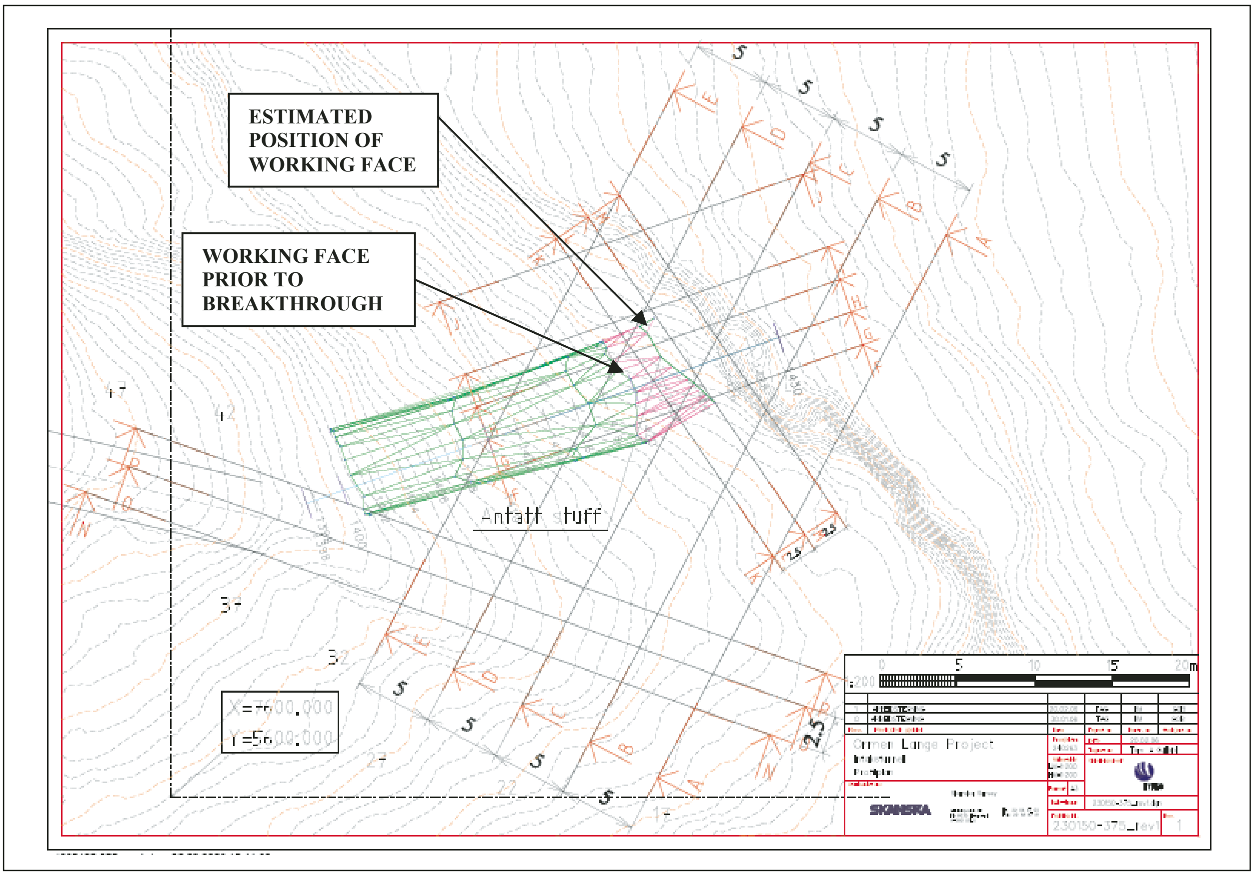

Figure 4. Working face measured in prior to firing the breakthrough round (green). Estimated position of working face after the breakthrough round (purple)

This method was fully investigated, and was in fact the method chosen in the end.
- **Divers.** The use of divers was considered as unsuitable because of the depth and the limitations it set.
- Use of a **demolition ball.**
- **Wiresawing**
- **Freezing**

As **plan B**, a method was investigated that involved plugging the existing tunnel from the shore and driving a new tunnel past the plug using the existing tunnel. If the chosen method was not successful, the intention was to mobilise for plan B.

The team working on the solutions under consideration consisted of highly skilled engineers from Skanska, but some external experts in the field were also brought in. The main aim was that we should succeed in obtaining breakthrough, and this was foremost in our minds throughout the process of finding a suitable solution.

The methods of drilling from a barge using a hammer drill and guiding pipe, and the method involving drilling equipment mounted on a tool carrier emerged early on as likely solutions. A detailed investigation and planning of both methods in parallel was started and was in progress from November until February 2006. However, at an internal meeting in February the method of drilling from a barge was abandoned as there was a great deal of uncertainty as to its practicability. In principle, we were left with one realistic method which entailed drilling with equipment located on the seabed but remote-controlled from a barge.

The safe completion of the operation was of paramount importance for both Hydro and Skanska. Therefore, an extensive documentation programme had to be gone through before we were given the go-ahead to start the operation. All details around the operation and the equipment that was to be used were analysed and carefully evaluated.

The following approaches relating to gates and blasting were discussed:
- Blasting with the same water level on each side of the gates
- Blasting with unilateral water pressure on the outside of the gates
- Removing gates and filling the intake reservoir with water
- Lowering of the water level and subsequent increase of water head.

It became clear at an early stage that blasting with water pressure against the gates was not suitable because the shock wave from the blasting would propagate through the tunnel to the gate. The last alternative of lowering the water level to a defined level was the most suitable. But here too there was a need for modelling to check what stresses the gates could withstand when the wave came.

No conclusion was reached until immediately before the blasting date. After the increase in water head had been verified, it was decided that before blasting:

- The water level should be lowered to below the gates (contour 74, equivalent to the 13-metre contour below sea level).
- The uppermost gate stop log should be removed.
- To verify models, measuring equipment should be mounted on the gate stop logs (pressure sensors and accelerometers).

Calculations showed that if these measures were implemented, the gates would withstand the stresses. It was reckoned that about 400 m3 of water would come across the uppermost gate stop log and into the intake reservoir. The conclusion after breakthrough was that the calculations made in advance corresponded well with the actual observations.

5. DESCRIPTION OF THE CHOSEN DRILLING METHOD

The method chosen for drilling the breakthrough round combines several disciplines which are mutually independent, but which in the case of this project had to work together. The development of the method can be said to be pioneering work.

Skanska Norge contracted the following main companies for the preparation and implementation of the chosen method:

• Dyno Nobel AS	Explosives and blasting plan
• ScanMudring AS	Tool carrier for drilling unit and extraction of masses
• Sperre AS	ROV (Remote Operated Vehicle, mini-sub)

The principle of the method was that a conventional drill should be mounted on a tool carrier located on the seabed in the piercing area, see Figure 5. The drilling operation was to be monitored from cameras on the tool carrier, and also by a mini-sub (ROV) which was to be controlled from the surface. The tool carrier was to be lifted into position from a drilling barge and control of the drill, the tool carrier and the ROV was to be effected in a coordinated manner from the same barge.

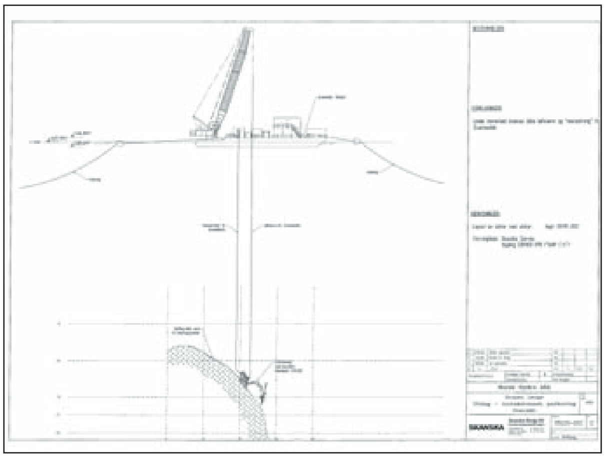

Figure 5 Principle of the chosen method

Figure 6 Model of the piercing area on a scale of 1:100

After the round had been drilled, the holes were to be charged from the surface with individually tailored charges that were to be guided into place by the ROV. The round was to be fired from a separate blasting barge. Prior to the firing of the round, the water level inside the tunnel had to be lowered to below the lowermost gate stop log on the intake reservoir. Test pumping had been done in advance using two large pumps (capacity 4 - 5 m3/min) to check that it was possible to lower the water level by pumping.

The pumps were in operation throughout the drilling operation to keep the water level at a predefined level about 5 metres below mean water level. Calculations were made showing that at this level water would not splash over the gates if there was a sudden collapse at the breakthrough point. All work inside the intake reservoir could therefore continue throughout the drilling operation without any interruptions.

After the round had been fired, the opening had to be inspected and rubble from the last round had to be cleared away. It was intended that this should be done by water suction, grabbing or digging.

On the basis of available measurement data of the seabed, a model of the seafloor in the piercing area had been made on a scale of 1:100 (see Figure 6). This was of great help when the drilling and blasting plan was to be drawn up.

A drilling and blasting plan was drawn up (see Figure 7) on the basis of existing sea contour maps, measuring-in of the face before the firing of the "breakthrough round", drilling lengths of blast holes for the breakthrough round and more "accurate" depth measurements. See Figure. 7.

All the holes were to be drilled using a Ø4" bit and a 1.3 x 1.3 metre drilling pattern as a starting point. A separate table of hole lengths for the different boreholes was

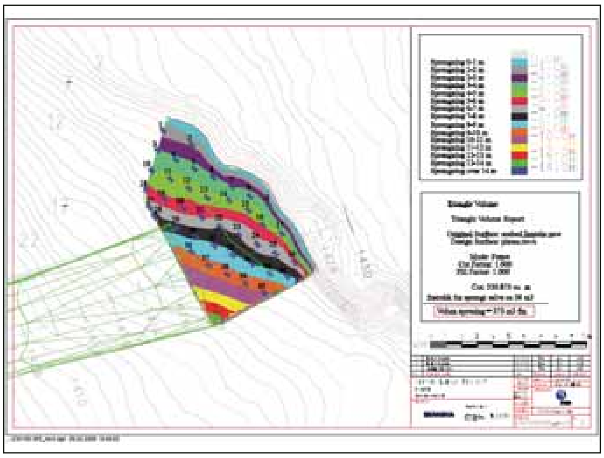

Figure 7 Drilling and blasting plan for piercing round

drawn up. This was done with a view to drilling through to the tunnel.

As can be seen from the drilling plan, the intention was to drill and blast a larger section than the actual tunnel opening in the breakthrough area. This was planned having in mind that the excavator was to have an area to land on after the round had been fired.

The following equipment package was used:

Drilling barge:
The Balder was used as drilling barge, and was chosen on account of its crane capacity and size. The vessel was rerigged with extra living quarters and units and equipment adapted for the operation (Scanmachine SM03, ROV), see Figure 8.

Tool carrier for the drilling unit:
The tool carrier supplied by Scanmudring AS is known as SM03. The machine is constructed around the chassis of an ordinary excavator. The machine without drilling equipment has a free-air weight of about 13 tonnes and is designed to work at depths of as much as 1,000 metres (see Figure 9). The machine is basically constructed for

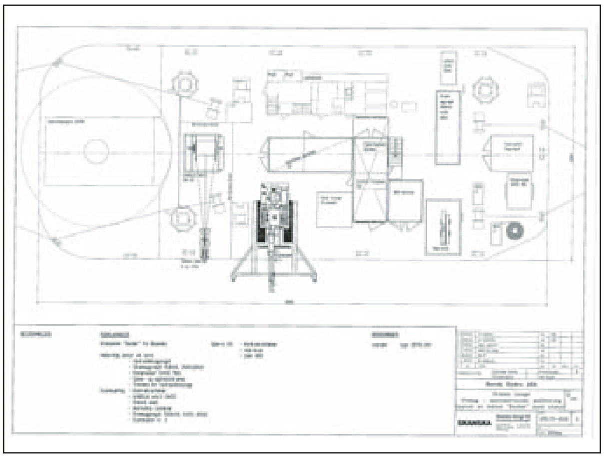

Figure 8 Rig plan for the Balder

digging and dredging on the seafloor, but for this project it was modified to be able to carry the drilling unit. To enable the tool carrier to work in steep terrain with a fall of 45 - 600 without tipping over, it had to have rigid support legs welded onto the front (see Figure 10).

Figure 9 The SM03 tool carrier for the drilling unit.

The SM03 was also to be used to dredge or dig out the masses remaining after the last round once the breakthrough round had been fired. Scanmudring AS had its own crew who operated this machine.

Figure 10 Lowering of SM03 with drilling unit

Drill:
A standard drill of the Montabert HC 40 type was used. A thorough check of the equipment was made, and critical seals and points at which it was undesirable to have water ingress, were greased and sealed. There was a reserve drill that could be used in the event of a breakdown. Hydraulic hoses were extended and passed up to the deck of the barge. They were kept floating in the sea by floats.

It was decided to drill using a Ø4” pointed bit and an extension rod. Maximum drilling depth for the equipment was therefore about 7.4 metres.

ROV (Remote Operated Vehicle):
Sperre AS provided a ROV and a crew to operate it. The ROV was of vital importance throughout the operation as it was used to monitor the work. It had to be in service in basically all operations that took place under water. The danger of entanglement with cables, anchor chains and, later on, charge hoses was considerable. The equipment used was a Subfighter 7500 with installed sonar (see Figure 11). Apart from being used to “see”, the ROV was also used to move slings on support bolts, to install funnels for the subsequent charging operation and, during the charging operation, to guide the charges into place.

Figure 11 ROV Subfighter 7500

Tripod:
Because the rock surface outside the opening area had a fall of from 45 - 70o, it was impossible to get the tool carrier with drilling equipment to stand stably on the seafloor during the drilling operation. A number of alternatives were evaluated. A solution that involved establishing support bolts on the ridge above the piercing site was considered the optimal solution. To drill holes for the bolts, a tripod fitted with a drill hammer taken from a Commando 300 rig was constructed (see Figure 12). The tripod had legs that could be adjusted to the fall of the rock. Two locating points were identified

so that slings running from these points down to the tool carrier covered the piercing area. The equipment was tested in open air before it was used at a depth of about 40 metres. The drilling went well at 40 to 62 metres below sea level and the bolts were installed as planned (bolts with a diameter of 60mm)

Figure 12 Lowering the tripod

Positioning system:
There was some uncertainty associated with the bathymetric map that was available, and therefore prior to the operation a detailed survey of the piercing area was made using a multi-beam echo sounder. In addition, "fixed points" were set in the area around the piercing site so that location of drilling points and positioning could be as accurate as possible. Transponders which sent signals to the ROV were located at the fixed points. A system known by the abbreviation GAPS (Global Acoustic Positioning System) was used for this. This system worked well at times, but proved to be unstable or to have insufficient accuracy. A more traditional method using GPS, sounding and the use of measuring tapes from known points was therefore used extensively during the drilling operation for the collaring of boreholes.

Explosives:
Today there are no civilian explosives or initiators that are approved for use at depths as great as 80 metres below sea level. Explosives able to withstand the pressures in question can be made to special order at Dyno's Gullaug plant. However, it was decided to try out standard explosives adapted for 4" boreholes before having any specially made. The choice fell on Dynoprime 1.7. The explosives and initiators were lowered to 90 metres and stored for 24 hours, 48 hours and 96 hours prior to detonation. It was found that the initiators might be a weak point. Attention was focused on this possibility and measures were taken during the manufacturing process, so that this problem too was solved. However, to be on the safe side, it was decided to use 2 - 3 initiators per hole depending on the length of the hole. The initiators were equipped with 120-metre long Nonel lines.

Figures 13 and 14 are schematic drawings of the design of the hole charges. A charge containing Dynoprime is shown in Figure 15, whilst Figure 16 shows a charge containing Hexotol explosive.

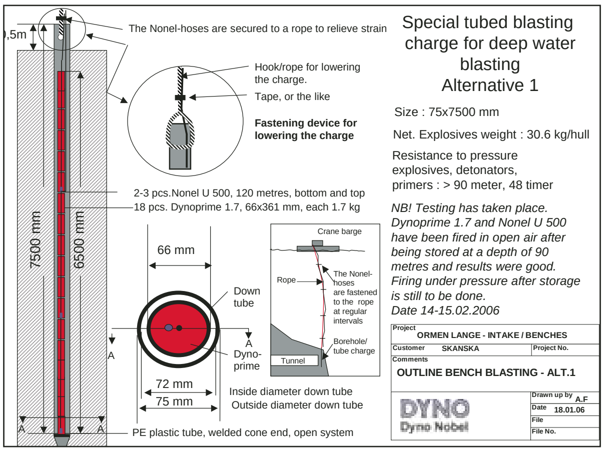

Figure 13Charge tube containing Dynoprime 1.7

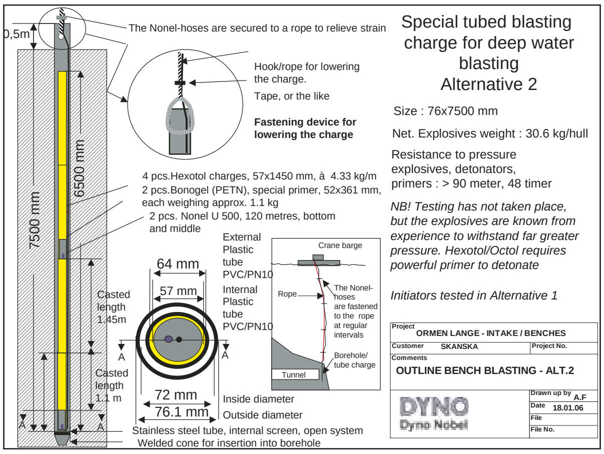

Figure 14Charge tube containing Hexotol

6. EXECUTION OF THE DRILLING AND CHARGING OPERATION

6.1 Preparatory work

After all the necessary documentation concerning the feasibility of the method had been examined and approved by Hydro, mobilisation of equipment was started at Aukra in the week beginning 13 March. Prior to mobilisation some dry runs with the equipment were made separately (tripod, drilling unit on tool carrier).

During the operation, the weather was cold and stable and the sea was still, which was something we were completely dependent on.

The first week was taken up with assembling the equipment and completing the rigging of the Balder. All necessary materials were taken on board or put in store for possible later use. Systematic filming of the piercing site and the area for the positioning of the anchor bolts was done. Transponder points (two positions) were established (see Figure 15) in order to obtain a good reference for collaring the boreholes.

Figure 15 Transponder point

Before drilling could start, the two anchor bolts had to be positioned. This was done using the tripod. The holes were drilled 2 metres deep and the bolts were mounted using the ROV. The bolts were lowered from a boat and guided into place in the hole by the ROV. The modified drill worked well at the depths in question (53 to 62 metres below sea level).

Once the bolts were installed, the SM03 was taken on board and made ready for drilling. The barge was fastened to a mooring point at sea and bollards or anchoring points ashore and positioned by means of winches on board. To ensure that the SM03 did not slide during drilling, loading slings (capacity 20 tonnes) were passed from the SM03 back to the established anchor bolts. Suitable lengths were used so that the position of the drill could be adjusted by adjusting the sling around the anchor bolt. The ROV was used to slip the sling on and off the bolt. In some cases, there was a need for assistance with a rope from the surface. However, by raising the SM03 in the water, the ROV could on the whole ensure extension and shortening on its own. After a successful "wet test", the drill was lowered to the piercing site for the first collar. Hydraulic hoses and communication cable (umbilical) had floats mounted thereon to keep them upright in the sea. When the equipment was on the deck, the hoses lay floating on the surface.

6.2 Drilling

The drilling started at the very front and an attempt was made to move backwards line by line. It was quickly found that the terrain was steeper and more undulating than maps and models indicated. The tool carrier more or less hung from the slings throughout the drilling operation. Despite the difficult collaring conditions, the operators managed in general to drill the holes where they were planned (see Figure 16). The holes were planned to be drilled vertically or with a slight backward fall (8 - 10°).

Figure 16Drilling a blast hole

As the holes were completed, funnels were inserted into them, see Figure 17. These were intended to prevent cuttings or detritus from entering the holes and, to facilitate the planning and locating of holes. The funnels were numbered consecutively in ascending order. The funnels were a great help for orientation during the drilling operation. Because of the steep terrain, the tool carrier had to be moved frequently. At best, it was possible to drill as many as three holes per move, but usually only one or two holes per move were drilled.

An accurate log was kept of drill lengths for each hole which could then be used for preparing the charges.

The ROV operator had a very central role in the operation. He had to check collaring points, collaring and the location of the tool carrier. He also had to monitor drilling and the moving of the machine, and the raising and lowering of the machine.

As the drilling progressed, several through-drillings through to the tunnel within were detected. The through-drillings were partly made intentionally to map the posi-

Figure 17Funnels for marking boreholes

tion of the tunnel and were partly unintentional. Holes that were drilled right through were plugged with steel cones to reduce inleakages into the tunnel, see Figure 18. The shortest drilling length during the through-drilling was about 1 metre (in the roof of the tunnel).

Figure 18 Steel cones for plugging boreholes that were drilled right through

Based on the new information that gradually came to light, a revision of the drilling plan was made. It was decided not to go as far out to the left as planned, see Figure 19. The through-drillings indicated that the face of the tunnel was quite close to the breakthrough point. By drilling further back in the roof of the tunnel, it would be possible to land the SM03 inside the tunnel instead. This also gave a reduction in the number of holes drilled.

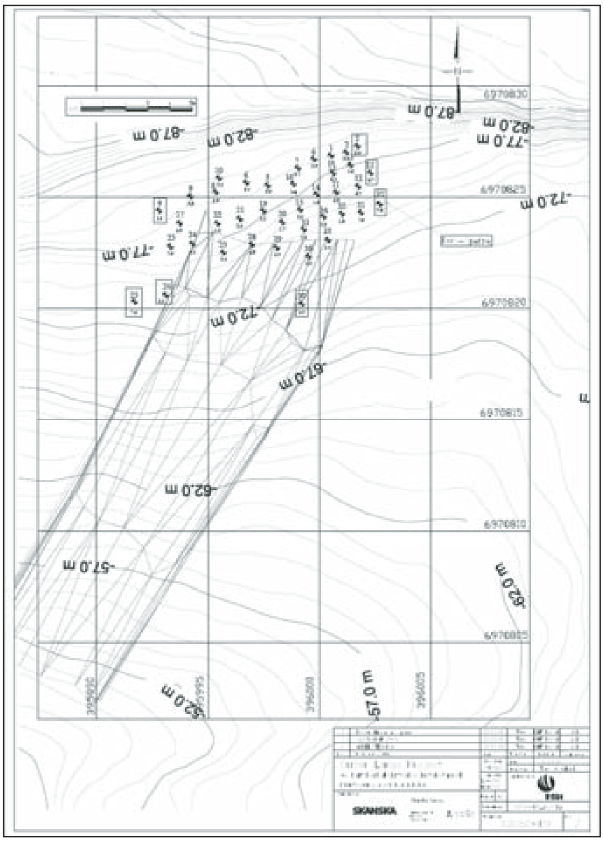

Figure 19 Plan for the fully drilled piercing round

In the period 24 March - 30 March a total of 39 holes having a length of from 1.2 metres to 7.6 metres were drilled. All told, 127.6 metres were drilled.

6.3 Charging

Prior to the start of charging, all holes that were to be charged were logged and cleared of any cuttings or crabs. For each hole, charges were prepared in 76 mm diameter PEH plastic tubes with lengths adapted to the length of the boreholes. The tubes had a tip welded on at the ends to facilitate entry into the funnels or boreholes. The charges were prepared on the deck of the Balder in a typical production line operation. The charges were lowered down using 6mm diameter floating rope to a defined depth. At that depth, the ROV was waiting to guide the charges into the hole. The charging started at the back of the round. This was to prevent the ROV from becoming entangled with the charge lines. With 2 - 3 lines from each hole, this became in time a rather demanding operation, see Figure 20

Figure 20 Charging boreholes

Once a charge had been placed in the holes, the Nonel lines were passed back to a blasting barge and fastened to numbered pegs corresponding to holes on the bottom, see Figure 21.

Figure 21 Blasting barge

With a few exceptions, the charging of the holes took place without any problems. The charging was started in the evening of 31 March (2010 hours) and the last hole was charged on 1 April (1202 hours). A total of 32 holes were charged with a total explosive quantity of 493 kg. The round was fired at 1900 hours on Saturday 1 April 2006. This time water appeared as expected.

The reason for not firing the round immediately the charging was completed was related to calculations made prior to the firing. These calculations showed that the round ought to be fired at low tide to reduce the load on the gates as much as possible.

After the round had been fired, 60 seconds passed before we heard that the water began to rise in the intake reservoir. The water rose steadily until it ran over the gate after about three minutes. It seemed as though the water came in two pulses, first a gentle pulse and then a more powerful one. The water flowed over the gates for about one minute before gently drawing back. After about eight minutes the water column had settled completely.

6.4 Finishing operations

After the round had been fired and the water had cleared, a ROV investigation of the piercing area was made. The inspection revealed that the blasting operation had been successful. It showed that between 1 - 2 metres of rock mass remained before the first breakthrough round had broken through. There was extremely little rock cover in the left half of the roof seen in the order of ascending chainage number. The margins had clearly not been in our favour.

There were very few large rocks in the old rubble and much of the old rubble had been blasted away. However, some rock removal was required in order to obtain sufficient opening (20 m2 over a length of about 5 metres). The Balder was therefore rerigged to take a grab and the SM03 with dredging equipment. Some grabbing was done and the SM03 used blades on the machine and suction equipment to remove some of the mass in the actual tunnel mouth.

Once this work had been completed and sonar scanning of the entrance portion with a ROV had been done, the job was considered to be finished. The equipment was taken ashore and demobilised.

The on-site operation was carried out in the period from 13 March - 4 April 2006. This was well within the overall time schedule allowed. However, the planning of the operation had been underway since 26 October 2005.

7. CONCLUSIONS

The reasons we found ourselves having to implement extraordinary measures in order to achieve breakthrough are many.

The most obvious reason is that the blast holes in the breakthrough round were not drilled long enough in critical areas. The cut had not managed to break through to the sea and create the necessary opening to allow the rest of the round to break through as planned. This in turn was connected to the fact that the number of certain through-drillings to the sea had been too small to be able to form a reliable picture of the rock surface beyond. The insufficient number of through-drillings had both a practical and a psychological cause. With a pressure of 8 bar it is difficult to get packers into the borehole. The crew at the working face knew that we would be faced with a serious problem if the drilling caused leakages which could not be sealed. Naturally, this meant that the face crew was reluctant to make too many through-drillings.

On the basis of the data we had from the ROV investigation, sectional drawings and drill-through lengths which were identical with sectional drawings, we believed that we had an adequate picture of the seabed topography. Experience has shown that the accuracy of map documentation is crucial. In steep terrain and at great depths it is difficult to produce good depth charts. This is something that requires more attention.

Another reason that the breakthrough round failed may be the rock quality. The rock in the area is hard, dark and solid amphibolitic gneiss with cross foliation (60 - 80o) which crosses the tunnel axis in a gentle diagonal. Later ROV observations showed that the rock had fresh fissures and cracks after the blasting, which may suggest that the rock mass had been about to collapse. However, the external pressure had been too great to allow the mass to be blown out and given the required opening, and a situation involving a sort of lid effect had thus arisen. As the tunnel is horizontal, we were not helped by gravity during the blasting. If the tunnel piercing had been more upwardly directed, the outcome may have been different.

Based on the ROV observations made after the successful breakthrough round on 1 April 2006, and observations made during the drilling of the breakthrough round, it is clear that the margins were not in our favour.

When we found ourselves confronted by a failed breakthrough, we were in a situation that was anything but desirable. However, it has been impressive to see the creativity and enthusiasm on site to find solutions to complex problems. Cross-discipline cooperation has turned a fiasco into a success. The development of the method has been pioneering work, and Skanska has shown that it is possible to succeed.

8. FINANCES

An operation of this kind, involving a large number of people, disciplines and equipment has a price tag. The total cost of the operation came to NOK 18 - 20 million. It is therefore not a solution that will be first choice for breakthrough, even though from a safety point of view it can be deemed a success. The method has at least proven to be practicable.

9.1 NORWEGIAN HIGH PRESSURE CONCRETE PLUGS

Jan Bergh-Christensen
Einar Broch

ABSTRACT:
A study on design, construction and operation of high pressure plugs was carried in the period from 1987 to 1991. Details of more than 30 plugs with static pressure head up to about 1,000 meters were collected and analysed. While the concrete length varies from 2 to 5 % of the static water pressure, the steel lining may be as short as 0.4 % of the water pressure head.

INTRODUCTION
A research project to document and analyse gained experience in the design, construction and operation of high pressure concrete plugs in Norwegian hydro power plants was carried out. The purpose was to evaluate the technology, and to develop guidelines of good practise in the planning and construction of concrete plugs for high pressure gas storage caverns.

Information of about 150 concrete plugs was collected. The data base includes some 30 plugs with water heads above 400 meters constructed during the period 1970 - 1990. During the 1980's, many high pressure plugs were constructed for static water head up to 1,000 meters. The study was concentrated to the newest plugs which were designed and constructed in line with improved quality standards and grouting technology. The newer plugs were more expensive, but also more efficient in terms of reduced leakage as compared to older cement-grouted plugs.

SITE	WATER HEAD[1] m	YEAR	CROSS SECTION m^2	LENGTH CONCRETE m	LENGTH STEEL m	WATER LEAKAGE l/min
NYSET-STEGGJE	964	1987	25	55	Penstock	< 60
TJODAN	880	1984	17	45	Penstock	2
TAFJORD K5	790	1982	18	88	Penstock	50[3)]
SKARJE	765	1986	25[2]	20	5.5	< 15[3)]
MEL	740	1989	22	27	27	1[3)]
SILDVIK	640	1981	26	35	12	< 240
JOSTEDALEN	622	1989	35	20	5	6[3)]
LOMI	565	1978	20	15	9.5	190
LANG-SIMA	520	1980	30	50	Penstock	120
SØRFJORD	505	1983	20	20	12	10[3)]
KVILLDAL	465	1982	31	30	4	[4)]
TORPA	455	1989	32	20	6	< 1[3)]
EIKELANDSOSEN	455	1986	20	20	5	8
STEINSLAND	454	1980	20	20	10.2	[4)]
KOLSVIK	449	1979	23	20	10	30
SKIBOTN	445	1979	18	12	7.6	96[3)]
LEIRDØLA	441	1978	26	30	Penstock	< 54
SAURDAL	410	1985	49	40	1.5	5[3)]
ORMSETFOSS	373	1988	22	22	7	< 3
DIVIDALEN	295	1972	10	13	4.5	< 120

Table 1: Key figures for some major plugs.
1) Max. static head 2) Varies from 20 to 30 m2 3) Remedial grouting at first water filling or later 4) Within accepted limits

PLUG TYPES

The two main types of concrete plugs used in hydroelectric power plants are shown in Figure 1. The penstock plug is located at the upstream end of the steel penstock, at the transition to the unlined pressure tunnel. Access to the unlined tunnel system is usually provided by an access gate plug located in the access tunnel adjacent to the pressure tunnel.

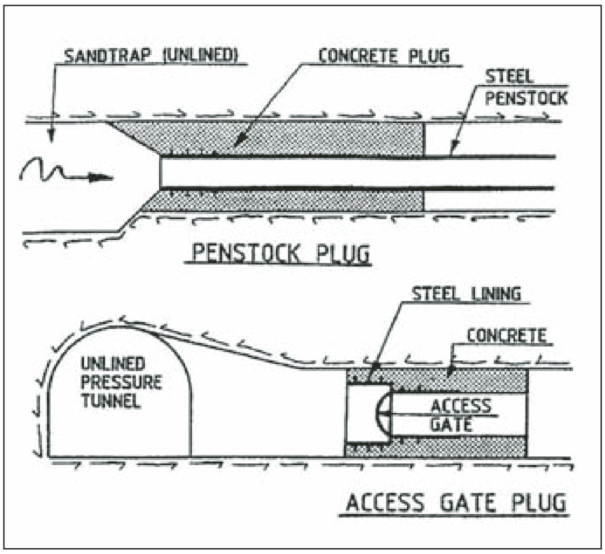

Figure 1: General layout of penstock plug and access gate plug (from Bergh-Christensen 1988).

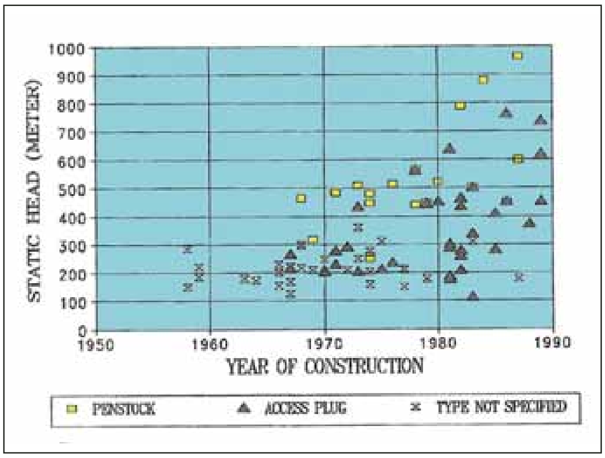

Figure 2: Max static pressure at plug vs. year of construction.

Figure 2 shows the trend towards higher water pressure for unlined pressure tunnels in Norway.
The tendency of increasing water head since 1970 is related to more extensive use of unlined pressure tunnels, especially after introduction of the air cushion surge chamber technology, see Gomnæs & al. (1987) and Goodall & al. (1989).

DESIGN

There are two fundamental requirements for the design of a concrete plug. Primarily, it must have the structural capacity to carry the static load from the water or gas pressure. Secondly, specific requirements must be satisfied in terms of leakage. Both in the design and the construction, there are normally few problems related to the load capacity. The length and layout of the concrete structure, however, often seem to be a subject for discussion. Less attention seems to be paid to the leakage problems, although one conclusion from the study is that efforts to achieve the optimum tightness are very important, both in terms of functioning of the plug and in terms of the total construction costs.

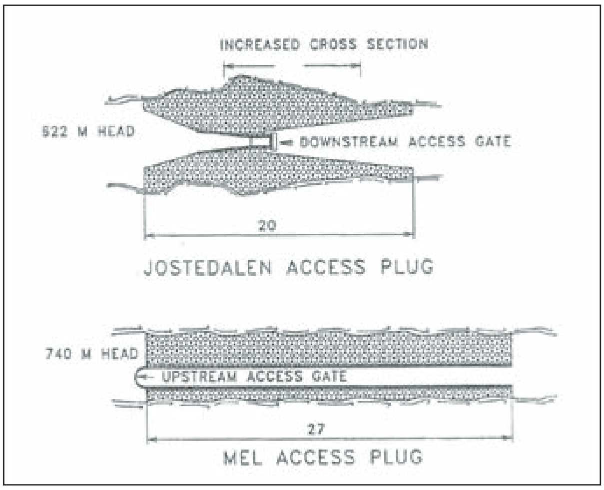

Figure 3: Sketch of Mel and Jostedalen access plugs.

The plug design may vary with respect to the length of both the concrete structure and the steel lining. Figure 3 illustrates the design of two different access plugs constructed in 1989. For access plugs, the steel lining is normally shorter than the concrete lining, and may be located in the upstream, intermediate or downstream part of the plug. The access gate may be located anywhere along the steel lined section. The shape of the plug may be simple or it may vary along the length axis in agreement with the established stress distribution.

PLUG LENGTH

It is commonly accepted that the plug length should be related to the actual water head or gas pressure. As shown in Figure 4, the length of both the concrete structure and the steel lining
(for access plugs) may vary within wide limits, even for the same water head. The steel lining is usually shorter than the concrete lining, the extreme being the Saurdal access plug with a steel lining of only 1.5 meter at a static head of 410 meter. Sometimes the steel lining of the access plug may even be of the same length as the concrete structure (Mel plug).

Figure 4 shows that the length of the concrete structure for an access plug ranges from about 2 to 5% of the maximum static water head (in meter). For tunnel cross sections ranging from 8 to 50 m2, this represents a maximum shear stress of about 0.4 MPa at the plug circumferential area, assuming a uniform shear distribution in the rock to concrete interface. This used to be the maximum accepted shear stress for uniaxial situations according to former standards for concrete structures (for uniaxial concrete strength 25 MPa, i.e. C25).

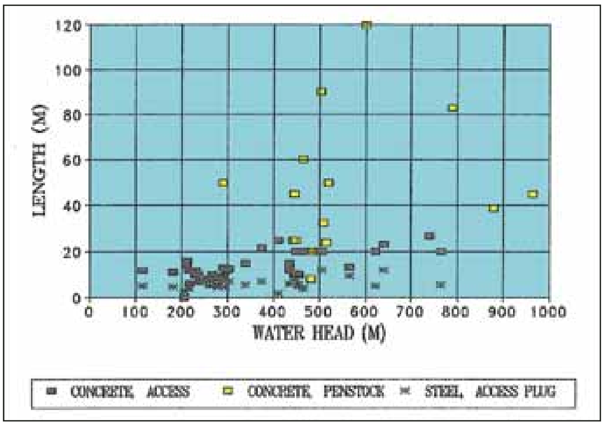

Figure 4: Length of steel lining and concrete structure vs. static water head.

The maximum linear hydraulic gradient along the plug axis (ratio of water head to concrete length) that may be calculated for a shear stress of 0.4 MPa will be ranging from 20 to 50 for the tunnel cross sections in question. This complies with a traditional rule of thumb for plug design in Norway, which is based on the assumption that higher gradients may lead to unacceptable high leakage. This gradient criterion may be considered radical. Benson (1989) has for instance suggested that the maximum hydraulic gradient should be as low as 20 for massive, hard and widely jointed rock types.

In reality, the uniform shear distribution presupposed in this design principle is not valid. Numerical modelling carried out during the research project showed that the shear stress will be concentrated to the first five meters of the upstream part of the plug (assuming steel gate located upstream so that the water pressure is not acting from inside the plug structure). The shear stresses rapidly decrease further downstream along the plug. Therefore, if one considers the actual stress distribution within the concrete body as calculated by numerical methods, relatively short plug lengths could be allowed. In practical design, however, one should also consider the three dimensional water flow regime and the limitations with respect to grouting. In this context, it is the authors' opinion that the minimum plug length for high pressure plugs that are supposed to act as water tight constructions should never be less than five meters.

CONSTRUCTION AND OPERATION

There has never been reported any plug malfunction or failure related to overloading in Norwegian hydropower projects. The only "failure" experienced is unacceptable high leakage. Normally, remedial grouting will be carried out during the first water filling or at a later stage. But the criterion for remedial grouting may vary a lot among the plug owners.

Grouting

A description of the grouting methods for concrete plugs has been presented by Bergh-Christensen (1988). The quality and the extent of rock and concrete grouting are of great importance both for the final construction costs and the leakages at the plug. This is illustrated in Figure 5, in which the construction costs are given for the two plugs shown in Figure 3. As can be seen, the grouting costs are in the order of about 40 to 45% of the construction costs for both plugs (both constructed in 1989). Grouting of the rock mass prior to concreting works amounts to about 10 % of the total cost. Remedial grouting during or after the water filling of the tunnel, accounts for another 10 %.

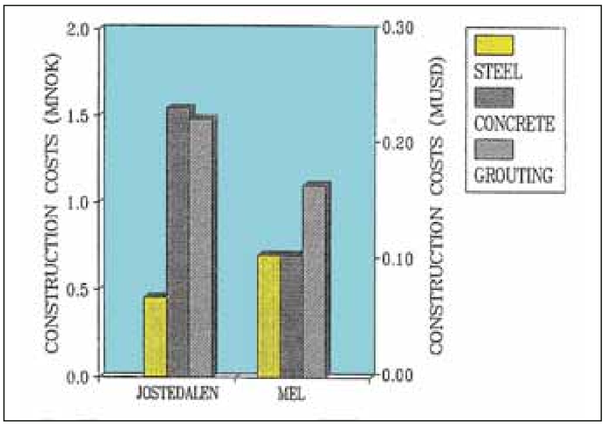

Figure 5: Construction costs for Mel and Jostedalen access plugs (in million Norwegian kroner and US dollars).

Whereas the steel lining at Mel is 5.5 times longer than for Jostedalen, the lining costs were only about 50 % higher. This is due to the more complicated design at Jostedalen, in which case especially the gate construction is expensive. The total costs are higher for the Jostedalen plug than for the Mel plug, even though the Mel plug is longer than the other. The concrete volumes of the plugs are about 600 m3 and 700 m3 for Mel and Jostedalen respectively. The simplicity of the Mel construction as compared to Jostedalen is probably the main reason for the cost differences.

A comparison of grouting costs for several plugs is shown on Figure 6. The costs are actual costs at the year of construction. At both Jostedalen and Mel, the most modern grouting technique with both polyurethane and epoxy injection at high pressure through grouting hoses has been used. At Ormsetfoss, this was done at a less ambitious extent. Much of the grout was injected through boreholes immediately before the first water filling. At Sørfjord, epoxy was not used and all grouting was done through boreholes.

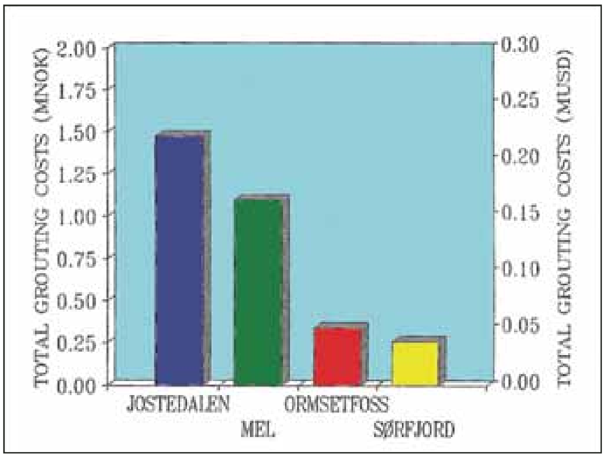

Figure 6: Grouting costs for some access plugs.

The consumed grout mass as documented for some plugs is shown in Figure 7. As can be seen, several tonnes of (fine grained) cement are normally injected. Most of the cement mass is used to fill the voids that normally will develop in the contact zone between the rock and the concrete at the tunnel roof. If cement grouting is neglected or not well performed, large quantities of the far more expensive chemicals will be needed.

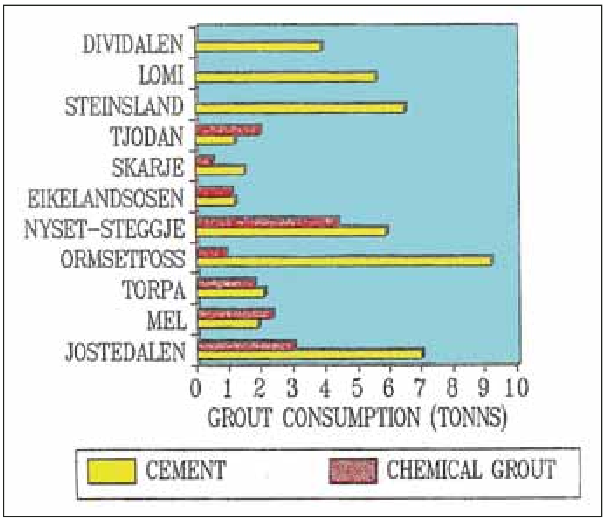

Figure 7: Grout consumption at some concrete plugs.

Often one will observe that the plug is constructed at the very latest stage before the power plant is put into operation. The plug construction period must therefore be short.. The cast concrete temperature will often rise to about 60 to 70° during the curing period. The plug will then cool down gradually, but slowly. Efficient grouting must not take place too early. It must be delayed until the concrete temperature has reached an acceptable low level. Because the construction of the plug is on the critical path of the overall timetable, it is a trend that grouting takes place too soon. Both the tightness of the plug and the grouting expenses will suffer. Careful planning and control with the concrete temperature is the solution of this problem.

The efficiency of the grouting works is believed to be dependent on the grouting pressure in relation to the water head and the rock stresses. For several plugs, the grouting pressure has been considerably higher than the water pressure. Figure 8 shows how the grouting pressure for some plugs is related to the static water pressure.

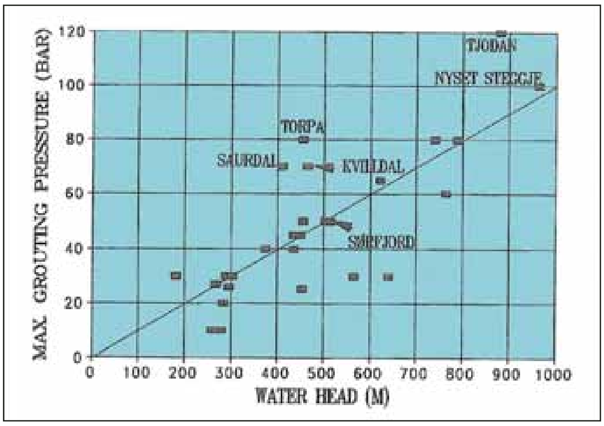

Figure 8: Grouting pressure vs. static water head.

At Torpa and Sørfjord, the grouting pressure was higher than the minor principal rock stress as indicated by overcoring measurements. At Torpa, the grouting pressure was even higher than the hydraulic jacking pressure measured at the plug location.

Leakages

When relating the water leakages to the water head, there apparently is no connection. In theory, the leakage should decrease with decreasing pressure gradient (Darcy). However, linear regression analysis does not correlate the leakage to the hydraulic gradient (Figure 9). Nor has there been found any correlation between the leakage and the length of the steel lining or the linear gradient at the steel lining.

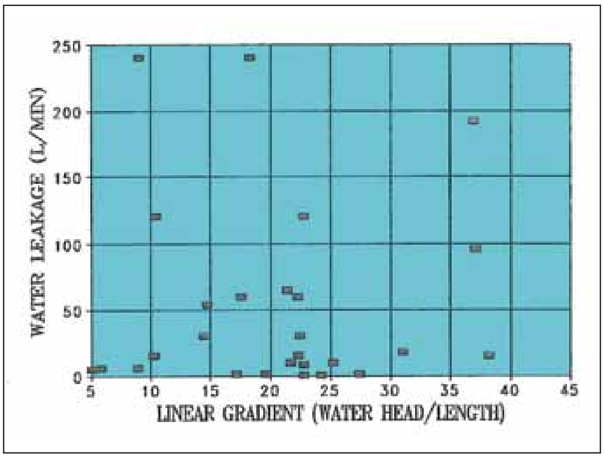

Figure 9: Leakage vs. linear hydraulic gradient.

To illustrate the latter, the Saurdal hydropower project access plug, with a steel lining of only 1.5 meter at a water head of 410 m (gradient 273) has a leakage of 15 l/min. In comparison, the Sildvik hydropower project plug has a leakage of about 240 l/min. at a gradient of 53 (water head 640 meter and 12 meter steel lining).

The leakage is best correlated to the year of construction. The modern plugs apparently are better sealed than the older ones. This is a consequence of the introduction of high pressure chemical grouting in plug construction.

The leakage changes with time. Detailed information is given from Saurdal, Tjodan and Tafjord. At Saurdal, the leakage was about 140 l/min. after the first filling. Additional grouting by polyurethane at a pressure of 6 MPa (410 m water head) through a curtain of drillholes from the downstream end about two weeks after filling reduced the leakage to about 15 l/min. Later on, the leakage decreased further by 60 to 70% within the next year.

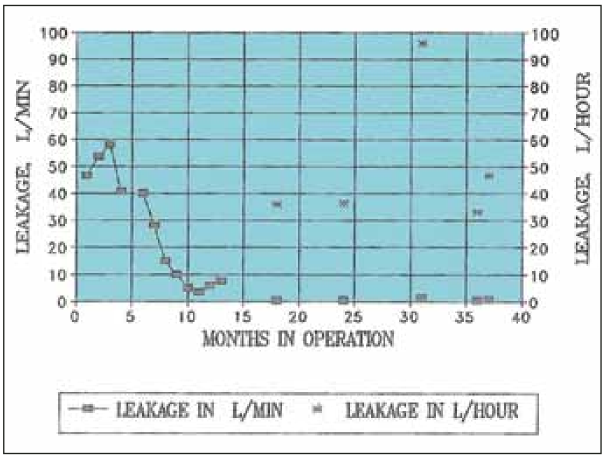

Figure 10: Leakages at Tjodan (880 meter water head) 1984-1987.

Even stronger reduction of the leakages occurred at Tjodan (Figure 10). No remedial grouting has been carried out. The initial leakage after the first water filling was about 50 l/min., which was reduced to about 5 l/min. during the first year of operation. In the next four years, the leakage decreased further, and was only one per cent of the initial leakage at the beginning of 1990. During the first seven years of operation, the pressure shaft was emptied twice. The owner believes that because of the emptying, suspensions with fine grained materials may have infiltrated the plug and caused the self sealing that have been observed.

CONCLUSIONS

Analysis and observations from the design, construction and operation of 150 high pressure concrete plugs in Norwegian hydropower projects have shown that the traditional design basis work well. For plugs located in tunnels with cross sections up to 50 m2 a total plug length between 2 and 5% of the static water head may be recommended.

The final leakage through the plug will to large extent depend on the quality of the concrete and the grouting work. Most of the leakage occurs along the rock to concrete contact zone and mainly in the roof section. The layout and design of the concrete and the steel lining will influence the plug behaviour and hence the extent of the grouting and construction costs.

In conclusion, the current design, construction and grouting technique of plugs for Norwegian hydropower plants have proven successful for operational pressures up to 100 Bar.

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(The present paper is re-written and condensed, but contains basically the same results as in above mentioned Hydropower 92 paper)

9.2 APPLICATION OF CONCRETE PLUGS IN THE OIL AND GAS INDUSTRY

Oddbjørn Aasen
Egil Ronæss
Ola Woldmo

1 BASIC DESIGN REQUIREMENTS FOR PLUGS

Cavern design operating conditions, pressure and temperature will set the conditions for the plug design.

Functional requirements:

- Separate stored product from the outside
- Ensure no/minimal inflow of water to the storage during operation.

Plugs between parallel storages sharing access tunnel:

- Ensure no leakage between stored products independent of level differences and operating pressures in parallel caverns.
- Leakages between parallel caverns storing different products may lead to off specification products.

Main plug:

- Strong enough to withstand maximum pressure difference (access tunnel totally filled with water and minimum operating pressure in the cavern

Design with or without access for equipment removal and/or inspection:

- Products to be stored at low temperatures (propane, propylene requiring cool-down of the caverns surrounding rock mass using a 2 stage cool-down,| may find it beneficial to be able to remove installations and make inspections during first phase of cool down using circulating air cooled from heat exchangers installed in the cavern.

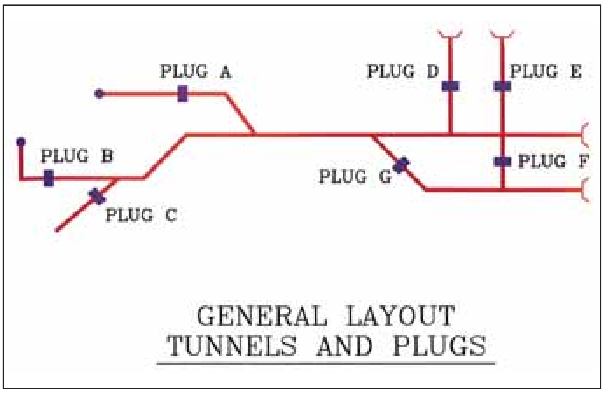

Figure 1 General layout tunnels and concrete plugs:

2 EXCAVATION AND ROCK SEALING / GROUTING

- Tunnel profile wedge shaped or not.
- Rock grouting before concreting of concrete plug.
- Deep curtain drilling and grouting in two steps, primary/secondary fans.
- Split space method, based on Lugeon test and grouting criterias.
- See example sketches, Phase 0. (Figure 2)
- Grouting with Rapid cement, afterwards Micro-cement.

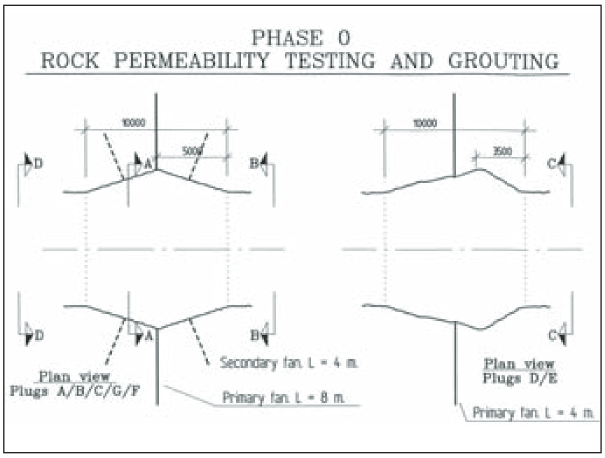

Figure 2 Rock grouting, phase 0:

3 CONCRETE PLUGS CASTING

Concrete plug casting

Before and during plug construction and casting preparations for grouting it is essential to get a proper plug sealing.

Temperature rise during concrete hardening:
- The concrete hardening process is an exothermic reaction – releasing heat.
- Theoretical temperature rise during the concrete hardening process is 40 to 45°C.
- The concrete mass is very large and surfaces to remove heat are small – consequently most of the released heat remains in the concrete.

Hardening period:
- Hardening period is defined as the time required to gain full strength
- Hardening time for the concrete is start temperature dependant (temperature of the delivered concrete), (1), (2) and (3).
- Hardening time for a start temperature of 15°C approximately 3 days, (1), (2) and (3).

Option to remove heat during hardening:
- Cooling during hardening is to reduce the maximum temperature in the plug and consequently reduce the gap between the plug and the surrounding rock when temperature is back to ambient temperature – then the plug shrinks.
- To reduce the maximum heat in the plug during the hardening cooling by circulation of a coolant (in Norway normally water will be selected as coolant). The coolant will be circulated in installed circulation pipes.
- For caverns to store liquefied gases far below 0°C the cooling tubes also will be used for active cooling during the cool-down process.

Casting of Access opening:
- Access opening will be coned towards the cavern.
- By installing a closure in the cavern end – this may be used as sealing barrier and inner formwork when the access is to be closed (by concrete casting).

- Steel pipes are installed in all depressions (=potential cavities) in the roof of the plug area, see figure no. 4 Phase I. The pipes will act as air evacuation pipes during concrete pouring operations.
- The concrete mixture should be designed to minimize the maximum temperature in the construction.
- Thoroughly compaction and recompaction of the concrete is required in order to minimize the plastic shrinkage, and to fill all openings within the formwork.

Termination of concrete pouring operation:
- After plastic shrinkage and curing of the concrete, cement mortar with expanding admixture is grouted through the pipes, resulting in a complete filling of all potential cavities in the plug area.

Pipes for cavity filling and air evacuation

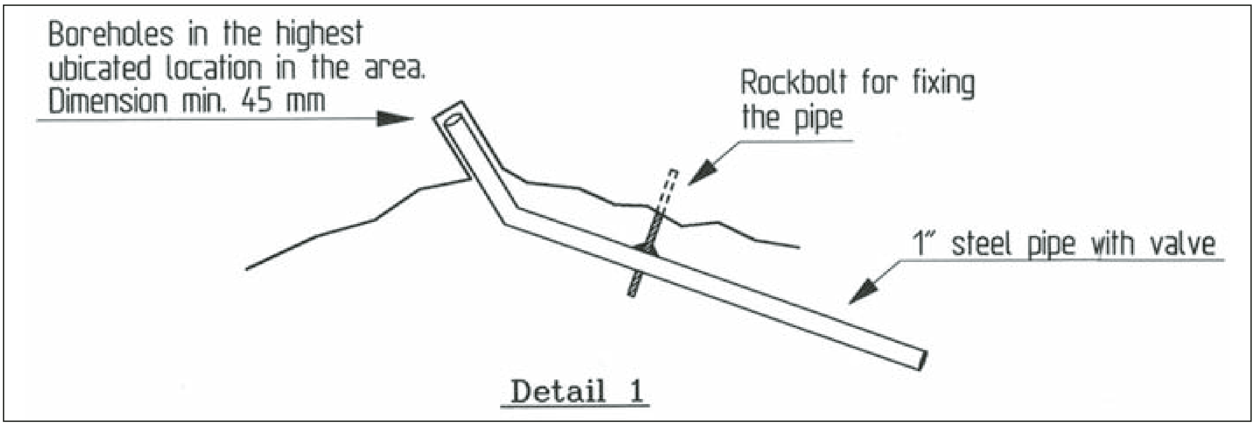

Figure 3 Cavity filling pipes

Contact grouting with cement:
- Contact grouting between rock and concrete could be executed by means of grouting tubes, see example in figure no. 4 phase I. These tubes will guarantee continuous contact with the rock surface in the complete cross section. It is used grouting tubes designed and applicable for cement grouting. Rapid cement and/or Microcement to be used, low pressure, 3-5 bars.

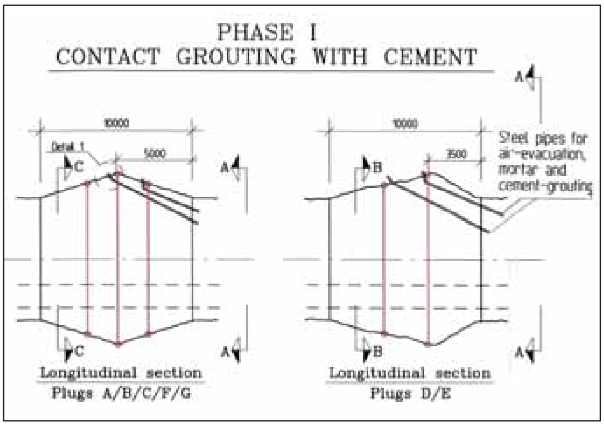

Figure 4 Rock grouting, phase I:

4 SEALING BETWEEN SURROUNDING ROCK AND PLUG AFTER HARDENING AND COOLING TO AMBIENT CONDITIONS

The plug will shrink during cooling to ambient temperature after hardening. This will result in gaps between the plug and the surrounding rock.

For caverns to store liquefied gases at temperatures far below 0oC the shrinkage will be considerably larger, and water leakage through the gaps will freeze inside the cavern reducing the storage volume capacity.

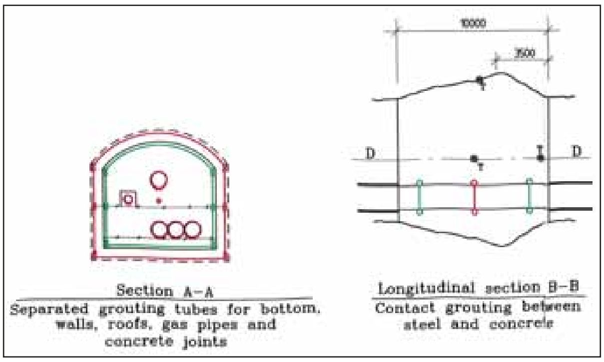

Figure 5 Sealing of pipes in the plug

Chemical contact grouting of tubes:
- The gaps are sealed with chemical contact grouting when permanent temperature in plug construction is reached.
- Grouting is executed by means of grouting tubes installed in plug area before pouring concrete.
- This yields all contact zones in the plug, i.e. between rock and concrete, concrete joints, steel-concrete surfaces, around pipes, etc., see figure no. 5 and 6 phase II: chemical grouting of tubes.
- As grouting materials, epoxy products with long potlife is applied in central parts of construction, polyurethane products in border sections, see figure no. 7.
- Maximum permissible grouting pressure is used.

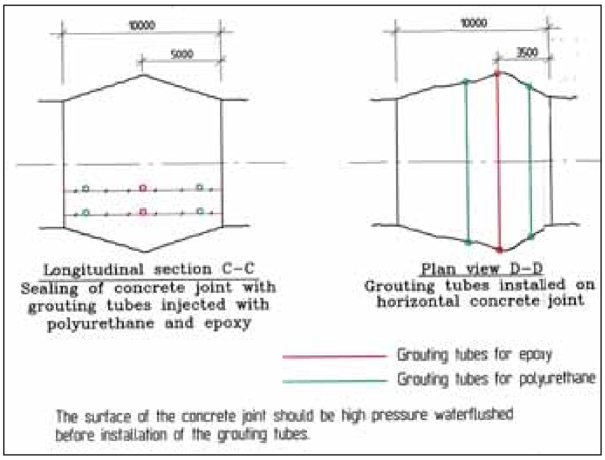

Figure 6 Principle of using polyurethan as a barrier for epoxy

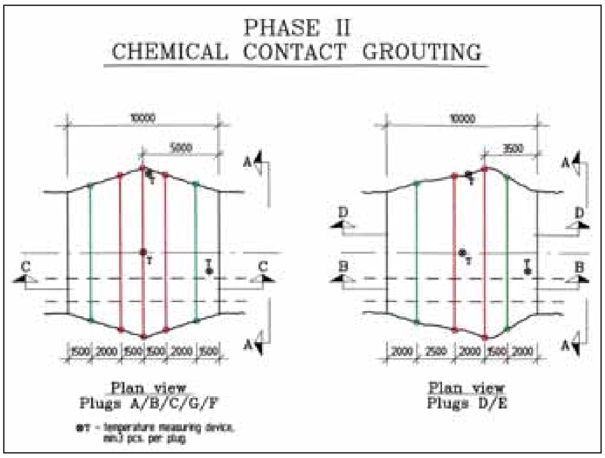

Figure 7 Chemical grouting, phase II:

General requirements – concrete and grouting works
- Detailed working procedures should be prepared for both the planning and execution of the concrete and grouting works.
- All grouting works should be executed and supervised by personal with documented experience from similar applications.

5 OPERATIONAL EXPERIENCE

1. Experience from 250,000 m3 storage in 2 parallel caverns storage for fully refrigerated propane at Kårstø Natural Gas Processing plant put in operation year 2000:

- Proper sealing and active cooling of the main plug is highly required for caverns to store liquefied gases as propane.

- To reduce required heat to freeze the outside of the plug storing liquefied gases well below 0oC – crushed rock / sand or Leca blocks should be filled outside the plug.
- Due to not proper sealing after casting of the plugs installed in the side access tunnels to separate the caverns for different qualities storage the leakage experienced are too large to isolate the storages from each other, quality wise.

- Experience from 60000 m3 refrigerated Propane cavern no. 2 at Mongstad Refinery, using 2 stage cool-down and a concrete plug with access opening. The Plug was cast with pipes for coolant circulation.

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10.1 CAVERNS, OPERATIONAL EXPERIENCE

Egil Ronæss

1 INTRODUCTION

Numerous petroleum products are stored in caverns, some in heated caverns compared with normal ambient conditions and other under refrigerated conditions.

Products correctly stored in caverns are:
• Environment friendly
• Products are safely stored
• At storage temperatures above 0oC stored products is exposed to ground water
• Stabile ambient conditions through the whole year.

Products stored in caverns are often for strategic storage, protected from land or air based attacks.

For large storage volumes (above 30 000 – 50 000 m3 storage volume) cavern storages are cheaper per m3 storage volume than similar storage volume in tanks.

Large cavern storage volumes opens for favourable market prices by:
• Seller: Sell at high price and not at the moment when you have to sell because of full storage.
• Buyer: Buy large volumes in season with low market prices and not frequent buying due to empty storage – independent of market price.

1.1 Products Stored in Caverns

Known petroleum products stored in caverns in Norway are:
• Crude Oil, normally large caverns at atmospheric conditions.
• Refinery distillation fractions and refinery complete products:
• Fuel oils
• Diesel oil
• Gasoline fractions
• Naphtha
• Condensates
• Butane – slightly pressurised to avoid hydrate forming conditions
• Propane at atmospheric conditions (-42^{o}C) or slightly pressurised (-30 to -38^{o} C) and pressurised (above hydrate forming condition – above 6^{o}C)

1.2 Operating Conditions

For product stored at conditions below normal boiling point the atmosphere above the stored product will consist of a harmless inert gas saturated with vapour from the stored product.

For products stored at “boiling” conditions the atmosphere above the stored product will only consist of gaseous components from the stored product. Inert gas will only be used in situations to avoid below design pressure conditions – to avoid air leakage into storage

1.3 Load in Facilities and other piping inside the Cavern

Bottom Load-in pipe terminated near far end to ensure circulation of the stored product. It is extremely important to avoid surging in the load in line by selecting the pipe diameter in the vertical section so large that it always will be two-phase flow (gas and load-in product). Dimension shall be based on maximum load-in volume.
Load in facilities for liquids, with storing temperature below boiling point, with lower explosion level (LEL) value below 6oC should only be loaded into the cavern via the bottom load in line to avoid sparks due to static electricity.

Top Load-in pipe, ending in the far end. The pipe is installed on a rack from cavern roof sparged (with holes downwards) to ensure product distribution. Main task Top Load in Line is for loading in warmer product than the one already stored to avoid roll-over.

Pressure control shaft is terminated in the cavern roof. This shaft will supply gas to maintain defined minimum operating pressure and release gas (to flare) when exceeding maximum operating pressure.

Leakage Water Pump shafts (pipes) will host the leakage water pump-out pipe.

Stored Product Export Pumps shafts (pipes) will host the stored product export pipes or be the export pipe.

Chemical Injection Pipe shaft to be able to supply chemicals preventing emulsion in the interphase between stored product and leakage water and for hydrate braking (optional)

1.4 Pump-Out Facilities

Pumps are normally installed in dedicated caissons in a common pump pit housing leakage water pump-out pump(s) and export pumps. Leakage water pump(s) ensures via level control a ground water level to avoid ground water into the product export pumps and to avoid stored product into the ground water pump(s) – through level interphase (water – hydrocarbons) measurement and control.

Leakage Water Pumps with drivers (el.motor or hydraulic) normally installed in shafts (pipe allowing the pump installation inside). The installation shafts in pump sump, well into the water phase. Normal is 2 pumps (1 spare). Automatic Start / Stop based on given interface levels between water and stored product. Leakage water pumps capacity must exceed the normal leakage volume with a large margin to cover for max leakage water volumes possible.

Stored Product Export Pumps with drivers (el.motor or hydraulic) installed in shafts / caissons (pipe allowing the pump installation and removal from the cavern top). The shafts in pump sump, ends well above the water phase. Number of pumps: minimum2 pumps and required number of pumps shall meet the maximum load out volume. Due to allowed installation level below the cavern floor due to the pump pit almost the full cavern storage volume is active. Export pumps must be designed for all actual operating conditions like the cavern pressure and product levels. Mini-flow system is required.

Submerged pumps installed in casings from surface are the normal way.

1.5 Instrumentation

Instrumentation related to Storage Caverns is normally related to level, pressure, temperature and flow.

Level Measurements will be for monitoring:
- Interphase level between Water and product (to avoid water exported with the stored product and stored product pumped out with the leakage water.)
- Level of stored product (interphase between product and gas phase) to give operating information on stored volume – to give information on volumes available for export or import.

Flow measurements are installed above ground for monitoring:
- Correct export volumes.
- Pumps mini-flow control.

Pressure measurements are installed above ground and will be for monitoring of:
- Pressure control inside the cavern – (releases to flare (or safe location))
- Operating pressure above the stored product, (avoiding pressure above design pressure and for pressure below approved operating pressure to avoid air into the storage)
- Monitor export pressure (and temperature) of stored “boiling” products.

Caverns operating with variable pressure from pressures below to pressures well above normal operating pressures (storage with variable pressure gas pillow) will require vacuum and overpressure valves (protection) setting the operating pressure limitations. This operating philosophy results in minimum requirement to flare off gas volumes from the storage as well as use of required inert gas to maintain the requireds operating conditions.

Temperature measurements will be for monitoring of:
- Cavern rock temperature.
- Stored product temperatures at different levels in the cavern (roll-over prevention and for certain product also monitoring operating temperatures to be well above hydrate formation).

1.6 Leakage water

Pre-injection into the rock massive prior to start up of blasting to ensure acceptable ground water leakage into the cavern is the basis for successful operation. Ground water inflow will vary with ground water level and cavern operating pressure.

Ground water leakage into the cavern requires collection and pump-out facilities to avoid ground water in-mixing into stored product. Ground water is normally collected in the pump pit and removed by ground water pumps. Leakage water pump capacity must by far exceed the leakage water inflow.

The contact between ground water and stored products must avoid emulsifying conditions.

The contact between ground water and stored products will bring impurities into the ground water and also small droplets of stored product.

Leakage water cleaning of pumped out ground water is a requirement to maintain acceptable environment.

2 TESTING WHEN STILL ACCESS

Ground Water leakage incl. test of ground water pump capacity is important to test prior to sealing off access and possibilities to modify. In cases with leakage water ingress larger than leakage water pump acceptable operation, it may be necessary to perform injection sealing of water channels. Leakage water inflow will vary with pressure inside the cavern and the ground water level – varying with precipitation conditions.

Perform tests of installed instruments and qualification of measurement accuracy. All instruments with no access after put in operation must have the final tests (example: surrounding wall temperature measurements, water level in pump pit).

Test installation and withdrawal of export pumps are highly recommended.

2.1 Cool Down for Caverns operating well below 0°C – Two (2) stage cool-down, Stage 1 - Air Cooling Stage.
Refrigerated caverns may be cooled down in 2 principally different methods:

1. One-stage cool-down using the stored product's heat of evaporation as coolant using nozzles to create small liquid droplets to give efficient evaporation.
2. Two-stage cool-down:

- First stage using circulating cooled air
- Second stage using product to be stored and similar principle as for one-stage cool-down.

The two-stage cool-down - stage one – normally uses circulating air inside the cavern. The circulating air is normally cooled using brine circulated from a convenient location on ground level. The circulating brine is cooled by a refrigeration cycle. The air coolers require de-riming in intervals to maintain average heat transfer efficiency.

The air-cooling MUST also include active main plug cooling.

3 COMMISSIONING

Commissioning is normally defined as testing and control of all vital functions prior to introduction of the product to be stored. Many of the commissioning activities take place after the main plug has been sealed off.

Some tests may be completed after the product has been introduced into the cavern.

Level measurement of the water level may be completed. Control of product level measurement at levels below normal operating levels may be checked against the ground water level measurement. Finalisation of product level measurement will be performed during introduction of product into the cavern.

Pump pit water level control / pump-out start and stop of pump as function of level.

Pressure and leak testing is normally related to seal tightness of the main plug and riser shaft(s) at design pressure conditions (ground water pressure surrounding the cavern shall at all cavern rock surfaces be higher than pressure experienced inside the cavern).

Oxygen must be removed by inert gas prior to introduction of hydrocarbons to avoid any danger for explosion inside the cavern and in the pressure control / flare system outside the cavern. Nitrogen (N_2) -normally or CO_2 are used for inertising the cavern. Inertising is understood as bringing the oxygen content in the cavern to an acceptable level:

- The inertising of an air filled cavern depends on the possibility to avoid intensive mixing of the inert gas and the air inside the cavern. Required inert gas volume varies between 3 and 4 times the cavern volume. Distance between inert gas inlet and gas outlet from the cavern, through pressure control route is vital to minimise the required inert gas volume.
- The inertising may be performed during the leak test and using the cavern 80 to 90 % water filled to reduce the volume of inert gas required to establish acceptable oxygen level in the cavern, and then by water pump-out replace water with inert gas. (This will be based on cost of water + cost for water pump-out against reduced cost of inert gas.)

Purging is understood as change-out of inert gas with vapours of stored product and is only fully relevant for products stored at boiling condition (vapour pressure equals storage pressure).

Installation of export pumps.

3.1 Caverns Operating at ambient or higher temperatures
No special commissioning activities in addition to the activities already described.

3.2 Commissioning including Cool Down for Caverns operating well below 0°C
Air must be removed by inert gas prior to introduction of hydrocarbons to avoid any danger for explosion. Prior to introduction of liquid hydrocarbons into the cavern for cool-down purposes the atmosphere must be changed to stored product vapour. To avoid local temperatures far below stored product temperature.

During the early phase of the cool-down process leakage water inflow will continue even after average wall temperature is below 0°C, partly due to the water flow and calorific value required to freeze water.

During cool-down water ending in the pump pit must be removed to ensure a free board between frozen water level and product export pump suction level. Pump-ability of water entering the pump pit is provided by addition of methanol to the pump pit. Methanol addition is related to pump pit temperature to ensure pump-ability of liquid content.

3.2.1 Cavern – ONE stage Cool-Down Using Medium to be stored as Coolant
Inertising and change-out of inert atmosphere to the stored product atmosphere in the cavern, see principles for inertising given in section 3.

Cool-down starts when water level is under control in the pump pit and the cavern atmosphere is changed to cool-down medium atmosphere. Cool-down medium is sprayed trough atomising nozzles installed from headers suspended from the cavern roof.

Active cooling of the main plug is extremely important to avoid water leakages to be ongoing after water leakages else in the cavern has stopped. The water leakage trough the plug – rock interface e will pile up in the cavern reducing the bulk storage capacity in the cavern.

The injected fluid vaporisation creates the cooling. The vaporised fluid are withdrawn from the cavern and compressed, dried (to remove water vapour from the gas), liquefied and re-injected through the nozzles.

It is important that the injected liquid in the piping to the nozzles has a temperature above the freezing point to avoid ice build-up on the piping to the nozzles. Two cases are known using sub-zero temperature in injection liquid where the spray system piping have fallen down and the injection system not possible to use. Back-up spray system has made cool-down completion possible.

3.2.2 Cavern – Two stage Cool Down, Second Stage Using Medium to be stored as Coolant
Main plug must be sealed prior to inertising and change to hydrocarbon atmosphere.

Inertising the atmosphere in the cavern and Change from inert gas to hydrocarbon phase atmosphere, see text under chapter 3 and Cool-down using medium to be stores as coolant, see chapter 3.2.1.

3.3 Above Ground Facilities
Pressure control valves and piping for release to flare alternative atmosphere or gas supply to cavern to maintain operating pressure.

Product import pipeline connected to cavern load-in lines with valves (bottom or top load in).

Product Export manifold with valves connection to each export pump and Export pipeline.

Product Export Pumps mini-flow controllers pipe and valves connected to cavern load-in line.

Leakage Water line with valves supplied from Leakage Water Pump(s) for transfer of leakage water to effluent treatment plant.

Facilities for safe lifting (normally in a dedicated shaft pipe for each pump) of pumps with drivers, cabling and piping from the cavern. This also includes safe way to handle the cable(s).

Facilities for withdrawal of level measuring facilities from the cavern for maintenance and inspection.

Utilities as Inert gas, Instrument and Plant air, water and chemicals shall be available on the cavern top.

Instrumentation Junction boxes for all instruments inside the cavern.

Above ground facilities for Water curtain including water supply and ground water pressure monitoring (normally not located on the cavern top).

4 START OF OPERATION
Normal operation of the cavern starts with first load in of product into the cavern.

4.1 Caverns Operating at ambient or higher temperatures
Water level control in pump pit has been established and in normal operation for a period ensuring proper design.

Mode of operation with respect to use of inert gas and flared volume or use of storage on water bed and no atmosphere above stored product must have been selected during engineering. The following pros and cons are relevant when selecting principle of operation:
- Cavern operation with atmosphere above product and constant operating pressure:
  - When product pump-out liquid must be replaced by inert gas
  - When filling gas must be released to flare / atmosphere
- Cavern operation with atmosphere above product and variable operating pressure:
  - When product pump-out pressure reduces – inert gas replacement not necessary within accepted pressure levels.
  - When filling pressure rises without any release to flare within accepted pressure levels
  - Reduces inert gas consumption and products vapour losses to flare / production of greenhouse gases.
- Cavern operation with product on a water bed and NO atmosphere above product:
  - Export pump located above water level in cavern (high up in the cavern.

- All supply of product requires water pump-out – preferably to buffer storage (to reduce water flow to effluent treatment.
- All pump-out of product are replaced with water.
- Ground water level control by total level above cavern roof top.

4.2 Caverns operating above 0oC and hydrate formation conditions

Products stored at temperature above hydrate formation conditions at product boiling point at temperatures below ambient temperature leading to product boil-off.

Boil-off can be recovered by compression, condensation and re-injection into the cavern or used as fuel to fired energy consumers.

This storage method is applied for:
• Propane at the Rafnes Noretyl Ethylene Cracker, start-up year 1977.
• Butanes at Mongstad refinery, Vestprosess Project (start-up year 2000)

4.3 Caverns operating well below 0°C

During cool down some of the leakage water will freeze and create an ice layer on the rock surface. From this ice layer some ice will sublimate into the caverns atmosphere. If the cavern has vapour recovery facilities with recycling of recovered product to the cavern the recovered product must be dried prior to cool-down.

During the cool-down process inflow of leakage water stops due to sub-zero conditions in the surrounding rock massive. This seals the surrounding rock.

Products stored at temperatures near normal boiling point (1 atmosphere abs.) at temperatures far below ambient temperature leading to product boil-off.

Boil-off can be recovered by compression, condensation and re-injection into the cavern or used as fuel to fired energy consumers.

This method is applied for Propane storage at:
• Kårstø natural gas and condensate processing plant, start up year 2000 – 1 stage cool down. Problems with water leakage between concrete plug and surrounding rock. This leakage lasted until long after “completed” cool down. Reason:
  - Main plug cool-down due to inadequate cooling intensity of the plug region.
  - Lack of adequate sealing injection between concrete plug and surrounding rock.
• Mongstad refinery:
  - Cavern no. 1 put in operation year 2000. Reduced storage capacity due to frozen ground water intrusion during cool down – 1-stage cool down.
  - Cavern no. 2 put in operation year 2003 – 2 stage cool down. Full storage capacity maintained.

5 OPERATION

Load in from adjacent production facilities for export or from unloading ships for storage and distribution:
• Load in from production - relatively small load in flows and high export flows
• Load in from unloading ships – high load i9n rates and smaller export rates.

Pressure control depends on operating philosophy as basis for mode of operation, see chapter 4.1.
• Minimising use of seal gas and vapour release to flare / atmosphere (minimise component losses).
• Constant vapour / gas pressure above stored product – gas flow in and out depend on product level changes in the cavern.
• Vapour / boil-off gas recovery when products are stored at boiling conditions.
• Caverns operating with variable operating pressure requires very low consumption of inert gas and releases to atmosphere / flare.

5.1 Caverns Operating at ambient or higher temperatures

Ground water level control must be working in pump pit to avoid in-mix of water into stored product and stored product into leakage water pump-out.

Modes of operation for pressure and inventory control, see chapter 4.1.

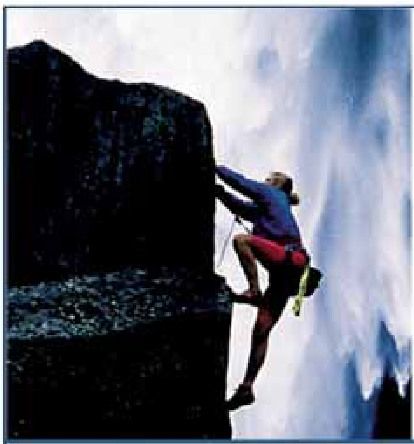
Leakage water pumped out from the cavern contains dissolved and occasionally also a water - hydrocarbon emulsion. This pump-out requires a cleaning (normally a biological cleaning process) prior to release as approved effluent from the storage facilities

5.2 Caverns operating well below 0oC

Roll-over prevention:
• Only fully refrigerated product to be loaded into the cavern bottom load-in line.
• Not fully refrigerated product should be loaded into the cavern via a roof suspended line with limited size drain holes (sparged) to allow the product cool down to storage condition by a flash.

To maintain stable product storage temperatures:
• Fully refrigerated product load-in vie bottom load-in line.
• Maintain stable storage pressure.

Vapour recovery facilities and optional drying facilities must be in operation to avoid operating problems. Operating problems created by ice / hydrate formation can be removed by methanol injection just upstream the problem points.



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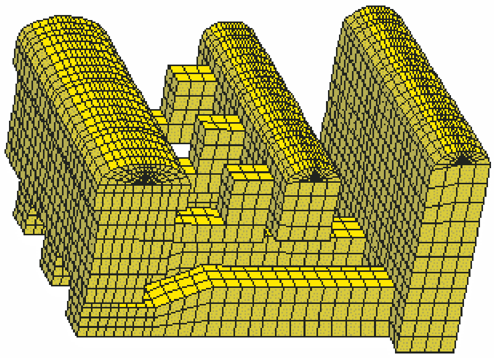

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10.2 COOLING WATER TUNNELS, OPERATIONAL EXPERIENCE

Egil Ronæss

1 INTRODUCTION

Sea water system design including capacity must include minimum tidal sea level and maximum sea water flow influence on intake basin water level. The minimum water level in the intake basin are influence by sea water level, pressure drop in the inflow tunnel (pressure drop is directly influenced by flow (consumption)) and required suction head for the circulation pumps (NPSH).

Pressure drop in the tunnel is proportional to the square value of the flow.

Tunnel transport capacity – important design / construction parameters, rock surface roughness and variable cross-section areas have clear impact on the tunnel water transport capacity.

Installation level of the circulation pumps, circulation pumps NPSH and intake tunnel entry level into the sea water intake basin sets the capacity limit. Do not save money on sea water intake basin bottom level – your capacity requirement may be increased and the cost of extension are more expensive without a reasonable-margin.

<u>Herøya cooling water tunnel from lake Nordsjø</u> put on-stream in the late 1960-ies (sweet water), once through system. Tunnel operation has since start in 1977 been very successful

<u>Rafnes cooling water tunnel from lake Nordsjø</u> put on-stream 1977 after a delay due to clay blocking by water softened and not covered clay zone in 1976 (sweet water), once through system. Tunnel operation has since start been very successful.

<u>Kårstø Statpipe cooling water tunnel in the fjord basin right South of the Kårstø plant</u> length approximately 600 meters, intake depth 25 – 30 metres was put on-stream (sea water) 1984. Operation terminated 1999/2000 by closing the sub-sea tunnel opening. The reason for termination was severe biological fouling and access to a new cooling water supply tunnel.

<u>Kårstø KUP (Åsgard) project cooling water tunnel into the fjord East of the Kårstø plant</u> length approximately 2.5 km, intake depth 75 – 80 metres was put on-stream 1999 / 2000 (sea water). Commissioning and operation experience so far is very good.

Not solid state rocky zones tunnel crossings (clay zones etc.) must be properly reinforced during the construction period.

10.3 SHORE APPROACH TUNNELS and FJORD CROSSING TUNNELS, OPERATIONAL EXPERIENCE

Egil Ronæss

1 INTRODUCTION

For use of bringing Pipelines onshore or offshore under rough conditions and rocky shore approaches.

The shore approach tunnels are normally sea water filled, resulting in problem free operation.

2 SHORE APPROACH TUNNELS

Concrete landfall tunnels was used for the Statpipe in and outgoing pipelines put in operation 1985 and still in operation. The landfall site experiences very rough conditions. There have been some structural problems during the period in operation, these have been successfully repaired.

Blasted tunnels from onshore rocky landscape to selected rocky structures offshore.

Sleipner Condensate pipeline project landfall rock blasted tunnel put on-stream 1993 at Kalstø on Karmøy island was a successful project. The tunnel was constructed with a short side tunnel branch for problem free construction of an optional side tunnel. This tunnel access start has been used for the "Åsgard Transport" pipeline project a 42" diameter pipeline put in operation year 2000.

3 FJORD CROSSING TUNNELS

Sub-sea tunnels to avoid any interference with ship traffic and ships anchoring, as well as possible difficult pipe-laying.

"Dry" tunnels allows frequent visual inspections for follow-up. The "Dry mode" requires water pump-out facilities at low points.

Rafnes to Herøya tunnel put on-stream 1977 for transfer of Chlorine, Vinyl-chloride and liquid mixture of Ethane, Propane / Propylene containing Butane as fuel and feedstock for the Ammonia plant N-II. This has been a water filled tunnel and has been successfully in operation since start-up. Modifications and eventual repairs will require pump-out for access.

Statpipe put on-stream 1985 has a concrete cast landfall tunnel at Kalstø, West side of Karmøy island. Between Karmøy landfall and Kårstø Gas Processing facilities there are 3 fjords where the Statpipe incoming "Rich Gas" pipeline, the outgoing "Sales Gas" pipeline and signal and communication cables are routed through 3 sub-sea fjord crossing tunnels. The tunnels have been "dry" – leakage water has been pumped out to sea via drain pipe from pump house site onshore. From 1993 the fjord crossing tunnels also hosted the Sleipner Condensate pipeline, for un-stabilised condensate, to Kårstø for processing to commercial products. The operational experience is very good.

10.4 EKEBERG PETROLEUM STORAGE FACILITY EXPERIENCE FROM THE EKEBERG OIL STORAGE AND EKEBERG TANK

Asbjørn Føsker

ABSTRACT:
After the Second World War the Norwegian Authorities started considering a safe storage system for fuel in the Oslo area. The rock mass in 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 still after 37 years of operation one of the main oil storage facilities for refined oil products in Norway. The experience after all these years of operation is remarkable. Only a few stops in operations and no major accidents confirm the efficiency and safety of the facilities.

INTRODUCTION

The main oil storage facility for refined products in Norway is located in rock under the Ekeberg hill in Southern Oslo in the harbour 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. In 1995, approximately 50% of the total annual demand for petroleum products in Norway passes through the caverns.

Figure 1: Aerial photo of Sjursøya Oil Terminal and the Ekeberg hill where the Ekeberg Oil Storage and the Ekeberg Tank are located (Photo: Fjellanger Widerøe AS, 2003)

Figure 2: The jet fuel train on its way to Oslo Airport from Ekeberg Tank and Sjursøya Terminal (Photo: Rune Fossum - Norwegian National Rail Administration)

Every day a direct train connection from Sjursøya to Oslo Airport serves the airport with 1 million litres of jet fuel.

For strategic reasons the detailed layout and information on total storage capacities are not available to the public.

HISTORY

Storage of petroleum products in Oslo can be traced back to 1898 when a tank farm was established by a local petroleum distributor, Østlandske Petroleumscompagnie, at Steilene (see Figure 3). Later the American oil giant Standard Oil and Esso Norge A/S took over the business. When the facility was moved to Sjursøya peninsula (closer to Oslo) in 1957, the tank farm at Steilene was totalling 23 oil tanks holding 70 million litres.

The terminal and tank farm at Sjursøya was developed in several steps from 1936 until the mid 1960's when capacity constraints and safety requirements for storage of petroleum products in the heart of Oslo City forced the facility to go underground.

Shortly after the Second World War the Norwegian Authorities had 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.

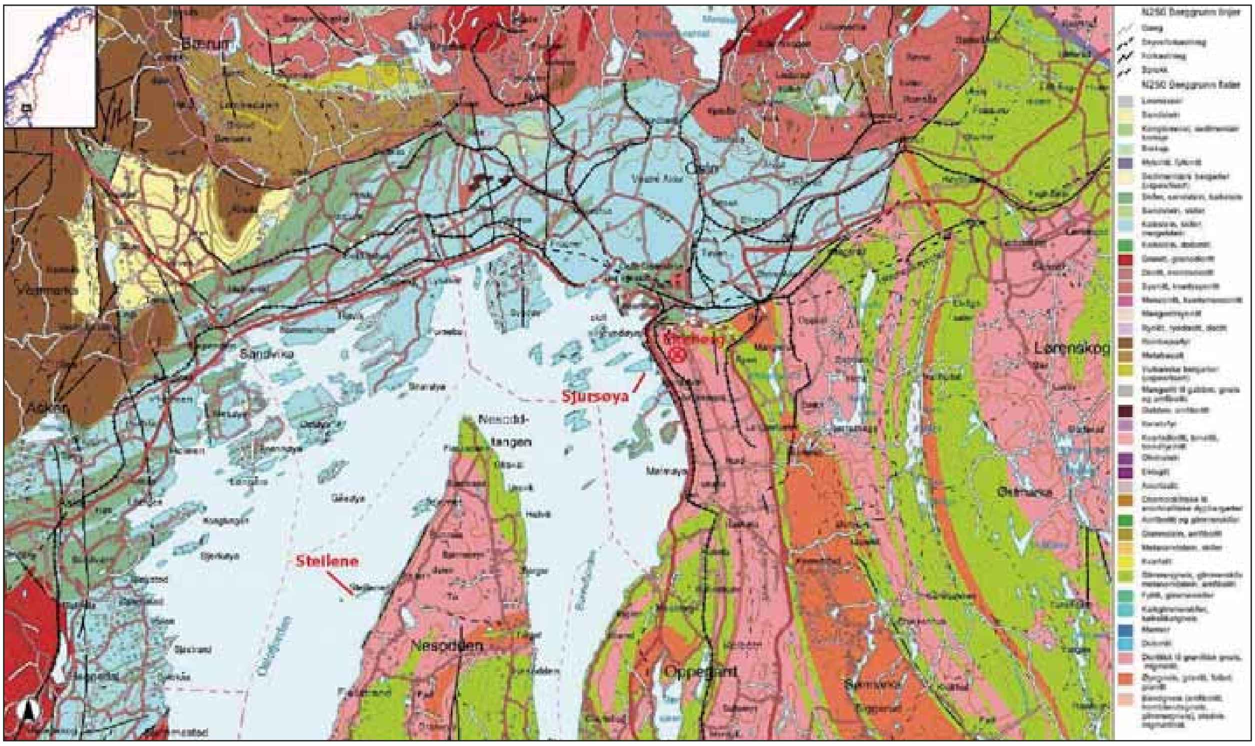

Figure 3: Geological Map of Oslo and inner part of the Oslo fjord (Kart: NGU)

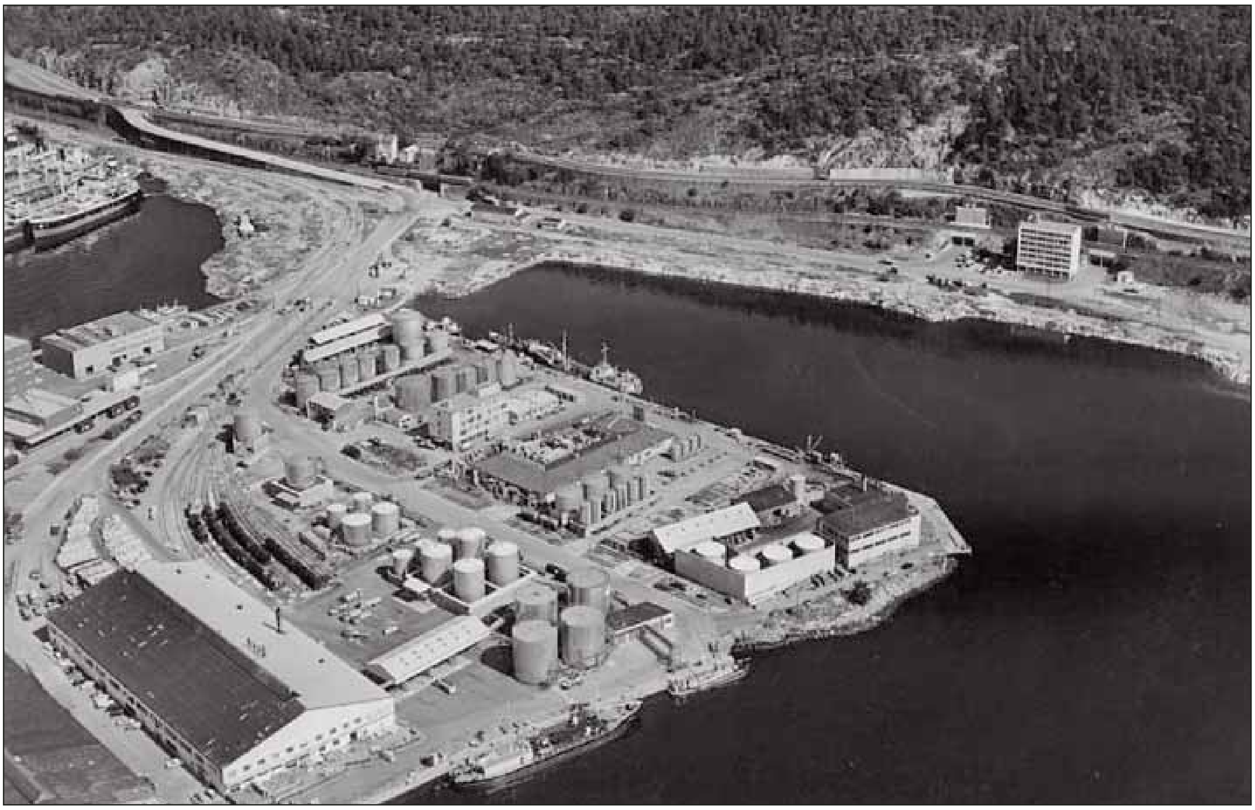

Figure 4: Aerial photo from 1968 of Sjursøya terminal with tunnel portals to the underground facility during the construction period for Ekeberg Oil Storage. (Photo: Widerøes Flyveselskap/Rolf Ingelsrud)

Similar storage facilities had already been developed in Sweden. By locating the storage close to the oil terminal at Sjursøya it was found that 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 split 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, Ekeberg Tank, was added in 1975-78. The construction costs were NOK 42 million and NOK 74 million, respectively.

The facility was built in joint partnership by A/S Mobil Oil Norge, AS Norske Texaco, Esso Norge AS and A/S Norsk Brændselsolje, the State Civil Defence organisation for fuel supply. Part of the stored petroleum is reserved for that purpose

Both facilities consist of a series of excavated unlined rock 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 is different, as shown in Figure 5 and 6 and further explained below.

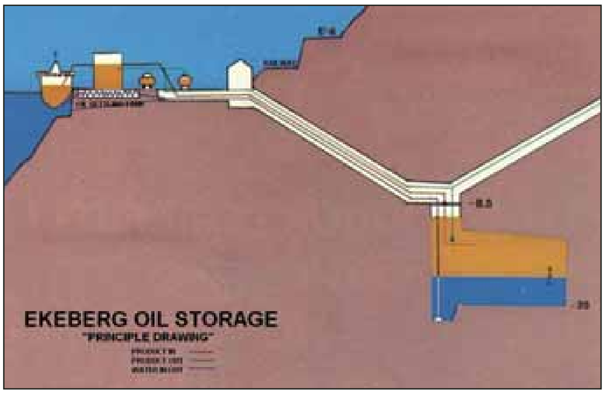

Figure 5: Storage principle Ekeberg Oil Storage (Illustration: Ekeberg Oil Storage ANS/Asbjørn Føsker)

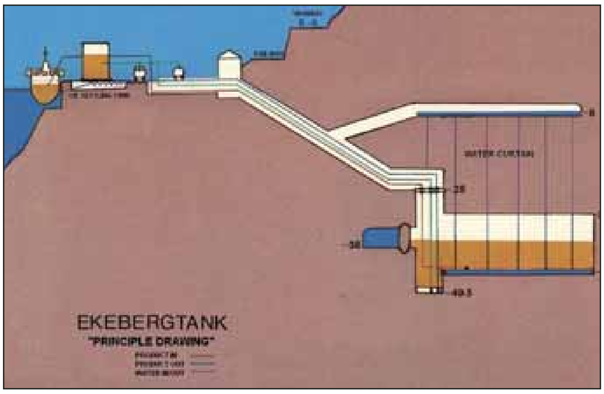

Figure 6: Storage principle Ekeberg Tank (Illustration: Ekeberg Oil Storage ANS/Asbjørn Føsker)

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. Petroleum is stored with 10 meters hydrostatic under-pressure relative to sea. The products float on a bed of sea water, which is pumped in or out from the fjord as the amount of oil varies, thereby maintaining the under-pressure.

The caverns are always filled to the top, with only a small petroleum surface area exposed to air. 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.

When receiving oil products, water is pumped out from the bottom of the cavern to avoid oil pollution in the fjord. As a safety measure this water passes through an oil separator before being discharged into the sea.

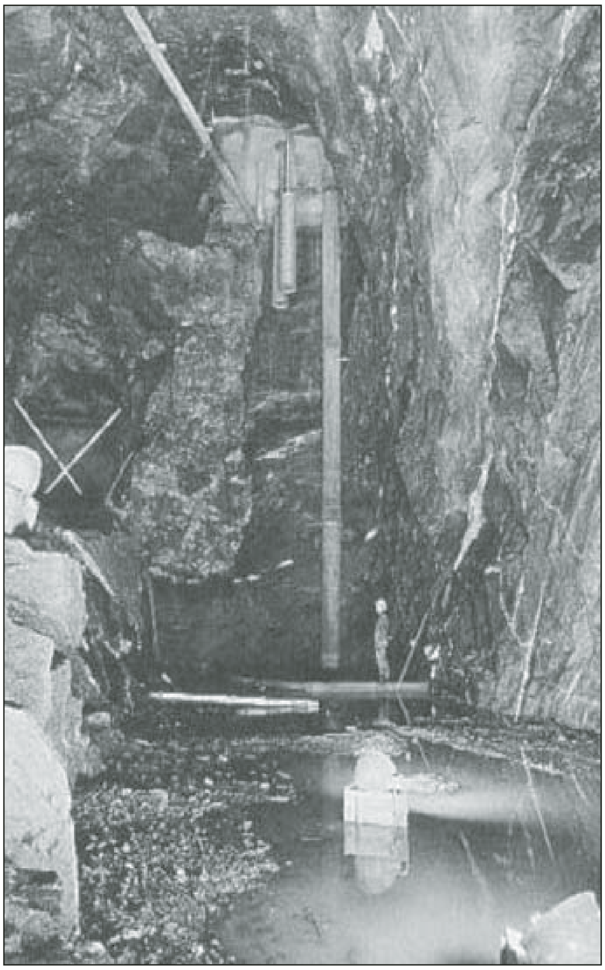

Figure 7: Storage cavern at Ekeberg oil storage. The import pipeline is coming in at the top, the product pumps are hanging at the top. The water pipeline goes all the way down to the bottom. The picture is taken during construction, before the cavern was filled with water and gasoline.
(Photo: Ekeberg Oil Storage ANS)

In Ekeberg Tank 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 Ekeberg Tank is used to store aviation fuel and gas-oils which are far less volatile than automotive gasoline.

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

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 Ekeberg Tank.

Groundwater leaking into the caverns of Ekeberg Tank is collected and pumped to the harbour basin through an oil separator.

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 the rock mass stability is maintained by bolts and shotcrete only. None of the facilities required concrete lining.

Over the 37 years of experience only one small piece of rock has been recorded falling from the roof in one of the caverns. Rock mass stability problems have not been observed in any other part of the facility.

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 within the facility.

All operations are highly automated. The control centre is located underground, from where the entire facility is 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.

EXPERIENCES FROM OPERATION

The first receipt of petroleum was on April 29, 1970. No difficulties were experienced that day and over the next 37 years, there has hardly been any stop in the operations and no major accidents.

The storage complex requires few people during operation, and the operating costs are very low.

These operational experiences confirm the efficiency and safety of the facilities. This is remarkable as tankers arrive almost every day, pending between 260 to 350 ships per year, and products are pumped out from the caverns continuously.

Corrosion in pipes and pumps has caused some problems. This has been experienced in product pipes as well as in sea water pipes. However this has been overcome by replacing equipment as required.

The equipment is to a large extent similar to that in ships. The Ekeberg Petroleum Storage Facility is a lot older than the normal life duration of ships. At the time of original design, experience with such facilities was limited. Hence equipment of less than the best quality was chosen for economical reasons, which over the long run has proved not to have been an optimum choice. However, the facility has served its owner's well for over almost four decades.

Delegations from many parts of the world have visited the facility.

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10.5 MONGSTAD OIL AND GAS FACILITY - OPERATION AND MAINTENANCE OF ROCK CAVERN STORAGE - LESSONS LEARNED

Jan Ulvøy
Arild Neby

ABSTRACT:
The oil and gas facility located at Mongstad near Bergen in western Norway is operated by Statoil and comprises Norway's largest oil refinery, a crude oil terminal and a gas processing plant. The oldest part of the refinery was built in the late 1970's. The sub-terrain of the facility area is characterised by its efficient utilisation of the underground space for rock cavern storage facilities. The concept of underground storage in rock caverns has proved superior to surface storage and has enabled the ever growing development at Mongstad. At the end of 2005 the facility had 27 hydrocarbon storage rock caverns in operation and one new cavern under construction.

INTRODUCTION

This article is based on Statoil's presentation of operational experiences at Mongstad held at the Rock Cavern Storage Seminar arranged by Norconsult in December 2005. The article summarises lessons learned after three decades of operation.

Figure 1: Aerial photo of Mongstad Oil and Gas Facility (Photo: Statoil)

Storage Capacity

The concept of underground storage in rock caverns has proved efficient and has enabled the steady extension of the facility. Statoil Mongstad had 27 rock caverns for storage of hydrocarbons in operation in December 2005.

Storage cavern number 28 - the New Naphtha Cavern (NNC) was then under construction with expected completion date in November 2006.

Requirements for Hydrocarbon Storage in Rock Caverns

The basis for safe and good operation resides in:

- Public rules and regulations
- Process conditions
- Operation and maintenance programme
- Appropriate choice of equipment
- Allowing equipment to operate as intended (as believed or known to operate if one has experience)
- Awareness of what can be done and where in the expected service life of the storage cavern
- Choice of simple, solid and robust solutions
- Choice of technical solutions that allow future measures to deal with corrosion and to upgrade the plant.

Input during Conceptual Design Phase to Construction Phase

- The people who operate and maintain rock cavern storages have a wealth of experience. But will those who design and construct the caverns take their experience into account?
- It is during the design phase and early in the construction phase that the principles are developed and the strategic choices made, thus creating the conditions for operation and maintenance.
- The planning and construction time of a storage cavern is from 2 to 3 years. The operation and maintenance period is from 0 to 100 years. Do we give this sufficient consideration?
- Experience has shown that early in the projects we shift our attention from general principles and instead become embroiled in details which may often be difficult to abandon even if it is later found that they were ill-advised.

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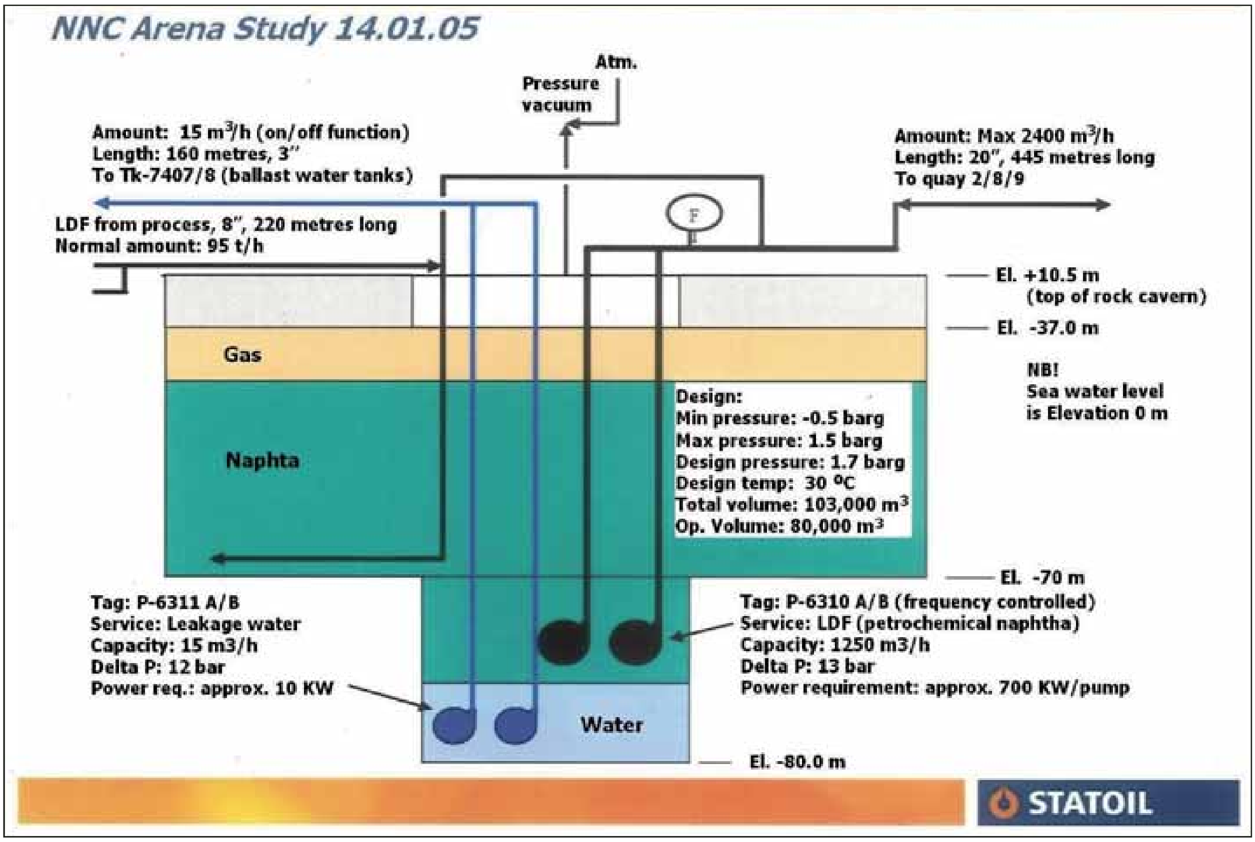


Figure 3: Operational flow chart for the detail design phase - New Naphtha Cavern Arena (Illustration: Statoil)

Figure 4: Example of utilisation of 3D design in the planning and detailing of a Cavern Top (Illustration: Statoil)

11. SUBSEA TUNNELLING FOR OIL AND GAS - CONCEPT STUDIES

Arnulf M. Hansen
Jan K.G. Rohde

INTRODUCTION

The Norwegian oil company, Statoil has through the years placed considerable efforts into finding the best suitable solutions to facilities and equipment in order to bring oil and gas from the reservoirs to the markets.

In the late 70-ies, beginning of the 80-ties the petroleum activity on the Norwegian Continental Shelf was extended to greater depths and exposed to more extreme environmental conditions, and consequentially leading to larger and more complex installations.

The need for alternative field development solutions increased and Statoil was considering tunnels as part of a field development as one alternative. Several concepts of application of field tunnels were studied:

- Platforms plus placing pipeline(s) in a tunnel to transport hydrocarbons from the oil field to shore terminals and thus overcome complex shore approach, as well.
- Connecting of a sub sea wellhead template to tunnel based equipment for processing and transportation to shore based facilities.
- Placing and operating all equipment for drilling, processing and transportation to shore in a tunnel system

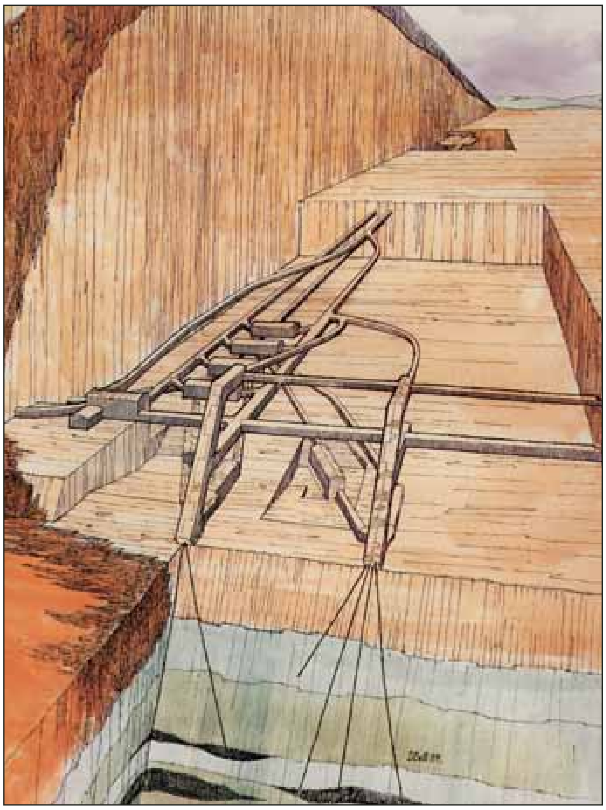
The Troll field was used as an example for the study. The Field Tunnel concept assumed a number of technical solutions which would demand extensive development of new technology.

In February 1984 Statoil entered into an agreement with a group of consulting engineers for a pre-feasibility study on a field tunnel concept. The group consisted of Ødegård & Grøner A/S (head of the group), A/S Geoteam, Jernbeton A/S, Resconsult A/S, A/S Gaute Flatheim, Department of Geology, Department of Mining Engineering and Department of Construction Engineering at NTH –University of Trondheim. Block 31 East at The Troll field was chosen for an investigation of a possible field tunnel system from Fedje Island to the oil field.

In the autumn of 1984 two separate joint ventures between contractors and consulting engineers were awarded contracts by Statoil for a feasibility study on tunneling to the Troll Field some 55km ashore.

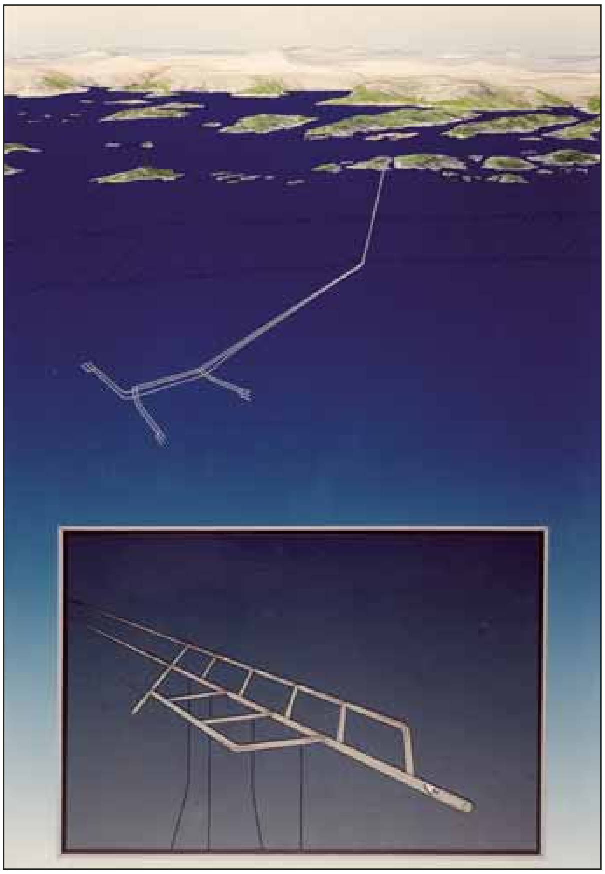
THE PETROMINE CONCEPT

In 1978 two Norwegian consulting engineers and a contractor started to explore the feasibility of an oil mine concept on the Norwegian continental shelf. In November 1984 two of the companies that started the oil mine studies in 1978, Ing. A.B.Berdal A/S and contractor Ing. Thor Furuholmen A/S founded The Petromine Company. In the end of 1985 the company was reorganized with additional partners, Norwegian Rig Consultants a/s and Norcem Cement A/S. The Petromine Company continued with the second phase of the study for Statoil as well as its own R&D work.



"TROLL I FJELL"

The other joint venture, The HAG Group consisted of contractor Astrup Høyer A/S and consulting engineer Grøner A/S. They named their concept study "Troll i Fjell" (Troll in Rock). The basic concept of the HAG Group was similar to the Petromine concept.



TASK OF THE GROUPS

The task of the groups was to investigate:

•-geology along the tunnel alignment from shore to the oil field
• construction technology of a tunnel system to the oil field
• oil production drilling from underground chambers
• oil processing underground at the oil field
• transport of the hydrocarbons from the field to shore based facilities
• safety and
• economical aspects of the concept

DESCRIPTION OF CONCEPT

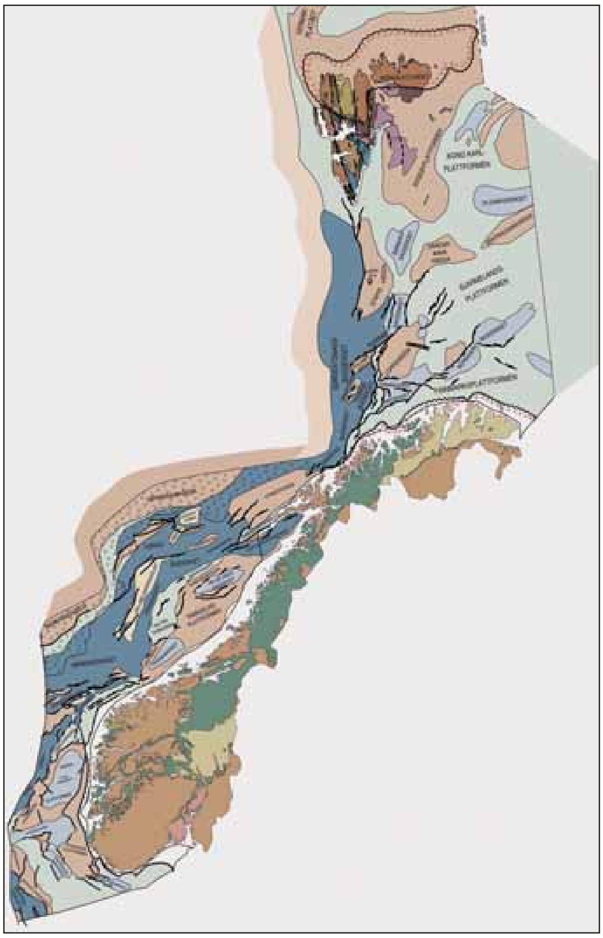
The tunnel concept comprises a network of tunnels containing and connecting the required equipment for drilling, processing and transportation to a shore based terminal.

The main tunnel system consists of three parallel tunnels excavated upward from a Base Station near the shore towards the oil field. At 8-10km intervals the three tunnels are interconnected enabling divided section to be established. From the main tunnel system two parallel tunnels to each drainage area at the field would be constructed. These tunnels are connected with transverse tunnels where the oil drilling equipment and a major part of the processing equipment are located inside pressure tight locks.

The three main tunnels are reserved for transport of hydrocarbon in pipes, transport of equipment and personnel, support facilities, ventilation etc. From the Base Station four tunnels lead to surface where the shore terminal is located.

GEOLOGY

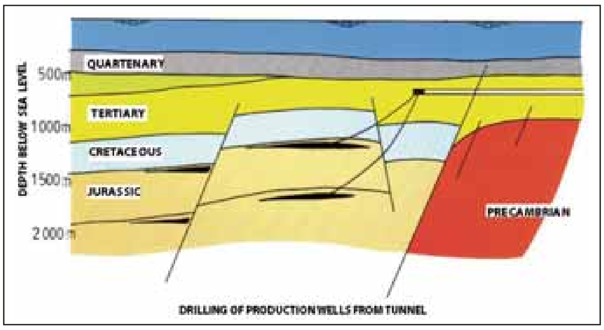
The geology along the oil field tunnels has a great variety from precambrian crystalline rocks to young and soft sedimentary rock formations, partly influenced by tectonical features, faults and fracture zones.



Geology of Norway and the Continental Shelf

In brief the crystalline bedrock consists of various types of granites, gneiss and schists while the younger sedimentary rock formations are layers of limestones, sandstones, shales and mudstones of various quality. The sedimentary rock formations are mainly from the cretaceous, paelocene and eocene periods. The subsea rock formations are covered by tertiary and quarternary sediments.

From more than 30 subsea strait crossings and several shore approaches in Norway, excavation methods and techniques are developed to cover the challenges expected for subsea tunnelling in the crystalline formations.



Tunnelling deep below sea level through soft sedimentary rock formation includes several challenges like high rock stresses, squeezing rock, structure collapse with water inflow at high pressure, flowing ground with sand and mudflow, gas pockets, mainly methane with high explosive risk. Studies were made to develop methods to detect soft structures and gas pockets ahead of the tunnel face.

CONSTRUCTION OF TUNNELS

The success of a field tunnel concept will highly depend upon the construction rate of the tunnels. In order to achieve a sufficient high rate of tunneling, use of TBMs (tunnel boring machines) was considered to be a must. Open Hard Rock TBMs and Single Shielded TBMs would be used for boring of the tunnels in the precambrian rocks (Gneiss, Granite) the first kilometers from the shore, and in the sedimentary rocks (Sandstones) respectively. The TBM would be equipped with rock drills for probing ahead of the cutterhead. Cement and or chemicals would be injected as required to protect the tunnels from leakage. Another important purpose of the probing is to get a pre-warning of shallow gas. Should gas be found, the rock would be injected with chemicals to lower its permeability and gas would be drained from the tunnel heading.

A major challenging factor for the feasibility of the field tunnel concept was the tunnel logistics. High TBM advance rates would consequently require large transport capacity of tunnel muck, materials and concrete segments for lining of tunnel and other support measures.

TUNNEL CONSTRUCTION DATA

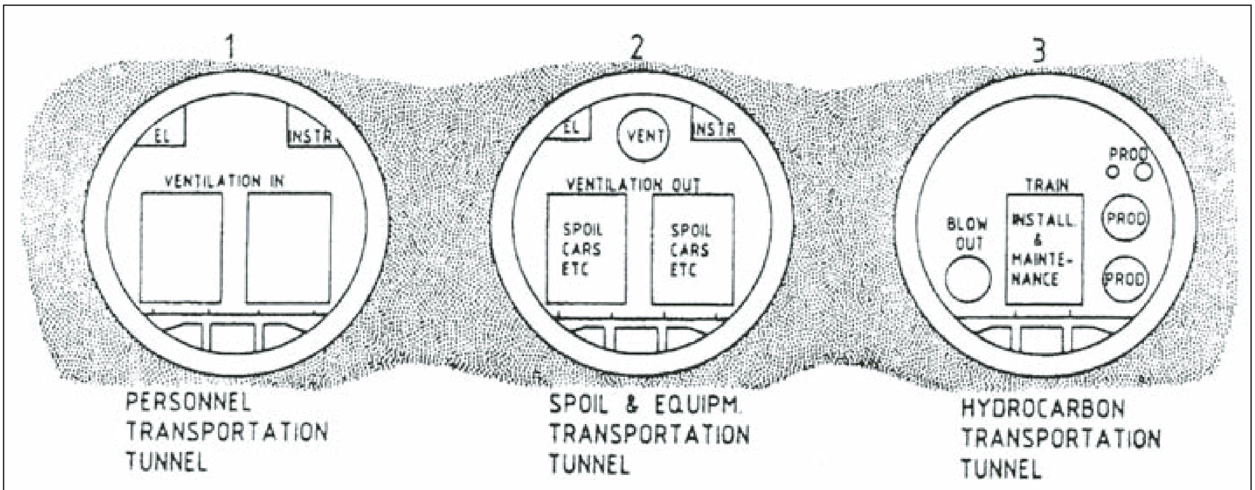
Distance from ashore: 55km
Total length of tunnels to be bored: 240km
Inside diameter of tunnel: 5m (after lining)
Depth below sea level at production area: 600m
Depth below sea level at base station: 700m
Number of tunnel boring machines: 8
Volume of bored rock (In-Situ): 6 Million cubic meters
Concrete lining: 1 Million cubic meters
Design load on lining: 10-11 MPa
Back fill: 200,000 cubic meters
Probe drilling, minimum: 800km
Construction time: 8 years

ADVANTAGES OF FIELD TUNNELS

Compared with fixed production platforms, the field tunnel concept offers the following advantages:
Low operating and maintenance costs
Not affected by weather conditions
Safer both for the personnel and for the environment
National security - Low sabotage risk – Protection from war actions
Reliability – low corrosion risk
Protecting a vulnerable environment from uncontrolled blow-outs
No conflict of interest with the fishing industry
No hazards to ship navigation

CONCLUSIONS

In 1985 after the concept studies, Statoil drew the conclusion that it is possible to construct tunnels from shore underneath the sea bed as far as 50-60 km in rocks of qualities equivalent to the Troll area and that it is possible to install and operate equipment for processing and transportation of hydrocarbons in the tunnel system.



To enable drilling of production wells from the tunnels would require extensive technology development of drilling equipment and procedures.
The concept including drilling of production wells from the tunnels was showing the most promising economical potential. Further, the concept is most suited for fields close to shore and for fields in deep and hostile waters.

FURTHER DEVELOPMENT

Beside methods and equipment for drilling and operating production wells in tunnel, Statoil listed the following topics for further development:
- TBMs for high advance rates and able to cope with high ground pressure in sedimentary rocks at great depths.
- Effective and reliable mapping of geology and monitoring of water and gas under high pressure ahead of the tunnel face.
- Grouting for stabilizing of rock and leakage prevention against water and gas under high pressure.

POSSIBILITIES FOR THE FUTURE

Statoil had through the concept studies established the feasibility of the major elements involved and identified the areas which would need further technology development in the future. They were of the opinion that the field tunnel concept was showing such promising economic and technical potential that it should be further developed.

In the mid 80-ies similar concept studies as for the Troll field were, as well, made for the "Haltenbanken" oil reservoir, 40-50km from shore west of Mid Norway.
Today these oil fields are operated by conventional platforms and sea bottom equipment.

Further north, outside the coast of Northern Norway and in the Barents Region where oil fields are closer to the coast line and the weather condition are extreme during winter time, field tunnels could be an alternative to consider again in the years to come.

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Name:..

Company:..

Address: ....

Place, Date and Signature:..


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