NORWEGIAN ROCK CAVERNS



NORWEGIAN TUNNELLING SOCIETY

PUBLICATION NO. 25

NORWEGIAN TUNNELLING SOCIETY



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REPRESENTS EXPERTISE IN

- Hard Rock Tunneling techniques
- Rock blasting technology
- Rock mechanics and engineering geology

USED IN THE DESIGN AND CONSTRUCTION OF

- Hydroelectric power development, including:
  - water conveying tunnels
  - unlined pressure shafts
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  - lake taps
  - earth and rock fill dams
- Transportation tunnels
- Underground storage facilities
- Underground openings for for public use

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FOREWORD

The present publication, No. 25 in the English language series from the Norwegian Tunnelling Society NFF, has - as always – the intention of sharing with international colleagues and friends the latest news and experience in the use of the underground. This time we want to show our various use of rock caverns, thus the title "Norwegian Rock Caverns".

One good reason for selecting this topic and title, is that next year, 2017, the World Tunnel Congress (WTC) will take place in Bergen on the Norwegian west coast. And the slogan for the congress is "Surface Challenges, - Underground Solutions." As we expect that many of the readers of this publication will take part in the WTC in Bergen, we have included examples of rock caverns located in or near Bergen, - Bergen known as "The city with seven mountains." Technical excursions to many of these caverns will be offered to the WTC participants.

On behalf of NFF we express our sincere thanks to the authors and the contributors of this publication. Without their efforts the distribution of Norwegian tunnelling experience would not have been possible.

Oslo April 2016

Norwegian Tunnelling Society, NFF - International Committee

Einar Broch *Elisabeth Grasbakken* *Øyvind Engelstad* *Gunnar Gjæringen*
Editorial Committee

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01. UTILIZATION OF ROCK CAVERNS IN NORWAY

BROCH, Einar

1 INTRODUCTION

During the last decades there has been a rapid development in excavation techniques for rock masses. Simultaneously there has also been a rapid growth of our cities, and an increasing awareness of the quality of our environment. This has led to an almost exponential increase in the use of the underground. This is clearly demonstrated by the fact that the number of cities with underground metros has increased to more than 100. Thus going underground is an everyday experience for a large number of people around the world.

Utilization of the underground for other purposes than traffic tunnels is not so well known for the general public. This publication will therefore first of all try to demonstrate how caverns excavated in rock may be used in urbanised areas for a wide variety of purposes such as:

- Storing of oil, gas, food, water, industrial waste and several other products.
- Municipal installations like treatment plants for drinking water and sewage, and car parks.
- Industrial installations like powerhouses, factories, telecommunication centres.
- Entertainment and recreation like sports halls, swimming pools, art centres, theatres, etc.
- War protection - Air raid shelters.

A factor of importance which should be kept in mind when Norwegian experience is being evaluated is the enormous amount of tunnelling and rock excavation which has been carried out in Norway during the last 50 - 60 years. A few figures may illustrate this:

- The number of railway tunnels is approximately 700, the longest being 14 km, and the number of road tunnels is approximately 900, the longest being 24,5 km, - the World`s longest road tunnel.
- Norway has today 200 underground powerhouses and 4000 km of tunnels for the hydropower industry.
- The World's largest cavern for public use, the Gjøvik Olympic Mountain Hall has a span of 61 m and can seat 5500 persons, - see Chapter 5.1

In a highly competitive market this has led to very cost-effective construction methods. In spite of high salaries to the tunnel workers, Norwegian contractors are probably producing the cheapest tunnels and rock caverns in the world. This clearly favours underground solutions for many purposes.

In the succeeding chapters examples of the different uses of the underground will be described. As the examples basically are from Norway, it is important that the reader is aware that the bedrock in this country is dominated by Precambrian and Paleozoic rocks. A difference to be aware of for an international audience is also that the zone of weathering is deeper in the rocks in most countries than in Norway as the Norwegian rock surface has been exposed to late glacial erosion.

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 rocks from this era are granites, gabbros and quartzites. Approxi-mately one third of the country is covered by rocks of Cambrian - Silurian age. The greater part of these rocks are metamorphosed, but to a very varying degree. Rocks such as gneisses, schists and greenstones as well as sandstones, shales, limestones and other nonmetamorphic rocks form a mountain range, which runs through the central parts of Norway. In the geologically unique Oslo region, the rocks are partly made up of nonmetamorphic Cambro-Silurian shales and limestones, and partly of Permian intrusive and extrusive rocks. These are the youngest rocks in Norway.

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 underground openings. Another complicating factor is the irregular stresses in the rock masses, which are due to the steep and irregular topography. Also high tectonic and residual stresses are encountered.

2 CAVERNS FOR STORING

Caverns are being used to store:

- Oil and other liquid hydrocarbons
- Pressurised gas and air
- Water
- Food
- Grains and vegetables at refrigerator temperatures.
- Fish, meat, ice cream, etc., at deep freezer temperatures.
- Industrial waste
- Others: Coal, sand, archival, art, wine, beer, seeds, etc.

Selected aspects of storage in caverns are described in this publication. Man has for centuries used the underground for storage purposes. Good protection and a constant climate have been important factors. Today some additional factors may favour the choice. It may be desirable to get for instance large tank farms for oil and other hydrocarbon products out of sight. There may also simply be a lack of land in built up areas. But the most important factor is, of course, the cost of the storage.

Excavation techniques for large rock caverns have been constantly improved over the last decades, and hence the relative costs have decreased. Cost comparisons carried out in Scandinavia between rock caverns and concrete or steel tanks for storage of liquids, indicate that when the volume to be stored exceeds approximately 5.000 m^3, the cavern gives the cheapest solution. Cost curves also show that the cost per m3 of cavern is reduced by 50% when the volume increases from 10.000 m^3 to 100.000 m^3.

2.1 Oil and other liquid hydrocarbons

Storing large quantities of oil in unlined rock caverns is a fully accepted technique all over the world today, and the interested reader can easily find related literature. (See also Article 2 in this publication). Let it only be briefly mentioned that oil caverns in reasonably good rocks normally has a span of 17 - 20 m, a height of 25 - 30 m and lengths from 200 to 500 m. Two to five parallel caverns are quite common. To prevent leakage of oil and/or gas through the rock mass, a so-called water curtain is usually established above and around the caverns.

2.2 Pressurised gas and air

There are basically two ways of storing large quantities of natural gas economically; either by compressing the gas, or by cooling down the gas,- in the ultimate case down to -160 - 170°C where the gas turns into liquefied natural gas, LNG.

So far storage of LNG in rock caverns has not been successful. There is some very challen-ging research ahead which has to be done. Our basic knowledge about how rock masses behave when exposed to these extreme temperatures needs to be improved. Such research can only partly be done in the laboratory, and even then with great difficulties. Reliable design parameters can only be obtained after testing done at a reasonable scale in the field. Also better knowledge about heat transfer in and around LNG caverns is needed. Storing gas in a compressed condition has for some time been done, but so far only for pressures up to approximately 10 bars. If natural gas shall be stored in rock caverns, pressures in the order of 100 - 200 bars will be needed to give economical solutions.

In Norway so-called air cushions have been used to replace surge shafts and surge towers at several underground powerhouses for 40 years. Unlined caverns with volumes of more than 100.000 m^3 and pressures of 78 bars are successfully operating without any air loss through the rock mass. Examples are described in Article 12 in this publication.

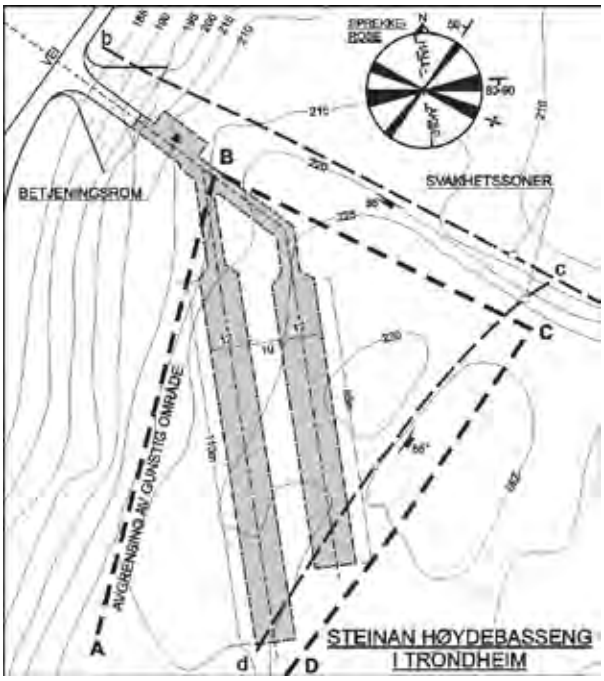


Figure 1: The Steinan rock cavern tank in Trondheim. Total capacity 22.000 m^3

The dotted lines indicate the topographical (A – B) and the geological (B – D) restrictions for a favourable location. (Broch & Ødegaard, 1983)

2.3 Drinking water

Next to the storage of oil and gas in caverns the most important is the storage of drinking water. Figure 1 shows the lay-out of an unlined rock cavern tank in Trondheim. The capacity of the tank, 22.000 m^3, was obtained by the excavation of two caverns with a width of 12 m, a height of 10 m and length 85 m and 110 m respectively. Also the service section is put underground. This is well accepted, but is not in daily use as the operation is remotely controlled.

Additional factors which may favour an underground solution for drinking water tanks are:

- High degree of safety, also against war hazards, sabotage and pollution.
- Constant and low water temperature.
- Low or no addition in price for a two chamber solution.
- The excavated rock masses may be used for other purposes.
- Very low maintenance costs.

2.4 Cold store caverns for food

Favourable temperature conditions are one reason for choosing the subsurface alternative. Another reason can be the favourable insulation that rock masses around a cavern can provide. The "walls" can, in many cases, be regarded as being of infinite thickness. Thus rock caverns have for some time been used as cold stores where, for instance, fruits and vegetables have been stored at normal refrigerator temperature, +(2 - 5)° C, and frozen food like fish, meat and ice-cream have been stored at so-called deep freezer temperatures -(25 - 30)° C.



Figure 2: Pictures showing the entrance to the Steinan rock cavern tank and the inside with water pipe, concrete dam wall and shotcreted rock walls

In Scandinavia the energy consumption for deep freezer stores is 75% and for refrigerator stores only 25% of similar surface stores. The peak energy requirements, and thus the in-stallations, are even more favourable. The deep freezer storage will need 50% and the refrigerator storage only 20% capacity of similar surface stores. Strongly reduced insurance rates are also favouring the subsurface solution for cold stores. This is due to the fact that the rock mass surrounding the storage caverns contain a big cold reservoir. In case of a breakdown in the cooling machinery, this will act as a reserve. Experience has shown that with cooling machinery out of function for a couple of weeks, an increase in the temperature of only 2 - 3°C is measured in underground deep freezer stores.

2.5 Industrial waste

In Odda, the Boliden company, a major producer of zinc, has by the Norwegian environ-mental authorities been instructed to deposit the residues from the production in a pollution safe place. The nearby steep mountains were considered to be the ideal place for the construction of rock caverns for storing of the residues. This project is described in details in Article 7 in this publication.

3 CAVERNS FOR MUNICIPAL INSTALLATIONS

Several municipal activities may be put underground. Today the following types of installations are found in rock caverns:

- Water treatment plants
- Sewage treatment plants
- Car parks
- Infrastructure

3.1 Water treatment plants

Several Norwegian cities have located their water treatment plants underground. One example is the Ulriken water treatment plant in Bergen which is described in details in Article 8 in this publication.

3.2. Sewage treatment plants

Also the sewage treatment plants have been put underground in many of the Norwegian cities and towns.

In the city of Trondheim with a population of 180.000 more than 90 % of the sewage is treated in two underground plants, one located in the eastern and one in the western part of the bay area. The first stage of the Høvringen sewage treatment plant was completed in 1978 as a primary treatment facility consisting of trashracks, sand traps and fat skimmers, see Figure 3. Later the plant was extended and chemical treatment was included. The marine outfall in the Trondheimfjord has a depth of 50 m and is located 100 m from the shore. Due to turbulence and mixing as well as the stratification of the sea, the outlet does not give any nuisance to the marine environment. The second underground sewage treatment plant in Trondheim, the Ladehammeren on the east side of the bay, was commissioned in 1992 and consists of three parallel caverns with a span of 15 m. The total length of the caverns is 450 m and the total excavated volume is 10,000 m^3.

Even though cost estimations may have shown that construction costs for underground treatment plants are higher than for similar on-the-ground plants, the underground solu-tions have been chosen in many of the cities and towns in Norway. Favouring this choice is first of all the wish to avoid the impact such big installations may have on the environment. A valuable additional benefit is the produced rock masses which there always is a need for in an urban area.

3.3 Car parks

Car parks located underground is a common solution in many Norwegian cities. Detailed descriptions of a car park in Bergen are given in Article 11 in this publication.

4 CAVERNS FOR INDUSTRIAL INSTALLATIONS

Caverns housing industrial activities or other activities related to industry may roughly be divided into the following categories:

- Powerhouses for hydro, thermal or nuclear power.
- Factories.
- Telecommunication centres.

Floor area is often more important than volume when caverns are designed for industrial installations. The shape may therefore be different from caverns which are typically designed for bulk storage (oil, gas, water). Low, wide and often relatively short caverns may be preferred by the planners. The exception is the modern hydro powerhouse with vertical axis for the turbine/generator where caverns with heights of 40 - 60 m may be needed.



Figure 3: Lay-out of the Høvringen underground sewage treatment plant.



Figure 4: Picture showing installations in one of the Høvringen caverns

4.1 Powerhouses for hydro

Topographical and geological conditions in Norway are favourable for the development of hydroelectric energy. 99% of a total annual production of 140 TWh of electric energy in Norway is generated from hydropower. It is interesting to note that since 1950 underground powerhouses are predominant. In fact, of the world's 600-700 underground powerhouses 200 are located in Norway, and during the period 1960 - 90 an average of 100 km of tunnels for hydropower projects was excavated every year.

Through the design, construction, and operation of all these tunnels and underground powerhouses, valuable experience has been gained. This experience has been of great importance for the general development of tunnelling technology, and not least for the use of the underground. The many underground powerhouses excavated in rock masses of varying quality are to a large extent the forerunners for the varied use of rock caverns which we find all around the world today. Examples of underground powerhouses are described in Articles 12, 13, 14 and 15 in this publication.

4.2 Factories.
As early as in 1954 an ammunition factory at Raufoss in Norway took into use nine big underground production halls with a total floor area of 25.000 m^2. An underground locomotive repair shop has also been used for a long time in Oslo.

4.3 Telecommunication centres.
Such centres are really the nerve centres of modern society, and should therefore be pro-tected as good as possible. The use of the underground is thus the evident solution. Excellent possibilities for climate conditioning are also favouring this solution. Today a number of underground telecommunication centres can be found in Scandinavia. One example is shown in the Figure 5: "Underground installations in Gjøvik" in Chapter 5 where cavern No. 3 is the local telecommunication centre.

5 CAVERNS FOR ENTERTAINMENT AND RECREATION

A particularly interesting development in the last 40 - 50 years with regard to the use of the underground, is the many rock caverns which have been excavated for entertainment and recreation. Many of them are dual purpose caverns. This means that, in the case of a fear of war, they are quickly converted to air-raid shelters. Thus part of the construction costs is covered by the National Civil Defence Authorities. In the Nordic countries one will today find caverns for:

- Theatre and cinema
- Concert halls
- Sports halls and gymnasiums
- Swimming pools
- Ice hockey rinks
- Restaurants.

Some examples will be briefly described in this chapter. Others are described in Broch and Rygh (1988).



Figure 5: Layout of the underground installations in Gjøvik

5.1 The Gjøvik underground complex.
In 1975, the first underground swimming pool with international standards was opened in Gjøvik, Norway. This swimming pool was part of an underground scheme which also in-cluded a telecommunication centre and headquarters for the local civil defence. As Figure 5 shows, the entrances to these subsurface installations are close to the main street of Gjøvik. From the entrance lobby with ticket office and cloakrooms, traffic is divided into two wardrobe/ shower sections for men and women respectively. Toilets and saunas form parts of these sections.

In the main hall with a span of 20 m is the swimming pool with 6 lanes of 25 m length and a children's play pool of 4 x 8 m. Separated by a glass wall is a small gymnasium (also used as a meeting room).

Of special interest is the fact that the energy consumption for running this underground public bath and swimming pool has been reduced to approximately 50% of what would have been necessary for a similar building on the surface. Both this and the fact that there were limited areas for building in the centre of the city were important factors when the decision to put the swimming pool underground was made. A total of 11,000 m^3 of solid rock was excavated and transported to a nearby marina under construction.

In connection with the Olympic Winter Games in 1994, the Gjøvik Olympic Mountain Hall was constructed next to the swimming pool.The cavern is 91 m long, has a maximum height of 25 m and the enormous span of 61 m. This is by far the largest span in the world for an excavated cavern which is being used by the public. Figure 6 gives an impression of the size of the hall and



Figure 6: Drawing giving an impression of the dimensions of Gjøvik Olympic Mountain Hall and the public swimming pool.

the swimming pool. The applied support of the cavern is given in detail in (Broch et al., 1996) and (Huang et al., 2002). When ice hockey games were played, the mountain hall had a seating capacity for 5500 persons. Today, the Mountain Hall is being used for a variety of activities such as concerts, hand ball games, exhibitions, etc. has a maximum height of 25 m and the enormous span of 61 m. This is by far the largest span in the world for an excavated cavern which is being used by the public. Figure 12 gives an impression of the size of the hall and the swimming pool. The applied support of the cavern is given in detail in (Broch et al., 1996) and (Huang et al., 2002). When ice hockey games were played, the mountain hall had a seating capacity for 5500 persons. Today, the Mountain Hall is being used for a variety of activities such as concerts, hand ball games, exhibitions, etc.

5.2 Holmlia sports hall and swimming pool.

In 1983, a combined underground sports hall and swimming pool was taken into use at Holmlia, a new suburban area in Oslo. During the planning of the area it was decided that a modern centre for varied sports activities should be built. In accordance with Norwegian civil defence regulations, blast and gas tight shelters for approximately 7000 people were needed near the centre of the new development area. Furthermore, large amounts of rock material were needed for the construction of roads and parking lots in the area.

Within easy walking distance from the Holmlia railway station and a shopping center, a small hill of gneissic rock was the obvious place for an underground sports centre. The rock cover was rather small, in certain places only 20 m above the roof of the sports hall, but was accepted by the Civil Defence Authorities. Figure 8 shows the general lay-out of the Holmlia Sports hall and swimming pool as well as the main dimensions. The sports hall is 25 x 45 m and is equipped for different ball games. The swimming pool has 6 lanes of 25 m length. A total of 53,000 m^3 of rock was excavated and the total floor area, including the swimming pool, stands, gallery and first floor above the entrances is 7,550 m^2.

When completed in 1983, the total costs (including civil defence installations) added up to 54 million NOK (7.5 mill. USD). Of this total, 8% was for design, supervision and administration, 67% for civil engineering works, and 25% for heating, sanitary, ventilation and electrical installations.



Figure 7: Picture taken during the excavation of the Gjøvik Olympic Mountain Hall

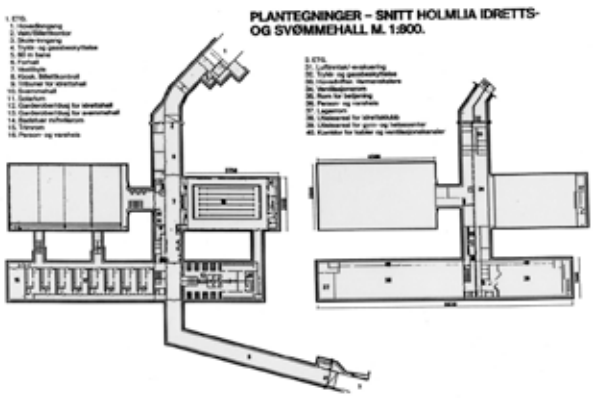


Figure 8: Plan view of Holmlia sports hall and swimming pool. (Rygh 1986)



Figure 9: The underground swimming pool at Holmlia



Figure 10: The underground sports hall at Holmlia

6 CONCLUDING REMARKS

Mountains and hills have often been regarded as obstacles by city planners. There is no doubt in this author`s mind that the underground will become a more and more valuable resource in cities and urban areas. Thus cities and towns with easy accessible rocks of a reasonable quality should have a great advantage. This introductory chapter has given an overview how the underground may be used in urban areas. However, only the fantasy sets a limit to how the underground may be used.

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Storage caverns
Boliden Odda AS. Courtesy: Anne Hommefoss

02. UTILIZATION OF THE UNDERGROUND FOR INFRASTRUCTURE IN THE BERGEN REGION

GJÆRINGEN, Gunnar

1 INTRODUCTION

Norway is a country with a lot of infrastructure installations underground – road and rail tunnels, caverns, parking facilities, water treatment plants and many others.

In, and near Bergen, the host city of WTC 2017, many infrastructure installations are situated underground.

Bergen is a city and municipality in Hordaland county on the west coast of Norway that is famous for being surrounded by seven mountains and is often referred to as the capital of Western Norway. It is the second largest city in Norway with a population of nearly 270 000 inhabitants.

Bergen is "The Gateway to the Fjords of Norway" and a well-established cruise port. It is an international city packed with history and tradition, a big city with small-town charm and atmosphere. Bergen likes visitors. And Bergen is worth a visit. Welcome! Bergen is surrounded by one of the world's most spectacular tourist attractions - The Norwegian Fjords, which have now been included on UNESCO's World Heritage List.

Bergen has given a warm welcome to its visitors for more than 900 years. Bryggen, the old Hanseatic wharf, has become a symbol of our cultural heritage and has gained a place on UNESCO's World Heritage List. It is architecturally unique and is perhaps one of the most familiar images in all of Norway.

Since Bergen is situated on the coast and is surrounded by a number of mountains, many tunnels of all kinds have been built here.

2 TYPES OF CAVERNS

- Maremine cave, Bergen centre. Mentioned in the novel written by Ludvig Holberg in 1741, The Journey of Nils Klim to the World Underground.
- Telecommunication. Telenor's telecommunications centre in Heggebakken, Bergen centre.
- Stad shipping tunnel under planning
- Oil caverns. Statoil's oil caverns with connection to the pipeline from the North Sea with installations at
  - Mongstad plant
  - Sture plant
  - Kollsnes plant
- Cod fish farms
- Water processing plant: Bergen municipality's water processing plant in Mount Ulriken
- Water supply system: Bergen municipality's water supply syste´m from Gullfjellet in tunnel
- Water and sewerage plants in tunnels: Municipal water and sewerage in Bergen





Fig. 1 and 2: City of Bergen.

- Wastewater treatment: Bergen's municipal waste-water treatment
- Parking facilities underground:
  - The Grieghallen-garasjen car park, Bergen municipality
  - The Klostergarasjen car park, Bergen municipality
  - Haraldsplass hospital, Bergen municipality
  - Sartor shopping mall, Fjell municipality
  - Skansendammen, Bergen municipality
- Warehouses: Minde
- Light rail tunnels, Bergen Light Rail
- Environmental waste storage: Bergen municipality's environmental waste storage facility at Stend.
- Subsea road tunnels with pumping stations:
  - Nygård tunnel
  - Bjorøy tunnel
  - Bømlafjord tunnel
- Roadtunnels
- Two-waylanes
- One-way lanes
- • "Discontinued" road tunnels:
  - Old road tunnels
  - Dale- Modalen - Built from 1920 to1939
  - Along the Eidfjord lake – Built in the 1880s Haukeli – Built in the 1880s.
- Railway tunnels
- Old railway tunnels
- Underground military base: Fjell Fortress: Built by the Germans during the Second World War ca 1941-1944.
- Powerplants:
  - Evanger
  - Modalen
  - Matre south
  - Matre north
- Water transport tunnels
- Cable Secure Tunnels for bridges
- Shelters in Bergen - Some combined with public transport tunnels
- Defence caverns

3 MAREMINE CAVE

Maremine cave (hole), Bergen centre, is where Nils Klim started his subterranean travels in the book The Jouney of Nils Klim to the World Underground, written by Ludvig Holberg in 1741.



Figure 3: The Maremine cave and the front cover of the book.

4 TELECOMMUNICATIONS CENTRE

Heggebakken Telecommunications Centre, Cappesvei 12a, Bergen is built inside the mountain Fløyfjellet and the decision to construct it was taken in 1973. From controlled FM and television transmitters in the Bergen area; the facility also includes telex and videotex exchanges (Data Torg), data networks and mobile and telephone exchanges for remote traffic and for parts of Bergen. Telenor sold the surface building in Cappesvei 12A in 2002 for the construction of housing, but still owns the underground facility in Heggebakken.

5 OIL AND GAS

Kollsnes gas processing plant

The Kollsnes Gas Processing Plant in Øygarden municipality north-east of Bergen, was put into operation in 1996 as a part of the Troll Project. Gas from the Troll, Kvitebjørn and Visund fields in the North Sea is treated at the plant, at a capacity of 145 Sm^3 NGL (natural gas liquids) and 69 000 Sm^3 condensate per day.

Shore Approach of the Troll Gas Pipelines

An important civil work at Kollsnes was the shore approach of the Troll gas pipeline as shown in the figure below. The work was executed from January 1991 to December1995, and comprised 7500m tunnels to a connection point for the Troll pipelines 3600 m from landside and 200 m below sea level.



Figure 4: Shore approach of the Troll gas pipelines at Kollsnes Gas Processing Plant.

The contract included also engineering and the installation of two 450-ton riser packets. These riser packets were installed and concreted into the piercing shafts from the tunnel system at 165 m below sea level. Piercing of a tunnel/shaft system into the sea at 165 m is one of the deepest ever executed in the world, possible the deepest. This contract was executed by AF Gruppen, Oslo, Norway.

The Sture Oil Terminal
At the Sture Terminal in Øygarden municipality, approximately 60 km north-east of Bergen, the construction work started in the autumn of 1984 with Norsk Hydro as Operator.

The Sture Terminal was able to receive the first oil brought ashore from the Norwegian Continental Shelf in December 1988. This was the first crossing of the Norwegian Trench by a pipeline at depths as low as 360 metres. The terminal was officially opened by the King Olav of Norway on 30th March 1989. The Sture Terminal receives crude oil and condensate through the Oseberg Transport System (OTS) from the Oseberg A Platform, via a 115 km long, 28" (71.1cm) diameter pipeline, and crude oil from the Grane field through Grane Oil Pipe (GOP) via a 212 km long pipeline, 29" (74.0cm) in diameter. The terminal comprises two large quay facilities able to handle large crude carriers up to 300 000 tonnes dead weight.

There are five crude oil caverns of dimensions (length*width*height of 19*314*33meter) with storage capacity of 1 million m^3 (plan and cross sections, see below).

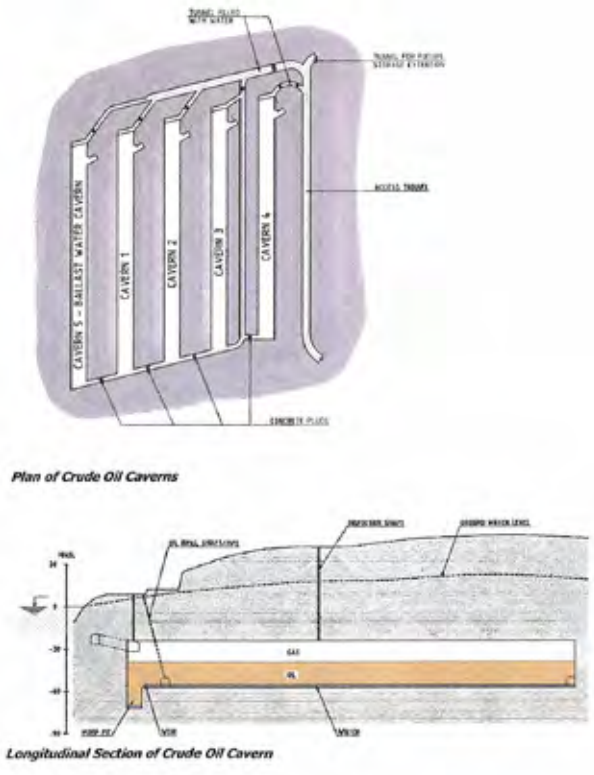


Figure 6: There is also a LPG-cavern of 60,000 m^3 and a ballast water cavern of 200,000 m^3.



Figure 5: Sture Terminal seen from east.
Photo: Øyvind Hagen, Statoil



Figure 7: Mongstad Production Plant.
Photo: Helge Hansen, Statoil

Mongstad production plant
The construction of the Mongstad Production Plant north east of Bergen in Lindås municipality was started in 1972. The facilities comprise a refinery, the Vestprosess NGL fractionation plant, a crude oil terminal, a combined heat and power plant and the world's largest CO2 technology centre (TCM). The refinery is the largest in Norway and has an annual capacity of near 12 million tons of crude oil.

The crude oil terminal plays an important role in Norwegian exports of this commodity to customers in North America, Europe and Asia. It provides intermediate storage for more than a third of all crude produced by Statoil on the Norwegian continental shelf, including the government's share. The Mongstad Production Plant has three quays able to handle VCCL (Very Large Crude Carriers) of up to 300 000 dwt, and has six oil caverns blasted out in the rock 50 m underground. The storing capacity of these caverns is 1.5 million tons of crude oil. VOC facilities are also in place. Every year 200 ships visit the terminal, and if product tankers are included the figure comes up to 1700 ships per year.

The crude oil terminal accommodates storage of oil from the Gullfaks, Statfjord, Draugen, Norne, Åsgard, Heidrun oil fields in the North Sea. The Mongstad Production Plant has in addition piercing of the pipelines from the Troll B and Troll C fields, where also Fram, Kvitebjørn og Gjøa are connected.

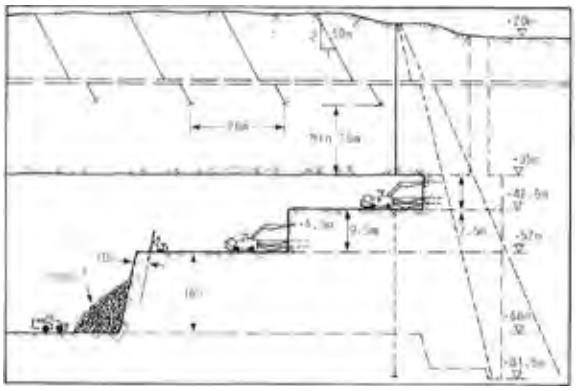


Fig. 8: Method for the blasting of caverns and the installation of water

In order to meet a very tough work schedule, the contractor was allowed to carry out the caverns on a "design-and construct contract" with the contractor, Aker-Høyer, responsible for the detail engineering. The caverns are 33m high and 18 m wide, and there are four caverns of 550 m in length and two of 330 m in length. The weekly production had to meet a schedule of 50 000 m^3 rock based on a two-shift arrangement. The drilled holes above the caverns for the water curtains (to hinder leakage of gas from the caverns into open air), were 50 m in length with a spacing of 20 m parallel in rows, and with internal distance to the next row of 68 m. The ground water system has been equipped with permanent non-stop automatic water infiltration system.

6 BRIDGE

The Nordhordland Floating Bridge

The Nordhordland Bridge consists of the world's longest free floating bridge (2005), 1246 m, and a high level cable-stayed bridge across the shipping channel of 32 x 50m. The floating bridge consists of a steel box girder which is supported on 10 concrete pontoons and connected to abutments with transition elements in forged steel. The pontoons were designed in LWA concrete LC55 in order to minimize the self-weight and draft of the pontoons.The floating bridge has the longest laterally unsupported span in the world (2005). Extension of the longest viaduct in Norway from 2 to 4 lanes by a separate bridge parallel to the existing bridge, linked together with an enclosure system beneath the superstructure.



Figure 9: The Nordhordaland Floating Bridge. Photo: Arne Årseth

7 RAILWAY

The Ulriken railway project with Full Profile Boring (TBM) for the new double railway tunnel through Ulriken.

The new Ulriken tunnel from Arna to Bergen, 7.8 km long, will be the first railway tunnel in Norway to be constructed using a tunnel boring machine.
The contract has a value of NOK 1.3 billion..The tunnel boring machine will have a diameter of 9.3 meters. This is to date the largest TBM machine that has been used to construct a railway tunnel in Norway.



Figure.10: The new Ulriken tunnel from Arna to Bergen

8 HYDRO POWER

The Dale power plant.

The Dale power plant is the lowest of the power plants in the Bergsdalsvassdraget river system. The water resources in the Bergsdalen valley are exploited in 'stages' with power plants all the way down the valley. The largest reservoirs,Torfinnsvatnet and Hamlagrøvatnet, are situated highest up. The drop from Hamlagrø down to Storefossdammen reservoir is exploited at Kaldestad and Fosse power plants. Storefossen is the inflow reservoir for Dale power plant, which is situated at the 'lowest stage'. In summer 1921, the year after the power company was formed, hydroelectric power pioneers started work on

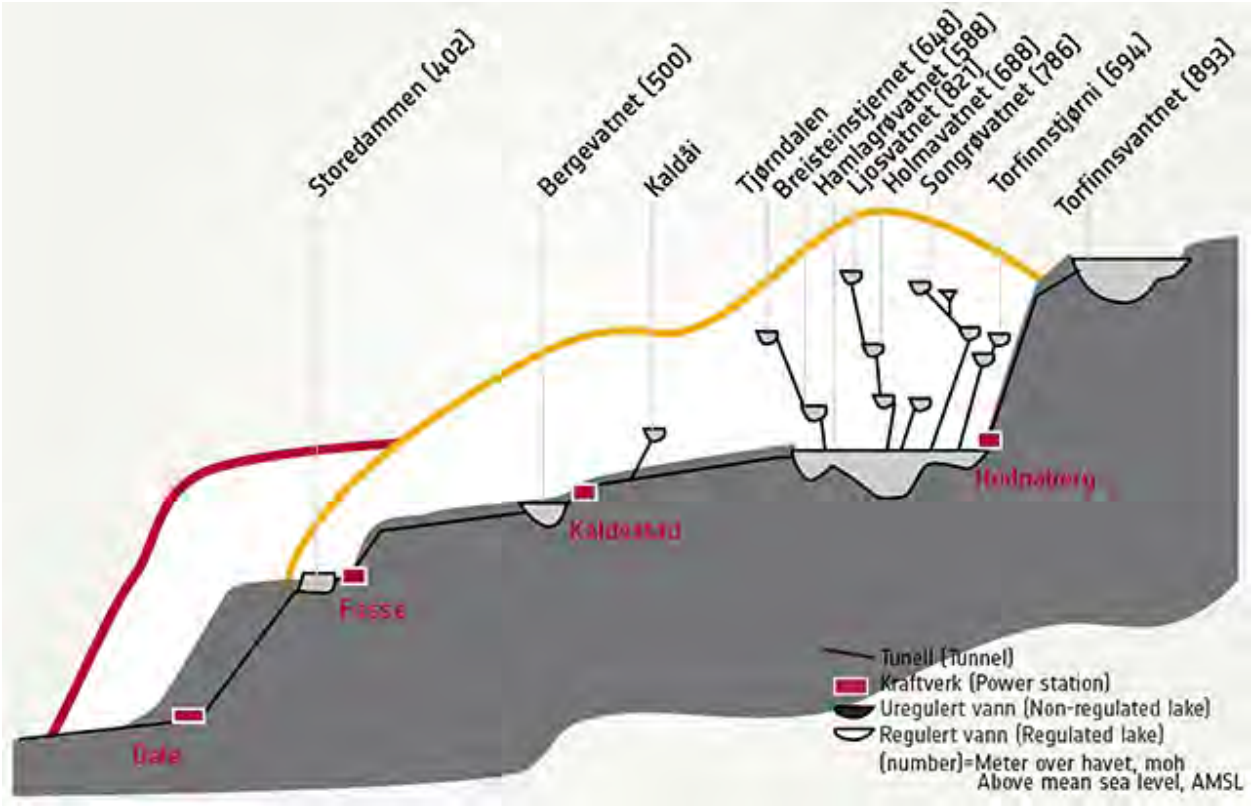


Figure 11: The Dale power plant is the lowest of the power plants in the Bergsdalsvassdraget river system.

Storefossdammen reservoir - the first stage in a very comprehensive and demanding development project. Power production started in 1927. Six generators were installed in all, the last of which started operation in 1951. From 1987 to 1990 BKK built a completely new power plant inside the mountain at Dale. Four of the generators at the old power plant were taken out of service and replaced by one new generator. In 2007, a new generator was installed in the new power plant, which meant that the old power plant was no longer necessary.



Fig. 12: The production in the old plant started 1927 and stopped in 2007 when the the new plant was established.

Technical data	
Power plant	Dale
Municipality	Vaksdal
Catchment area	248.6 Km^2
Inflow	680 Mill m^3
Penstock, length	2.18 Km
Penstock, cross section	45 M2
Height of fall	375 M
Output	146 Mw
Turbine	2 Francis
Average annual production	677 Gwh
Construction completed	1927 / 1951 / 1990 / 2007

9 ROADS

Eidslandet
The impressive scenery of Dale - Eidslandet with old road tunnels built between 1920 and 1939.

Construction start: 01 August 1920
Opened: 21 October 1939
Length: 10430m
Tunnels: 1443m
Total Cost: NOK 1,220,000
Cost per linear metre of road: NOK 116







Figure 13,14 and 15: Map, and pictures from the old tunnels.

The road construction of E39 from Rådal to Os
Purpose: The aim is to establish an efficient system for the transport of goods and passengers. Our society expects efficient traffic flow and regional accessibility, as well as road safety in the network of main roads.

Length: New E39: 16,250 metres
Extent: In total, 18,000 metres of main road and 1300 metres of new county road with pedestrian and cycle lane
Financing: Road toll, national government
Total cost: NOK 6.2 billion, 2012 price level.
National Transport Plan: Mentioned in NTP 2010-2019.
Start: 2015
Estimated opening: 2020



Figure 16: Map of the E39 Os – Bergen project.

The new connection from Bergen to the island of Sotra (west of Bergen).

- Proposal for mainland connection Sotra-Bergen along the stretch from Kolltveit to Storavatnet was drawn up in 2013/2014 and sent for further processing in a country of mountains and Bergen municipality around the turn of 2014/2015.
- We hope that the zoning proposal will be finally adopted by Fjell and Bergen municipalities by the end of 2015.
- Application for funding wil be sent when zoning proposal has been adopted - and approved in Parliament, hopefully during 2016.
- Work with land acquisition can start once the plan has been adopted.
- Start construction work around the turn of 2017/2018.
- Finishing of road project around the turn of 2021/2022.

The project will be one of three PPP projects in Norway, and includes one four-lane bridge and three two-tube tunnels.





Figure 17 and 18: The old and the new connection between Bergen and Sotra.

The Rogfast

Rogfast is the name of a 25-26 km long subsea tunnel being planned to pass under Boknafjord in Rogaland. The tunnel will contribute to shortening the travel time on the Coastal Highway Route E39 between Stavanger and Bergen by about 35 minutes. The project is now in the planning stage. In the Norwegian National Transport Plan, it is assumed that the start-up will be in 2017, while locally there is a desire to bring the start-up forward to 2014 if possible.



Figure 19: Rogfast subsea tunnels.– each of approx. 28km , total length 56km).

The Rogfast tunnel goes through an area of complicated geology that has several thrust nappes and faults. The tunnel will go through phyllite and mica schist and possibly also granitic and dioritic gneisses.

The projected tunnel is planned as a twin tunnel with a cross-section of 2 × T 9.5. There will be an arm up to Kvitsøy and a two-level underground interchange. The design speed will be 100 km/h and maximum gradient will be 7%. The maximum depth will be 390 m. The tunnel will be ventilated along its length and three double shafts are planned. The design fire will have a 100 MW effect at the outset. A complete risk and vulnerability analysis will be carried out to determine what standard must be incorporated to achieve a satisfactory safety level with respect to fire and other undesirable events. Roughly speaking, Rogfast will generate surplus rubble and spoil amounting to approx. 6 million m^3. This must be utilised for socially beneficial purposes and it is assumed that most of it will be used as fill in the sea near where tunnelling begins. The fill areas will be valuable maritime industrial areas after the project has been realised.

Municipal sub-plans have been approved in all three municipalities concerned (Randaberg, Kvitsøy and Bokn.) Work has now started on zoning plans in all three municipalities, and approved zoning plans are expected to be ready early in 2013 for the entire project.

Byfjord and Mastrafjord

The western coast of Norway consists of many long fjords. This article describes how two of these fjords are crossed by two subsea road tunnels, 5,860 m and 4,405 m long, respectively. The longer tunnel, the Byfjord Tunnel, is the world's longest and deepest subsea road tunnel, at 223 m below sea level. The paper describes the preliminary investigations and geological conditions for the tunnels, each of which has three lanes, and a total width of 11 m.



Fig. 20: Byfjord and Mastrafjord subsea tunnels.

10 WATER TREATMENT PLANT

The Ulriken water treatment plant
The work:

- Access tunnels and caverns to processing plant, intake pipe, clean water reservoir.
- New processing plant in mountain with full purification of drinking water
- New water intake in Svartediket
- New water pool in mountains 70 masl.
- Modification of existing pithead installations for access, a support staff, the chemical replenishment, viewing room and closed footbridge to underground facility.
- Tunnel (about 2 km)

Key data for the new facility
Average water production: 520 l / s
(equivalent to 45,000 m^3 / day)
Maximum water output: 925 l / s
(equivalent to 80,000 m^3 / day)
Volume water pool: 15.000 m^3
Volume Haukeland Brac: 30,000 m^3
Depth of new deep-water intake: 28 m
Water treatment process: Direct Filtration and UV radiation
Excavated rock volume: 120,000 m^3 (solid masses)



Fig. 21: Drawing of The Ulriken water treatment plant

11 THE BERGEN LIGHT RAILWAY

The Bergen Light Railway from the centre of Bergen to Bergen Airport, Flesland.
The city of Bergen in Norway is building a light rail system between the centre of the city and Bergen Airport at Flesland. The project, being undertaken in three phases, includes about 20 km of tracks. The first phase, with a length of 9.8 km and 15 intermediate stops, has been completed at an estimated investment of €175m. About 3.6 km will be added in the second phase and another 7 km in the third phase.



Figure 22: The Bergen Light Railway

12 UNDERGROUND PARKING FACILITIES

The Skansedammen underground parking facility
The new Skansedammen car park beneath the Skansedammen pond was officially opened at 4 pm on 1 September 2015. Some swimming area is no longer available, but the firehouse from 1903 is still reflected in the water in all its glory.The decision to construct the underground Skansedammen car park garage was taken in 2003, some two years after the car park company had proposed it. But then the Directorate for Cultural Heritage issued a protection order for the pond. Rounds of discussions followed, and eventually the Ministry of Climate and the Environment overturned



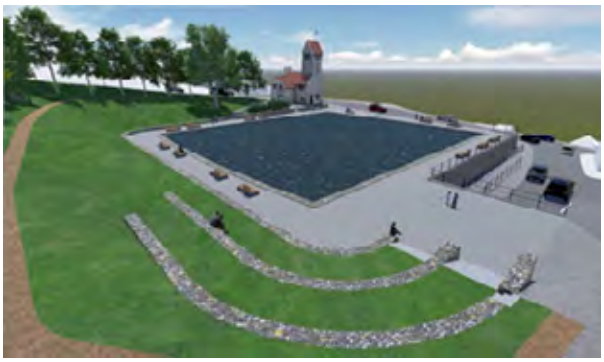


Figure 23: The Skansedammen underground parking facility

the order, allowing the city council, in 2010, to again pass the proposal to build the car park.

The Skansedammen pond had previously been swimming area and habitat of carp. In 2014 it was over. The Aquarium was given the rest of the fish from Skansekaia dam as alligator food. The new version of Skansekaia dam is only 19.5 centimeters deep. Under the pond is a car park with space for 193 cars. It is zoned for residential parking, and was built to eliminate street parking.

13 SHELTERS

In Bergen we have 26 shelters, and they are mostly situated under ground.

The Sydnes Tunnel

The Sydnes Tunnel is a road tunnel in Bergen, Hordaland. The tunnel runs in the city centre under the University of Bergen between Nygårdsgaten and Bredalsmarken. It is reserved for transit. The tunnel is also built to serve as shelters during a crisis situation. Should such a situation arise, the tunnel will be closed to traffic and used as shelters. Refuge rooms can accommodate 3,500 people. The Sydnes Tunnel replaced the older Nygård Height Tunnel which was completed in 1948.

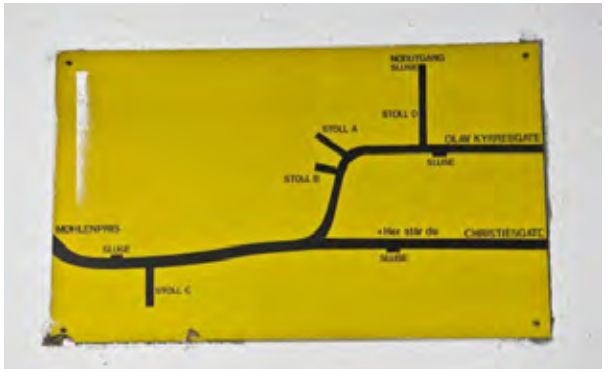




Fig. 24 and 25: Map and picture from the shelter used as bu tunnel.

Map of the shelter and a picture showing the massive concrete doors are covered and barely visible if you do not know that they are there. In a crisis situation, these will be closed in a v-shape that makes them resistant to pressure.

14 CONCLUSION

The caverns in the Bergen area meet many of the different needs of the community, and show that the underground has been used for centuries here. With all the new tunnels under both planning and construction we will continue to use the underground for many reasons and with many aspects and prospects.

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v. The Sture Oil Terminal – Statoil.
vi. The Mongstad Oil and Gas Production Plant The Nordhordaland Floating Bridge – The Norwegian Public Roads Administration.
vii. The new Ulriken Tunnel from Arna to Bergen – The Norwegian Rail Department.
viii. The Dale power plant – BKK – Bergenshalvøens Kommunale Kraftselskap.
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xi. The new connection from Bergen to the island of Sotra (west of Bergen) – The Norwegian Public Roads Administration.
xii. The Rogfast subsea tunnels – The Norwegian Public Roads Administration.
xiii. The Byfjord and the Mastrafjord subsea tunnels – The Norwegian Public Roads Administration.
xiv. The Ulriken Water treatment plant – Bergen Municipality.
xv. The Bergen Light Railway – Hordaland County Facility
xvi. Underground Parking Facilities in Bergen – Bergen Municipality.
xvii. Shelters underground in Bergen city – Bergen Municipality.



The Follo Line Project
Grouting of bench at assembly cavern for TBM at Åsland, the Follo Line project. Norwegian National Rail Administration. Courtesy: Elisabeth Grasbakken

03. PLANNING, DESIGN AND CONSTRUCTION OF LARGE ROCK CAVERNS

ENGELSTAD, Øyvind

1 INTRODUCTION

With continuous growth in population and increased urbanization, the efficient use of surface space has become more and more important. Utilization of the underground space for infrastructure development and storage offers a solution to some of the challenges. This is also a fact in Norway, and throughout the last 100 years an increasing amount of infrastructure has been moved underground as to free up space on the surface for other purposes.

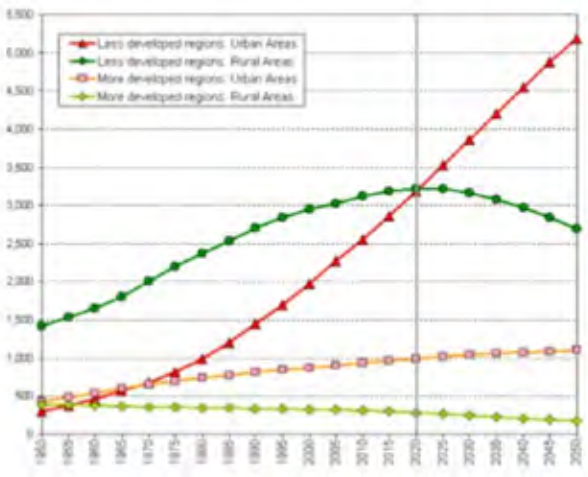


Figure 1: The prospected development in Urban vs. Rural population (Source: UN Department of Economic and Social Affairs, 2010)

Some of this infrastructure demand construction of large underground opening in form of caverns. The dimensions of these caverns span from small pump stations and defence facilities to huge sport facilities like the Gjøvik Mountain Hall. As the importance and dimensions of the caverns increase, the demand for good planning, appropriate design and efficient construction is essential. This article discuss the site investigation and planning process, the design challenges and construction techniques involved for safe and cost efficient development of large caverns.

2 PLANNING

2.1 Scope of planning

When planning an underground development like a rock caverns there are many issues that needs to be taken into consideration. Some of these issues are listed below:

- Purpose of the development
- Relation between the planned facility and other infrastructure
- Size and shape of the development as well as possible future enlargement
- Possible conflicts with existing underground and surface facilities
- Possible conflicts with future development in the area
- Property ownership
- Environmental impact
- Social impact
- Constructability and access to site
- Construction risk
- Operational risk
- Legal framework
- Economic framework

2.2 Planning process

The development of a project may follow different paths, but in many cases the planning process follows the steps defined below:

Screening Study

The main purpose of the screening stage is to clarify the purpose and key demands forming basis for the project as well as identifying the most suitable site for the development. Some of the key activities are:

- Clarify purpose and basic demands
- Identification of potential sites for development
- Screening of sites (cost/benefit)

Pre-feasibility Study

The main purpose of this stage is to ensure that there is sufficient potential to defend further site investigations and study costs. Some key activities are:

- Development of basic concepts
- Overall schedule
- Rough cost estimation and economic analysis

Feasibility Study
The purpose of a feasibility study is to clarify if the development is feasible of not both from an economic and impact perspective. Some of the key activities are:

- Basic studies within all relevant areas
- Site investigations
- Development of design basis (all relevant demands governing the design)
- Layout and optimization of size and shape of the development
- Identification and layout of construction facilities, spoil deposits and borrowing pits
- Constructability study
- Detailed cost estimation
- Risk assessment
- Schedule
- Economic and financial analysis
- Environmental and social impact assessment (ESIA)
- Clarification of access to land etc.
- Preparation of application for licenses and permits

Basic design and preparation of tender documents
The aim of this stage is to develop the project as to enable tendering for construction and supplies as well as processing of construction licences etc. some of the key activities are:

- Basic design of all the main components in the project
- Design calculations necessary for definition of key dimensions, support measures etc. The level of detail is governed by the risk assessment and demands from licensing agencies.
- Development of function description, scope of work, technical requirements, bill of quantities etc. for tendering purposes.
- Contract strategy and prequalification of contractors and suppliers.
- Contractual framework
- Tender documents
- Tendering and contract negotiations

Detailed design and construction
In the construction phase the design is adopted to the conditions encountered and input from the different parties participating in the construction. In many cases (depending on the type of development) the installations in the caverns will be subject to design by EPC suppliers, and there will be an initial phase where the final dimensions and arrangement is settled through an iterative cooperation process. As the contractor enters the area of the cavern(s), in-situ mapping, probe drilling, Lugeon tests, stress measurements etc. is performed and the geological/geotechnical information is updated. The final design of the rock support will be based on this updated information. Normally the support is installed as initial and permanent, where the initial support is installed to secure safe working conditions and the permanent support is installed to supplement the initial support as to secure long term stability and serviceability. For a cavern the dimensions are normally such that the permanent support should be installed successively with the benching, as to prevent excessive costs to provide access at a later stage. In addition to the support, one normally install and collect data from instrumentation such as extensometers, inclinometers, load cells and measured convergence as to analyse the stability and function of the installed support.

Commissioning
In the commissioning phase the installations are tested and put into operation. The as built documentation is prepared as to form basis for the asset management. Supplementary instrumentation for recording of data for monitoring in the operation phase is normally installed and tested in this phase.

Operation
In the operation phase the stability of the cavern and the condition of the installations are monitored and maintained based on an asset management plan. It is important that all elements of importance to the stability and integrity of the cavern is included as "maintenance objects" and are monitored and maintained as planned. Data from the instrumentation installed is monitored and recorded on a regular basis and a warning system based on triggers when the data pass certain thresholds is normally applied.

2.3 Ground investigations
Deviation from assumed ground conditions is one of the most common reasons given for cost and schedule overruns in underground development projects. The reason, is in most cases, insufficient ground investigations or that the parties have not put sufficient emphasis on the ground conditions during planning, design and construction.

Pre-construction investigation, often simply called pre-investigation, is thus very important for evaluating the feasibility of the project and for planning and design. Among other good reasons for having a high focus on pre-investigating the following factors are particularly emphasized:

- Give input for analysing stability and estimating rock support requirements

- Provide input for evaluating alternative excavation methods and selecting equipment/tools for excavation and rock support
- Provide a basis for predicting performance and capacities
- Provide basis for estimating cost and schedule.
- Provide basis assessing potential environmental impacts such as surface settlement, ground water lowering, vibrations etc.
- Give a basis for preparing tender documents

If the pre-investigations are insufficient or of poor quality, reports and tender documents will not reflect a correct picture of the actual geological conditions. Conflict between contractor and owner due to "unforeseen geological conditions" will normally be the result of this, and in worst case the project may end up in court with the extra costs implied by this. High emphasis on investigation is therefore very important for all aspects of the project.

There is no straight answer to the question "what is the correct level of ground investigations and analysis?". There are a many factors that influence the stability of the rock mass and many of these are not easily measured or quantified. In comparison with other construction material like concrete and steel etc., the properties of the rock mass are much more complicated to measure and understand. The properties also vary greatly within a rock type and between different rock types. The fact that the rock mass is jointed and can be protruded by weakness zones with varying degree of decomposition makes the picture even more complex. Extensive site investigations are often expensive and may times does not give the desired insight. An experience based approach has thus been the common approach in Norway.

It is common to distinguish between pre-investigations for planning purposes and investigations performed during construction for final design.

The goal of the pre-investigations is normally to obtain information that can form basis for:

- Preparation of geological/geotechnical maps and sections in the area of the project
- Obtaining data about the properties of the rock mass, e.g. distribution and composition of rock types, soil cover, orientation and density of the dominating crack systems, position and characteristics of weakness zones, groundwater conditions etc.
- Obtaining data about rock stresses (magnitude, direction, anisotropy etc.)
- Conditions in critical areas like cross cuts, areas with low rock cover etc.
- Assess uncertainty and need for flexibility in design
- Assessing the influence on the surroundings (environmental and social impacts, including vibrations, deformation, groundwater lowering etc.)

The main factors governing the magnitude of the ground investigations are:

- The stage of development
- Type of development
- Contractual arrangement for the construction (EPC, Unit rate based contract (Fidic Red Book or similar))
- The stability and safety demands during construction and for the operation of the plant versus the degree of difficulty. Eurocode 7 defines three Geotechnical Classes (RC) from 1 to 3 depending on complexity and consequence of failure, but does not take degree of difficulty into consideration. The basic principal is however the same as in the former NS3480 (Norwegian standard for Geotechnical works), with a classification based on degree of difficulty and damage consequence class (CC). This principal is still applied in Norwegian projects. The categorization is given as follows:

	Degree of difficulty		
Reliability class	**Low**	**Medium**	**High**
CC/RC 1	**1**	**1**	**2**
CC/RC 2	**1**	**2**	**2/3**
CC/RC 3	**2**	**2/3**	**3**

- Design life of the underground installation and the possibility to do maintenance
- Cost and schedule risk perspective, signifying the level of risk the developer is willing to carry into the next step of development. This is normally governed by a cost vs. benefit assessment, where the value of additional information have to be balanced with the cost of additional investigations and studies

Pre-investigation tools are available for practically any kind of ground condition and any kind of site characterization. However, it is important to realize that even when great effort have been made in the pre-investigation, some uncertainty will still remain regarding the ground conditions. Pre-construction site investigations therefore always have to be followed up by continuous engineering geological investigation during construction.

Below is presented a schematic representation of normal pre-investigations for an underground project.

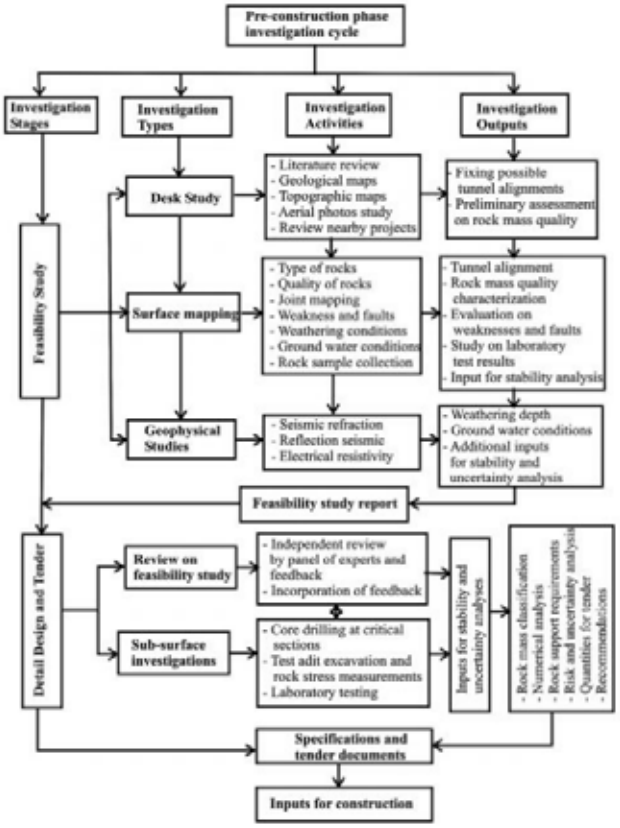


Figure 2: Schematic representation of typical ground investigations in the pre-construction phases of underground projects (Panthi and Nilsen (2007))

3 DESIGN

3.1 Basic approach to design of caverns

A basis for planning of underground structures in Norway, is to consider the rock mass as a self-supporting material. Hence the purpose of rock support is to enable the rock mass to be self-supporting, and not to establish a structure supporting a "passive rock mass".

Location and orientation

The choice of location for a cavern complex is the single most important decision in the whole design and construction process, as it determines the quality of the rock in which the underground space will be excavated. Factors other than engineering geology can constrain the choice of location, and a favourable site may have to be found within a limited area.

The constraints on cavern location that should be considered are frequently related to the location of portals, access tunnels, external traffic, and the size and operation of the cavern facility. Within such constraints the cavern installation should be optimised with respect to the topography and geology. The important factors in this optimisation are:

- adequacy of rock cover,
- avoidance of weakness zones or the crossing of them in the shortest possible distance
- avoidance of adverse orientation relative to major joint sets and weakness zones
- sufficient depth below the groundwater table (specifically for oil storage caverns)
- avoidance of rock with abnormally low stresses, giving reduced confinement and aching effects
- avoidance of rock with very high stresses giving rise to rock burst

When designing the layout for a cavern project, one normally seek to place the axis of the caverns as to have a favourable orientation in comparison with the major joint systems as well as the major stress.

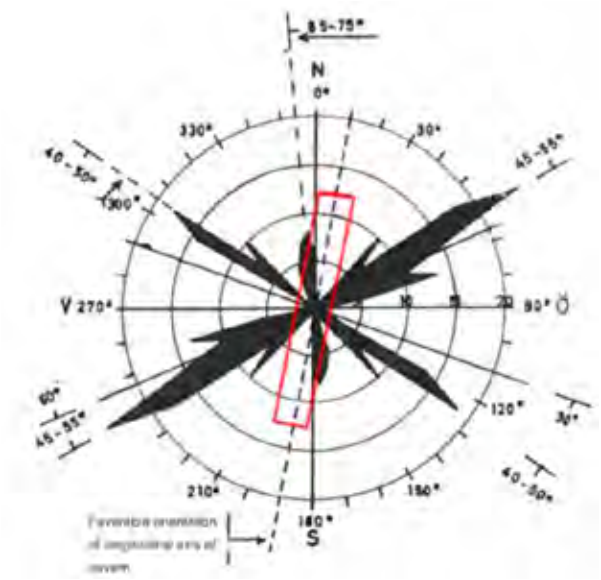


Figure 3: Favourable orientation of the longitudinal axis of a cavern or other underground opening in respect to the orientation of the dominating crack systems (Selmer-Olsen, R. And Broch, E. (1977)).

3.2 Layout, size and shape

The rock mass is a discontinuous material of low tensile strength. The basic design concept for an underground cavern is to aim at evenly distributed compressive stresses in the rock mass surrounding the excavation. This is best obtained by giving the space a simple form with an arched roof.

The cross-section of a cavern should be optimised, within given restraints, to produce the lowest combined excavation and support costs. For example, support costs increase with cavern span, excavation rates reduce with cavern height and wall support costs increase with cavern height.

The layout, shape and size of the caverns is normally governed by the purpose of the cavern. In general the more circular the shape is, the more stable the underground openings becomes. A rounded shape will however normally give a higher construction cost, as the excavation becomes more complicated (rasional benching is hindered). Depending on the orientation of the main stresses the most favourable shapes are illustrated below.

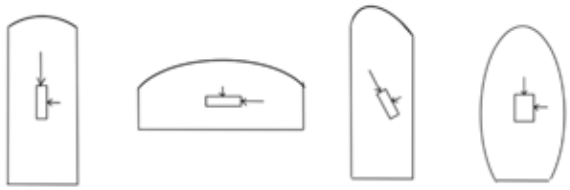


Figure 4. Optimum shape of cavern cross section depending on the orientation of the major stresses. a) σ_1 vertical > σ_3 horizontal, b) σ_3 vertical< σ_1 horizontal, c) sloping anisotropy and d) close to hydrostatic conditions σ_1 vertical = σ_3 horizontal.

Normally a large span of the cavern is more challenging than a more narrow cavern due to the need of securing an ache effect to support the roof span. The arching effect obtained is as function of rock stresses (magnitude and orientation), rockmass quality and shape of the roof. The arch shape of the roof of the cavern is commonly chosen with a roof arch height of 1/5 of the span. Reducing the roof arch height increases stability problems and fall-out during blasting but may be justified if the dominant joints have a shallow dip. Improved stability by increasing the roof arch height is in some cases favourable.

Cavern walls are normally vertical. This suits the method of excavation (benching) and yields little unusable space. Wall stability is a function of wall height, the in-situ stresses and the orientation and engineering properties of the principal joint sets. The flat wall surface precludes any substantial arching action and high walls tend to be unstable.

Major joints and seams can dominate wall stability and can affect the chosen wall height. The cost and scale of stabilising measures can increase substantially with wall height and this has to be taken into account in optimising cavern shapes.

Joints with shallow dip favour wall stability as the dominating vertical stresses in the walls increase joint friction. Conversely, steeply dipping joints with strikes parallel to the wall reduce stability as the horizontal stresses on the joints are small. The converse of this is true for the roof arch.

Many projects does not only involve a singular cavern, but several caverns, tunnels shafts etc. When caverns are placed in proximity of each other there will be an accumulation of stress in the rockmass separating the different openings. This might cause excessive strains in the rock mass causing instability, slabbing and spalling of the surface towards the openings.

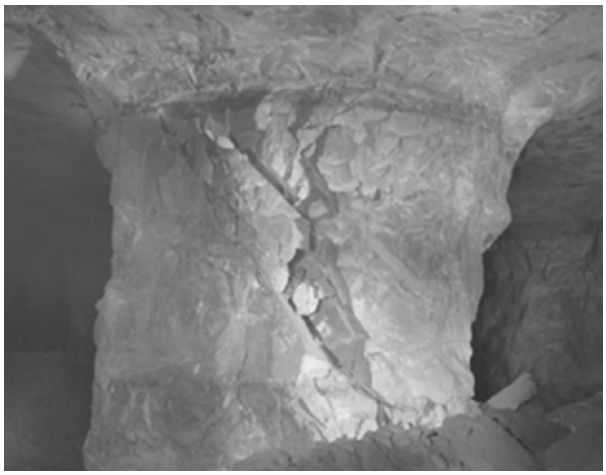


Figure 5: Instability of pillar between underground openings (G. S. Esterhizen et. Al)

As a general guide, vertical separation should be no less than the largest span or height of the adjacent caverns. Separations of less than 20 m should normally be avoided. For schemes with stacked caverns, detailed analysis will be required if lesser separations are desired. Pillar widths should in general be designed so generously that mathematical modelling is not necessary. Pillar widths designed to the theoretically defendable limit will commonly give rise to additional excavation and support problems.

Another challenging area of a cavern structure is areas with stress release and insufficient side support. This typically appears at intruding edges and corners at intersections between openings and where shape of the cavern is changing.

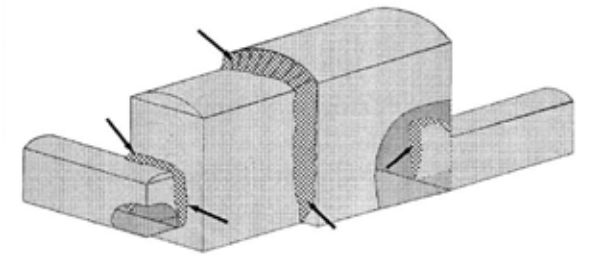


Figure 6: Instability of edges and corners (Selmer-Olsen and Broch, 1982)

The location, orientation and dip of weakness zones is of major importance. In general one should try to avoid placing caverns in conflict with major weakness zones. If a weakness zone is encountered in the cavern one should try to orientate the cavern as to cross the zone at an angle of close to 90 degrees. Making the influence zone as minor as possible.

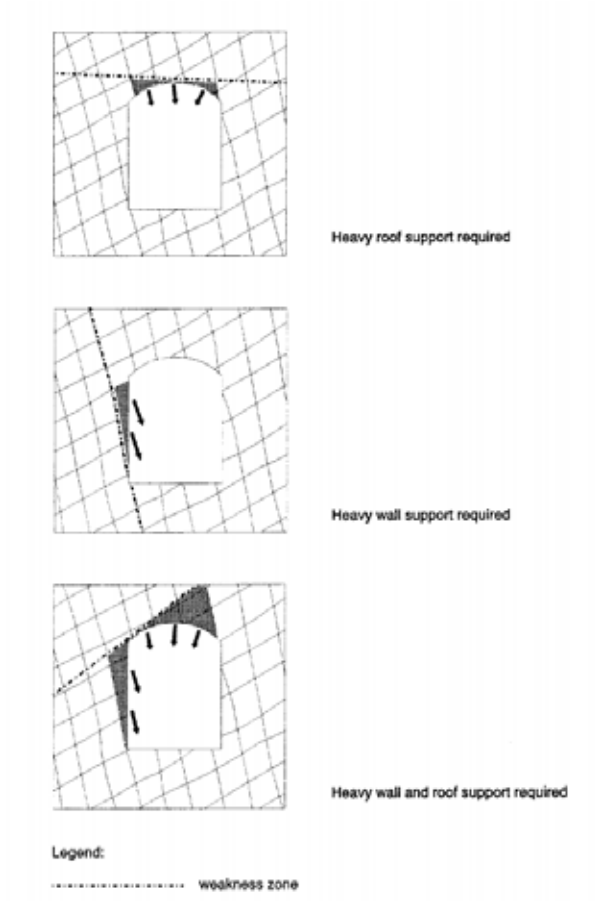


Figure 7: Influence of weakness zones in the rock mass surrounding the cavern

Weakness zones in the surrounding rock mass that does not protrude into the cavern volume must also be accounted for. Such zones can have a great influence on the stability and thus necessary rock support, as illustrated in figure 7.

3.3 Rock support design

The design of rock support for underground projects at a planning stage of a project is always a challenge, as both loads and bearing capacity is unknown. Owners and third parties with little knowledge regarding engineering geology often require absolute answers in the form of calculated rock support design, for instance like calculation of reinforcement in concrete structures. The bedrock however is not fully "opened" for inspection and investigations and will always provide surprises. Effect and interaction between different factors as low overburden, proximity to existing underground structures, unfavourable stress situation, poor rock mass quality, intrusions, weakness zones and ground water can be very difficult to predict.

The empirical approach to design of underground space in hard rock conditions is based on experience from numerous caverns and tunnels constructed in various types of rock mass under varying stress conditions . This approach is used for tunnels and caverns of conventional size, say up to 25 m span, constructed in good rock. Factors other than rock stresses normally govern design and sophisticated design tools are therefore not in common use in Norway.

The normal rock support system in Norwegian caverns is fibre reinforced shotcrete end-anchored or fully grouted bolts (depending on rock stresses and quality of rock) in the roof and walls. In cases where the rock mass is of high quality the walls are not covered with shotcrete. In some cases anchors are necessary to stabilize the rock mass where deeper seated failure surfaces are encountered (see Figure 7).

As a rule of thumb Palmstrøm (2000) give a suggested expressions for calculating the length of rock bolts from dimensions of the and the block size. The equations are plotted in the graphs in figure 8..

3.4 Modelling

In the case of large span caverns, caverns in difficult ground conditions and multi-cavern schemes it is common to perform FEM analysis to study the stress distribution and stability issues governing the design and rock support. Such analyses are used primarily to extrapolate experience (i.e. the empirical design rules) to cover conditions not previously encountered or to

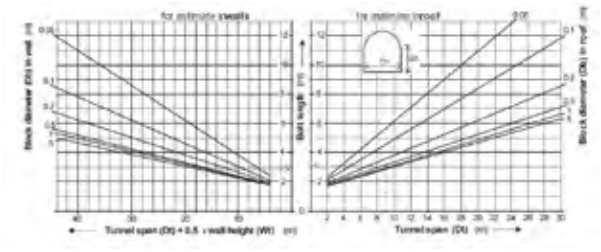


Figure 8: Suggested bolt lengths in roof (right) and wall (left) as a function of span and block size (Palmstrøm (2000))

better understand the mechanisms. Such mathematical analysis should under no circumstance fully substitute for experience and observations during construction.

Many material models exist on the market for modelling the behaviour of the rock mass. The more advanced the modelling become, the more demanding it is to specify appropriate input.

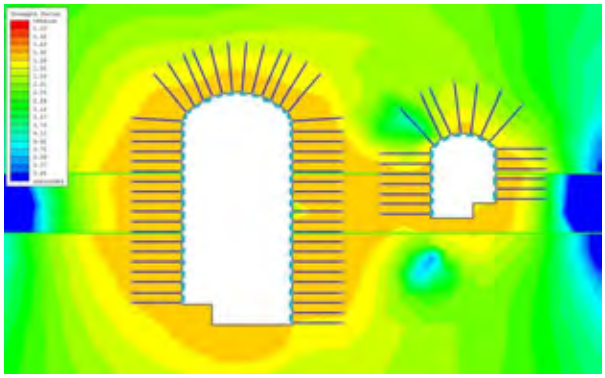


Figure 9. Example of simple FEM-analysis for a power house and transformer cavern complex (showing strength factor for tension in the rock mass) (http://www.mpassociates.gr/new/programs/alphabetical/phase2.html)

4 CONSTRUCTION

4.1 Excavation method

Drill and blast construction dominates the construction of underground space. In the last few decades, the basic methods have not changed, but improvements in equipment and blasting technology have resulted in increased production rates and lower costs. Although full face boring can be an attractive alternative to the drill and blast method for long tunnels and shafts of uniform cross-section, the drill and blast method is expected to remain viable for tunnelling for years to come, and will remain the only method of forming large caverns in hard rock.

The rock blasting of caverns can involve:

- face blasting with horizontal drillholes for tunnelling or top heading excavation,
- benching with horizontal drillholes
- benching with vertical drillholes.

All three methods are commonly employed in the excavation of caverns.

4.2 Excavation sequence

4.2.1 Top Heading

The top heading of a cavern excavation should normally be excavated first using
tunnelling techniques. This gives easy access to the cavern roof for installing support works. The secured roof gives safe working conditions for the excavation of the lower levels of the cavern.

The lower levels may be excavated using quarrying techniques, i.e. benching, which
Is normally cheaper than tunnelling.

The size of the top heading is governed by several factors which commonly require
the top heading to be divided into two or more sections. These factors are:

- the reach of the drilling jumbos and the platforms/ lifts used for support works, which limits the height to 10-15 m.
- the area of unsupported roof that can be exposed at any one time, which is primarily a function of rock quality and presence of weakness zones.
- limitations on the quantity of explosives discharged in a round given by; blast vibration acceptance criteria and the practical depth of blast holes, which is normally 3 to 5 m depending on the cross-sectional area of the top heading.

It is normally economical to excavate as large a face as possible, but the above factors may limit the size of the top heading to some 100 to 120 m^2.

In poor rock conditions, the initial top heading may need to be substantially smaller than 100 m^2 and should then be used to examine the ground conditions. Detailed excavation planning, design of rock support and other rock treatment works should then be done before the full span of the cavern is exposed.

The number of sections of top heading depends on rock conditions, the span of the cavern and the maximum practical size of heading.

The order of driving the sections depends on the rock joint orientations and the need for support. The sections should be driven in the order that avoids support of rock that will be removed by excavation of subsequent sections. The side of the cavern roof that is expected to have the least favourable stability should be driven first. Where no such stability and support problems are expected, the centre heading may be driven first . There are various options for the timing of the excavation of the second and subsequent sections of top heading.

4.2.2 Benching

Bench excavation is cheap because the large free surfaces allow the use of quarrying principles rather than tunnelling technology.

Production rates of bench excavation can be high. Maximum rates of 60 000 m^3 of rock excavated per week have been reported from a six-cavern crude oil scheme in Mongstad, Norway.

Bench excavation may be done with vertical drillholes as in a quarrying operation, or with horizontal drillholes. Normally vertical drillholes are preferred as the drilling of holes is then independent of other sections of the excavation cycle except during blasting and ventilation. Vertical drillholes will normally also leave a better contour on the wall of the cavern. The holes are usually made with drilling jumbos or with crawler-mounted quarrying rigs. However, horizontal drilling may be required if a clean floor is necessary. Modern drilling jumbos with high production rates have increased the competitiveness of horizontal drilling compared to vertical drilling. Caverns of limited height preclude vertical drilling because of lack of headroom near the walls.

The cavern excavation should be divided into benches of a suitable height. The height of the benches may be determined from consideration of the following:

- Access: The mucking-out must be through tunnels located at suitable levels.
- Blast-hole deviation: The longer the holes, the greater the deviation which has to be matched to desired tolerances.
- Reach of jumbo: Bench heights with horizontal holes are limited to the reach of the drilling jumbo, which is some 7 to 10 m; this is not a limitation for vertical holes.
- Stability of walls: In some conditions, walls may be unstable if too high, such that successive bench excavations and support will be required.
- Cavern use: Some uses, such as unlined fluid storage, do not require optimisation with respect to shape; only volume is important, and they are optimised primarily with respect to efficient construction. For other caverns like in hydropower, the shape of the cavern and the accuracy of the contour is important both to allow sufficient space for the installations to enter the cavern and to reduce concrete volumes.

When the quality and accuracy of the contour is important, emphasis should be put on reducing the bore hole deviation and special techniques can be adopted to reduce overbreak. Normally simple blasting is used with a reduced distance between holes and reduced charging of the holes in the contour. In some cases the contour is pre-splitted by drilling holes close together (8-12 times bore hole diameter) and blasting these holes up front to induce a crack between the hole around the contour. Another method used to secure a high quality and accurate surface is “seam boring”. In this method the drillholes are placed very close together and not charged with explosives.

The charging of the contour holes is also an important factor for achieving a good quality and reduce the blasting induced damage of the rock surrounding the cavern. The figure below illustrates this.

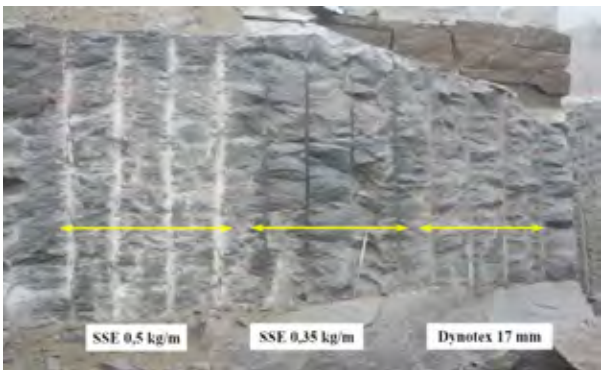


Figure 10: Contour quality vs. charging density. (Olsson m.fl.(2008)

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The Follo Line Project
Grouting of bench at assembly cavern for TBM at Åsland, the Follo Line project. Norwegian National Rail Administration. Courtesy: Elisabeth Grasbakken



04. CONTINUED ROCK STRESS AND DISPLACEMENT MEASUREMENTS COMBINED WITH NUMERICAL MODELING AS AN ACTIVE, REALISTIC ROCK ENGINEERING TOOL

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HOLMØY, Kristin Hilde
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ABSTRACT

The Norwegian University of Science and Technology (NTNU) and the Foundation for Industrial and Technical research (SINTEF) located in Trondheim, Norway have since the late 1960ies developed equipment for in-situ rock stress measurements, including 2-D and 3-D overcoring and hydraulic fracturing. This also includes equipment for monitoring of rock stress changes. SINTEF carries out stress measurements on a commercial basis, which has resulted in measurements at more than 150 locations in 18 countries around the world. This includes disciplines such as mining, hydropower projects, public caverns, military installations and infrastructure.

This paper presents recent a case study of a sublevel caving mining operation. SINTEF has developed a comprehensive procedure, combining continuous rock stress measurements and displacement measurements with 2-D and 3-D numerical modeling. Continued rock stress measurements and rock stress change monitoring are actively used to calibrate the numerical models. This forms a tripod of information, which gives reliable modeling results used for active operation planning.

Keywords
In-situ stress measurement, 2-D and 3-D overcoring, stress monitoring, rock mechanics, numerical modelling

1 STRESS MEASUREMENT METHODS AT SINTEF

The Rock Mechanics Laboratory of the Norwegian University of Science and Technology (NTNU) and SINTEF (Foundation of Technical and Scientific Research at NTNU) has performed in-situ rock stress measurements since 1964. Through SINTEF Building and Infrastructure, Department of Rock and Soil Mechanics, measurements are carried out on a commercial basis.

Several measuring methods have been applied over the years, including the USBM 2-D cell and the photoelastic 2-D cell. However, the present applied techniques are:

- Self-developed and highly improved versions of the originally South African CSIR 2-D Doorstopper
- 3-D CSIR overcoring cells.
- Self-developed hydraulic fracturing equipment

These methods have been continuously developed and improved through extensive research and application over the last 30 years.

Whenever possible, the procedures described by the International Society for Rock Mechanics (ISRM) as "Suggested methods for Rock Stress Determinations" are used to evaluate the measurements. However, it must be pointed out that these are only suggestions based on papers published in the International Journal for Rock Mechanics and Mining Sciences in 1989, and not by any means a norm or standard. Some of the ISRM suggestions are replaced by the SINTEF procedures, which we believed that they are more appropriate.

Three-dimensional (3-D) determination of stress tensors at a site is normally based on 7 – 10 successful single measurements well away from the influence of the tunnel in a sub-horizontal borehole. The stresses are calculated by the computer programme DISO (Determination of In-situ Stress by Overcoring) developed by SINTEF. By randomly selecting strain readings from different positions along the measuring hole, up to 35,000 groups of results can be achieved. From this, statistical calculations are carried out which results in the mean values and the deviation of the principal stresses. The programme automatically removes obvious erroneous strain values. The magnitudes of the principal stresses are presented graphically in a histogram plot, while the directions are given in an equal area lower hemisphere stereonet plot.

2-D overcoring stress measurement
A diamond drill hole (76 mm outer diameter) is drilled to the desired depth. The core is removed and the bottom of the bore hole is flattened with a special drill bit.

A two dimensional measuring cell (door-stopper) that contains a strain gauge rosette is inserted into the hole using a special installing tool and glued to the bottom of the hole. The door-stopper is fixed to the hole, then the initial reading (0 recording) is performed. The installing tool is removed and the cell is ready for overcoring.

A new core is drilled with the 76 mm outer diameter diamond drill, thus stresses are relieved at the bottom of the borehole. The corresponding strains at the end of the core are recorded by the strain gauge rosette. The core is removed from the hole with a special core catcher. Immediately after removal from the hole, the second recording is performed. From the recorded strains, the stresses in the plane normal to the borehole may be calculated. Supplement elastic parameters for the calculation are determined from laboratory tests.

The equipment and procedure for this stress measurement technique are presented in Figure 1.

3-D overcoring stress measurement
For the 3-D overcoring stress measurement, a diamond drill hole (76 mm outer diameter) is drilled to the desired depth. Usually, this depth is 1.5 times the span of tunnel/cavern. The bottom of the bore hole is flattened with a special drill bit, and a concentric hole with smaller diameter (36 mm o.d) is drilled approximately 30 cm further at the bottom of the hole into the rock mass.

A measuring cell with strain gauges and a data log unit are installed with a special installing tool containing orienting device. Compressed air is used to expand the cell in the hole, and the strain gauges are fixed to the walls in the hole. The cell is then ready to start measuring. Continuous logging of strain data is stored in the measuring cell. The installing tool is removed and the cell is ready for overcoring. The small hole is overcored by the larger diameter bit, thus stress relieving the core. The corresponding strains are recorded by the strain gauge rosettes.

The core is removed from the hole with a special core catcher. Immediately after removal from for the hole the recorded data is transferred to a computer. When the elastic parameters are determined from biaxial- and laboratory tests, the stresses may be calculated.

The described procedure is presented in Figure 2. Detailed descriptions of the measuring cell are not presented here. Interested readers are referred to the SINTEF's website to find more information.

The described techniques have been used widely in various different disciplines, including mining industry, hydropower, oil and gas storage, and infrastructures in many countries around the world. Our measurement comprises some 150 locations from 18 countries throughout the world. As each client or location may represent a number of measuring sites, the total number of sites/measuring holes are in the order of 330, corresponding to approximately 3000 single measurements. Interested readers can find selected activities and list of clients in a SINTEF brochure, on SINTEF's website.

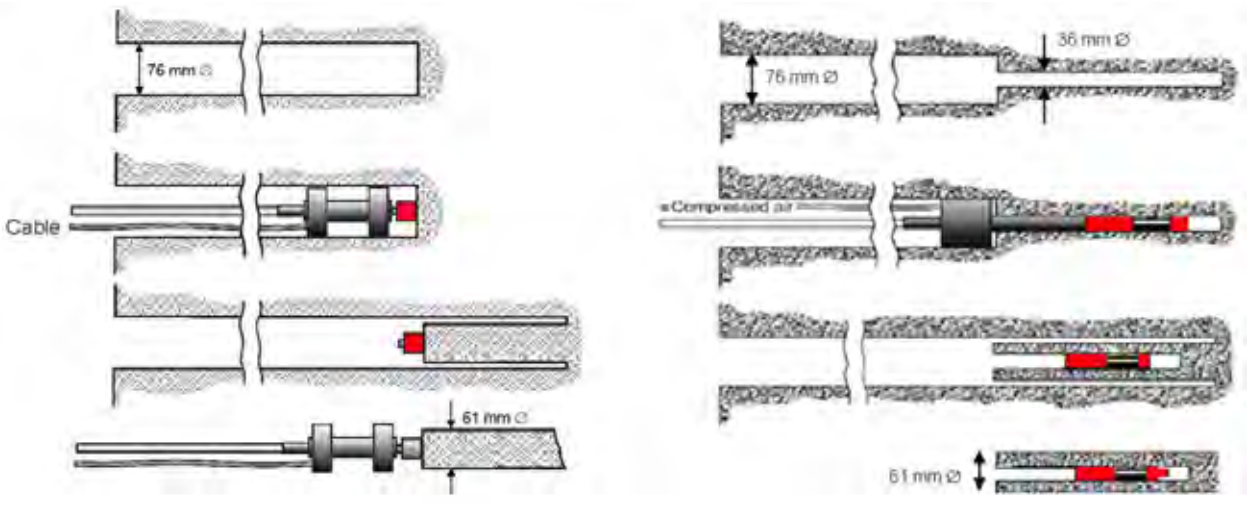


Figure 1: Equipment and procedure for 2-D stress measurement (left hand side) and 3-D stress measurement (right hand side).

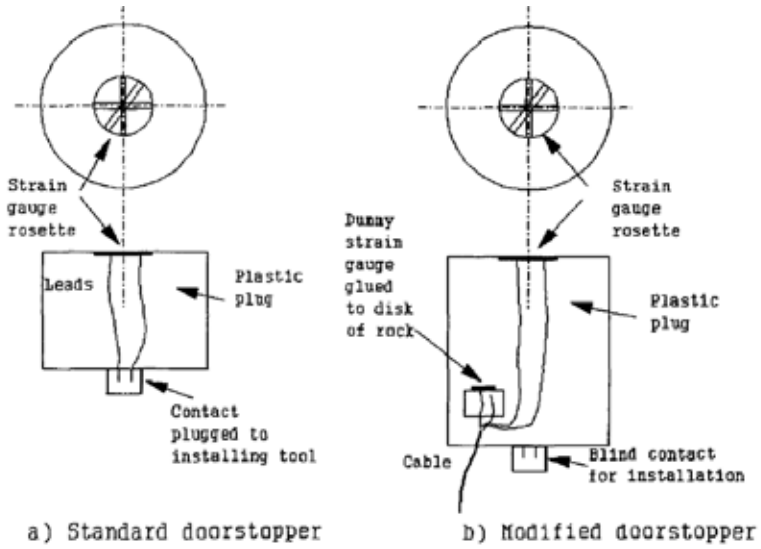


Figure 2:. (a) Standard and (b) modified doorstoppers.

2 STRESS MONITORING METHOD AT SINTEF

Development

Description of the stress monitoring development is taken from Myrvang et al, 1991 for Tverrfjellet – a multi-metal mine in Norway. The mine production started in 1968, and serious rock stress build-up occurred in certain areas of the mine over years, related to the mine progress. The need for monitoring rock stress increase was urgent. In 1988, in a very short time the Rock Mechanics Laboratory of SINTEF developed a special version of the "door-stopper" for monitoring purposes for this mine. The biaxial doorstopper gauge for rock stress measurements has for some 25 years been used for absolute rock stress determination in Norway. The standard doorstopper has a dummy gauge for temperature change compensation in the installing tool. In the modified version the traditional contact of the doorstopper has been replaced by a permanent cable cast into the plastic plug together with a dummy cell (Figure 2). The cell is cemented to the flattened bottom of a standard 76 mm diameter, diamond drill hole. During drilling of the hole, standard doorstopper measurements are taken at selected intervals to obtain the initial stresses (Figure 3). The last measurement is taken as close as possible to the selected depth of the monitoring doorstopper (normally within 10 cm).

Readings are taken by means of a standard, high quality strain gauge bridge, and the stress changes are calculated using the strain differences from the initial zero readings and the elastic parameters of the rock. The calculated stress change is then compared with the closest initial stress measurement from the standard doorstopper measurements.

A major concern in connection with long-time measurements is the stability of the measuring system as such. To control this, a similar doorstopper is glued to a destressed drill core from the same bore hole. This core is kept at the measuring site between readings, and the doorstopper is read simultaneously with the active one. After about 20 months of operation the general experience is that the long-term stability is very good. Provided that a high quality strain gauge bridge is used it may therefore be concluded that strain changes that are not induced by stress changes are negligible. However, to be on the safe side, a passive cell is always kept at each measuring site.

Since the first successful use in Tverrfjellet mine, the 2-D longterm door-stopper monitoring equipment has been used in many other sites both in Norway and internationally.

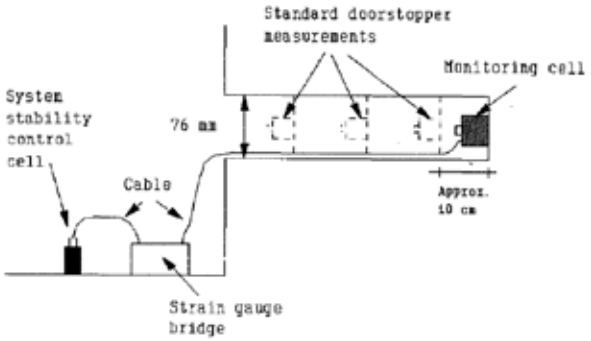


Figure 3: Measuring procedure during bore hole drilling.

3 COMBINATION OF STRESS MEASUREMENT

Numerical modelling – monitoring
Many rock engineering projects today may face rock mechanics challenges such as particularly complicated profile or excavation plan, and complicated geological conditions. There may be no similar existing experience to lean on. Thus, empirical methods have limitations and uncertainties in such cases. Therefore, it is a goal in SINTEF to develop a reliable rock engineering tool to deal with the challenges. The tool will be a combination of stress measurement-laboratory, numerical modelling, and monitoring.

With the growth of computerisation in engineering offices and fast development of software, the use of numerical methods is increasing rapidly. Today, numerical methods are considered to be the most flexible and complex method of design in many disciplines. However, quality of the method depends largely on the quality of the inputs. Thus, in order to make the numerical model to be a reliable and effective tool for rock engineering, in-house competences (stress measurement – laboratory, numerical modelling, and monitoring) in SINTEF are developed in the way that they complement each other. We believe that by using numerical model accompanied with proper measuring and monitoring tools (forming a tripod of information as shown in Figure 4 - left), it would create an effective tool to deal with most of the rock engineering challenges.

With long history of research and development in theory, equipment, and practical skills, our stress measurement can provide reliable information regarding stress condition at a given site. The measured stress could be in-situ stress condition, stress in a rock pillar, or stress condition around any rock opening. In-situ stress conditions obtained from the measurements together with thorough geological engineering mapping provides reliable inputs for numerical modelling. Based on these inputs, a comprehensive numerical model can be established and simulated to provide information needed for further evaluation and decision-making. To increase the reliability of the model even further, it is then followed, verified, and improved along the way by communicating with monitoring equipment. Any discrepancy between the model and in-situ observation or monitoring data must be studied carefully in order to detect the pitfalls and possible improvement. A recommended working model for combining stress measurement – laboratory, numerical modelling, and monitoring is shown in Figure 4.

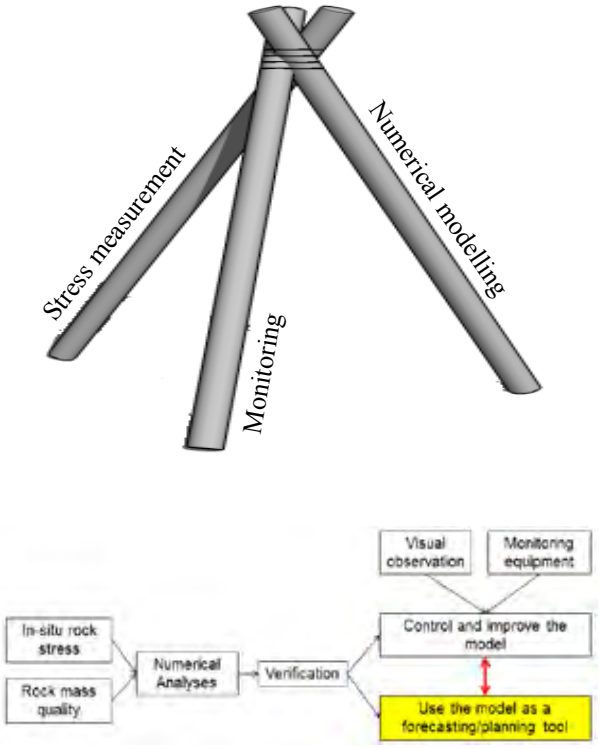


Figure 4: Recommended working model for combining measurement & laboratory – numerical modelling – monitoring.

4 APPLICATION AT RANA GRUBER MINE

General information
The Rana Gruber is an iron mine in the North of Norway, 30 km east of Mo I Rana. The mine is located in a foliated gneiss host rock, and the ore body is about 70 m wide and more than 300 m deep. The iron ore deposits in an area are well known prior to 1799. The mine has been owned and exploited by different owners in the 19th and 20th centuries. Different mining methods were used, including open pit mining and sublevel stoping. Open pit mining had been operated until 1980s, and from 1999 the underground mine was established with sublevel stoping mining method. The history and development of the mine was described in some publications by Sand (2010), and Sand and Trinh (2011).

Since 2009, due to increasing needs of production, Rana Gruber AS investigated methods to increase the mining capacity. The investigation resulted in the decision to change over from open stope, sublevel stoping to sublevel caving. Changing a mine from sublevel stoping to sublevel caving method requires many con-

siderations in rock mechanic aspects. Some of the typical considerations are:

- From the long experience of the mine development, it is well-known that the rock mass in this area has high horizontal stress. How will this stress condition affect the new mining plan?
- Stress, displacement, and the overall safety of the mine in associated with the pillar removal;
- Caveability of the hanging wall of the mine;
- Advantages and disadvantages of different mining layout in the sublevel caving: Complete mining 1 upper level before starting the lower one, mining from 2 ends toward centre; mining different levels at a time with different lagging distance; mining from centre toward 2 ends; mine the extra ore body in the middle part of the mine.

In order to provide inputs for the mentioned considerations, numerical modelling was extensively used for testing different mining plans. Different numerical models were established in both 2-D and 3-D. In the 3-D models, the entire mine was reconstructed numerically, and the future mining alternatives were numerically performed in order to pre-investigate advantages and disadvantages for each alternative from rock mechanics point of view.

Caveability of the hanging wall of the mine

In sublevel caving, caveability of the hanging wall is one of the important issues for the applicability of the mining method. Typical displacement around a mine with sublevel caving method is presented in Figure 5 – left hand side. In this figure, the zone below the limit angle line is considered to be stable and no displacement caused by the mining activity should be presented in this zone. Meanwhile, zone above the break angle line is considered to be the unstable zone with various types of instability modes such as visible cracks to completely caved rock. The zone in between these two lines is a transition zone.

The zone above the break angle line is called "Discontinuous Deformation" (DD) zone, which consists of Fractured Zone (FZ) and Caved Zone (CZ). Analyses performed in this paper are aiming to find the DD zone with different subsequence sublevel caving levels. The analyses follow similar analyses done for Kiirunaavara mine by Villegas and Nordlund.

To evaluate the displacements of the hanging wall at Rana Gruber, a Phase2 (Rocscience Inc., 2005) model is established and simulated as shown in Figure 5 – right-hand-side and Figure 6. The procedure for the work is as follow:

- Step 1: Collect information for the inputs, such as in-situ stresses and rock mass properties. Many stress measurements were carried out for this mine since 1970 by SINTEF. In addition to that many laboratory tests and rock mechanics analyses were carried out for the mine. Thus, the in-situ stresses and the rock mass behaviour of the mine are relatively well defined;
- Step 2: Pillar removed, mine bottom at level 280, top of waste rock at level 350 m. This is to simulate the current situation in the western part of the mine. Results from this model will be compared with site observations for verification and necessary adjustments;
- Step 3: Pillar removed, mine bottom at level 250, top of waste rock at level 350 m;
- Step 4: and subsequence: perform the gradually sublevel caving mining activity as they are planned;
- Surface monitoring system based on periodic total station and GPS measurements were installed and followed-up. The monitoring and surface crack mapping were used for model verification.

Evaluating different mining layouts for sub-level caving

When changing the mining method from sublevel stoping to sublevel caving, Rana Gruber considers several layout alternatives for the new mining method. With the sublevel caving method, the mine will be mined from level 280 m.a.s.l. (meter above sea level) to level 123 m.a.s.l.. The total thickness between these two levels is almost 160 m. Production drifts are arranged in four levels 221, 187, 153, and 123 m.a.s.l. There are many detailed layout proposals, however only a few of them are selected to be presented in this paper. The alternatives presented in this paper are (i) mining from two ends toward centre, one level at a time; (ii) mining from two ends toward centre, several levels simultaneously with certain delay between levels; (iii) mining from centre toward two ends, several levels simultaneously with certain delay between levels. Example of one of the mining alternatives is presented in Figure 7.

The objective for the numerical analyses is to identify advantages and disadvantages for each mining alternative. Results from the simulations will form a part of the decision basis for selecting the optimum mining alternative for the mine. The analyses focus on the overall stability of the mine. FLAC-3D program (Itasca, 2009) is used for the modelling.

Three mentioned mining alternatives are simulated. In order to highlight advantages and disadvantages of each mining alternative, principal stress values in the foot wall,

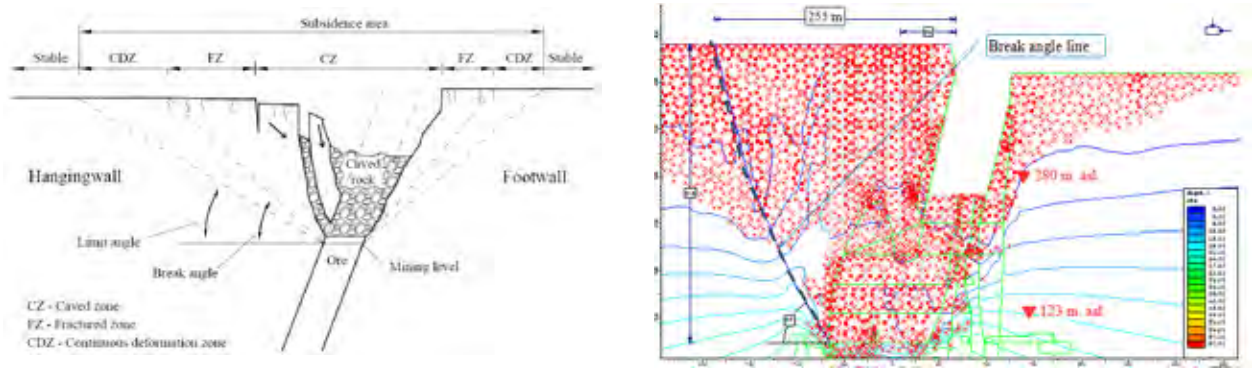


Figure 5: Surface deformation zones at a mine with sublevel caving mining method in the left-hand-side (Lupo, 1996 and Villegas and Nordlund, 2008) and 2D model for Rana Gruber in the right-hand-side.



Figure 6: Results of 2D model for Rana Gruber for different mining levels.

at some important locations in the infrastructure (main access ramp, transportation drifts) are collected and plotted for every simulation step. This stress development will be studied and evaluated to see when the infrastructure is in "in a dangerous condition". The criteria to define an "in a dangerous condition" for the infrastructure are when the following conditions are both fulfilled:

- The differential stresses (Sigma 1 – Sigma 3) are high; and
- The magnitude of the minor principal stress (Sigma 3) is reduced significantly.

A typical stress result for one of the simulation is shown in Figure 8. Stress results of the simulation case number 1 (mining one level at a time, starting from two ends toward centre) are shown in Figure 9. In the figure, the vertical axis is the stress value, and the horizontal axis shows the mining steps. The three digits following the "SIM3-" in the name of the mining step denote the mining level. The one or two digits after the mining level denote the mining step for that level. According to the mining configuration, 13 mining steps are required to complete one mining level.

According to the criterion and the graphs in Figure 9, the infrastructure at level 221 is "in a dangerous condition" at mining step SIM3-187-12 or 13. At this stage, the minor principal stress is reduced to almost zero and the differential stress is significant (about 18 MPa). The stress condition indicates that the infrastructure at level 221 may well be in yielded zone in this mining step (SIM3-187-12 or 13). It can also be seen that after step SIM3-221-13, the Sigma 1 is at maximum (about 27 MPa) and the Sigma 3 is low (only about 2 MPa). This may indicate that the access in level 221 can be "in dangerous condition" already from this step. However, at both mining steps (SIM3-221-13, SIM3-187-12), the mining activity at level 221 is already completed. According to the mining plan, the infrastructure at level 221 will not be used any more after mining step SIM3-221-13, thus, it is not a problem whether or not it is "in dangerous condition".

Similar stress development for other mining alternatives are collected, plotted, and evaluated in the same way. Results from the analyses suggest that to enable a safe mining development, the following mining alternatives are recommended:

- Mining is preferred to take place at one level at a time
- Mining more than one level at a time should only be done with 80 m lagging or more, commencing at two ends progressing towards the centre of the ore-body;
- Mining from the centre of the ore-body towards its two ends is not recommended even with 80 m lagging or more

As mentioned earlier, in order to increase the reliability of the model so that it can serve as a reliable planning tool, a following-up program to verify the

numerical model is established. In order to perform the following-up program, 2-D stress sensors and extensometers were installed in different locations of the mine to record the stress changes and displacements. A total of 11 of the 2-D stress sensors and 7 extensometers have been installed so far at key locations in the mine monitoring the stress changes and displacements. Many verification analyses have been carried out, and a typical comparison of stress was made as shown in Figure 10. As can be seen from the figure, the model predicted that at certain mining steps, the stress in the drift can reach up to 45 MPa and this prediction is fairly consistent with the recorded value in a corresponding 2-D stress sensor. Results from the instrumentation support the conclusion that the results from the numerical models are reliable.

5 CONCLUSIONS

By combining measurement & laboratory – numerical modelling – monitoring, SINTEF is able to provide a practical tool for Rana Gruber during their planning process. Results from the numerical models played an important role in providing necessary information for decision making. The mine was successfully changed from sublevel stoping to the sublevel caving method as planned, and the monitoring equipment installed in the mine gradually shows that the results from the numerical models are realistic.

6 ACKNOWLEDGEMENTS

The authors wish to thank Rana Gruber AS for permission of publishing information in this paper.

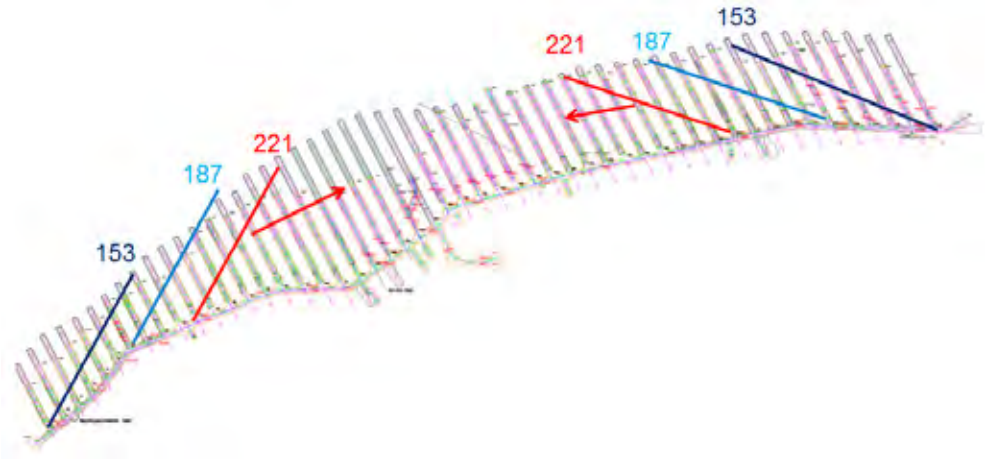


Figure 7: Mining alternative with mining different levels at the same time and mining from two ends towards centre.

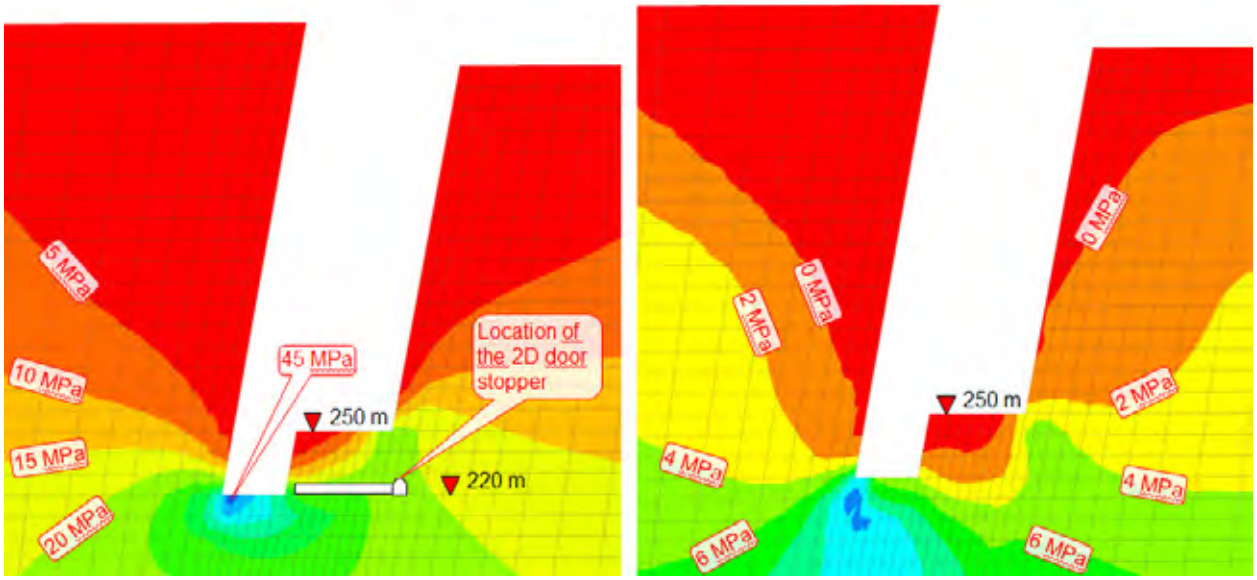


Figure 8: Stress condition near the infrastructure (10 m into footwall) at level 221 in an intermediate mining step in this level.

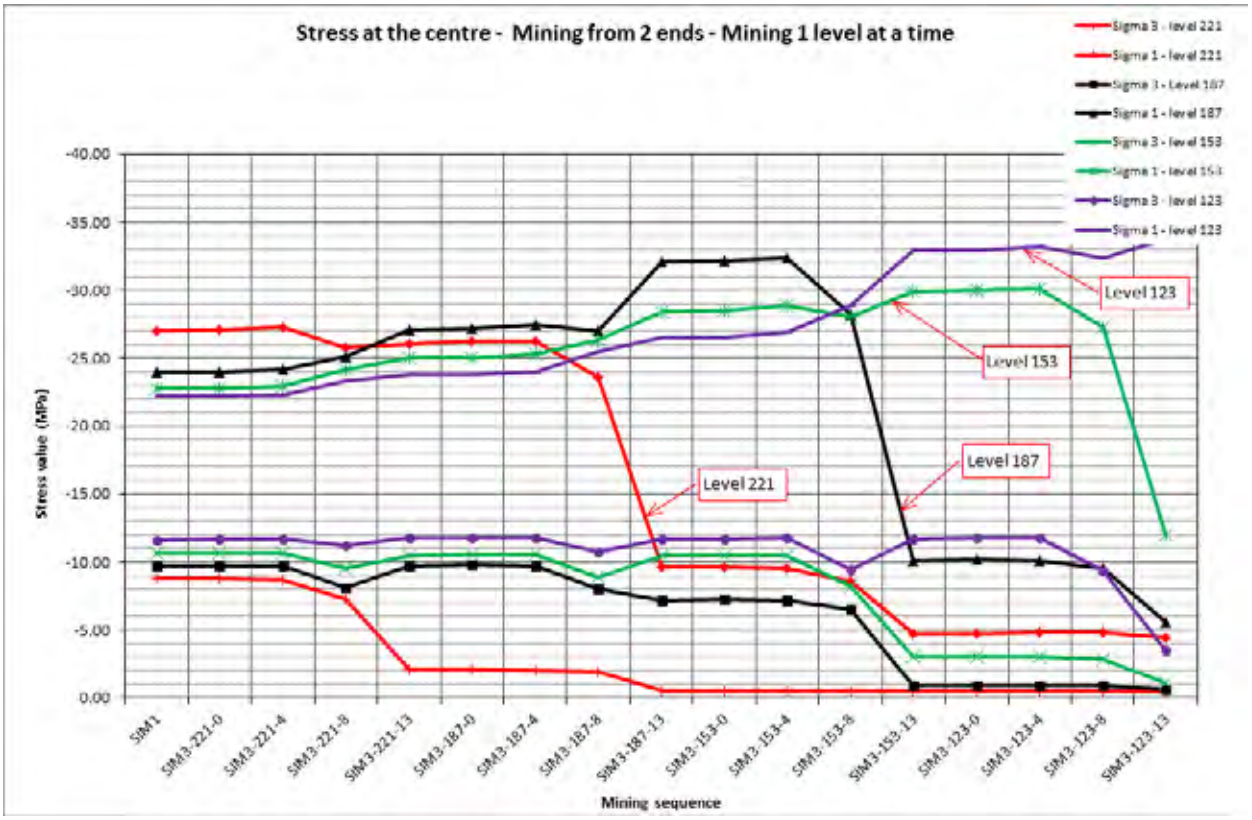


Figure 9. Stress condition near the infrastructure (10 m into footwall) at different levels in relation to different mining steps.

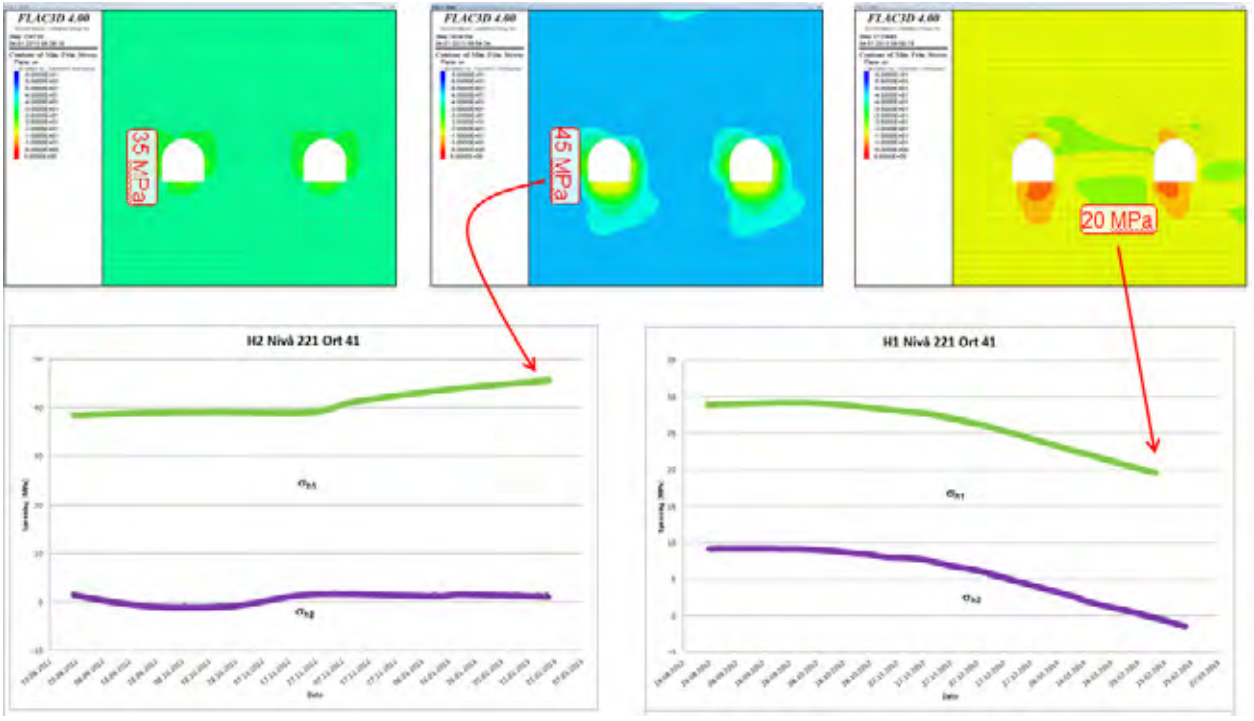


Figure 10: Results of the numerical model for different mining steps comparing to the recorded data from a 2-D stress monitoring.

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Sture crude oil caverns
Courtesy: Norsk Hydro ASA

05. EXPERIENCES FROM UTILIZING NORWEGIAN PROJECT EXECUTION PRINCIPLES IN JRC ROCK CAVERN PROJECT - SINGAPORE

FAGERVIK, Finn

Project Management has for decades been considered as the most valuable contribution to the success of a project. Despite this accepted statement, Project Management has up to recently been identified with the person named Project Manager and his technical competence in the specific type of project, which is still the situation in Singapore. Complexity of projects and requirement for timely delivery motivated the Norwegian Offshore Oil & Gas industry to establish management tools to support the Project Manager to succeed, and the Project Execution Model became mandatory.

This article elaborates the experiences of implementing such Project Management principles in the Jurong Rock Caverns project in Singapore. Due to limited length of this article, we apologise for not having been able to detail the principles specifically. However, we trust the article will contribute to further interest for Project Management Execution Models, and how cultural awareness is not only related to geography.

Keywords:
Jurong Rock Cavern, JRC, Project Management, Project Execution Model (PEM), Zero-Philosophy, Planning, Risk Management, Culture Awareness.

1 JURONG ROCK CAVERNS (JRC)

Phase 1 Jurong Rock Caverns (JRC) project is located at Jurong Island beneath Banyan Basin in Singapore involving the construction of rock caverns and associated tunnels to store 1.47 million m³ of hydrocarbon products. The project also includes the construction of two access shafts, associated aboveground and underground facilities, including a new Multi-purpose Product Jetty. Budget is approximately one billion SGD. Date for starting Phase 2 is not yet decided.



Figure 1: Location of Jurong Rock Cavern

2 JRC LAYOUT

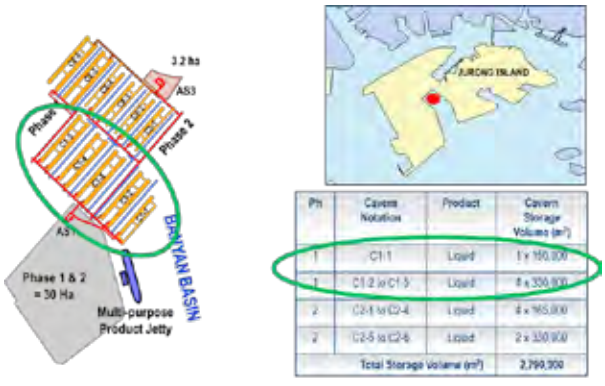
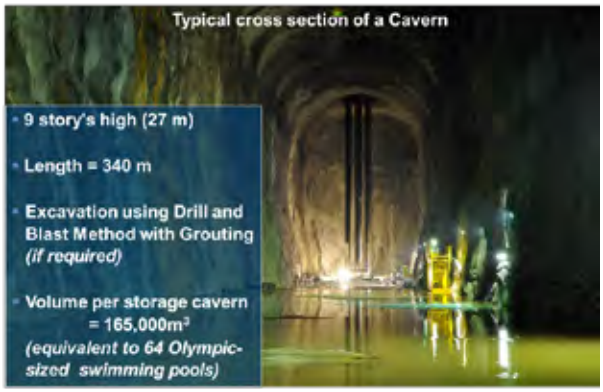


Figure 2: JRC layout



3 JRC KEY MILESTONES

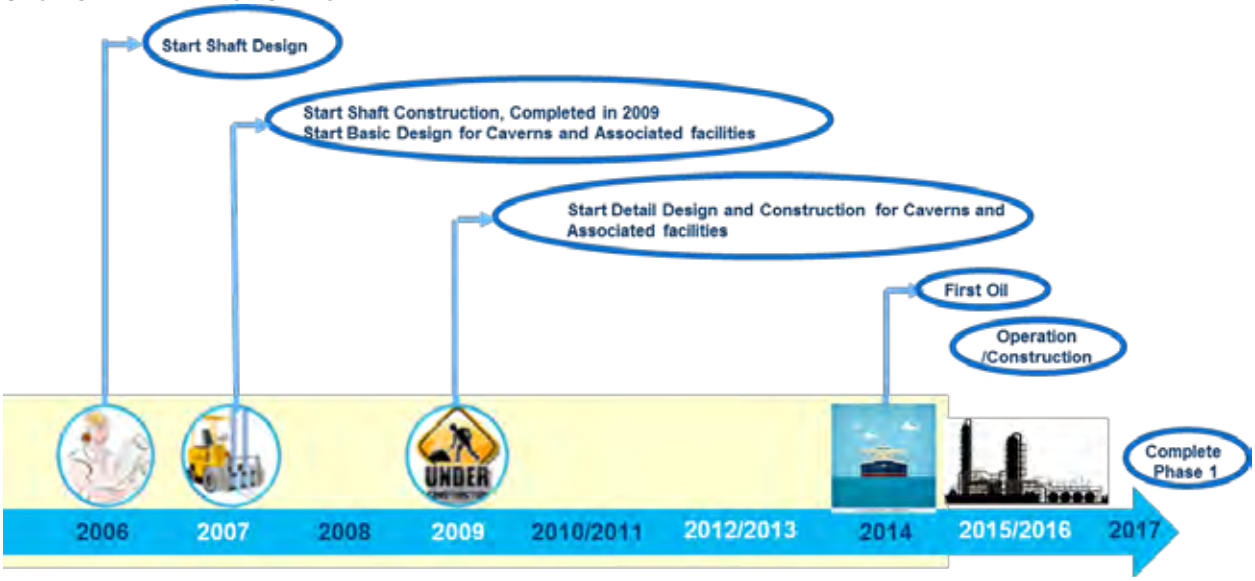


Figure 3:JRC Key Milestones

4 STM – THE PROJECT MANAGER

STM is a cooperative entity established jointly by SINTEF, Tritech and Multiconsult in 2006 for providing project management on behalf of JTC Corporation, Singapore to the Phase 1 Jurong Rock Caverns Project. SINTEF is a Norwegian research institution and one of the largest in Europe.

TRITECH is a listed company in Singapore, specialising in civil, geotechnical and rock engineering.

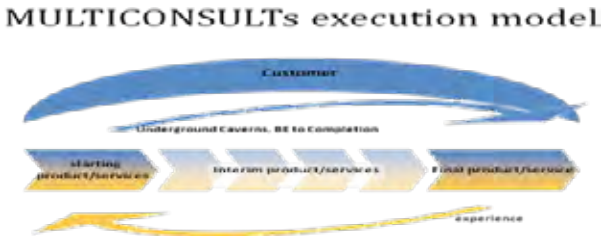
MULTICONSULT is a leading multi-disciplinary consulting and engineering company listed in Norway, specialising in Buildings, Oil & Gas, Transportation & Infrastructure, Industry, Energy, and Environmental.



Figure 4: STM, the project manager

5 STM PROJECT MANAGEMENT PRINCIPLES

The STM Project Management principles, hereafter called the "STMPEM", is based on Multiconsult Project Execution model, which was developed during early 2000 and based on more than 20 years of experiences from Norwegian Project Execution Models originally developed for Offshore Oil & Gas projects. It is not possible to describe this in this article, hence only some of the most essential parts contributing to the STM Management Team's success, are mentioned. It is essential to underline that since implementing the STMPEM ten years ago, new technology has enabled extensive improvements, which may make our achievements look like minors.



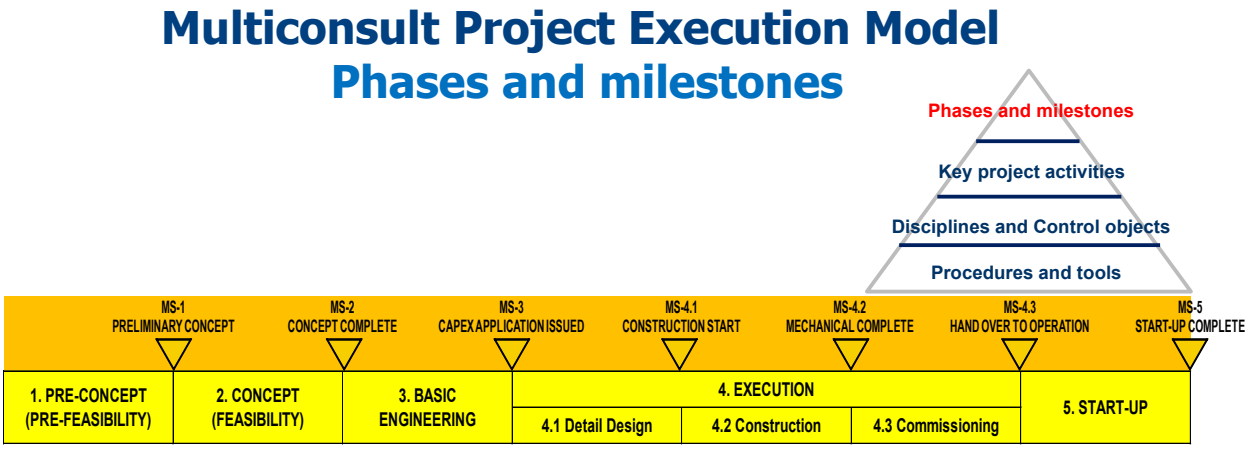


Figure 5: STM Project Management Principles

6 STMPEM IMPLEMENTATION CHALLENGES

The establishment of the STM Project Management Team (PMT) included taking over already started project activities. When we started implementing the STMPEM, the PMT needed to evaluate and consider the status of activities such as:

- Status of started/ongoing activities.
- Contractual agreements and implemented routines.
- Time span prior to commencement of start of new activities.
- Training of the PMT, adjusting of procedures, and Culture Awareness.
- Client, Consultant and Contractors acceptance of management tools, terms and routines.
- Potential contractors' capability to respond to planned new tender requirements.
- Existing and new contractors and consultant's capability of project management through project execution models.

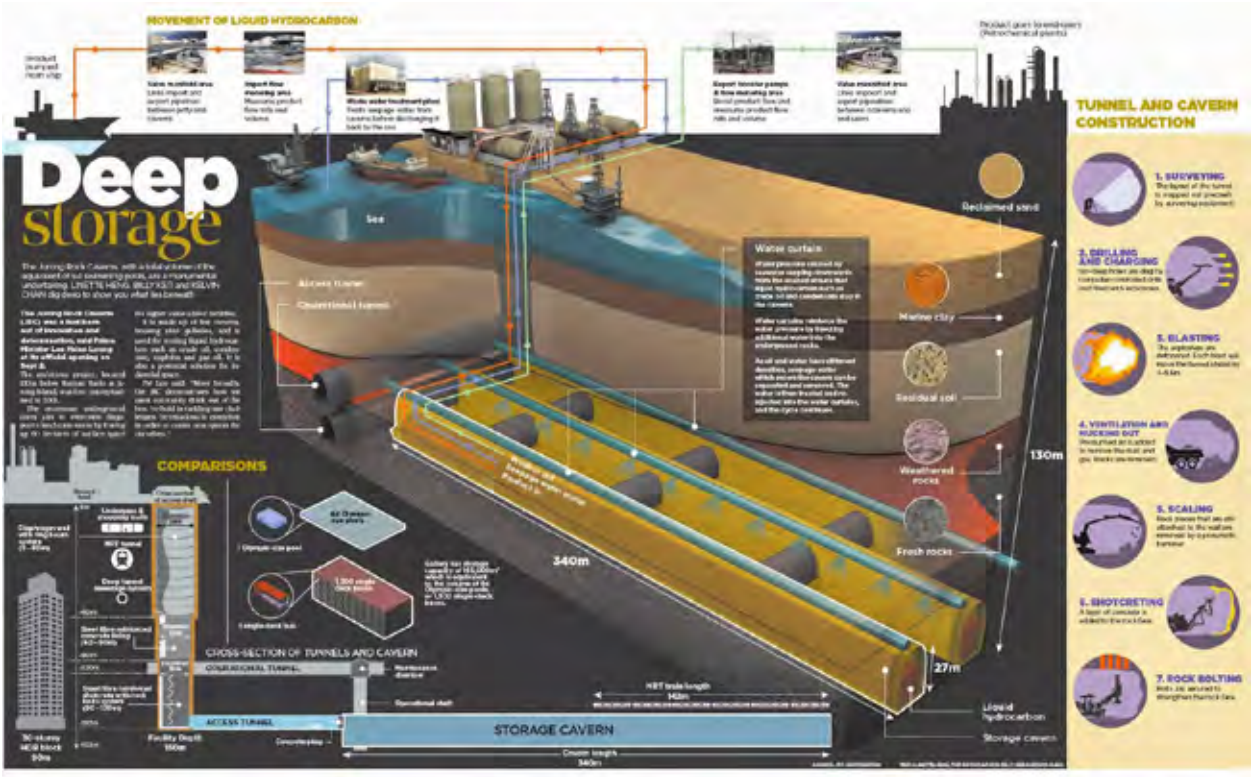


Figure 6: Project overview

During this establishing of "Status", we realized that Singapore project culture was much more unlike Europe, than we had been able to plan for. We recognized that the Multiconsult PEM as adopted in previous projects could not be implemented in the project as planned, and a new challenge was brought to the table. The STMPEM had to be redesign to fit into existing local project execution practice and culture. We realized that our mindset had to be changed from assuming we could "instruct" the JRC Project to adopt the STMPEM. Implementing one culture into another culture cannot be achieved successfully without accepting that this takes time and that several routines could never be accepted. The challenge was how this could be changed without losing the benefits of the STMPEM, including how we now could ensure to meet the forthcoming project milestones. Based on above the restructured STMPEM ended briefly as:

- 30 day Start-Up Plan.
- Web based Quality Plan.
- HSE & Risk Management.
- CTR based Project Control system.
- Work Breakdown Structured planning system.
- Review, Surveillance & Monitoring procedures.
- 3D design system integration.
- Management through meetings.
- Web Hotel based document control system.

7 STMPEM CONTRIBUTIONS

The structured generic 30 days plan, enabled a clear and professional communication with the Client and trust was established for STM capability to achieve demanding deliveries/milestones during the first months, which when achieving the milestones successfully, the trust for the years to come was established.

The WEB Based Quality System was at that point of time (2006), a "State of the Art" system. The effectiveness of this system enabled internal communication between locations all around the world. The system eliminated time zones and geography, and made 24/7 availability to information for Client, Consultants and Contractors, which never before had been possible for this type of projects in Singapore.

JTC had in the Tender specification for the new contracts to be tendered, required a high focus on HSE. Based on this strong JTC HSE commitment, STM presented and implemented the requirement of Zero Injury and Impact based on what we in Norway had been practising in the Offshore Oil and Gas industry, widely named as the Zero Vision Philosophy. These practical requirements became mandatory for the entire Project.

An aligned Risk Management System has ensured a systematic follow up of all types of risks and uncertainties throughout the entire project from early planning and design phases to construction, commissioning and Start-Up. This includes technical and progress challenges enabling corrective action to take place in due time.

8 SUMMARISING OF ACHIEVEMENTS

Implementation of the Norwegian Project Management principles in the Design and Construction of Phase 1 Jurong Rock Cavern Project, have been much more challenging than expected. However, the good cooperation between JTC and STM has overcome the challenges in a productive way and the value added from the STMPEM to the JRC project, is considered extensive. In addition to the examples of yearly achievements listed below, availability of electronic information, transfer of technical competence, are underlined as success factors. The extreme positive HSE achievement, e.g.; the LTIFR (Lost Time Injury Frequency Rates) has since STM mobilised the PM Team for the Shaft Contract, been improved from 4.6 to end 2015, after more than 25 million man hours below 0.5. Singapore average is 2.2.

For STM internal performance, the STMPEM has also been challenging. At a certain point of time, the STM Team consisted of personnel from six different countries with different culture and background. Based on extensive transfer of Norwegian experiences and daily supports during the first years, and positive attitude in the team, the implementation of the STMPEM became successful.



Figure 7: Access via shaft, JRC



Figure 8: Rock cavern for hydrocarbon products, 165.000m³

For those parts of the Norwegian Project Management principles that we found not possible to implement, the sheer awareness of the importance of not being implemented, resulted in successful mitigation activities performed through systematically daily surveillance.
Below is listed some examples of yearly achievements considered as a result from the STMPEM and transfer of knowledge and experiences from Norway:

2006: Prepared contract requirement for the BEDCM Contract introducing the Zero Injury and Impact Philosophy.
2007: Client's acceptance for risk shearing principle for the underground works when establishing the Design and Build Tender.
2008: Sato Kogyo (the Shaft Contractor) acceptance to change their grouting technique to "European" method.
2009: Sato Kogyo acceptance to introduce "Cycle Planning" on Level 6 Planning, to improve overall progress and productivity.
2010: Design and Build Contractor's acceptance to prepare detailed overall ventilation planning considering full flexibility in faces, including later MEI work.
2011: Design and Build Contractor's acceptance to include contingency planning as part of Under Ground excavation planning.
2011: Design and Build Contractor's acceptance to change Under Ground layout to avoid areas with adverse conditions.
2012: Design and Build Contractor's acceptance to start planning construction activities based on how operation would affect construction and vice versa.
2013: Clients acceptance to implement Norwegian progress mitigation methodology to meet the milestone date for import of First Oil, (Hot Plant).
2014: Risk Management activities to ensure merging of the phases could be performed without affecting operation.
2015: Design and Build Contractor's acceptance for how to transfer experience from start-up of Stage 1 to reduce risk for Stage 2.

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Svalbard Global Seed Vault
Three underground caverns and an entrance tunnel constructed in permafrost. Natural temperature -3°C permanently cooled to -18°C. Huge number of samples from all continents safely stored. Courtesy: LNS AS



06. ROCK CAVERN IN PERMAFROST ON SVALBARD NORWAY TO HOUSE THE GLOBAL SEED VAULT

BARLINDHAUG, Sverre

ABSTRACT

The Norwegian government has built a Global Seed Vault for the International Treaty on Plant Genetic for Food and Agriculture on the arctic island Svalbard. The storage consists of 3 underground caverns excavated in weak sedimentary rock and located about 40 m below the surface.

The cold climate on Svalbard makes all soil and bedrock permanently frozen down to 200 m below the surface. The permafrost was utilized during construction as soil and weak rock could be excavated vertical and thus limit the excavated volume. The permafrost is also helpful for the long term stability for caverns excavated in weak rock.

The temperature in naturally frozen ground is 5 °C below zero and is favorable for seed storage. However the temperature in the Global Seed Vault will be 18 °C below zero. There is no insulation in the caverns and a large amount of frost will be stored in the surrounding bedrock.

1 INTRODUCTION

The arctic island Svalbard is located close to 80o north (figure 1). Norway has been mining coal on Svalbard for about 100 years. Most of the bedrock on the island is weak sedimentary rock and due to the cold climate the soil and the bedrock is permanently frozen down to about 200 m below surface. During the few "hot" summer months only a little more than 1 m is thawed.

For more than 20 years Norway has used abandoned mine openings to store seeds from Scandinavia as the permafrost creates good storage conditions for the seeds. When the International Treaty on Plant Genetic Resources for Food and Agriculture was adapted by the United Nation (FAO) in 2004, they looked to Norway and Svalbard for a possible Global Seed Vault. The Norwegian government agreed to house the facility and in 2006 they finalized a location study that ended up with a site close to the main Norwegian settlement on Svalbard, Longyearbyen (figure 2).



Figure 1: Arial Map (Google.com)



Figure 2: Detail location (Photo: Sverre Barlindhaug)

In November 2006 the Norwegian Directorate of Public Construction and Property awarded Barlindhaug Consult and Multiconsult the contract to complete the planning and the detailed design of the underground storage for global seeds. Less than one year was given to finalize both planning and construction of the whole facility.

2 GEOLOGY

Due to the tight schedule to complete the project forced us to depend mainly on our own experience from other projects in the area. Geological investigation was therefore limited to a study of available geological maps. The given location had mainly focused on acceptable access. The detail site selection had to focus on geology as the caverns had to avoid coal layers in the bedrock. Coal formations could contain methane gas and create problems during the construction work.

According to the geological map the selected site for the caverns was in a rock formation with layers of sandstone and mudstone, and below any known coal layer. A tunnel project had been excavated through the same rock formation a few years earlier. In that project the different layers of sandstone and mudstone varied in thickness from a few cm up to 2 meters. The sandstone had medium mechanical strength while the mudstone was so weak that the rock mass started to decompose soon after it was exposed to air.

At the given location the bedrock was covered with moraine and there was no available information about the actual thickness. During the planning stage we assumed that the moraine cover could be about 8 meters.

3 PLANNING STAGE

The rock caverns were planned at a depth where the permafrost would not be affected by the annual temperature variation at the surface. No detail investigation had been carried out to identify this optimal location for stable permafrost. According to earlier experience the cavern should be about 40 m below surface and the temperature in the permafrost at this level was estimated to about 5 ^{o}C below zero the year around.

The Global Seed Trust had up to now used normal freezers around the world for seed storage where the temperature is 18 ^{o}C below zero. The same temperature would be used in this underground storage and additional freezing was therefore required. The Global Seed Trust had suggested an insulated storage erected inside the rock cavern. We recommended caverns with exposed rock surface. The energy required to create low temperature would however be higher in a cavern with exposed rock compared to cavern with an inner insulation. But the cost of this extra energy would however be less than the combined construction cost of larger caverns and the erection of an insulated building inside the cavern.

There is also an important side effect by letting the rock surface be exposed to cold temperature. The frost zone will travel far into the rock mass. To convince the owner a modeling study was carried out to see if the 18 oC below zero in the cavern had any effect on the rock mass outside the cavern after 50 years. The modeling showed that with an insulated storage inside the cavern, the rock mass 20 m from cavern would hardly be affected by the low temperature inside the cavern (figure 3a). With only exposed rock in the cavern the rock mass in the same distance from the cavern showed close to 12 oC below zero (figure 3b). This means that without insulation a large amount of frost will be stored in the surrounding rock. The large amount of frost in the surrounding rock will result in less energy to maintain the low temperature in the future. If any long term disturbances in the energy supply occur the frost stored in the rock masses will help the caverns to maintain low temperature for months.

Thick layer of moraine was expected above rock surface in the area and made a special challenge to the design of a safe and functional entrance. The thick moraine overburden will require a large open cut excavation to achieve sufficient rock cover for the tunnel. As there is no avalanche risk in the area the open cut could have remained open, but on Svalbard such opening could represent serious problem during winter as snow drift frequently will fill up the open cut. Large open cut was however not wanted for environmental reasons.

To obtain a free access to the storage all year round a steel tube was chosen from surface to a tunnel portal in rock. To bring the terrain back to original shape the whole open cut above the steel tube, should be backfilled. At the end of the steel tube a monumental concrete structure will work as entrance to the seed vault.

4 CONSTRUCTION

As the rock quality and the overburden thickness was not known in detail at the planning stage, adjustments were expected during construction. We were also allowed to do the necessary changes according to actual ground conditions. To achieve the optimal project for the users, close supervision was required during the whole construction period.
The construction work started in April 2007 (figure 4a) and was carried out by LNS a Norwegian contractor with

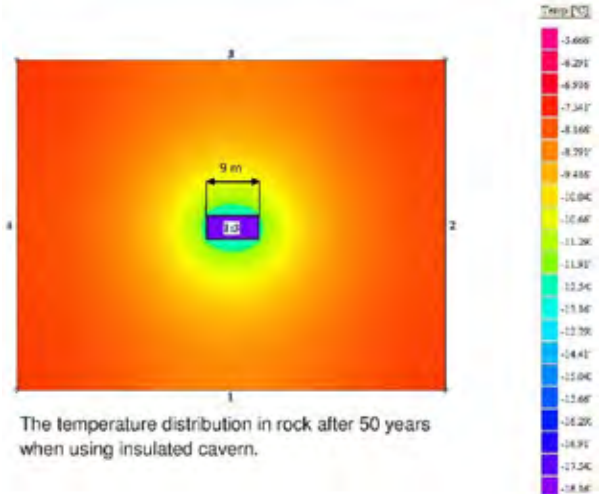


Figure 3a: Temperature in the surrounding rock with insulated cavern (Multiconsult)

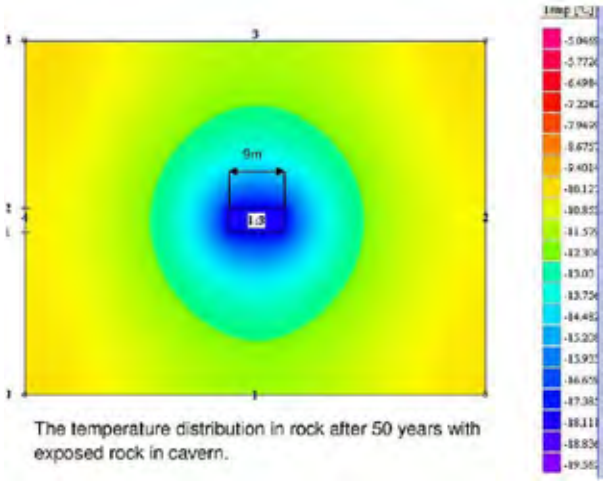


Figure 3b: Temperature in surrounding rock with exposed rock in the cavern (Multiconsult)

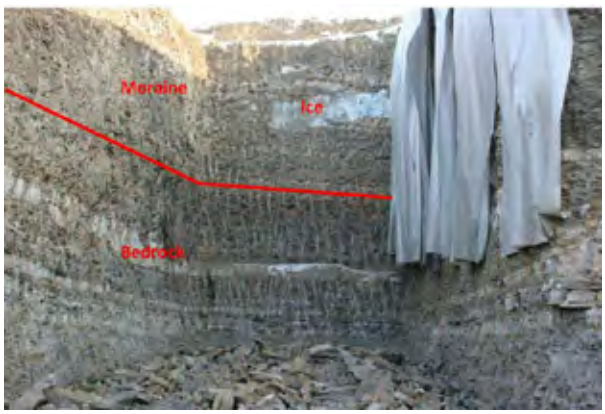
long experience in tunnel works on Svalbard. All open cut work was completed when it was still full winter and completely frozen soil and rock. The whole open cut was excavated by use of ripper and hydraulic hammer.

As all materials were frozen during excavation even the soil could be excavated vertical in the same way as the bedrock beneath. Layers of ice were found buried in the moraine.

According to our estimate of the overburden thickness the open cut was planned to be 40 m long where the steel pipe was going to be erected for permanent access and backfilled. The overburden however turned out to be thicker than expected and we were therefore forced to either make a longer open cut or an inclined access. The open cut was kept 40 m long as originally planned, but had to be with sloped invert. The total height of the open cut at the tunnel portal was then about 20 m (figure 4b) more than 4 m higher than planned.



Figure 4a: Construction site in April 2006 (Photo Sverre Barlindhaug)





4b: Open cut excavation in moraine and weak rock (Photo Sverre Barlindhaug)

For the permanent use of the seed vault only a small access was required. The contractor was therefore free to select the diameter of the access tunnel and the steel tube that was required for the tunneling equipment. They decided to make the access 6 m wide. The open cut was then 10 m wide to have space for proper back-filling around the tube.



Figure 4c: Cross section (Barlindhaug/Multiconsult)

By making the whole slope in the whole open cut vertical the excavated volume could be kept to an absolute minimum, but the excavation had to be completed while the air temperature was below zero. As the open cut was facing north the heating from the sun should normally be a minor problem. But on Svalbard with a midnight sun just as strong as the sun during daytime, we faced problems during night shift. The "heat" from the sun resulted in thawing of the surface material in the open and resulted in rock fall. Just after the open cut was completed and the tunnel work was going to start, the steel tube had to be erected and back-filled to obtain safe access to the tunnel.

Tunnel excavation was carried out with normal drill and blast method. The tunnel excavation confirmed the expected rock quality with layers of reasonable strong sandstone and weak mudstone. The thickness of the different layers varied between a few cm and up to 50 cm. The foliation was close to horizontal.

Normally equal spacing between the perimeter holes and gently blasting is required to obtain a smooth surface. In this project it was more important to place the perimeter holes in sandstone. If the blast holes were set in mudstone all blast energy was absorbed in the mudstone and there was hardly any effect on surrounding sandstone.

Just after blasting the rock surface was still frozen and remained stable, even the weak mudstone. Shortly after the mucking operation had been completed, heat generated from all the equipment and the "hot" fresh air, made the rock surface thaw and layers with the soft mudstone started collapsing. This resulted in a flat roof unless the rock support was installed immediately.

The rock support contains a combination of shotcrete and fully grouted rock bolts. Normally shotcrete support would not be suitable on frozen rock surface, but the heat generated from the equipment and the "hot" fresh air, made the rock surface sufficiently warm for hot shotcrete to cure. The rock support is however a temporary requirement until the permafrost is reestablished. In permanently frozen ground the rock caverns would be stable even if the rock is weak. A light collared coating was sprayed on top of the final shotcrete layer to create a brighter surface.

The initial design had a 60 m long access tunnel and 2 caverns for seed storage, each 15 m wide and 25 m long. As we experienced stability problem in the 6 m wide access tunnel, larger problem were expected in a 15 m wide cavern unless extensive and expensive rock support were installed. The span in each cavern was therefore reduced to 9.5 m and extended to 27 to maintain the same storage area (figure 4c).

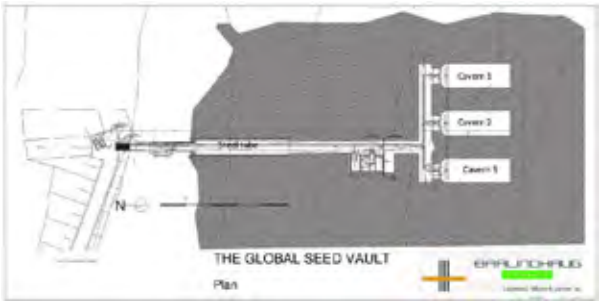


Figure 4d: Plan (Multiconsult/Barlindhaug)

By reducing the width of each cavern the roof was also lower. This design change saved an excavated volume which was nearly equal to one complete 9.5 m wide cavern. The Global Seed Trust had assumed more storage would be needed in the future. As the new design had saved volume we recommended using this saving to make an extra cavern while the excavation was going on. This extra excavation was almost free of charge in the present contract.

In the beginning only one of the caverns will be used and it may probably take close to 20 years to fill it up. The freezing process was therefore only required in one of the caverns. The freezing started in cavern 2 the middle one (figure 4d). When the next cavern will be put to use, the temperature in the surrounding caverns would then probably have reached close to the correct level.

With the global seed vault located at Svalbard it is expected to be on a very peaceful place on earth. In the planning we had however also foreseen a possible grenade attack. Therefore none of the caverns are in the line of the access tunnel. A small hole is made in the



Figure 4e: Outside and inside cavern 2 (Photo Sverre Barlindhaug)

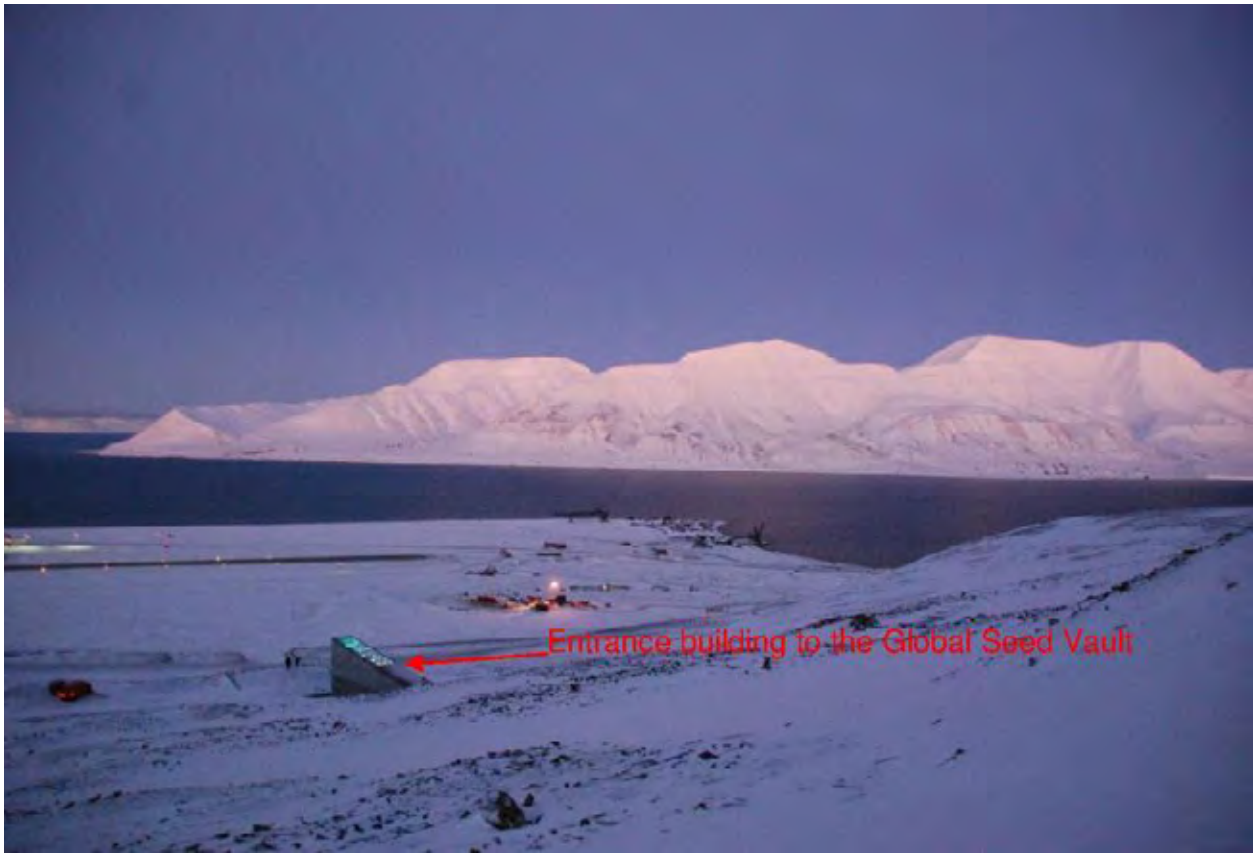


Figure 5: The portal structure in a wide landscape (Photo Sverre Barlindhaug)

wall in the end of the access tunnel. Any missile send in through the access tunnel will therefore not harm any of the seed vault.

5 ACKNOWLEDGMENTS

The project constructed for the Global Seed Vault has no impressive dimension especially not from outside (5 1), but it has a great importance for humanity. We could sens the global interest during the design and construction stage from international media, politicians and environmental organizations. International journalists visited the site frequently. When the construction work was completed and the freezing process could start, National Geographic called this event one of the most important this century.



Storage caverns, Boliden Odda AS
Courtesy: Anne Hommefoss

07. UNDERGROUND WASTE DEPOSITING CONCEPT AT ODDA

HOMMEFOSS, Anne
ARNESEN, Frode

ABSTRACT

Previously the jarosite residue was discharged directly into the sea near Odda. This created a big pollution problem in the fjord and it was necessary to develop an alternative method for disposal of the residue.

Today the residue is stored in rock caverns approximately 2 km north of the zinc plant, Boliden Odda AS. The storage operation has been carrying on since 1985, and experiences from construction and operation has contributed to maintaining a high level of safety and security while lowering the cost per unit store volume. During the 30 years of operation, a continuous optimization of the design with regard to excavation cost and storage efficiency has taken place, giving higher and wider caverns and introducing new transport tunnels. The design of the caverns has been adapted to the rock quality and stresses. During the construction phase, systematic approaches to training and reporting and a co-operative approach to safety and environmental issues has been successful.

1 INTRODUCTION

The Boliden Odda AS zinc refinery plant is located in the southern end of Sørfjorden. Sørfjorden is a side branch of the Hardanger Fjord in the southwest of Norway.

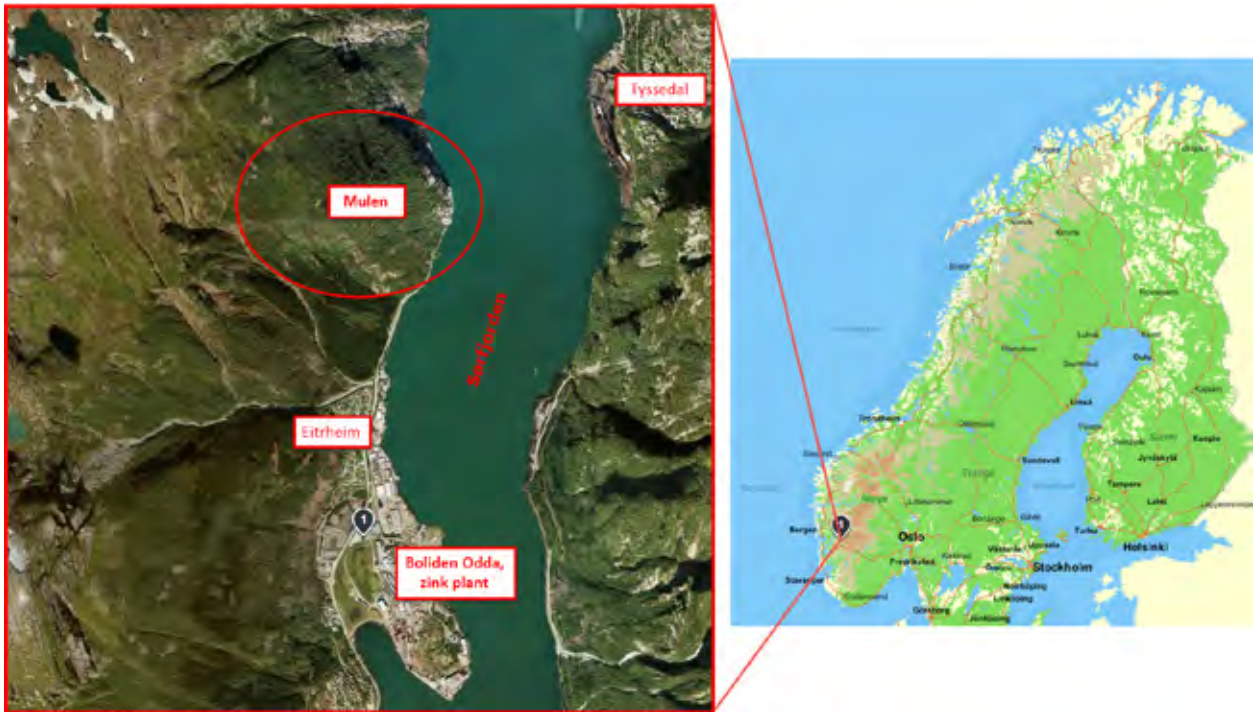


Figure 1: Location of Boliden Odda zinc plant, and the area of the rock caverns (Mulen). (http://kart.gulesider.no/).

Boliden Odda was founded in 1924 and is located in the west coast of Norway. About 55 percent of the raw material comes from Boliden's mines in Sweden and Ireland, the rest comes from mines in Canada and Peru.

Previously, the waste from zinc refining was discharged to the fjord suspended in water to a dry substance content of 10%. With the addition of other industrial waste products from the area being discharged to the fjord, this created severe pollution problems in the inner part of Sørfjorden. Due to the pollution problems, the State Pollution Control Authority asked Boliden AS (formerly named Norzink AS) to develop an alternative method of waste disposal. After 1st of July 1986 no discharge of residue to the sea was permitted.

Studies for alternative disposal of waste from the zinc production started in 1975. Depositing in rock caverns was the most attractive alternative among several other alternatives considered, as solidification in cement and other chemical treatment methods

The residue from the production consist of the following main constituent:

SO4	30%
Fe	20-28%
Zn	3-5%
SiO2	2-6%
Pb	2-3%
Cu	Less than 1,0%
CaO	
MgO	
As	
Sb	
Cd	
Ag	

The Boliden Odda Zinc smelting plant produce pure zinc, zinc alloys, sulphuric acid and aluminum fluoride. In 2014 they processed 302 kilotonnes zinc concentrate, and produces 166 kilotonnes zinc metal. And produced 123 kilotonnes sulphuric acid and 35 kilotonnes aluminium fluoride.

93% of the production at Odda is exported, mainly to steel industry in Britain, France, Germany, Benelux and Scandinavia. Boliden (with the two zinc plants; Kokkola in Finland and in Odda, the annual production exceeds 450.000 metric tons of zinc) is the world's fifth largest producer of zinc metal from smelters and the eighth largest producer of metal in concentrate from mines.
Boliden Odda employs 321 people (in 2014), and is the largest private employer in Odda municipality.

In 2013-2014 the two largest caverns at Boliden was excavated. Both of them, hall 17 and 18, has a measured volume from 285.000 m^3 to approximately 300.000 m^3 solid rock. They are ca. 25 m wide, 60 m high and 250 m long.

2 LOCATION

There were several alternative locations for placing the rock caverns, and the rock formation approximately 1,5 km from Boliden Odda was selected. The selection of location and the design of the rock caverns are based on geological and hydrogeological evaluations while also considering the geographical and topographical aspects. The overburden varies from approximately 200-600 m above the caverns. The mountain rises with an inclination of 40° up to an elevation of 1000-1500 m above sea level.

At the industrial area, the solid waste mixed with water form a slurry, which is pumped through a 2 km long transport pipeline to the storage caverns. Today the storage plant consists of 18 rock caverns and about 1 km of connected tunnels.

3 GEOLOGY

Rock quality has been a key factor for location and design of the storage.

The rock formation is a massive, medium to coarse-grained monzogranitic gneiss, with occasional veins of pegmatite, amphibolite and granite of precambian age belonging to the Kinsarvik nappe from the Caledonian orogeny.

The foliation of the rock strikes in a NW direction dipping steep towards NE. Besides the joint system parallel to the foliation, a joint system occurs with strike in a NE direction and near vertical dip.

The geological conditions, as observed in the tunnel systems, increases in quality when mowing further north and west into the mountain. From coredrilling joint spacing oftens is between 1 and 3 meters, giving RQD value of 100%, and corresponding Q- values of 50 and above.

Rock stresses

The rock overburden varies from 200 m towards 600 m above the caverns. The Folgefonn peninsula with a large glacier have mountain tops of 1488 within 4 km from the site.

Measurements of the gravitational and tectonic stresses indicate a dominance of gravitation induced stress

from the high mountains, resulting in a major stress direction with an inclination of 50o to the horizontal, directed towards the deep fjord, combined by a weaker tectonic stress in the north-south direction. The stress level induced around excavated opening will inevitably lead to formation of new joints and instability. Proper design and orientation of the caverns and tunnels is important for the stability and the design of the rock support system.

Hydrology
The main reason for storing the material inside rock caverns is to prevent hazardous heavy metals from reaching the biosphere. Leakage models are established to analyze the drain pattern around the caverns. For monitoring the water regime, leakages and pollution to the fjord, piezometers, sampling and test wells are established. In addition, a measuring weir is installed at the outer end of the access tunnel for leakage measurements. The existing road tunnel in the area represents a complication of the system, but is also used as a sampling location.

Water samples are taken regularly from the sampling wells and selected locations at the seashore for testing, analysis and reporting. Chemical analysis is carried out regularly sampled and analyzed, and reported annually.

The results generally show that rock outside major joint system is practical without water leakage. However some major joint systems leads water and is connected to the groundwater table. Due to the hilly terrain, groundwater is fed from snow and rain at altitudes above 1000 m, ending at sea level. The results confirm that freshwater in the rock is present at lower level than seawater, and that there exist a positive gradient of water directed into the cavern. The low permeability of rock is proved by the fact that it has been possible to excavate a cavern with 55 m wide pillar, having no observed leakage from the adjacent caverns with 60 meters water head above the cavern floor. The results from the surveillance program confirms the effect of a more or less continuous inwardly directed hydraulic gradient, into the drained caverns. The water from the mountains, 1000 m above, are distributed along the surface and by surface parallel joints and is acting as a water barrier between the caverns and the sea.

This effect is also taken in account when analyzing the long term environmental stability of the storage. The current plans show that very few additional measures is necessary for conserving the existing situation for the future after the eventual cessation of the zinc refining activity, and closure of the plant.

4 INVESTIGATION, MODELLING AND MONITORING

During the studies for location of the plant, in 1982, core drilling with Lugeon- and leakage testing as well as stress measurements were carried out to verify the rock conditions and the hydrological state of the rock formation.

For the 2nd generation caverns, the program included stress measurement at two sites, extensometer measurement of wall deformation during excavation of cavern no. 13, and a 120 m core into an area for future extension of the plant.

After an initial rock mechanical study with promising results, a new round of investigations were carried out.

Computer modelling by Phase 2Tm and other computer programs were carried out. The results from this showed that rock stress conditions were favorable for large cavern concept as shown by numerical simulations, based on geotechnical data acquisition and analysis, experience from caverns no. 13 and 14, back analysis and monitoring results. Analysis of different excavation phases show the roof stability is most critical during the early stage, at later stages have a reduction of tension stresses in the roof area.

The direction of the measured largest main stress governs the optimum orientation of the caverns. The mapped joint directions have less influence. The vertical high walls are the most critical areas with respect to stability during excavation.

Extensometer readings from cavern no 13 showed that the cavern deformations during excavation included only the outer 3 meters of rock adjacent to the wall. For later caverns, reference points in the cavern walls installed at the second level of blasting were regularly measured until completion of the cavern. A small increase of deformation for each level of blasting were registered. The total deformation was in the range of 5 to 11 mm. This confirmed that the bulk mass modulus are somewhat better than derived from laboratory testing.

First generation of caverns
The first generation of caverns (no. 1-12) are inclined, bottle shaped, with various lengths and with the lowest side at sea level at the inner end. The total volume of these caverns varies from 65.000 m^3 to 140.000 m^3. Excavation of the caverns are by utilizing conventional drill and blast method, top heading and benching. The rock support are mainly bolts, and a mini-

mum of shotcrete. A pipeline system in the cavern roof transports slurry into the cavern, and a concrete wall at the outer end equipped with valves catches particle-free overflow water for return to the slurry transportation process in a closed circuit back to the industrial plant and the process. The cavern volume also serves as auxiliary storage of excess polluted fluid from the purification plant for the zinc refinery. This is very important, giving a safety barrier to the process system.

Second generation of caverns

The second generation of caverns was a result of an engineering process using all the experiences from the previous stages, comprising the need to increase the storage capacity and productivity, while maintaining a high safety level. The added cost of establishing a stable and safe wide and high cavern could be compensated by the lower cost for an efficient bench blasting, excavation and mucking and transportation operation of the volumes in lower part of the cavern. Additionally, more efficient storage capacity the by "self- compaction "of the deposited sludge due to the increased height have a positive effect.

Excavation of the new caverns are carried out to a deeper level by introducing an additional access tunnels 30 m below the earlier main level, and establishing an infill tunnel access at the inner side of the tunnel. The lower tunnel serves as transport opening for the rock mass, enabling the cavern to be excavated considerably deeper than the first generation of caverns, and access for drilling rigs, hydraulic hammer excavators, shotcrete rigs and personnel lifters for rock support. The inner access tunnel supplies consumables for drilling and blasting and personnel entrance and emergency opening through a temporary scaffold staircase tower. Dismantling and re-erection of the tower for each level of benching, currently two times, is part of the operation. The inner tunnel opening is also important for ventilation, supplying pure air from the inner and lower openings, venting polluted air out at the upper high end.

Cavern no. 13 is 22 m wide, 41 m high and 200 m long. Cavern no. 14 is 22 m wide, 43 m high and 250 m long. This gives a storage capacity of 180.000 m^3 and 210.000 m^3 respectively. Caverns no. 17 and 18 completed in 2013 and 2014 was approximately. 25 m wide, 60-63 m high and 200 m long. This gives a storage capacity of 280.000 m^3.

In the future, these caverns will be even larger, a preliminary goal are cavern volumes of 400.000 m^3.

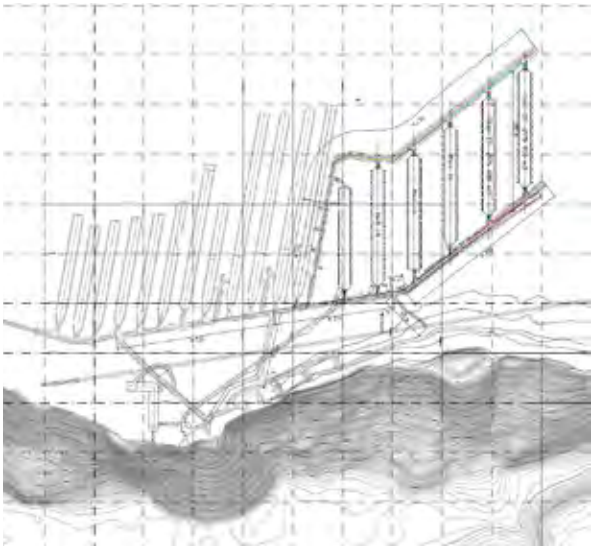


Figure 2: An illustration of the plant. Caverns from no 13 to 18 are "new generation"

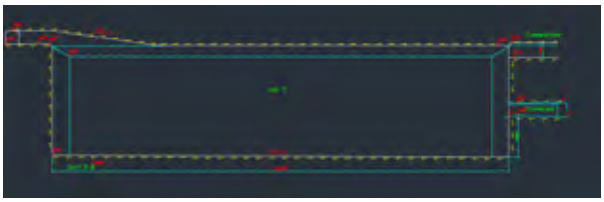


Figure 3. Longitudinal and transverse sections of rock cavern number 17.

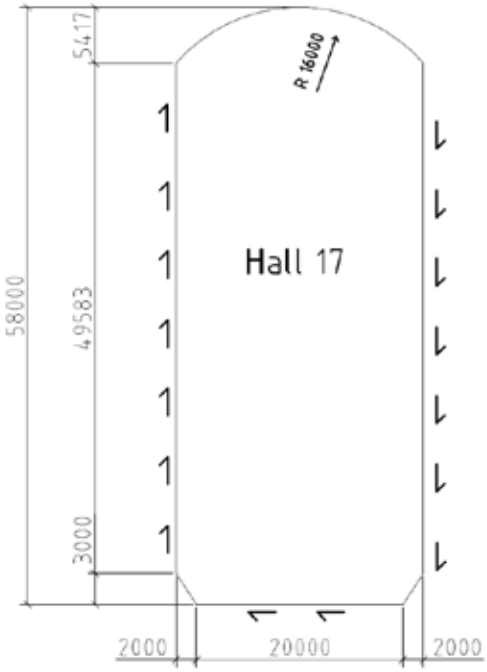


Figure 4: Cross-section of cavern no 17. 24 m wide and 58 m high.

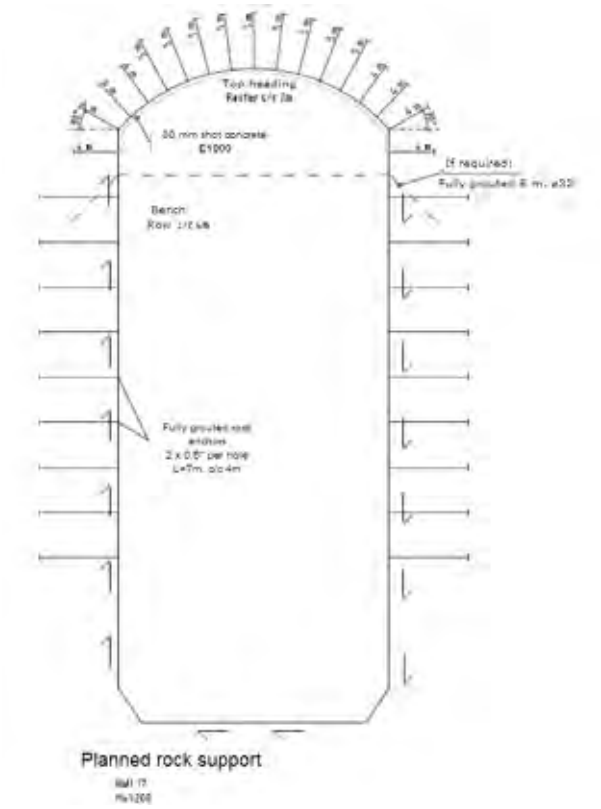


Figure 5: Cross section of cavern no 17, showing planned rock support with rock bolts and 7 m horizontal rock anchors.



Figure 6: Barge used for controlled rock dumping into the 130 m deep fjord.

5 CONSTRUCTION OF CAVERNS NO. 17 AND 18

We have excavated the top bench by drill and blast with 6 m long rounds, having one 10 m wide pilot header 12-24 meters ahead of a horizontal bench round to full cross-section. The rock stress level is appearing as rock spalling and some cracking sounds, but no dramatic rock burst events has been reported.

Subsequent scaling and installation of systematic patterns of end anchored bolts and steel fibre reinforced shotcrete stabilizes the situation. Grouted rebar bolts and thicker shotcrete are used for stabilization of local jointing and weakness zones.



Figure 7: A completed blast of the top bench in cavern 17

During and after second level of benching, the roof is checked manually from lifting platforms. If necessary, rock support in the roof is possible using crane operated equipment.

Systematic rock support of the wall area is done by 7m long grouted 2 pcs 16 millimeter steel strands per hole, with a spacing of 4m. Additional support of jointed rock at the surface are ensured by end-anchored 2-4 m long rock bolts, supplemented by fiber reinforced shotcrete. The level of detail rock support is largest in the lowest part of the benches where large bottom charges are necessary.



Figure 8: Installation of rock support in cavern 17.

Visual control supported by geometry measurements are the main methods for control of roof stability. All this requires a comprehensive system for inspection reporting and execution of rock support carried out by the client and the contractor.



Figure 10: Scaling after excavating two benches in cavern 17

Contour drilling accuracy have a major impact on the level of rock support. 64mm drill diameter is chosen. With direction stabilizing drill bit and rods, drilling deviations are generally too high when the drilled lengths exceeds 15 meters. Bench height above this level is not recommended without special considerations and safety measures.



Figure 11. Fourth bench is starting in cavern 17.



Figure 12: Typical stress- induced jointing patters in cavern no 18, necessitating use of shotcrete.



Figure 13: Cavern no 17 completed. Frode Arnesen from Multiconsult ASA as a reference point in the picture.

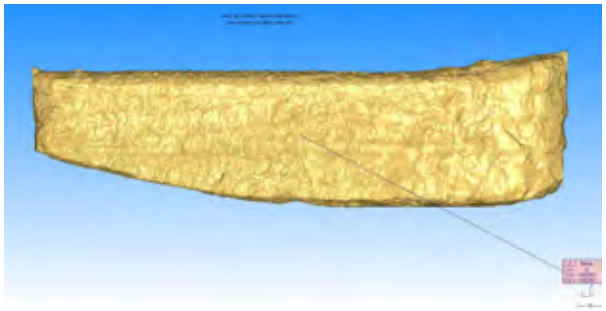


Figure 14: Longitudinal section from scanning of cavern no 18 with volumes.

6 CONSTRUCTION COST

Excavation cost for high caverns with transport tunnel at different levels and extensive use of bench blasting has the same elements as above ground, particularly when excavating simultaneously in two or more caverns. Use of 60- to 80-ton excavators and six wheel 40-ton trucks have been successful.

A tunnel drill rig and necessary equipment for installation of rock support as rock bolts- rock anchors and shotcrete will have to work alongside the bench blasting and excavation operation.

The cost level of the operation is governed by:
1) general cost for transportation and excavation,
2) cost of tunneling and rock support, and
3) general cost or running an underground construction operation.
The cost are close to one third each.

Cost variations during three different contracts have had highest variation in factor no 2, as the quantities of rock support have increased as the tunnels have been located further into the mountain with increasing rock overburden.

Due to technical and market development, Boliden has been able to keep the total cost of depositing at a nearly constant level during the last ten years.
Future development trends

Future caverns will be larger in width, height and length, adapted to even more efficient excavation and transport. The rock conditions and stresses will eventually limit the dimensions. We should avoid further increase of overburden. Technological development, enabling more operations to be performed by remote control can add more to the possibilities for further development regarding cost and safety.

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Bekkelaget water treatment plant

08. ULRIKEN / SVARTEDIKET WATER TREATMENT PLANT

FLATIN, Asle

1 NEW ADVANCED DRINKING WATER TREATMENT PLANT FOR BERGEN CITY CENTRE

Lake Svartediket has supplied the centre of Bergen and its immediate surroundings with water since 1855, making it the city's oldest water source. Of the five waterworks in the Municipality of Bergen, the Svartediket plant is the largest, supplying clean water to about 50 000 persons, businesses and public services. Work has now been done to convert the plant from the earlier minimum treatment plant to an advanced treatment facility, and the new process is to be put into operation in June 2007. As the plant is situated very close to the built-up area of the city and there is a shortage of space, the new parts of the facility have been built underground and are connected efficiently to the old plant on the surface.

The main features of the development project are:
- A new underground process plant equipped to provide advanced treatment of drinking water
- A new deep-water intake in Lake Svartediket
- A new treated water basin underground at 70 masl
- Upgrading and alteration of the existing surface plant, including access/entrance, operations/personnel department, chemical addition, viewing room and enclosed footbridge to the underground construction
- A new Haukeland water reservoir underground at 125 masl.

The plant has been constructed with direct filtration and UV radiation as two independent hygienic barriers. The direct filtration process is the Molde process, which involves chemical precipitation / carbonation in a triple-media filter and the addition of iron chloride and CO2 ahead of the filters. Chemical precipitation/direct filtration removes particles and colour, whilst carbonation increases pH, alkalinity and calcium content of the water. The purpose of carbonation is to improve corrosion control of the water mains.

The sludge treatment method used is gravity thickening. The sludge fraction is passed to the public sewage network, whilst the reject water is passed back to the source.

The planning of the plant was started in 2000 in the form of a draft project competition carried out ahead of the preliminary project and later detailed planning. The construction period for the plant has been just over three years from April 2004 to June 2007. The first two years were spent on building underground constructions and structural works, whilst in the last year the focus has been on the technical contracts. Prior to commissioning, the system will be tested / started up and the filter media loaded.

The Haukeland reservoir has been constructed as a part of this development. This reservoir is a tunnel of about 2 km blasted out between Svartediket and Landås. The water-filled portion will be approximately 1.3 km in length, and the reservoir will be a part of the municipality's general drinking water transport and distribution system located in the mountains surrounding the city.

Key data of for the new plant:
- Average water production: 520 l/s
- Maximum water production: 925 l/s
- Volume of treated water basin: 15 000 m^3
- Volume of Haukeland reservoir: 30 000 m^3
- Depth of new water intake: 28 m
- Process: Direct filtration and UV radiation
- Budget: NOK 300 million
- Excavated rock volume: 120 000 m^3 projected solid mass
- Transportation of excavated rock through residential area: truck every 4 minutes
- Concrete works cast on-site: 5000 m^3



Figure.1: Site plan

Project participants

- Owner and construction management: Municipality of Bergen, Agency for Water and Sewerage Works
- Construction management: Sivilingeniør Erik Fylkesnes AS, Bergen, Bergen Vann KF
- Principal consultant: Asplan Viak AS, Sandvika
- Sub-consultant, plant engineering: Smidt & Ingebrigtsen AS, Bergen
- Sub-consultant, rock engineering geology: Geo Bergen, Bergen
- Sub-consultant, HVP: Opticonsult AS, Bergen
- Consultant, traffic engineering: Halldor Havsgård, Consultant Engineer
- Contractor, underground, HVP and building engineering works: NCC Construction AS, Bergen
- Contractor, mechanical and process equipment: PMI Pindsle AS, Sandefjord
- Contractor, electrical engineering systems: Profitek AS, Bergen
- Contractor, process control equipment: IKM Automasjon AS, Bergen
- Contractor, filter materials: Franzefoss Miljøkalk AS
- Contractor, safety systems: YIT Building Systems AS
- Contractor, telephone systems: Tele-Com Bergen AS

2 UNDERGROUND WORKS

Hilmar Lilleøren, Constructional Engineer, Smidt & Ingebrigtsen AS and Geir Bertelsen, Engineering Geologist, Geo Bergen.

Design of the plant

The underground works comprise about 2.8 km of tunnels with a cross-section varying from 17 to 73 m² and caverns having a total volume of 50 000 m³. NCC Construction AS has been executing contractor.

The tunnels consist of the main access to the plant itself, the intake tunnel from the water source Lake Svartediket, access to the upper part of the process cavern and access to the Haukeland reservoir.

The Haukeland reservoir is in the form of a 22 m² horizontal tunnel of about 1300 m in length and is dimen-

sioned for about 30 000 m³ of water. To allow access to the Haukeland reservoir when in operation, a 140 m-long tunnel has been constructed, which runs around the northern dam in the reservoir.

A connection has been established between the Haukeland reservoir and Landås by means of a tunnel of about 130 m in length. The plant at Landås is a part of the previously developed Gullfjellet waterworks.

The largest rock caverns in the plant consist of the process cavern, the treated water basin and the pumping station / backwash water basin, and of these, it was the process cavern that presented the greatest construction engineering challenges. This cavern has a span of 22 m, a rise of about 4.0 m and wall heights from 10 to 19 m. It was excavated conventionally using the top heading and bench approach, which involves top heading excavation and installation of permanent support followed by bench excavation.

The water treatment plant is connected to the water source by an intake line running through a tunnel. This intake tunnel runs out into Lake Svartediket at a depth of about 20 m, whilst the intake line has been laid with the intake at a depth of 28 m.

Throughout the construction period, Lake Svartediket was in service as a water source and was subjected to restrictions as regards both facility operation and minimum water level. Breakthrough therefore had to be carried out under water. To protect the water from harmful pollution, a certain area outside the breakthrough point was ”fenced in” using a silt curtain.

Preliminary surveys of the breakthrough point conducted by divers found rock, but because of poor visibility in the water it was difficult to determine its quality. However, probe drilling from the tunnel showed possible large blocks of rock or rock with substantial fissures. Before the final procedure of carrying out the breakthrough blast was performed, the contractor, together with the project owner, conducted a comprehensive safe job analysis.

An external consultant was engaged to calculate the pressure on the tunnel plug initiated by the breakthrough blast, and, if necessary, propose special measures to reduce the pressure to an acceptable level. After considering a number of special measures, it was decided to carry out the breakthrough blast with a ”dry” tunnel (apart from the receiving pit) and employ a positive pressure corresponding to a pressure that was about 5 m lower than the water pressure above the breakthrough point.

The breakthrough blast went according to plan. However, as the probe drilling had indicated, the rock at the breakthrough point was more fissured than had been established beforehand. Scaling and installation of support at the breakthrough site was therefore a much more extensive job than projected.

Potential transport routes on the surface are relatively narrow and run very close to established buildings, including a primary school. It became clear quite early on in the planning phase that transport of rubble could become a bottleneck and limit the progress of the tunnel and cavern excavation work. This was taken into account when planning and setting work schedules and deadlines. Excellent cooperation between contractor, project owner and residents in the area helped to ensure that also this part of the construction project was carried out in a satisfactory manner.

Engineering-geological conditions

The project has required extensive blasting close to dwellings and other building structures, necessitating the implementation of a number of restrictive measures. Limits for blast-induced vibrations were defined in accordance with NS 8141. The contractor was required to conduct vibration measurements, but at no time during the blasting operation were vibrations in excess of the limits registered.

The housing development in Bjørndalen is very close to the plant, and at the foot of the steep slope on the northwest face of Ulriken. It had been determined previously that there was some danger of landslide here, and a new assessment was made. The extent to which the blasting operations might impact the landslide danger was also assessed. As a result of these evaluations, it was decided to carry out a number of landslide protection measures, including the construction of an embankment and the erection of a catch fence.

The bedrock in the development area consists of green schist, mica schist and amphibolite. Schistosity, other jointing and some zones of metamorphic rock have presented a number of challenges as regards stability. During the blasting of the process cavern, there were some large accidental rockfalls. Here, support has been provided using long bolts and reinforced shotcrete arches.

The tunnelling work was for the most part carried out without any particular problems. A weakness zone roughly parallel to the Haukeland reservoir caused difficult excavation conditions and more water ingress than anticipated. However, this was not entirely unex-

pected, as similar problems had been encountered when driving a tunnel further south, in the same rock mass, in connection with the Gullfjellet waterworks. In the Haukeland reservoir, the problems were solved by changing the direction of the tunnel, so that it ran outside the weakness zone.

In general, it has been necessary to implement extensive support in tunnels and rock caverns using bolts and fibre-reinforced shotcrete

One of the aims was to limit to the maximum ingress of fissure water into the basins. Procedures for probe drilling and grouting were devised in order to keep the leakages below defined limits. In the Haukeland reservoir, a number of seepage-bearing zones were encountered during probe drilling, and some grouting was necessary.



Figure.2: September 2005. The breakthrough blast went according to plan.

3 MECHANICAL AND PROCESS EQUIPMENT

Thomas Frydenberg, Agronomist and Jon Brandt, Civil Engineer, Asplan Viak.

This contract has been supplied and installed by PMI Pindsle AS, Sandefjord, with subcontractors for machinery and equipment. The main elements are:

- **Process pipelines.** These are primarily stainless steel pipes and pipe sections inside the new underground process plant. Dimensions are up to DN900 mm. Pipework was prefabricated and installed by PMI Pindsle AS.

- **Valves and fittings.** KSB AMRI butterfly valves from Lindflaten have been chosen for the process lines. Valves that are to be controlled have pneumatic actuators with either on/-off function or regulating function. To allow water to flow down from the 125 masl zone to the 70 masl zone, two Erhard annular piston valves with Auma electric actuators from Sigurd Sørum have been installed.

- **Pumps.** The pumps in the plant have been supplied by ABS. The table shows a complete list of the main pumping stations in the water treatment plant.

- **Blowers and compressors.** Two Aerzen blowers have been installed to provide air for filter backwashing, and two compressor systems have been installed to supply compressed air to valves. The supplier of this equipment was Granzow.

- **UV station.** Consists of four medium-pressure disinfectant units of the type Berson InLine 15000+. The four UV units have been installed in separate lines with individual quantity control and energy stepping. A fully automated washing system has also been installed for cleaning the UV chamber and quartz glass. This equipment was supplied by HOH Birger Christensen.

- **Measuring equipment.** Comprises primarily electromagnetic water flow meters, level indicators, pressure gauges and equipment for continuously measuring water quality parameters such as pH, turbidity, colour, UV transmission and iron content.

- **Chemical dosage.** All addition of chemicals will be done in the existing plant at the surface. Storage tanks and dosage equipment have been installed in the process plant for precipitation chemicals and extra chlorine. Existing lime silos are to handle marble (calcium carbonate) in the new plant, and the existing CO2 tank is to be used in the future with new dosage control panels. Transport lines have been installed to convey precipitation chemicals, marble and CO2 from the filling station in the existing plant to the underground process plant.

In the autumn of 2004 there was a giardiasis epidemic in Bergen, and the drinking water from Svartediket was identified as the source of infection. The construction of the plant was in full swing, but there was still about two and a half years to go before the new plant would be ready, and until there would be two hygienic barriers against Giardia. As a solution to this problem it was decided to install the UV disinfection units for the new plant temporarily in the ”chlorine chamber” on the lower side of the old water treatment plant with chlorine and screens. Four UV units were ordered instead of three as originally planned for the new plant, and two pairs were installed in series in the temporary installation.

	No. of pumps	Q_{dim} pr pump l/s	Lifting height at Q_{dim} total (mVS)	Motor output kW	Function
Untreated water pumps	3	550	5.1-17.3	132	Raw water into filters.
Backwash pumps	2	688	19.7	160	Backwashing of filters
Pumps 125 zone	3	182	66	160	Treated water from 70 masl to 125 masl.
Pumps 204 zone	3	50	93.3	75	Treated water from 125 masl to 204 masl
Return water pumps	2	80	17.8	22	Return water to source or raw water side.

Table: Pumps

A new transformer for power supply to the temporary UV unit and a mobile back-up power generator were installed. The temporary installation was incorporated in the existing process control system for monitoring and for controlling the number of units and power stages according to water volume.

When the new plant is put into operation, one of the lines with two units will be moved into the new process plant and put into operation first, and then the second line will be moved. This means that there will be no interruption in the UV radiation of drinking water for Bergen city centre.



Figure.3: The untreated water pump station



Figure 4: Pumps in the 125 masl zone



Figure 4: UV station temporarily installed in the old chlorine chamber

4 DESCRIPTION OF CONSTRUCTIONAL WORK ON THE SURFACE PLANT

Kristian Asdal, Architect MNAL, Asplan Viak

The original surface plant at Svartediket – located very close to the dam and watercourse – was built in several stages, and the entire structure was given an exterior brick facing and a lapped roof.

The new project generated a need for a new main entrance and a general upgrading of existing functions in addition to establishing several new ones. A viewing room located in a new storey above the ground floor was described separately in the tender document, and it was eventually decided that it was to be built. This room, complete with kitchen and necessary support rooms, allows an excellent presentation of the water treatment plant to visitors, due in part to its strategic location with good visual overview of the different sections.

The upgrading and alteration work on existing buildings included personnel rooms/locker rooms, vestibule, operations room, laboratory, offices and various technical functions. Good storey height in the building allowed for new technical ducting above ceilings.

In the added entrance structure to the west, there is a new lift and main staircase (which provides access to the viewing room). The main entrance features a large west-facing glass facade.

The structural addition is in keeping with the old building as regards the materials used, whilst its form and detailing clearly express a new and modern style.

Furthermore, a footbridge has been built to connect the surface plant to the underground facility. This footbridge, constructed in steel and glass, is 39 metres in length, and crosses the overflow from the dam, with dramatic effect when the water gushes down beneath it.



Figure.5: The footbridge connecting the surface plant to the process plant

5 DESCRIPTION OF CONSTRUCTIONAL WORK ON PROCESS PLANT

Erik Nøtland, Civil Engineer, Asplan Viak

The process plant is mainly located in rock caverns. All the basins and other structures are constructed in in-situ concrete. The executing contractor for all constructional works is NCC Construction AS.

As the plant has been designed with relatively large internal height differences (about 20 m between upper and lower level or floor), all load-bearing structures are founded directly on the rock. This has been done to avoid damage to settlement-sensitive water structures. All water basins have been constructed as watertight slack reinforced concrete structures. The water pressures are in the range of 5-8 m.

Draining textile has been used on the formwork in the filter basins as an alternative to epoxy treatment to increase the durability of the concrete. All other concrete surfaces that are in contact with water are untreated, except for the upper part of the filter basins, which are tiled to facilitate cleaning in the splash zone.

The rock caverns have been clad internally with PCV fabric to prevent ingress of water from the rock, which also results in light surfaces that are attractive and save on lighting.

The floors are partly covered with ceramic tiles and partly coated with an epoxy coating. Walls and ceilings in dry rooms are painted with acrylic paint. Some of the process rooms where noise production is high have been soundproofed with sheeting in walls and the ceiling.



Figure.6: Casting nozzle bodies

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Typical crude oil cavern in Norway (Sture)
Curtesy: Norconsult AS

09. STENDAFJELLET ROCK QUARRY AND UNDERGROUND WASTE DISPOSAL SITE

HOMMEFOSS, Anne
ARNESEN, Frode

ABSTRACT

In Stendafjellet, 8 km south of Bergen center, and 5 km from Bergen Airport, Fana Rock and Recycling A/S (Fana Stein og Gjennvinning A/S, FSG) is operating an underground quarry and crusher plant. Excavated rock volumes are later used for depositing sites for contaminated soil and masses with low organic content.

350 000 tons of rock material for construction purposes is produced yearly in up to 13 different grain size fractions. A considerable quantity of this material, including fines are tarmac additives. Previous experience of excavating caverns 25 m wide and 50 m tall, in the same type of geological conditions, help in planning and design.

A qualitative cost comparison between excavating the top header and a 35 m tall bench is examined. The results from this cost comparison describes some characteristic differences in cost elements between underground and above ground excavation.

Favourable rock quality enables construction of tall caverns with high degree of stability with relatively little rock support. Together with a normal level of preconstruction investigation, the Q-system gives acceptable basis for design and construction, if continually supervised by an engineering geologist on site.

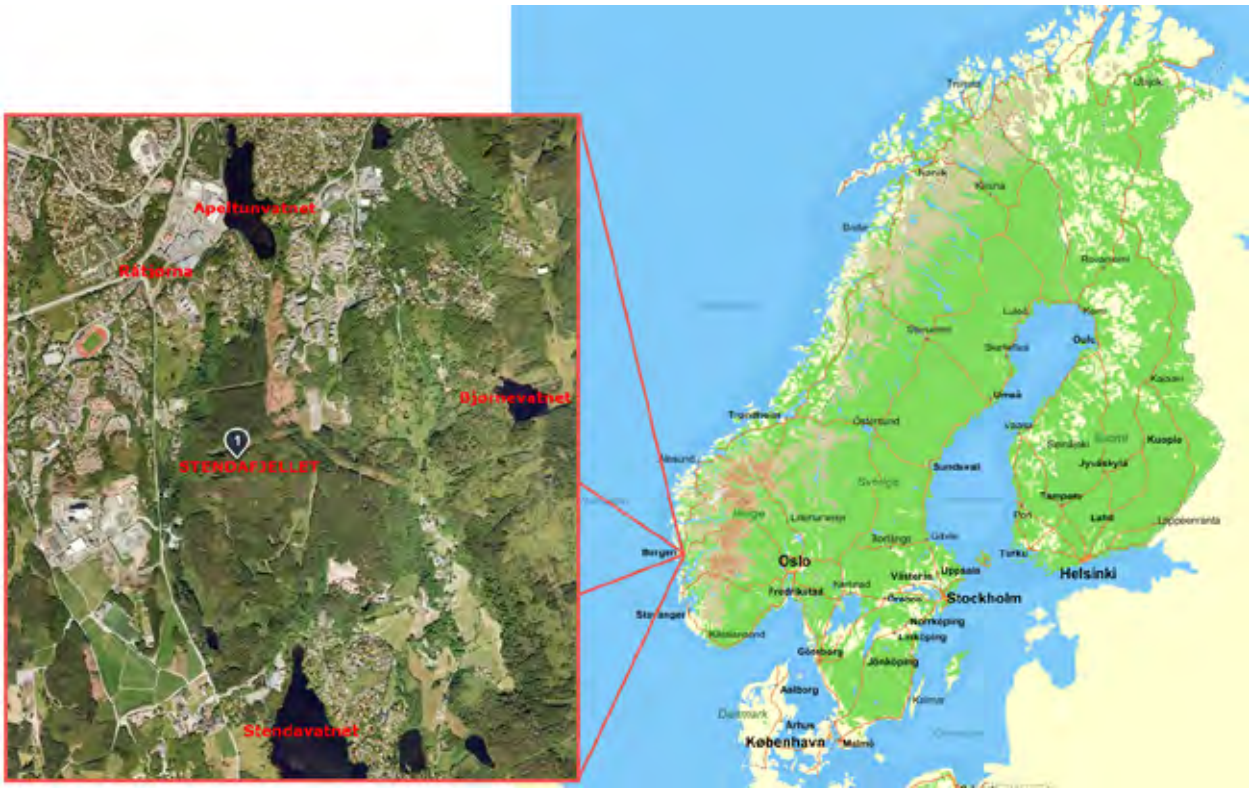


Figure 1: Location of Stendafjellet, and Fana Rock and Recycling A/S. (http://kart.gulesider.no/)

The permits for depositing polluted materials in underground caverns depends on identified waste categories according to waste regulations.

It is recommended to map rock material quality fit for quarrying and possibilities for underground depositing. And including these in urban planning. Supplying good quality rock material for construction purposes, near the consumer market, result in low transportation costs for clients nearby.

1 INTRODUCTION

Fana Rock and Recycling (Fana Stein og Gjenvinning, FSG) was established in 1954. The company has two separate businesses: quarry/chrusher/sorting plant and waste disposal for contaminated substance/mass. They also work with entrepreneurship and tunnel excavation. The quarry and crushing plant mainly produce gravel. They also sell natural products as foundry sand, scouring sand, shell sand and bark.

The waste disposal is a business idea, exploiting the volume of the excavated rock caverns. The purpose is to safely handle contaminated masses and prevent pollution of the environment.

The company has 20 employees. Daily operation is mainly automated. Subcontractor perform excavation and transportation of rock to the main crusher.
Today the plant comprises eight rock caverns for rock mining/waste disposal and an underground plant for crushing, fractioning, storing and loading of produced materials. There are a system of tunnels connecting the ends of the caverns at two to three levels, allowing rational access for both the excavating rock operation, rock support work, and infilling of material.

In 2003 FSG was given permission to deposit waste. Later they received extended permissions due to changes in the legal framework and state of the marked. The authorization applies to deposition of waste in the existing rock caverns. According to the authorization

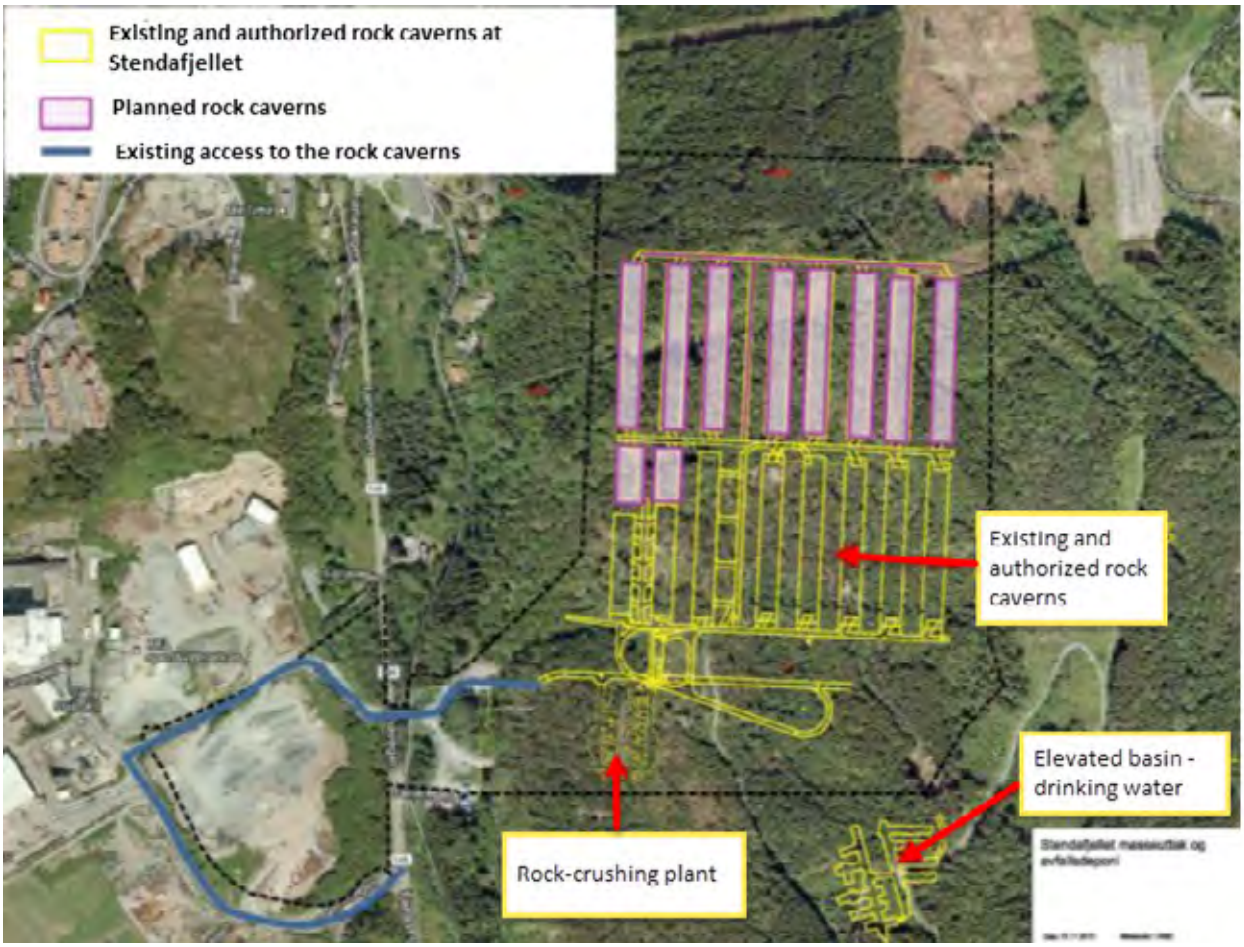


Figure 2: An illustration of the existing and planned rock caverns and plant at Stendafjellet.

given by the county administrator, 27th of July 2012, FSG can accept waste in category 2 (ordinary waste) and category 3 (inert waste). The authorization applies to a yearly waste quantity of 350 000 tons.

2 GEOLOGY

Stendafjellet is a hill of medium to fine-grained granitic, syenitic and mangeritic rocks of Precambrian age, generally altered and deformed to gneisses by Caledonian processes. Except for a few distinct shear zones, the rock mass quality in the area of the underground quarries generally seem to be of good quality. The brittleness and flakiness classifications are within category 1, which is the best category according to testing methods in Norwegian Road construction codes.

Favorable rock quality help in construction of tall caverns. This makes a high degree of stability with relatively little rock support possible. Together with a normal level of pre-construction investigation, the Q-system gives acceptable basis for design and construction. In addition, the site needs continual supervision by an engineering geologist.

Joint systems

The structure and degree of fracturing of the rock mass varies in the area. The jointing system are mainly dry, but there have been registered some areas with water from gouges in the system.

The existing rock caverns are in an area with few weakness zones. During excavation of the rock caverns, a small number of weakness zones was encountered. None of these contained any large amount of water. Heavy jointing, as well as infillings with altered swelling clay materials have challenged the rock support operations as well as the productivity of the last caverns.

3 HYDROLOGY AND WATER TREATMENT

Stendafjellets highest peak is 232 meters above sea level. The water drains naturally northwest towards Råtjørna (40 meters above sea level), northeast towards Apeltunvatnet (32 meters above sea level) and west towards Grønnestølen, to a creek and bog that follows the bottom of Rådalen valley and further to Råtjørna from the highest point of Stendafjellet. Apeltunvatnet run-off to Nordåsvatnet (sea), and Stendavatnet drains towards the Fanafjord.

A meteorological station at Stend show normal yearly precipitation at 2 041 mm. The Flesland meteorological station show 1 815 mm of normal precipitation during one year. It is anticipated a higher precipitation at Stendafjellet due to topography, compared to Flesland.

NVE (The Norwegian Water Resources and Energy Directorate) has measured the water run-off at two locations at the foot of Stendafjellet. They found yearly water run-off to be 1986 mm/year and 1936 mm/year in 2013. The yearly water run-off, calculated by NVE, is 60-65 liters per second per km^2. From the entire Stendafjellet area it is presumed that the water run-off is about 36-40 m^3/hour. This constitute about 16-18 m^3/hour above the existing rock caverns, and 19-21 m^3/hour above the planned rock caverns.

The terrain at Stendafjellet is relatively steep and water run-off therefore probably happen as rapid surface run-off. The ground conditions appears dry, and there is no ponds, lakes, rivers, creeks or bogs on the mountain. Water flow happen within joint systems and channels in the rock mass.

The permeability of the rock mass is low, apart from the joints- and weakness zones. These zones are considered "watertight". At the current climatic conditions.

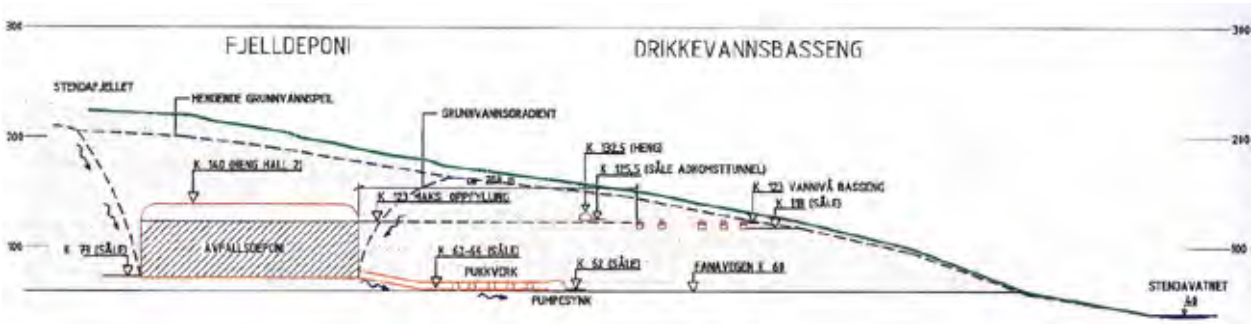


Figure 3: Cross section a long a north-south profile through the cavern.

As mentioned before, the rock mass at Stendafjellet consist of few joints- and weakness zones. During excavation of the first rock caverns, there were two areas with considerable water leakage. The leakage at these locations decreased rapidly and stopped after some time.
In weakness zones crossing the plant, there is only registered small amounts of water leakage. This is due to filling material, which is powdered and fine structured rock material, partly clay fractioned.

A common problem is the connection between excavation of rock caverns and the drawdown of the water table, and/or changes in water level and discharge of lakes, rivers or bog areas. Because of the great distance from the closest recipients and the absence of surface water, wetland areas and bogs, this is of no concern at Stendafjellet.

Measurements around the existing rock caverns show ground water levels lie deep in the rock mass at Stendafjellet. The measurements also show that the water level varies in correlation with precipitation and seasons. The ground water level has not been affected by previous excavation.

Water enters into the rock caverns with a very low flow rate. The gravitational water from the rock caverns are sampled, analyzed and categorized as contaminated according to the law on pollution, but not dangerous.
Extensive monitor programs for registering the ground water and chemical quality in and around the rock caverns have been established. Registration started before development of the waste disposal site, and therefore this is a good basis for evaluation.

Initially, the ground water level was measured at eight locations. Today, the monitoring program consist of sampling from seven wells for inspection and one reference point. Two wells are inside the rock caverns to survey the area between the waste disposal site and drinking water basin. Five wells are at potential effluent water areas around Stendafjellet. Two wells are located between the waste disposal site and water wells at Rådalen.

The water level in the wells outside the rock caverns have been continuously supervised over the last couple of years. Measurements show that water level vary rapidly with strong correlation to precipitation.

Sampling tests 4-6 times a year analyze for about 200 different environmental toxins. In addition, quarterly samples of the seepage water is tested before it is transmitted to the municipal water grid. As mentioned before, there is no surface water near the plant, and the county administrator in Hordaland has given exemption from the instruction of The Norwegian Environment Agency (1st of July 2013) concerning the collecting of samples from the surface water.

Analyzes show that the ground water is not influenced by seepage water from the waste disposal site. Water level measurements confirm that the ground water gradient has direction towards the rock caverns.

The existing rock caverns make an artificial barrier. The water can not flow uncontrolled from the caverns to the surface. The water is collected and lead to a suction pit at the lowest point of the plant. This water is lead to the municipal treatment plant.

4 LOCATION AND DESIGN

The first generation plant consisted of crushing- and sorting plant, access tunnel and storage chamber for rock. The rock cavern is excavated by top heading, and deep-hole drilled benches. The bench heights are more than 25 m.

The lowest level is a ring-shaped tunnel, connected to the south and north access and transport tunnels. Operation of the quarry have high level of automation, requiring one person for running the operation, and two persons driving a wheel loader and dump truck delivering stone to the crusher.

Caverns for mining and storage

Today, the caverns are excavated with top heading, access to the middle level and access from the lower level in one part of the cavern. This is due to the loading and transportation logistics. The bench heights are normally 12 m. The top heading rock support consist of system bolting and fiber reinforced shotcrete. The benches are supported by end-anchored bolts, and shotcrete if found necessary. The access for transport vehicles at the lowest level is restricted with safety zones between transport roads and the cavern walls.

The caverns are orientated north south. The width of the caverns is 25 m, the height is approximately 50 m. The lengths varies between 120 and 200 m, limited by existing shear-zones. The distance between the caverns is 25 m, and the volumes vary from 120.000 m^3 to 200.000 m^3.

It is applied for excavation of eight similar and parallel caverns from the west side to the east side of Stendafjellet. The length of the remaining, and planned caverns, might increase up to 200 m and 400 m. It is also possible to increase the height of the caverns up to 100 m.

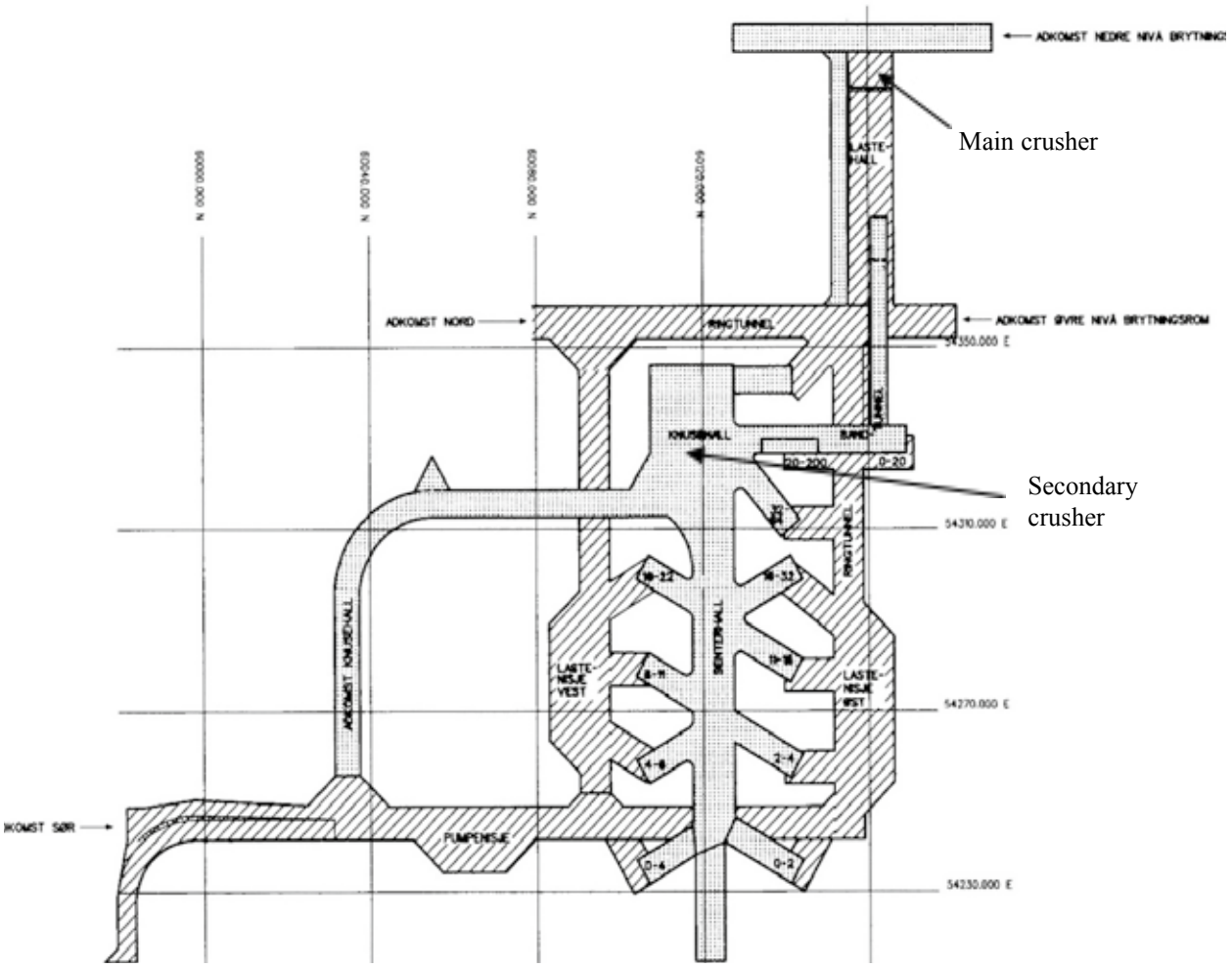


Figure 4: The mining area and the crushing plant are connected through the loading area at the lowest level where the primary crusher is located. The secondary crusher is located at the upper level and connected to the primary crusher and storage shafts and niches at the lowest level by conveyor belts. The lowest level are shaped as a ring tunnel, connected to the south and north access and transport tunnels.

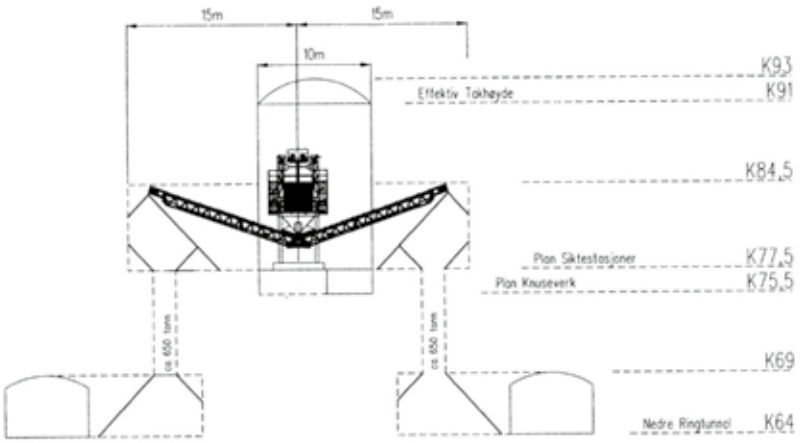


Figure 5: Cross section of the secondary crusher/sorting cavern with conveyors to storage niches.



Photo 1: Rock cavern, 200 m long and 25 m high (http:// www.fsg.no/index.php/bilder/bilder-deponi)



Photo 4: Close up photo of zone with infilling of swelling clay (light gray). Photo by FSA, Multiconsult.



Photo 2: Lorry-mounted inspection platform for mapping and installation of rock support. Photo by FSA, Multiconsult.

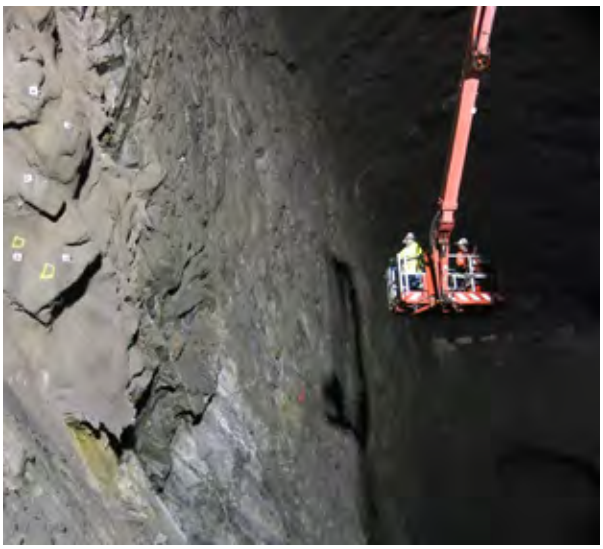


Photo 3: Inspection of weakness zone in east wall of cavern no. 7. Photo by FSA, Multiconsult.



Photo 5. Installation of rock support in wall by drilling with long stick excavator with hydraulic hammer. Bolt installation is carried out by operator in a separate lifting platform. Photo by FSA, Multiconsult.



Photo 7. Inside the rock cavern, upper side, for waste disposal of contaminated masses. (http://www.fsg.no/index.php/bilder/bilder-deponi)



Photo 6. Photo showing access tunnels at different levels at the south end of the cavern. Photo by FSA, Multiconsult.

5 OPERATION OF THE WASTE DISPOSAL PLANT

All incoming material is controlled and checked. The caverns are filled with remote controlled equipment from the upper level access tunnel. Only a small amount of manpower is required for operations. It is possible to do filling from separate tunnels from above or the side of the caverns.

The existing permit for depositing polluted materials in the caverns are summarized by a list of identified waste categories.

Regular control of the drainage water is carried out in order to check for contamination and for analyzes purposes. The low permeability of the rock result in very small leakages (0,1-0,2 m^3/min) for the whole cavern system.

6 ENVIRONMENTAL BENEFITS AND MAIN CONDITIONS FOR SUCCESSFUL QUARRY

The excavation increase the surface altitude, and reduce air pollution, noise and vermin in the adjacent areas with respect to both the crushing plant and the waste disposal site. It minimizes the risk of ground water contamination. The underground openings are designed to allow water from the production areas and the disposal areas to drain

naturally towards an effluent control pit with pump instalations. This pit is located in the south access tunnel, at the lowest point in the entire system of underground excavations. After passing through an oil and mud separator, the drainage water connects to existing pipeline system for collection of drainage water from old waste disposal sites at Rådalen, and subsequently to existing sewage plant and approved sea discharge.

All urban societies are consumers of crushed rock and produces waste and polluted materials. At Stendafjellet, short distance to the consumer compensates partly for the more expensive method of quarrying for construction quality stone material by underground methods. In filling of polluted materials, which otherwise would have to be transported long distances to other approved deposit sites, contributes to a profitable operation. Looking at the environmental side of the operation, no agricultural or forest area have been altered, the noise and dust from quarrying is negligible. CO2 – emissions from the transport distances for materials in and out are also considerable when comparing to existing alternatives.

7 FURTHER DEVELOPMENT

Whether the quarry/deposit is to be extended further, will depend on the market for both aggregates and depositing. And on the taxation and permits for competing quarries and depositories in the area.

The operation also shows that it is possible to excavate caverns with large vertical dimensions at a low cost in an area with a high level of urban development. Such compact low-cost volume storages can replace storage methods which normally require larger areas, e.g. container storage and unmanned robotic storage installations.

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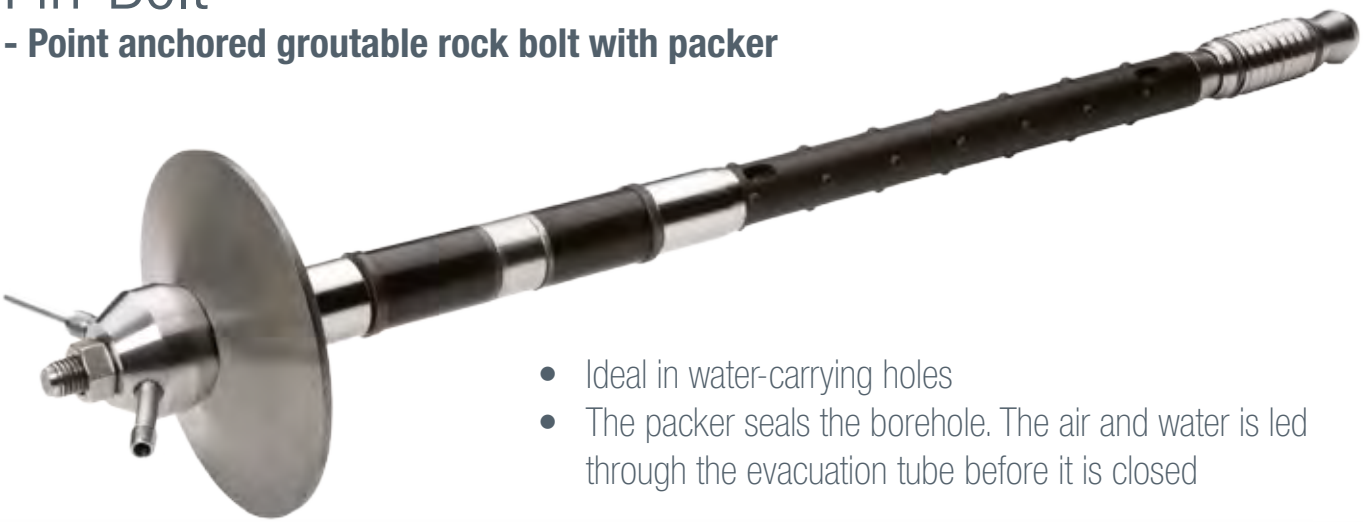
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Drill rig at Holmestrand station
Courtesy: Norwegian National Rail Administration

10. HOLMESTRAND UNDERGROUND RAILWAY STATION – SITE INVESTIGATIONS, EXCAVATION AND ROCK SUPPORT

WIIG SAGEN, Hanne

SUMMARY

A new overground railway station in Holmestrand, Norway, was planned in the 1990s. The need for higher speed involved more stringent requirements and a whole range of compensatory measures, which resulted in the planned station being moved underground. The station hall is unique worldwide, with four tracks passing through the station, two of which are through-tracks, designed for trains travelling of speeds of up to 250 km/h. The other two tracks are for boarding and alighting of passengers along the platforms. The total length of the tunnel is 12.3 km, and the station hall is located near the middle.

We know of no other projects of this nature elsewhere in the world, and the design and construction of the state-of-the-art solution have posed many challenges. A variety of models were used to calculate pressure/suction forces, air speeds, the fire/evacuation concept and acoustics in the station hall, and a series of site investigations were carried out to develop theoretical models for mapping the rock conditions and hydrology. This article will have focus on site investigation, excavation and rock support of the cavern, which has a cross section of 550 m^2.

1 INTRODUCTION

The Norwegian National Rail Administration (NNRA) is modernizing the railway system in the county of Vestfold south of Oslo. The Vestfold line was put into operation more than 130 years ago. The Vestfold line was divided into twelve sub-sections and the order of the construction work was decided on the basis of the effect that each sub-section would have on today's train traffic. Modernization and Construction of double tracks on ncertain sub-sections would increase capacity and remove bottlenecks. Three sub-sections have been completed, two are under construction and the rest are being planned successively and are scheduled to be completed by 2030, see Figure 1.



Figure 1: The Vestfold line. Under constriction, planned and completed subsections

The present-day 16.7 km long railway from Holm through Holmestrand to Nykirke is the section of the Vestfold line that has the poorest geometry and technical standard. New double tracks along this section will reduce travel time, increase capacity and punctuality, and reduce the need for maintenance where there are risk of landslides along cuttings and in areas of poor ground conditions on the current route along the Holmestrandsfjord. The construction work started up in august 2010 and is scheduled to be completed by the end of 2016. The budget is NOK 6,600 million (2015).

The project was originally planned and zoned as two separate tunnels with an open-air zone through Holmestrand. In a report from the Norwegian Parliament in 2009, the

Norwegian National Railway Administration (NNRA) was asked to reconsider the route past Holmestrand, for the purpose of increasing the design speed to 250 km/h. Based on the applicable track geometry requirements, this meant that the new Holmestrand station would have to be moved into a 12.3 km long tunnel through the Holmestrandfjellet mountain, see Figure 2. The plan is to construct the station in Holmestrand inside a rock cavern west of the existing station area and as close to the rock face at Holmestrandveggen as possible.

The station is planned with two exits: one leading to the present-day station area and one to the center of Holmestrand. There will be a distance of 100 m from the nearest platform to the open air. Holmestrand station will have four tracks — two through-tracks in the middle, designed for speeds of 250 km/h, and one track on either side, for trains stopping at the station. In total, the station hall will be 35 m wide and 16 m high, with a cross section of as much as 550 m^2, see Figure 3. Cross section of the station hall, 550 m^2 The main hall will be 250 m long to allow for the establishment of platforms on either side. In addition, there will be cone-shaped sections at each end to absorb pressure/suction forces, so that the total length will be 860 m. The normal blasting profile for the double-track tunnel will be 133 m^2.

2 GEOLOGICAL AND GEOTECHNICAL CONDITIONS

2.1 Rock types

Several geological investigation and other tunnels nearby have given knowledge that the rock types in the area are mainly sandstone (also known as the Ringerike sandstone), the Asker group (sandstone, schist, clay and conglomerate) and basaltic lava flows containing layers of red silt- and sandstone, tuff, agglomerate and lava conglomerate, the latter named the B1 group. The thickness of the lava flows varies from 5-15 m and have different mechanical properties. The Holmestrand plateau is part of this basalt formation. There are also several dykes in the area that mainly consist of syenitic porphyry, diabase and rhomb porphyry. These follow the main joint directions in the area and have intruded in the various faults as well. The dykes are often more jointed then the country rock, housing calcite or clay zones which traverse parallel to the border of the intruded country rock.

2.2 Geological formations

The most characteristic geological feature in the area is the long-stretched basalt wall which will house the station hall, see Figure 4.

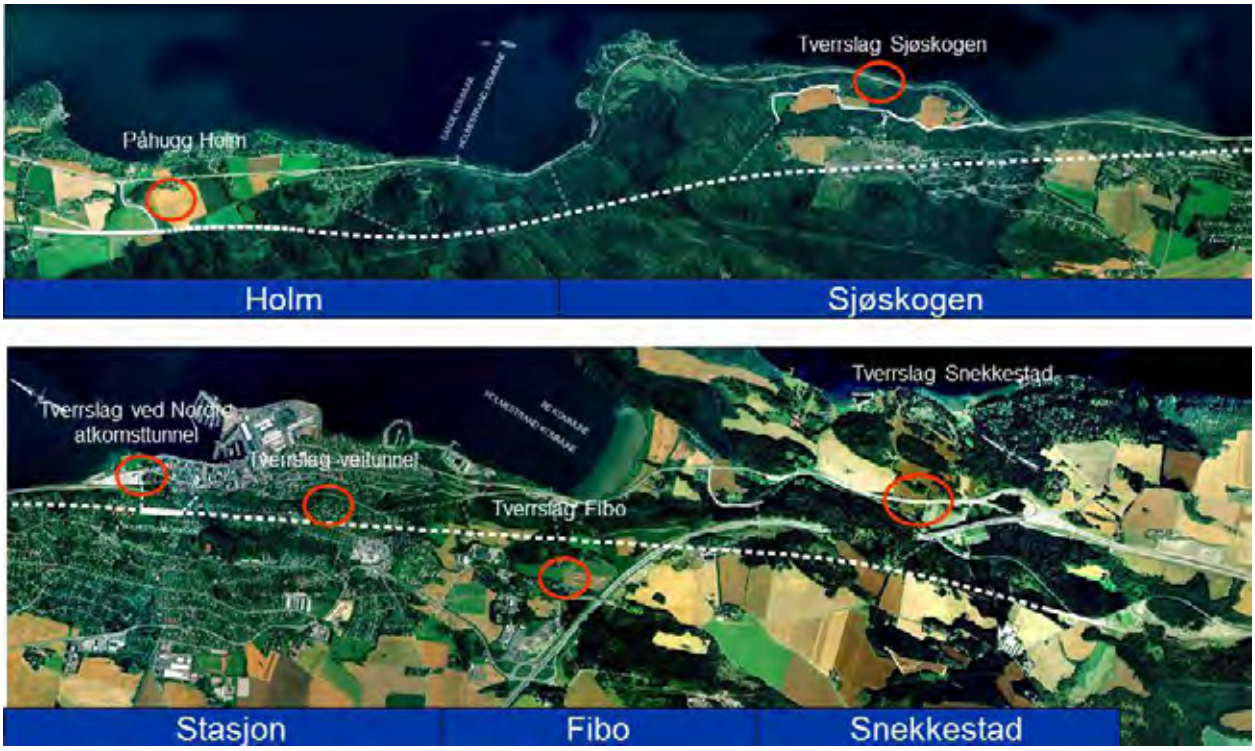


Figure 2. New double track from Holm to Nykirke. The route, contracts and stating points for tunnel excavation.

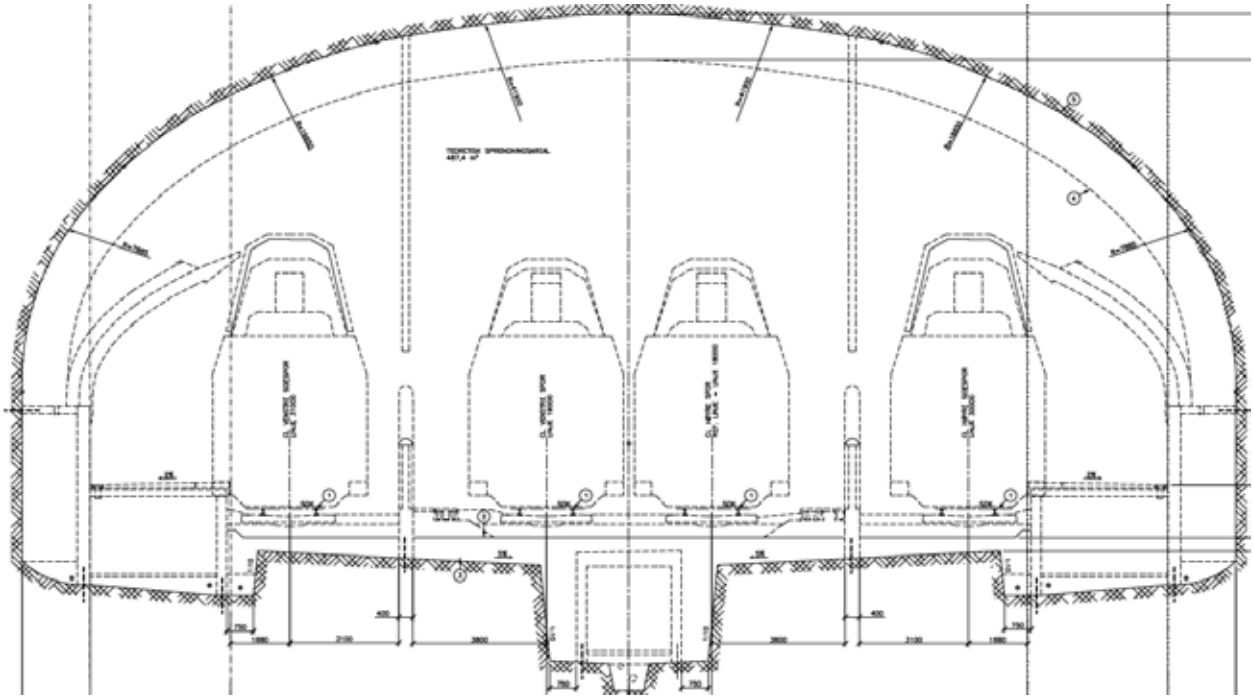


Figure 3: Cross section of the station hall, 550 m²



Figure 4: The up to 70 m high rock slope consist of volcanic rock. The cavern floor is at about the same elevation as the road just below the slope.

Miscellaneous faults exist along the planned railway tunnel, varying in size and they are expected to contain clay. A major fault will influence on the cavern stability over a length of up to 100 m.

2.3 Soil masses

Above the tunnel we find what is known as the Holmestrand plateau. The plateau is partially covered by vegetation and cultivated fields. The whole area lies below the marine limit and the soil consists mostly of marine sediments. The soil thickness varies, consisting of a non-continuous cover intercepted by bare rock in many places, see Figure 5. The area is characterized by parallel rock ridges and intervening valleys filled with loose material.

2.4 Overburden

The total overburden along the 15–35 m wide cavern section is between 24 and 60. The soil thickness straight above the cavern is generally 0–15 m, while the rock overburden for most of the tunnel is more than 40 m. The lowest rock overburden exists where the total overburden is 24 m, which is above a major fault zone, see Figure 5 for placement of the major fault zone.

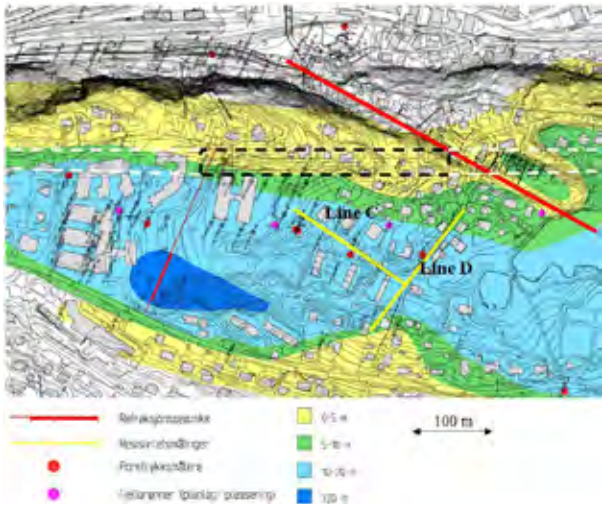


Figure 5. Soil map above the cavern. The Station hall is marked with black and white dotted line and the major fault zone is marked with red line.

3 SITE INVESTIGATION

Detailed site investigations involved on site mapping, drilling, geophysics, laboratory testing, and analysis of historical information. All of which have been conducted to create a thorough understanding of the in situ ground conditions and rock mass parameters. The design modeling is based on information generated though these investigations and expert knowledge. Excavation through the major fault zone, an about 15 m wide fault just south of the station hall which will influence on the stability of the 31 m wide tunnel over a length of 80 – 100 m, is considered to be the main challenge of the project from an engineering geologist's point of view.

An overview of the pre-investigations — field mapping, core drillings and stress measurements — is shown in Figure 6. More stress measurements (3D overcoring and doorstopper) were subsequently carried out during excavation. In addition, two horizontal triple tube core drillings were carried out through the major fault from the tunnel face, before the tunnel was excavated through the major fault zone.

The field mapping included fracture orientation and spacing, fracture length, and fraction friction parameters. In general, the core drillings were used for exploring the bedrock geology, measuring fracture orientation and spacing, estimating rock mass quality, to provide samples for laboratory testing and for estimating rock mass hydraulic conductivity (packer tests – Lugeon values). In addition, two of the vertical boreholes were used for stress measurements (hydraulic fracturing).

Electrical resistivity tomography (ERT), Refraction seismic CPTU and oedometer measures test have been conducted to give an understanding of the soil and groundwater conditions. The main purpose of ERT was to measure the thickness of soils, but data on the rock mass (electric conductivity) was also measured, see Figure 5.

3.1 Stresses

As stress conditions are particularly important for the stability of tunnels with wide spans, several measuring methods were used, and measurements were carried out in several places, as described above.

In conclusion, the stress measurements were interpreted as follows:

- Only gravitational stresses exist south of the major fault and in the fault itself.
- Locked-in (residual) stresses exist north of the major fault:
- The direction of the smallest horizontal stress is almost parallel to the tunnel axis.
- The direction of the largest horizontal stress is almost perpendicular to the tunnel axis.
- The magnitude of the smallest horizontal stress is not smaller than 1.7 MPa.
- The magnitude of the largest horizontal stress is not smaller than 3 MPa and may be about 11 MPa.
- The vertical stress corresponds to the weight of the overburden.

The stress measurements (3D overcoring and doorstopper) carried out during excavation confirmed the results above.

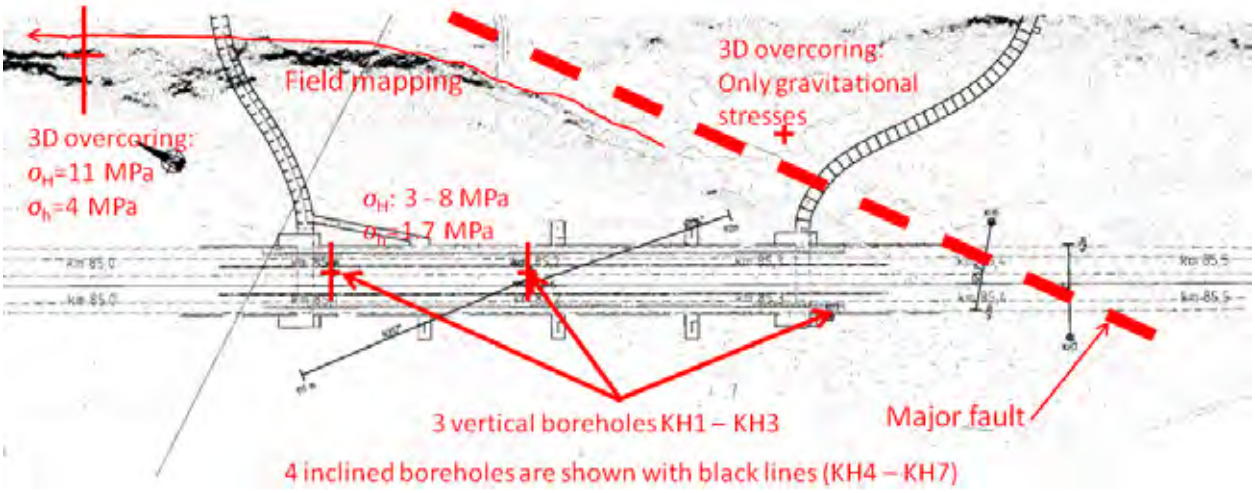


Figure 6: Overview site investigations inside and near the station hall, where KH denotes standard core drilling.

3.2 Overburden

The lowest rock overburden exists where the total overburden is 24 m, which is above a major fault. Rotary drillings carried out during the pre-investigation phase indicated a minimum rock overburden of approximately 8 m in this section, while drillings carried out from the tunnel face during excavation indicated a minimum rock overburden of approximately 6 m.

3.3 Mechanical Properties of Intact Rock

The bedrock is of volcanic origin and Permian age and the thickness of the various lava flows (layers) is 5–15 m. The main body of a lava flow consists of basalt and the tops of the lava flows include fragments of solidified lave that were entrained with the liquid lava. Original gas bubbles in the top layers have subsequently been filled by carbonate minerals. Between the different flows, time breaks are visible as disintegration of the rock mass. This is especially evident in the top layers that also include foreign material added by water and air transportation. Overall, the lava flows can be grouped into two main rock type categories: (i) basalt and (ii) lava breccias/sediments, see Figure 7 and Figure 8.

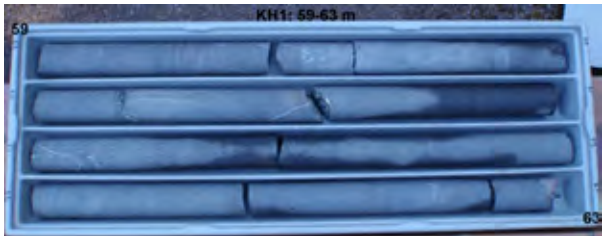


Figure 7: Basalt.

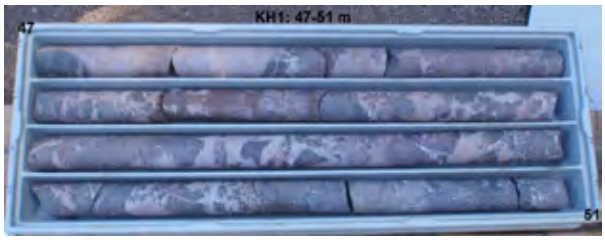
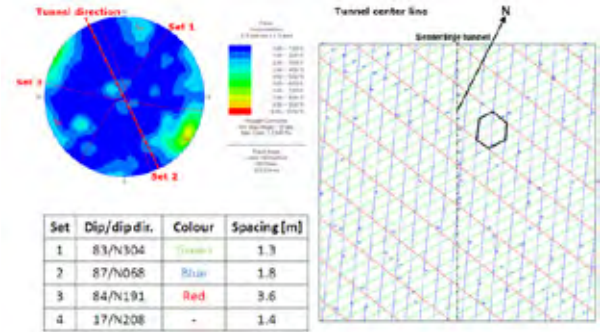


Figure 8: Lava-breccias/-sediments.

The uniaxial compressive strength varies from 35 MPa (weakest lava breccias/sediments) to 197 MPa (strongest basalt), and the average value of all tests is 102 MPa. Young's Modulus varies from 15 Pa to 79 GPa. The tensile strength varies from 4 MPa to 12 MPa. These values are based on a large number of laboratory tests carried out on core samples (intact rock) during the pre-investigations.

3.4 Fractures

Fracture orientations and fracture spacing, as derived from the pre-investigations, are shown in Figure 9. It can be seen that there are three sub-vertical fracture sets and one sub-horizontal fracture set. Geological mapping during excavation showed that the numbers of fracture sets that form blocks together, and their spacing, vary along the length and across the width of the tunnel.



Set	Dip/dip dir.	Colour	Spacing [m]
1	83/N304	Green	1.3
2	87/N068	Blue	1.8
3	84/N191	Red	3.6
4	17/N208	-	1.4

Figure 9: Fracture orientations and fracture spacing, as derived from the pre-investigations. Upper left: stereo plot. Right: section of the station hall viewed from above with traces of the three sub-vertical fracture sets shown with their true average spacing. The black polygon indicates one of many columns formed by the three sub-vertical fracture sets.

Data that can be used to estimate fracture friction profiles were available before the detailed planning of the station hall started. Q-value parameters had been mapped in 87 places, in basalt over a distance of about 10 km, including in the station hall area. The most frequent friction angle, derived from joint roughness (Jr) and joint alteration (Ja) values, was 45°. During the pre-investigations for the station hall, the joint roughness coefficient (JRC) of a large number of sub-vertical fractures was measured together with the joint compressive strength (JCS), the latter by using a Schmidt Hammer. These data gave a friction angle of 44° using average values for JRC and JCS and normal fracture stresses as assumed from the stress measurements.

3.5 Rock Mass Classification

Based on the pre-investigations the typical Q-value was estimated to be 13, corresponding to "good" rock: Qtypical = RQD/Jn × Jr/Ja × Jw/SRF = 95/15 × 2/1 × 1/1 = 13, where RQD is the degree of jointing, Jn is the number of joint sets, Jr is the joint roughness factor, Ja is the joint alternation factor, Jw is the joint water reduction factor and SRF is the stress reduction factor. The geological strength index (GSI) was estimated to be 67.

3.6 Weakness Zones

The pre-investigations showed a few minor weakness zones and a major fault. The tunnel has a 31 m wide span where the major fault intersects the tunnel over a length of approximately 80-100 m. The major fault is approximately 7 m wide, and the poorest part consists of rock mass with a Q-value of approximately 0.01, corresponding to "extremely poor" rock. The total overburden in the area of the major fault is 24 m and the minimum cover of poor rock is about 6 m. The leakage from both horizontal boreholes (triple tube core drilling) drilled through the major fault from the tunnel face was 0.23 liters per meter borehole. The average Lugeon value derived from packer tests was 40 L ("medium") in one of the boreholes, and 85 L ("high") in the other. Results from core drilling conducted from the tunnel face stated that the width of the fault zone is smaller than estimated from the pre-investigations, see Figure 10.



Figure 10: KH 11 drilled from the tunnel face through the major fault.

3.7 Soil cover, waterproofing requirements and hydrology

Soil cover maps have been prepared on the basis of 2D resistivity measurements and seismic refraction surveys in addition to probe core drilling, see Figure 5. Water loss measurements have been carried in core boreholes showing an estimated water ingress in the range of 850 l/min per 100 m of tunnel for the station hall, where the overburden is 50 m. This indicates that, without injection, the tunnel will quickly drain the surrounding rock and soil. Waterproofing requirements have been defined out of consideration for the natural environment, the lifetime of the tunnel structure and maintenance. Considerations relating to reliability, availability, maintainability and safety (RAMS) were decisive in defining the requirements.

A waterproofing requirement with a maximum water ingress of 5 l/min per 100 m tunnel has been defined for the station hall. This involves extensive and systematic pre-injection through a large number of injection holes (at intervals of no more than 1.5 m) with overlaps of at least 10 m between the grouting fans, in addition dry inspection holes prior to each blast. The grouting fans are 22 m long.

4 MODELING OF TUNNEL STABILITY

4.1 Modeling of the rock mass behavior in an unsupported tunnel

The distinct element codes UDEC (two-dimensional) and 3DEC (three-dimensional) were used to model the rock mass behavior of an unsupported station hall. The main conclusion from the distinct element modeling is that both the stress conditions and the location of the intersection between fractures belonging to fracture set 4 and the tunnel crown are very critical for the overall stability of the tunnel.

4.2 Modeling of supported tunnels

The two-dimensional finite element code Phase2 was used to model supported tunnels under different stress regimes. A number of models were computed for tunnel cross sections located at four different tunnel locations: (i) at the center of the station hall, (ii) at the intersection between the access tunnel and the station hall, (iii) at the location of the major fault where the span is 30.5m and (iv) just south of the major fault where the span is 31 m. (i) and (ii) were computed for different stress regimes, all including locked-in stresses, and (iii) and (iv) were computed for gravitational stresses only. Design of the rock support is based on results from modeling.

5 ROCK SUPPORT

The rock quality was as predicted in the pre-investigations, with Q values of 15<Q<40 north of the weakness zone and of 4<Q<15 south of the weakness zone. Based on the results from pre-investigation and modeling, the largest theoretical wedge at a width of 34.5 m was calculated to be 1,840 m^3. Based on these calculations and the modeling, the minimum rock support north of the major fault zone should be 6-12 m long rock bolts spacing c/c 2.2 m and fiber reinforced shotcrete minimum 20 cm thickness according to Figure 11.

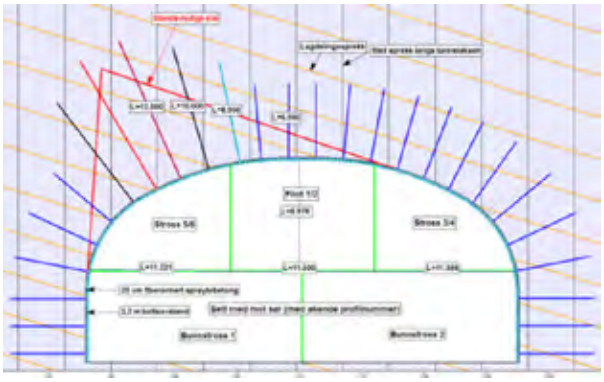


Figure 11: Minimum rock support north of the major fault zone.

South of the major fault zone, the minimum rock support should be 6-12 m long rock bolts spacing c/c 2 m and fiber reinforced shotcrete minimum 20 cm thickness according to Figure 12.

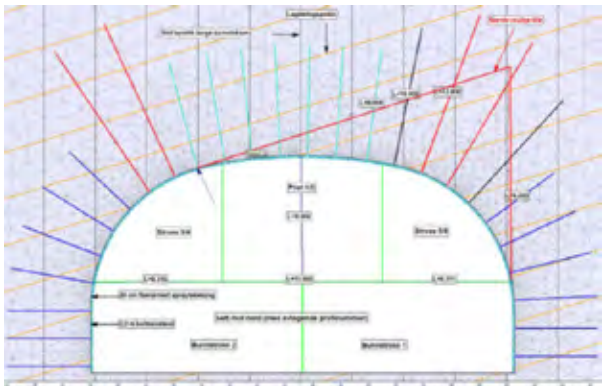


Figure 12: Minimum rock support south of the major fault zone.

Rock support through the fault zone consist of spiling, fiber reinforced sprayed concrete, rock bolts and lattice girders spacing c/c 1 m incased by sprayed concrete (reinforced lining).

6 EXCAVATION AND GROUTING

6.1 Excavation

The station hall was excavated from two directions, from two cross-cuts approximately 600 m apart. This meant that the work could be carried out from four faces, see Figure 13.



Figure 13: Excavation directions.

The cross section was excavated in several stages and the whole gallery was excavated before starting with the lowest part of the cavern, the bench. The order of work for the gallery was: center of the gallery is driven 1 or 2 rounds (5-10m), one of strosses is driven to the same chainage as the pilot, and the other stross was driven to the same chainage as the pilot. Rock support is carried out after each blast round.



Figure 14. One round in the center of the gallery is excavated.

The bench was excavated in two parts, see Figure 15.



Figure 15. Excavation of the bench.

Once the bottom bench had been excavated, excavating for emergency stairwells, lift shafts and permanent access tunnels started. The work on the stairwells and lift shafts was very extensive, as the geometry was unfavorable from an engineering geology perspective and involved up to 50 m wide spans.

6.2 High Pressure Grouting

As mentioned above, there are residential and recreation areas straight above the station hall. There are also areas of great soil thickness. Lowering the groundwater could have major consequences in this area and must be avoided. Injection was planned as continuous pre-injection throughout the length of the tunnel, regardless



Figure 16: Excavation of the northern access.

of whether water was encountered or not. Generally, standard cement would be used for injection, but there was also the option of using finer injection cements. High injection pressures of 45–90 bar were used, and the injection fan was designed so that the holes would be no more than 2-2.5 m apart and would not be punctured by bolts.

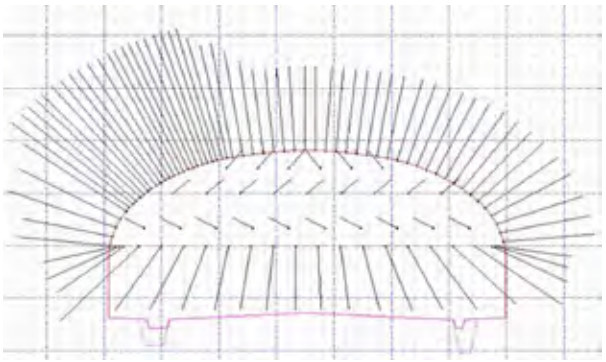


Figure 17: Drill pattern for grouting fan.

Drill pattern for grouting fan north of the main fault zone is shown in Figure 17. The extra holes on the left are at different angles and of different lengths avoid puncturing the grouting fan when drilling the 12 m long rock anchors. A fan like this consists of 100 holes and approximately 2,300 drill meters. It would take two jumbos 10 hours to drill a fan like this. In the station hall, injection was carried out every 10 m and grout was applied to the whole cross-section from the same position. Grouting was also carried out in the sole and face of the tunnel, and also in niches, stairwell and lift. The injection holes were 22.5 m long, so as to provide an overlap of approximately 10 m between the injection fans. The achieved end pressure, the injected amount, or a combination of the two, was used as the stop criteria.

6.3 Twelve meters Rock bolts

Rock anchors with lengths of up to as much as 12 m have been installed. This was unique in a global perspective and a major challenge in the project. The bolts were required to withstand loads of up to 40 tons. Rock anchor similar to what is known as “CT bolts” was developed in collaboration with Vik Ørsta. Despite the length and 12 m it was also possible to install the bolt at a smaller angle to the rock face.

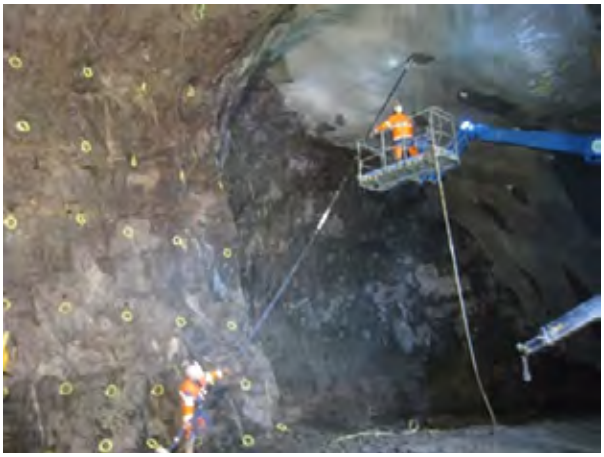


Figure 18: Installation of 12 m CT bolt.

6.4 Crossing the Main Fault Zone

The main challenge of excavating the station hall was crossing the major fault zone south of the cavern.
A successful grouting very often improves the rock quality by reducing the flow of water and increasing friction in joints. That is why effective grouting is also called “half a concrete casting”. As there was less rock overburden, it was necessary to reduce the grouting pressure, and it was decided to grout in two steps, using finer grouting cement in the second step.

Excavation was done with the following procedure; the pilot was excavated along one side until it reached the zone. Grouting was done every 10 m, every second hole was drilled in the first round and grouted with standard cement to fill the main joints and create a kind of umbrella for the second round. The second round of holes were drilled at a slightly smaller angle and grouted with micro-cement to close the finer joints. The pressure and volume were adjusted to the prevailing conditions. The grouting holes in the side towards the fracture zone were adjusted to penetrate into and through the zone, see Figure 19.

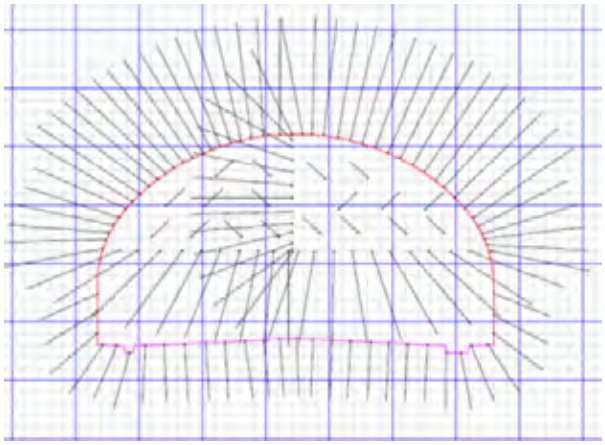


Figure 19: Grouting fan through the main fault zone.

Blasting length was reduced to 2.5 m and horizontal spiles consisting of c/c 0.3 m self-drilling hollow bars were installed in lengths of 9 m after every second blast round. After each round 25 cm of shotcrete was applied, and the surface was modeled before installation of lattice girders spacing c/c 1.0 m, see Figure 20. The lattice girders were covered with 8-10 cm of shotcrete. Temporary vertical lattice girders were installed to support the permanent lattice girders against the side blasts. Rock bolts were installed between every lattice girder.



Figure 20. Lattice grider spacing c/c 1.0 m.

For every grouting fan, probe holes were drilled to verify rock overburden. The lowest measurements showed an overburden of 5–6 m. The lengths of bolts were adjusted to the rock overburden.

The same procedure was followed when opening the side bench. After the top gallery had been excavated, the lattice girders were extended to the permanent sole of the cavern, in step with the blasting of the bench, see Figure 21.



Figure 21. Excavation of the bench through the main fault zone.



Figure 22. The Station hall after completion of the excavation.

7 PROJECT COMPLETION

As soon as civil works in the station hall was completed, the railway-technical work started along with the fitting out and interior decoration of the new station. The station will be fully fitted out with track system, contact lines, electrical works, HVAC, telecommunications and signaling systems before the end of 2016, when this spectacular facility is scheduled to be completed and opened.

REFERENCES

General reference for the geotechnical work described in this article and photo/illustrations are from reports and articles that have been prepared on behalf of the Norwegian National Rail Administration.



Underground car park in Arendal
Courtesy: Veidekke ASA

11. UNDERGROUND CAR PARKS IN BERGEN

SALOMONSEN, Astrid

1 INTRODUCTION

There are a number of underground car parks in Bergen, in particular in built-up areas, where the object is often to free space for other urban activities or structures. As the city is aiming to become car free, it is essential to utilise the possibilities underground, and in Bergen, a city surrounded by seven mountains, there is no shortage of possible locations for such parking facilities.

Underground car parks provide good space utilisation in cramped urban environments and help to inject new life into city centres.

Veidekke Entreprenør AS has twenty years' experience of building subterranean parking facilities. Since 1996 the company has improved and developed good technical solutions – taking an individual and tailored approach to each project. The company's Bergen branch (Veidekke AS, distrikt Bergen og Samferdsel) has constructed several of city's underground car parks.

2 STRAUME CAR PARK – 2014

Norway's largest underground parking facility is the Kystbygarasjen car park in Straume, just outside Bergen. The car park has 1500 parking spaces, and is located partly under the Sartor shopping mall. The construction period was from May 2012 until November 2014, and the contract sum was about NOK 330 million.

The developer is the Sartor Holding company Straume Parkering AS. Sartor Holding is working on a development in Straume called Sotra Kystby. In addition to the car park, this project includes the construction of housing, commercial buildings, a cinema and a hotel, and the extension of the local Sartor mall.

The parking facility is linked to its surroundings via four shafts, each about 40 m in length, which come to the surface in the centre of Straume, with one emerging inside the Sartor shopping mall. The shafts were constructed using the slip form method and were built in two stages, each lasting about 15 days.

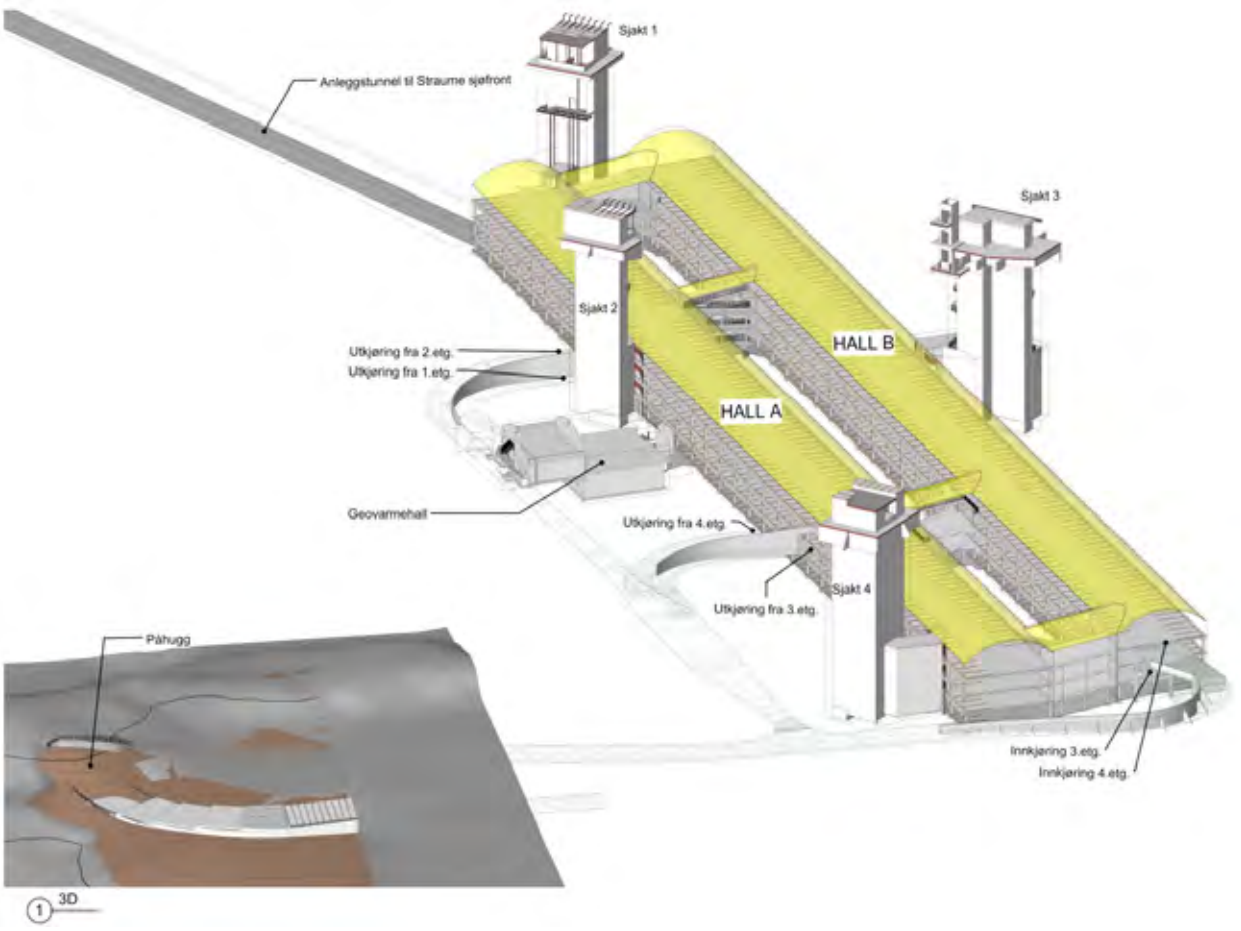
The facility consists of two parallel caverns that are 250 m long, 16 m wide and 17 m high, and which are connected via four cross adits. In all, about 215 000 m^3 solid mass was removed, the equivalent of 22 000 truckloads.





Tunnels	Length (m)	Cross-section (m^2)	Volume (m^3)
Access – entrance	250 m	60 m^2	15 000 m^3
Access – exit	240 m + 60 m	60 m^2	18 000 m^3
Cavern A	250 m	266 m^2	66 500 m^3
Cavern B	250 m	266 m^2	66 500 m^3
Site access tunnel	365 m	45 m^2	16 425 m^3
Shaft 1	39 m	120 m^2	4 680 m^3
Shaft 2	40 m	90 m^2	3 600 m^3
Shaft 3	37 m	145 m^2	5 365 m^3
Shaft 4	38.5 m	115 m^2	4 427.5 m^3
Geothermal heating plant	35 m	215 m^2	7 525 m^3
Total volume			**216 663 m^3**

Table 2: Amounts of excavated rock removed from the Straume car park





Each cavern has four levels, the upper three of which consist of concrete elements that are 16 m in length. This gives a column-free parking facility. The car park floor has been coated with Flowcrete, which helps to give the facility a light and agreeable appearance. The ceiling vault in white aluminium prevents dripping and problems with water and frost.

On the ground level, 163 energy boreholes have been drilled to a depth of 200 m to collect geothermal energy. The geothermal energy is collected via water wells in the geothermal power plant. The pond in the plant stores 998 m^3 water. It is intended that the geothermal power plant in the parking facility will supply geothermal energy in the form of heating and cooling to the shopping mall and the whole of the local area. Its construction was an extra job that was carried out without any changes in the work schedule, at a cost of about NOK 100 million.

The portal has been constructed in concrete and natural stone, and has been covered with soil so as to blend into the terrain.

To protect the local environment from heavy transport, a 350 metre-long site access tunnel was built, through which excavated material was transported for disposal. This tunnel runs from the end of the parking facility to the Straumsund sound, where most of the material was deposited as fill. Veidekke Eiendom is going to build 250 dwellings, a park area, a jetty and commercial buildings on this fill.

Geological conditions during excavation of the facility were good. The rock encountered was basically sound, hard rock, with the exception of surface rock at the tunnel entrance and local areas of weak rock and clay. Reinforced concrete arches were used to provide support in these areas. The facility lies at a relatively shallow depth, and high groundwater, together with many fissures, in the rock has resulted in substantial water seepage in the facility. Grouting or other form of heavy support has not been necessary.

3 HARALDSPLASS CAR PARK – 2016

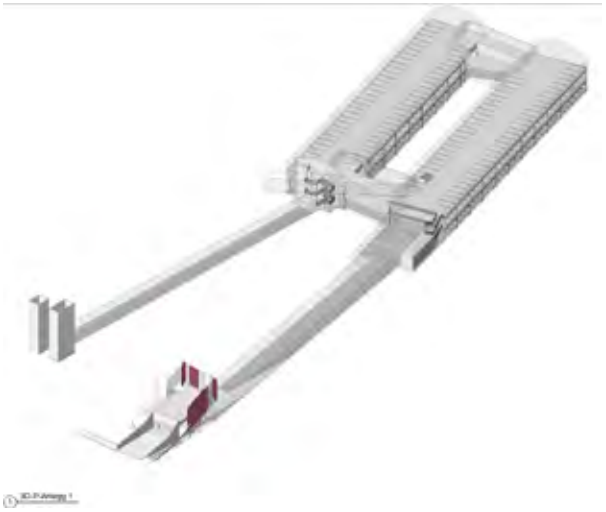
The Haraldsplass Diaconal Foundation is in the process of building an extension to Haraldsplass Hospital. To free space for the construction of a new ward wing, it has been necessary to move all parking spaces. An underground parking facility is being built behind the hospital, inside Bergen's highest mountain, Ulriken. The contract sum for this facility is about NOK 100 million. The construction period is November 2014 to March 2016.

The facility will have a common entrance and exit and a pedestrian tunnel leading to the new ward wing.

Just above the portal lies the longest linden tree-lined avenue in the Nordic region. The avenue is a listed heritage site and its trees are insured for NOK 100 000 each. The position of the portal was dictated by this avenue, and because the rock was lower than anticipated, sheet piling has been set up 5 m from the tree trunks. This gives an entrance with a gradient of 1:6.4.

The car park consists of two parallel caverns that are 82





Picture X 3D-drawing of the parking facility behind Haraldsplass Hospital

m long, 16 m wide and 12 m high. The access tunnel is 140 m long whilst the pedestrian tunnel is 90 m in length.

In total about 45 000 m^3 solid mass has been removed, which is equivalent to 4 500 truckloads.

Many of the technical solutions are similar to those used in earlier parking facilities. The method of excavation is conventional tunnelling. The levels or floors are constructed of concrete elements and the ceiling vault is aluminium. Project organisation and technical partners are almost the same as for the Straume car park.

Lift shafts and technical rooms have been delivered in prefabricated concrete elements, giving the project gains in terms of progress.
A ventilation shaft has been cut out using wire. The shaft is about 15 m high and has a cross-section of 8 m^2. It runs from the access tunnel up to the surface just behind the hospital.

At the start of cutting, there was a great deal of surface rock, and this extended throughout the access entrance/ exit. There were challenges in view of the shallow overburden underneath the hospital.

Inside Ulriken the rock was very good, solid gneiss, and there was little water ingress in the caverns.

Just above the entrance and technical rooms there is a German tunnel dating from the Second World War, and minimum overburden between the new structures and this tunnel was 2 m. This was a problem area that required extra support in the form of spiling and concrete reinforcing arches. The presence of many fissures gave substantial water seepage in this area.



Tunnels	Length (m)	Cross-section (m^2)	Volume (m^3)
Access tunnel	140 m	38 m^2	5 320 m^3
Cavern A	82 m	195 m^2	15 990 m^3
Cavern B	82 m	195 m^2	15 990 m^3
Adits 2	12 m	120 m^2	1 440m^3*2=2 880 m^3
Pedestrian tunnel	90 m	25 m^2	2 250 m^3
Shaft	15 m	8 m^2	120 m^3
Total volume			**42 550 m^3**

Table 4: Amounts of excavated rock removed from Haraldsplass car park



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Lysebotn II power station
Courtesy: Norconsult AS

12. THE DEVELOPMENT OF UNDERGROUND HYDROPOWER PLANTS IN NORWAY

BROCH, Einar
EDVARDSSON, Sverre

1 INTRODUCTION

Climatic, topographical, and geological conditions in Norway are favourable for the development of hydroelectric energy. The rocks are of Precambrian and Paleozoic age, and although there is a wide variety of rock types, highly metamorphic rocks predominate. From an engineering point of view they may in general be classified as typical hard rocks.

Approximately 99% of a total annual production of 140 TWh of electric energy in Norway is generated from hydropower. Figure 1 shows the installed production capacity of Norwegian hydroelectric power plants. It is interesting to note that, for new plants constructed since 1950, underground powerhouses are predominant. In fact, of the world's 600-700 underground powerhouses about one-third, i.e. over 200, are located in Norway. Another proof that the Norwegian electricity industry is an "underground industry" is that it today has close to 4500 km of tunnels. During the period 1960 - 90 an average of 100 km of tunnels was excavated every year. During the 1990s the activity was low, but it has picked up again after the turn of the millennium due to increased activity on upgrading and enlargement of existing schemes.

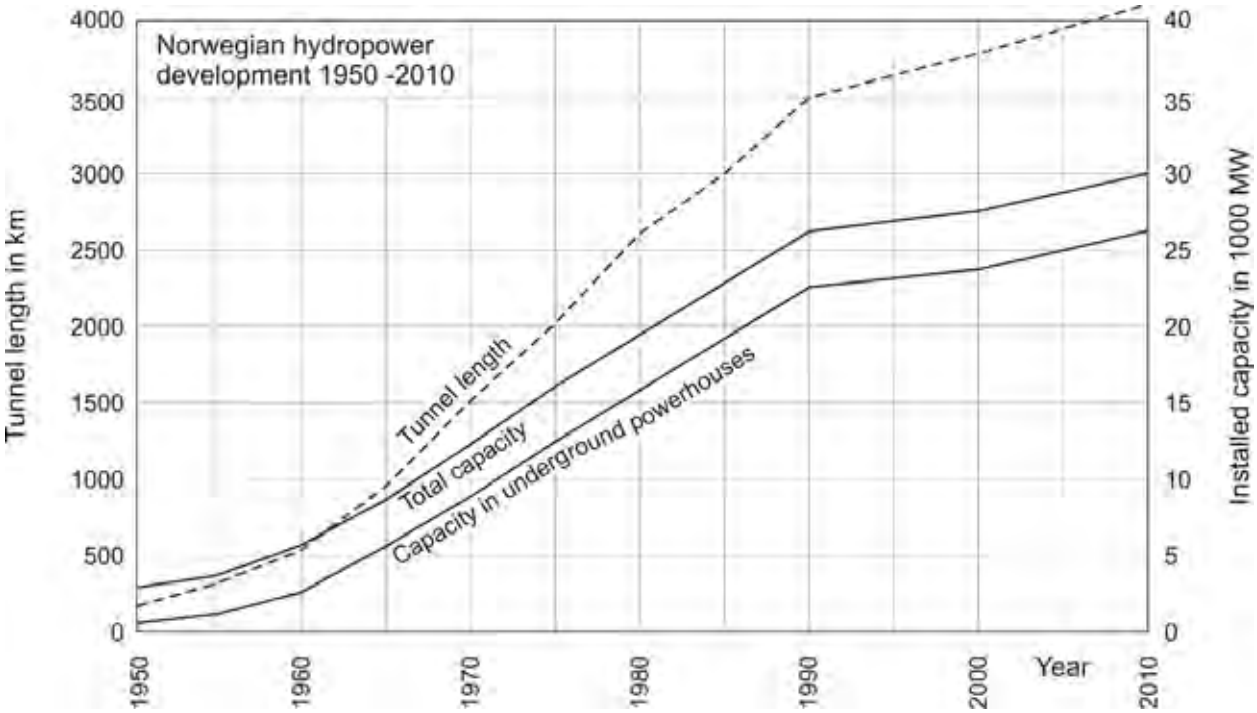


Figure 1: The development of Norwegian hydroelectric power production capacity and the accumulated length of tunnels excavated for the period 1950 -2010.

Through the design, construction, and operation of all these tunnels and underground powerhouses, valuable experience was gained. This experience has been of great importance for the general development of tunnelling technology, and not least for the use of the underground for other kinds of facilities. The many underground powerhouses excavated in rock masses of varying quality are to a large extent the forerunners for the varied use of rock caverns which we find all around the world today.

Example of an underground powerhouse from the early 1950s is shown in Figure 2. In this case a concrete building has simply been constructed inside a rock cavern. The powerhouse has in fact false windows to give the operators a feeling of being at surface rather than underground.



Figure 2: The Aura underground hydropower station, commissioned in 1952

Later operators and other personnel became more confident in working and staying underground, and powerhouses were constructed with exposed rock walls, often illuminated to show the beauty of rock such as demonstrated by two powerhouses commissioned around 1970 and shown in Figure 3.

Also, special techniques and design concepts have over the years been developed by the hydropower industry. One such Norwegian speciality is unlined, high-pressure tunnels and shafts, Broch (1982B, 2000). Another is the so-called air cushion surge chamber which replaces the conventional vented surge chamber. A number of power plants with air cushions were constructed in Norway in the 1970s. These specialities are described in an earlier volume of this publication series, Broch (2002).

Most of the Norwegian hydropower tunnels have concrete or shotcrete lining along only 2-4% of the length. Only in a few cases it has been necessary to increase this to 40 - 60%. The low percentage of lining is due not only to favourable tunnelling conditions. It is first and foremost the consequence of a support philosophy which accepts some falling rocks during the operational life of a water tunnel. A reasonable number of rock fragments spread out along the tunnel will not harm the tunnel or disturb the plant operation provided a rock trap of adequate design is located at the downstream end of the headrace tunnel. Serious collapses or local blockages of the tunnel must, of course, be prevented by the use of heavy support or concrete lining where needed.

2 EARLY REASONS FOR GOING UNDERGROUND

During and shortly after the First World War there was a shortage of steel leading to uncertain delivery and very high prices. At that time the traditional design was to bring the water from the intake reservoir, or from the downstream end of a headrace tunnel, to the powerhouse through a steel penstock at surface. Both the penstock and the powerhouse were above ground structures as shown in Figure 4.

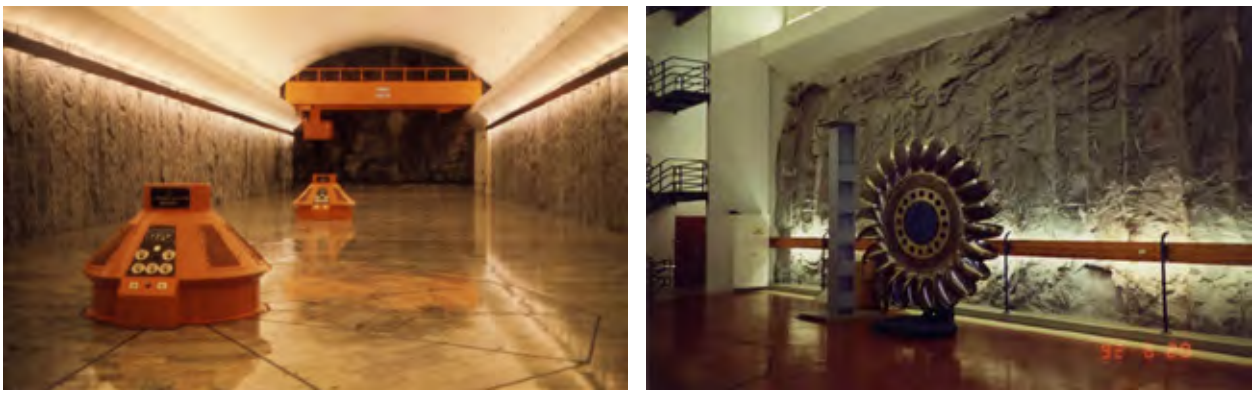


Figure 3: Åna-Sira (left) and Tafjord K-5 (right) underground powerhouses

During and shortly after First World War there was a shortage of steel. With the lack of steel for a penstock, the obvious alternative was to try to convey the water as close to the powerhouse as possible through a tunnel or a shaft. As a result, four Norwegian hydropower stations with unlined pressure shafts were put in operation during the years 1919-21, with water heads in the range 72 to 152 m. One (Skar) was a complete failure due to too low overburden of rock, only 22 m rock cover where the internal water head was 116 m. Another (Toklev) has operated without any problem ever since it was first commissioned.

Although three out of four pressure shafts constructed around 1920 were operating to satisfaction after some initial problems had been solved, it took almost 40 years for the record of 152 m of water head in unlined rock a (Svelgen) to be beaten. Through 1958, nine more unlined pressure shafts were constructed, but all had water heads below 100 m. Until around 1950 powerhouse and penstock located at surface was the conventional layout for hydropower plants as demonstrated in Figure 4.

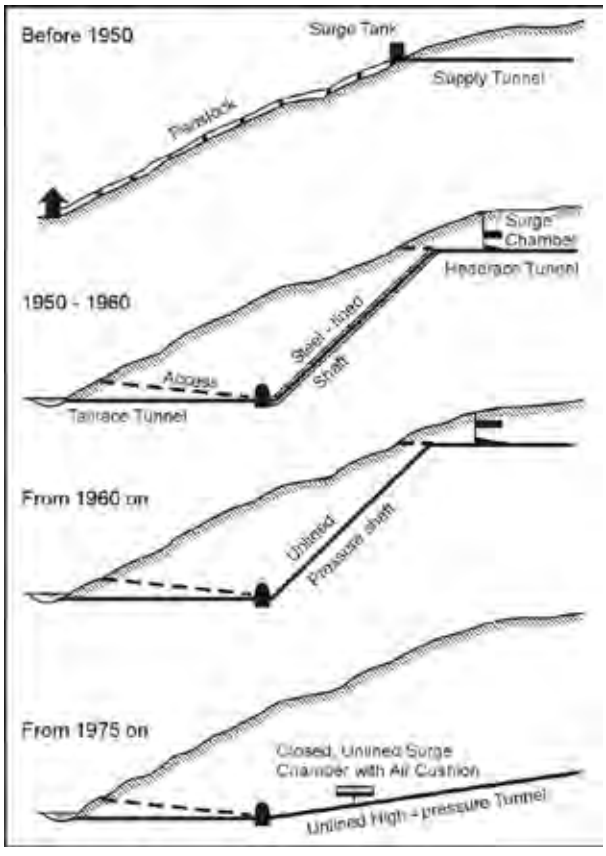


Figure 4. The development of the general layout of hydroelectric plants in Norway

3 DEVELOPMENT AFTER 1945

In a few early cases, underground location of a powerhouse was for topographical reasons the only possible option (Bjørkåsen,1921). During and after the Second World War, the underground was given preference by considerations of wartime security. But with the rapid advances in rock excavation methods and equipment after the war, and consequent lowering of the costs, underground location became the most economic solution. This also tied in with the development of steel-lined, and later unlined, pressure shafts and tunnels, giving the designer a freedom of layout quite independent of the surface topography.

Except for the smallest hydropower stations, powerhouses are now located underground whenever topography and rock conditions are suitable.

Frequently the overall project layout requires a deep-seated powerhouse where in situ rock stresses may be substantial, and with more than 1 km long access tunnels. This requires assessment of the rock stress regime in advance of the excavation of the powerhouse complex. Based on a preliminary rock stress model, mainly derived from the topography, together with strike and dip of foliation of the rock mass and mapping of any detectible fault zones, the most favourable location and orientation of the powerhouse cavern and the optimum shape and alignments of ancillary tunnels and caverns, are determined.

Even extensive investigations and analyses in advance may leave uncertainty as regards the most favourable placement and orientation of the power house complex. In such cases the final decisions are left until the excavation has started, the approach known as “design as you go”. At intervals during the advance of the access tunnel, the in situ rock stresses are determined by hydro-fracturing or the over core drilling method. From the measurement results, the initial stress model will be modified, a realistic pattern of the rock stress regime established and the powerhouse location will be confirmed or modified. Exact location of fault zones may be determined by exploratory drilling of long holes (50 – 100m) from the face of the tunnel as it approaches the powerhouse cavern, and final adjustments made of orientation and position. The above procedure requires some initial flexibility in the layout of the powerhouse complex, which will be finally fixed during construction. The approach has to be properly reflected in the civil works contract to secure a smoothly managed period of construction.

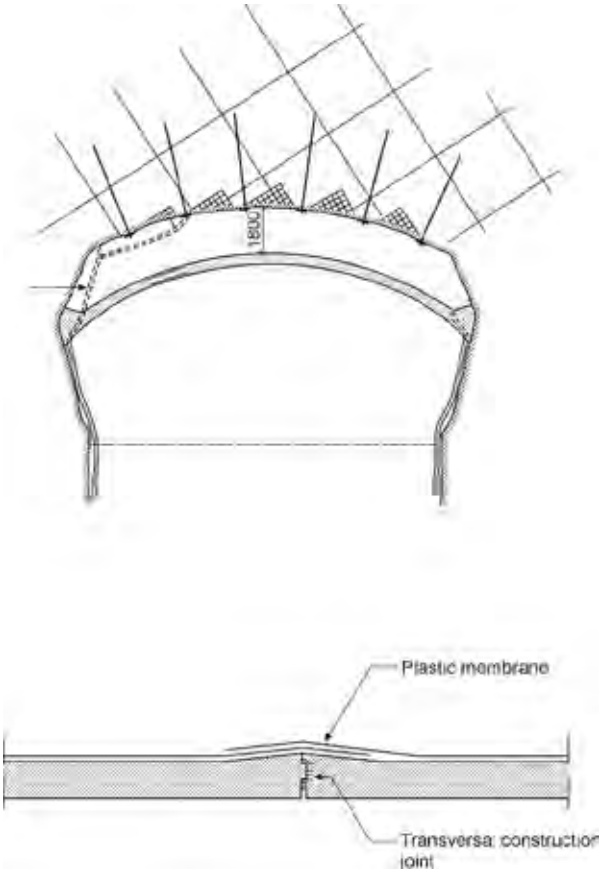


Figure 5: Typical design for a free span concrete arch.

In the early powerhouse caverns the direct rock support of the ceiling was in most cases limited to rock bolting. To safeguard equipment and operators in the machine hall against rockfalls, a 25 – 30 cm thick free spanning cast in place concrete arch was constructed below the ceiling, see Figure 5. In poor rock mass, the ceiling was in some cases reinforced by a compact concrete arch in combination with rock bolts, directly supporting the ceiling. In that case a light arch ceiling would be suspended below the roof arch to improve appearance and to intercept any water leakage.

The present-day solution prescribes systematic bolting of the rock ceiling following immediately after excavation of the top heading, followed by fibre-reinforced shotcrete, with thickness adapted to rock conditions. A common solution for the supporting structure of the powerhouse crane is the rock-bolt supported crane beam. A massive concrete beam anchored to the cavern wall by long grouted rock bolts is constructed right after the excavation of the top heading, see Figure 6.

In this way the powerhouse crane can be installed and be available for the concrete works and installation of draft tubes or turbine pit linings etc. as soon as excavation

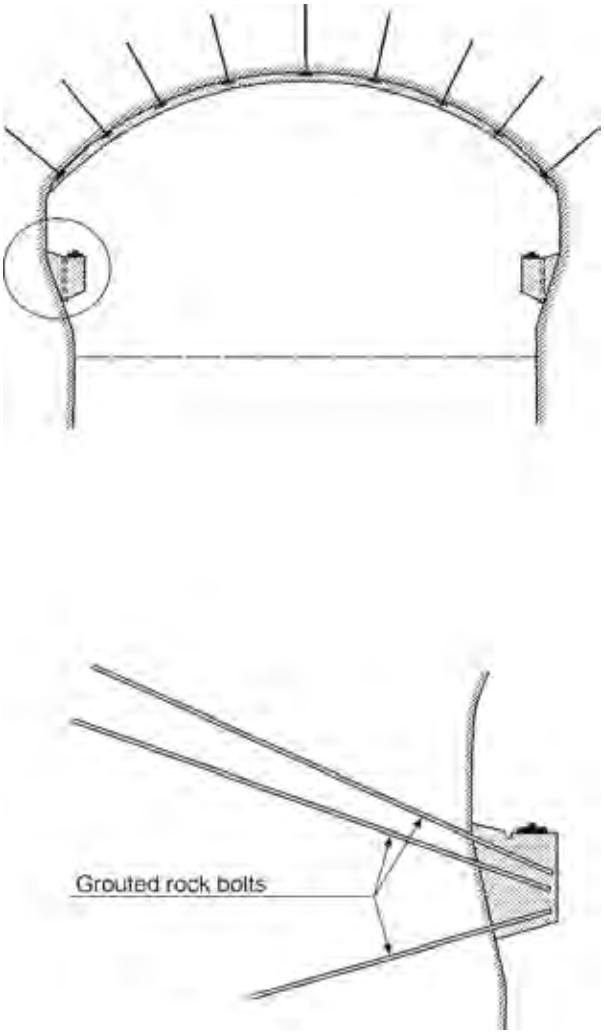


Figure 6: Steelfibre reinforced shotcrete arch and rock bolt supported crane beam

and support of the powerhouse cavern is completed. The rock bolt supported crane beams may either be designed for carrying the heaviest generator loads during erection, or be designed for loads occurring during earlier stages of erection only, for later being underpinned by columns to the machine hall floor level.

In the early stages underground powerhouses were designed with one rock cavern only as the most cost efficient arrangement. Typical transformer arrangements for one-cavern layouts are shown in Figure 7 (a, b and c).

In later years operation and maintenance considerations, including safety aspects and consequences of possible transformer explosions, have been overriding the earlier main focus on savings in project costs. In an underground power plant the main transformers represent the most serious risk in this respect. Today therefore new

underground plants will be designed with a separate transformer cavern in parallel with the powerhouse cavern, at a sufficient distance in between to secure the overall rock stability of the complex without detrimental deformation of cavern walls. This layout gives much better control of consequences of transformer explosions with substantially reduced risks for damage of other vital equipment in the powerhouse.

The twin cavern solution also gives the option to locate the operation machinery for the draft tube gates, in the transformer cavern. See Figure 7 (d).

When the hydropower powerhouses for safety reasons went underground in the early 1950's, the surface steel penstocks went underground as well. Thus, for more than a decade most pressure shafts were fully steel-lined, in a few cases at the beginning as steel penstocks installed freely in the shafts, but soon the design changed to fully embedment in concrete. During the period 1950-65, a total of 36 steel-lined shafts with heads varying from 50 to 967 m (with an average of 310 m) were constructed.

In Norway the rock mass surrounding a sub-surface facility is considered to be part of the underground structure and thus as an integrated part of the design. Many places around the World the layout of an underground powerhouse will be more or less equal to a powerhouse located at surface. In Norway the walls of rock cavern walls will normally act as the outer walls without being covered by concrete. Loads and forces acting on the turbines and generators are transferred by concrete embedment and slabs directly to the surrounding rock. This allows for minimizing the excavated volume and reduction in concrete consumption. The volume of concrete in underground powerhouses in Norway is normally around 20 % of the excavated volume.

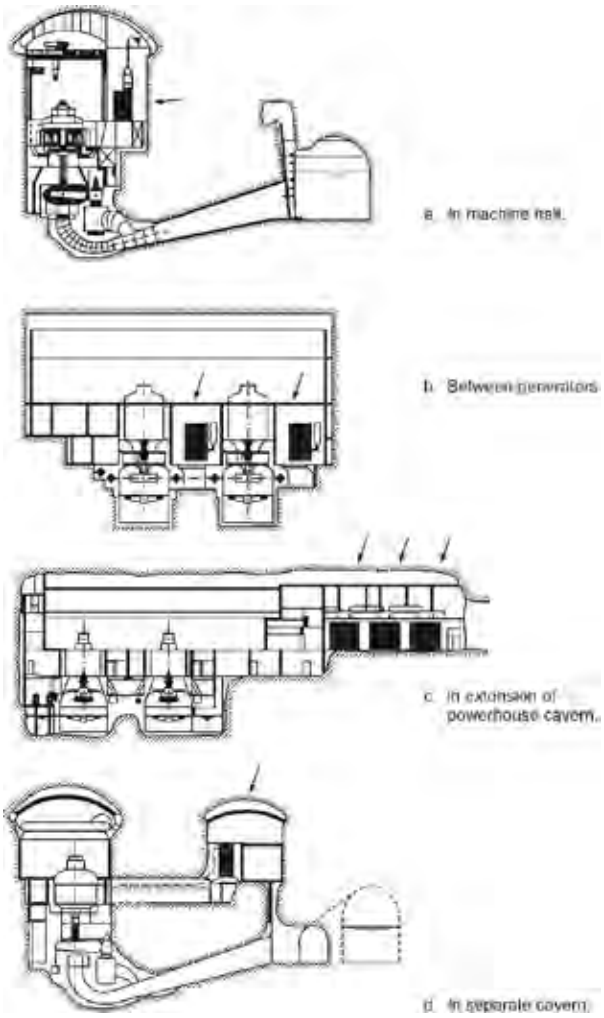


Figure 7: Common transformer locations.

4 UNDERGROUND HYDROPOWER PLANTS WITH UNLINED WATERWAYS

The new record shaft of 286 m at Tafjord K3, which was put into operation successfully in 1958, gave the industry new confidence in unlined shafts. As Figure 8 shows, new unlined shafts were constructed in the early 1960's and since 1965 unlined pressure shafts have been the conventional solution. Today more than 80 unlined high-pressure shafts or tunnels with water heads above 150 m are successfully operating in Norway, the highest head being 1000m. Figure 8 clearly demonstrates that increasing water heads reflect an increasing confidence in unlined pressure shafts.

The confidence in the tightness of unlined rock masses increased in 1973 when the first closed, unlined surge chamber with an air cushion was successfully put into service at the Driva hydroelectric power plant. This innovation in surge chamber design is described in detail by Rathe (1975). The bottom sketch in Figure 2 shows how the new design influences the general layout of a hydropower plant. The steeply inclined pressure shaft, normally at 45o, is replaced by a slightly inclined tunnel, 1:10 - 1:15. Instead of the conventional vented surge chamber near the top of the pressure shaft, a closed chamber is excavated somewhere along the high-pressure tunnel, preferably not too far from the powerhouse. After the tunnel system is filled with water, compressed air is pumped into the surge chamber. The compressed air acts as a cushion to control the water hammer effect on the hydraulic machinery and the waterways, and for ensuring the operational stability of the plant.

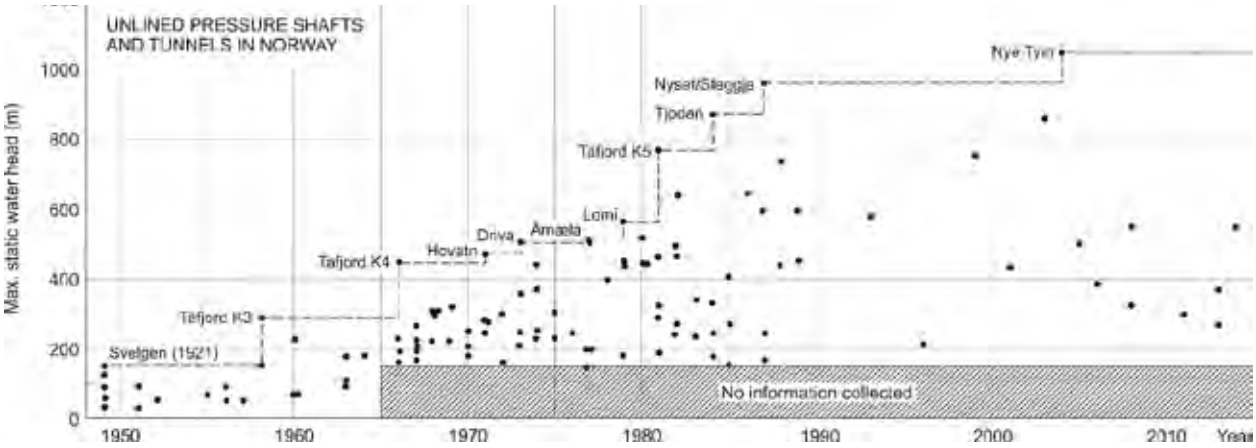


Figure 8: The development of unlined pressure shafts and tunnels in Norway.

To demonstrate the design approach an example of an underground hydropower plant will be shown and briefly described. Figure 9 shows the simplified plan and cross section of a small hydropower plant with only one turbine. No dimensions are given, as the intention is to show a system rather than give details. Similar layouts can be found for Norwegian plants with water heads in the range of 200 - 600 m.

The figure is to some extent self-explanatory. A critical point for the location of the powerhouse will normally be where the unlined pressure shaft ends and the steel lining starts. The elevation of this point and the length of the steel-lined section will vary with the water head, the size and orientation of the powerhouse, and the geological conditions, in particular the character and orientation of joints and fissures. Steel lengths in the range of 30- 80 m or approx. 15% of the water pressure are fairly common (varying with geological conditions and layout).

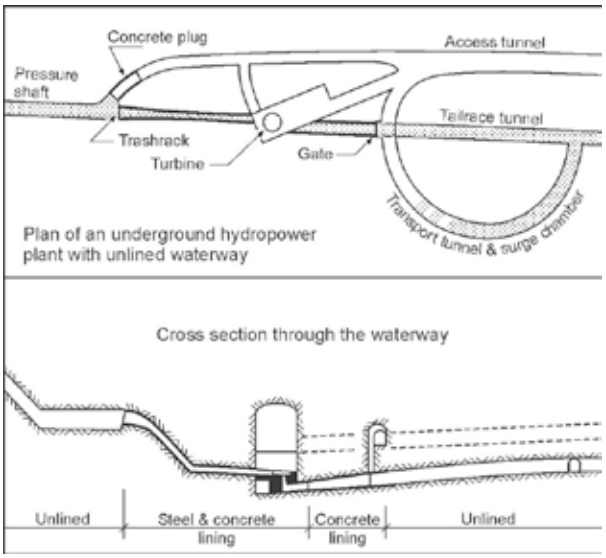


Figure 9: Plan and cross section of an underground hydropower plant with unlined waterways.

The access tunnel to the foot of the unlined pressure shaft is finally plugged with concrete and a steel tube with a hatch cover. The length of this plug is normally 10 - 40 m, depending on the water head and geological conditions. As a rough rule of thumb the length of the concrete plug is made 4% of the water head on the plug, which theoretically gives a maximum hydraulic gradient of 25. Around the concrete plug and the upper part of the steel-lined shaft a thorough high-pressure grouting is carried out. This avoids leakage into the powerhouse and the access tunnel. Further details about the design of high-pressure concrete plugs can be found in Dahlø et al.(1992) or Broch (1999).

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13. EXCAVATION OF LYSEBOTN II POWERSTATION

FARSUND, Tor Ø.
MATHIESEN, Thomas K.

1 INTRODUCTION

The new Lysebotn II power plant includes 11 kilometres of tunnels, two intakes, one outlet and a powerhouse complex (Figure 1). Lyse Produksjon AS is the owner and the tunnelling contractor is Implenia Norge AS. The project is scheduled to be completed in the spring 2018.

Today the existing Lysebotn I power plant leads water from Lyngsvatn and Strandevatn down to Storetjern (619-635 w.l), from where power is produced. The new tunnel system of Lysebotn II will allow water to be tapped directly from Lyngsvatn (637-687 w.l). The increased water head and new technology will result in an increased annual output of 180 GWh (15 %).

The bedrock within the project area is Precambrian Gneiss. The main cavern of the power station is approximately 20 metres wide, 35-40 metres high and 50 metres long with an overburden of approximately 650 metres. This paper presents the final positioning and the excavation of the Lysebotn II main cavern and transformer cavern.

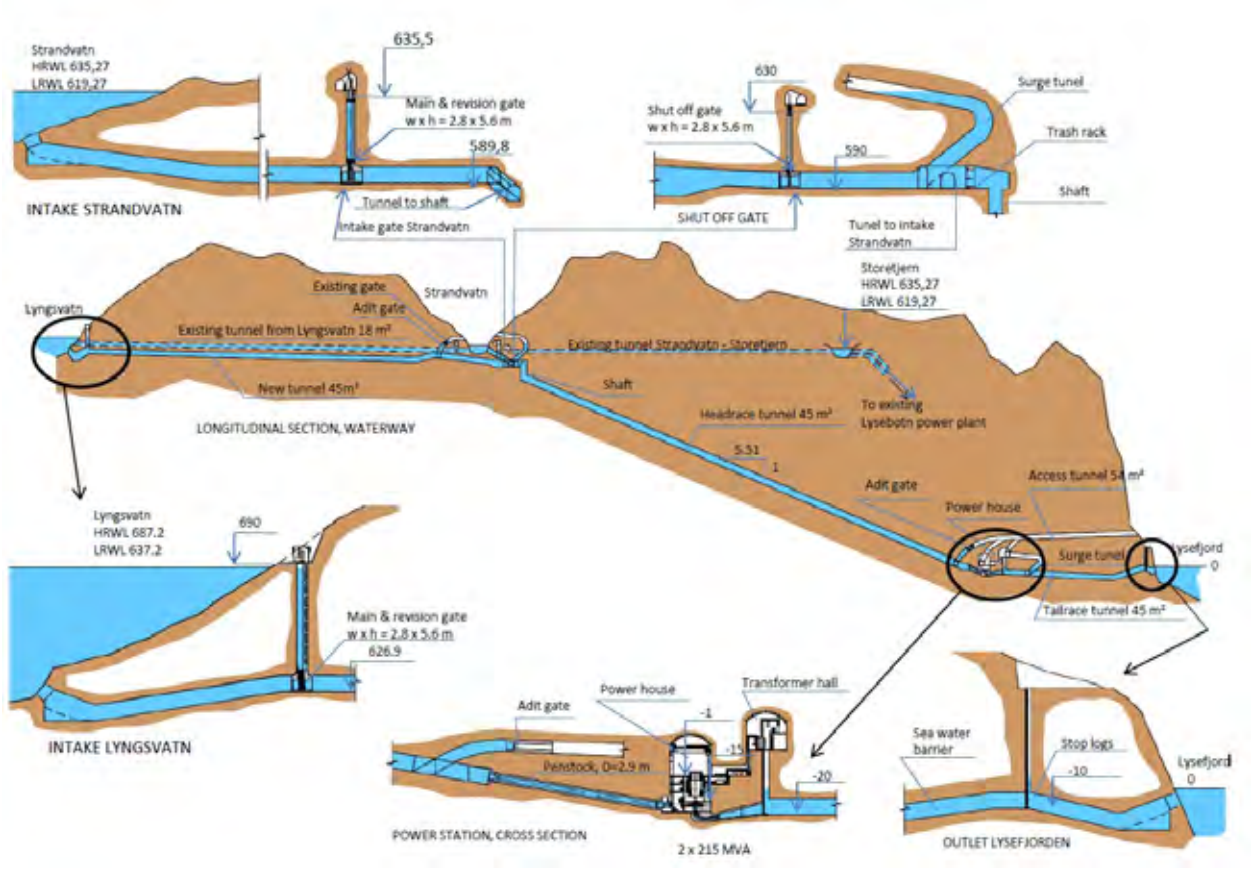


Figure 1: Schematic overview of Lysebotn II (Lyse Energi AS).

2 PRELIMINARY INVESTIGATION

The final positioning of the power station complex was verified with probe drilling and rock stress measurements. Hydraulic fracturing was performed by SINTEF the 2nd -3rd of December 2014 approximately 100 meters east of the two caverns. The results indicated that the minor rock stress o3 is higher than the hydraulic pressure from Lyngsvatn, with a suitable safety factor. Just before Christmas 2014 the access tunnels reached the two caverns. Probe drilling were performed by Implenia the 16th-18th of December 2014. The probe drilling verified rock mass of good quality in the station area. The foliation of the Granodioritic Gneiss is dipping 20-30 degrees towards the Southeast (strike almost parallel to the length axis of caverns).

3 EXCAVATION OF THE CAVERNS

The first blast into the main cavern was performed the 18th of December 2014. In total 38 000 sm³ and 17 000 sm³ rock mass was excavated in the main cavern and the transformer cavern respectively.

The permanent rock support of the caverns comprise fibre reinforced shotcrete and end anchored rock bolts in a dense pattern. The rock support system is designed to allow some degree of yielding without overstressing the support, considering the expected and observed high rock stresses in competent rock mass. Spalling and rock burst due to high tangential stresses was expected and did occur to some minor extent. Experience has shown that under such conditions the intensity of spalling may be increased if ridged rock support, such as fully grouted rock bolts, is introduced, and even re-activation far behind the face has been known to happen if stiff permanent rock support is added at a later stage. All the rock support bolts in the caverns and the surrounding areas are end anchored with resin. Figure 3-5 shows how the caverns were excavated in stages 1-3.

4 HIGH TANGENTIAL ROCK STRESS

During the excavation of the cavern roof (Stage 1) noises from the rock mass was commonly heard, as the rock stresses where re-distributed around the cavern geometry resulting in tangential stresses exceeding the strength of the rock. Slabbing did occur and regular temporary rock support was rock scaling, installation of end anchored bolts and shotcrete.

When the two cable tunnels were excavated in the pillar between the main cavern and transformer cavern

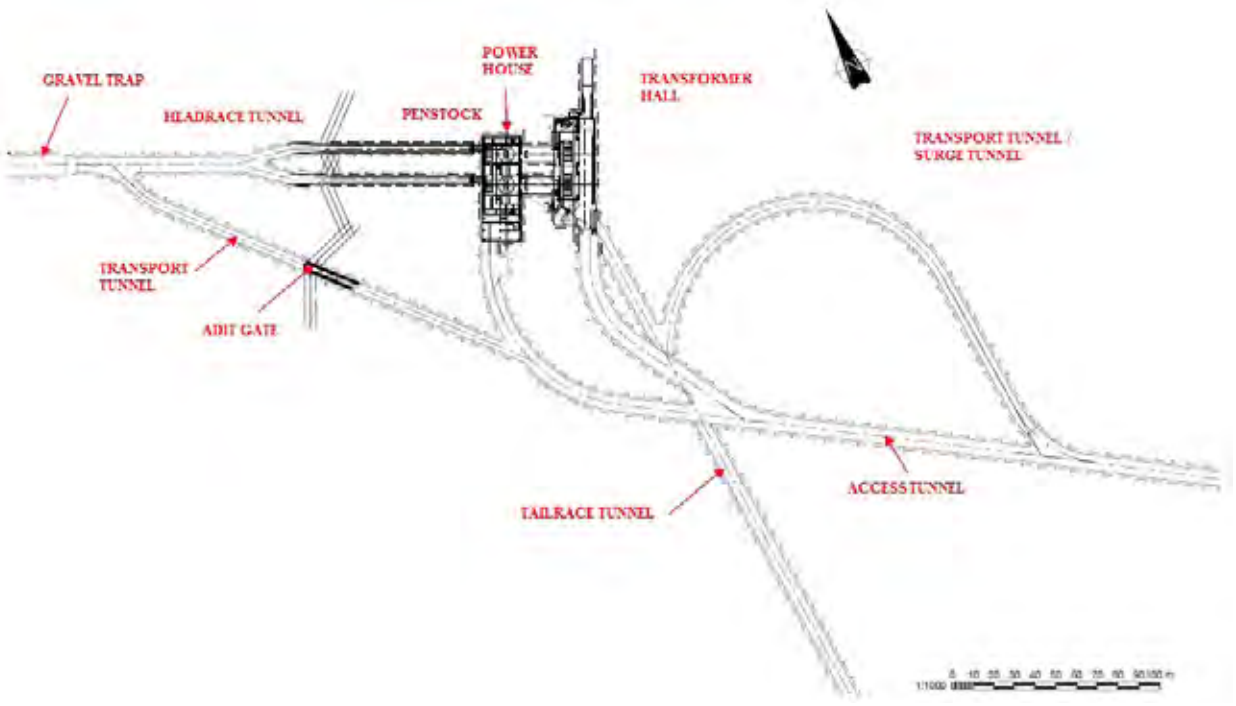


Figure 2: The Lysebotn II powerhouse complex (Lyse Energi AS).

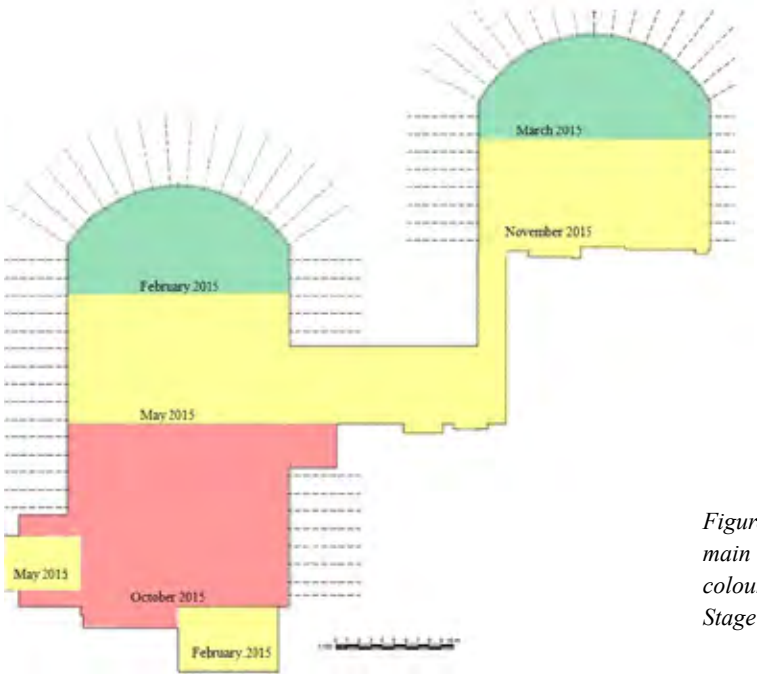


Figure 3: Excavation and permanent rock support of main cavern (left) and transformer cavern (right). The colours indicates stages of excavation; Stage 1 (green), Stage 2 (yellow) and Stage 3 (red).

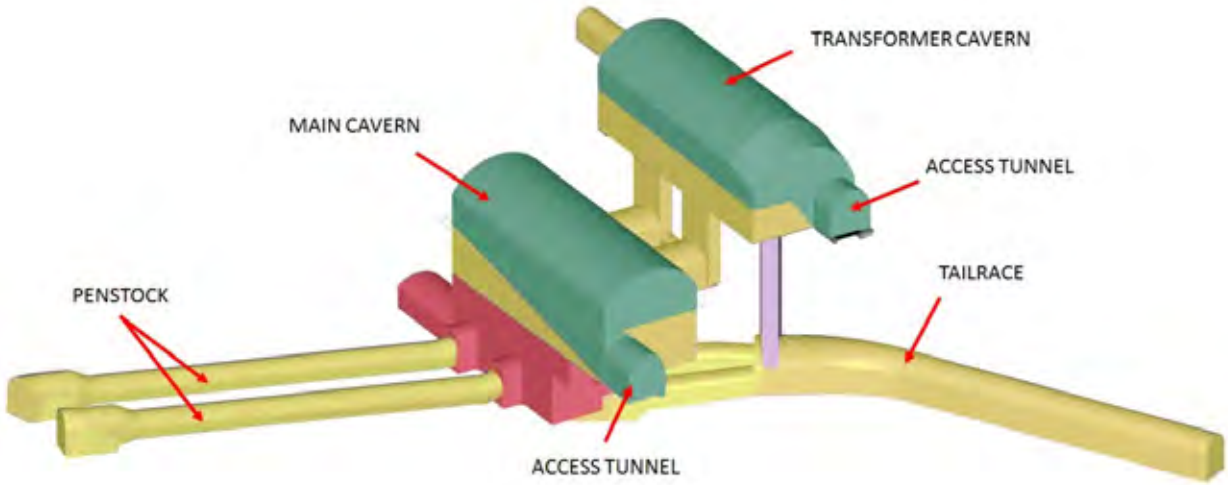


Figure 4: 3D-model of station area (Lyse Energi AS). The colours indicates stages of excavation; Stage 1 (green), Stage 2 (yellow) and Stage 3 (red).

		E6mØ20mm [pc]	E5mØ20mm [pc]	E4mØ20mm [pc]	E3mØ20mm [pc]	Shotcrete [m^3]
MAIN CAVERN	Stage 1	528	88			331
	Stage 2		458	322		284
	Stage 3		637		95	235
TRAFO	Stage 1	578	175			333
	Stage 2		534		83	182

Table 1: Permanent rock support of the main- and transformer cavern.

A



B



C



D



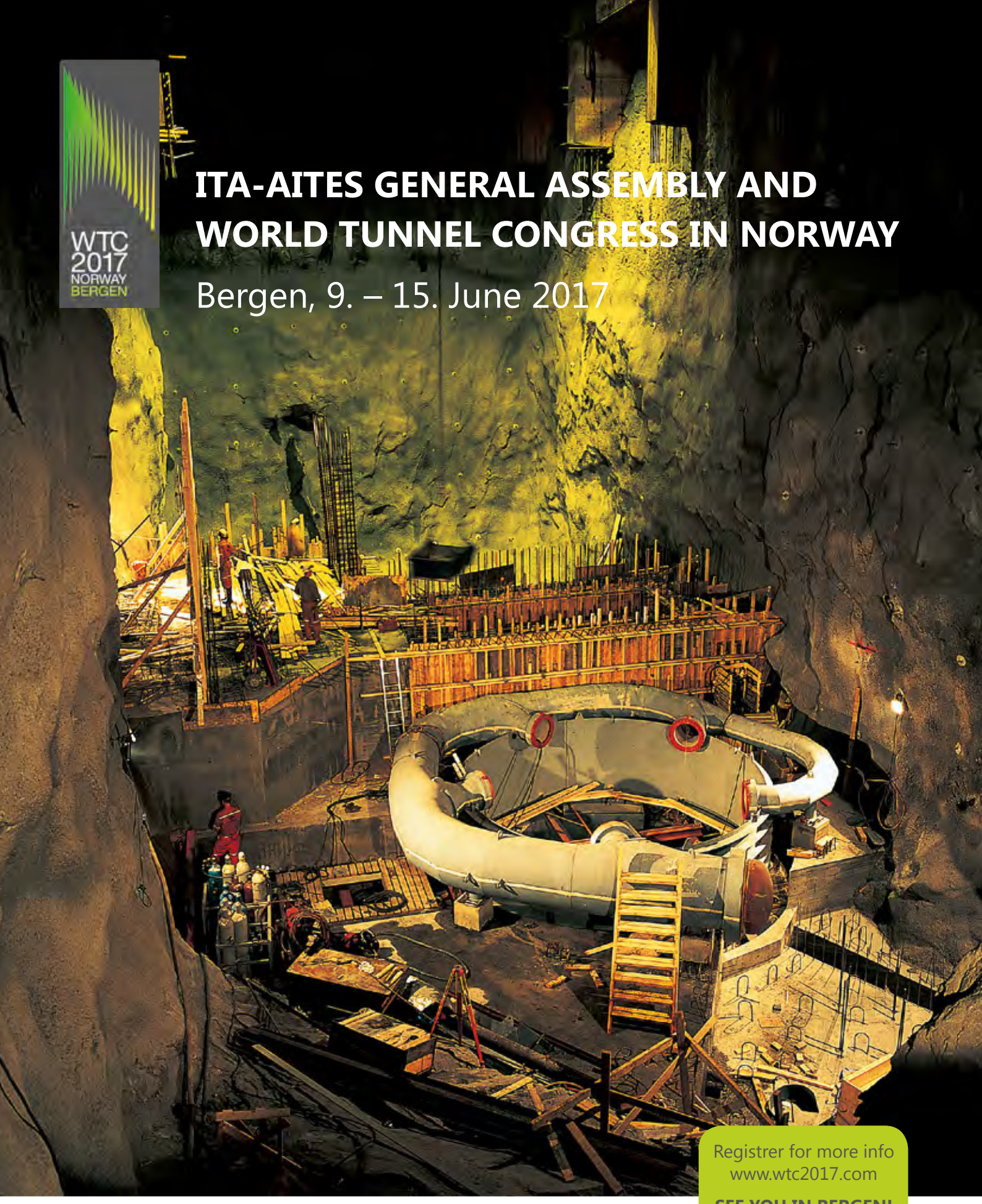
Figure 5. A: Ongoing drilling at the roof of main cavern (Stage 1). B: Excavating the cabel tunnels. C: The crane is installed and the drilling has started again. D: Excavation of cooling water tunnel at the invert of the power station (Stage 3).

(Stage 2) a highly anisotropic rock stress situation was expected. When the tunnels were excavated light slabbing did occur. The roof/wall were permanently supported by shotcrete and end anchored bolts.

As the excavation of the main cavern was reaching the final invert (Stage 3) the rock stresses concentration became more apparent. Immediately after blasting near the South-West lower corner, some "popping" in the rock mass occurred. The "popping" continued for approximately 30 minutes. Steel fibre reinforced shotcrete and end anchored bolts with high yield capacity was an important support to prevent or reduce the effect of the spalling.

The contractor completed the excavation of both caverns exactly on time with only minor problems due to the rock stress situation. Important factors for the successful excavation of the Lysebotn II caverns are considered to be:

- The powerhouse complex is located in rock mass of very good quality.
- The cavern geometry and orientation is favourable with regards to the in situ rock stress.
- The flexible rock support is installed early to prevent spalling.
- The drill and blast technique is specially designed to reduce blast vibrations and minimize the blast damaged zone.



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Lysebotn II power station
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14. NORWEGIAN DESIGN ADOPTED TO CHINESE HYDROPOWER CAVERNS

HUANG, Ziping

ABSTRACT

After the historical and successful practice in Lubuge and Ertan Hydropower Projects in China in early 1980s and 1990s, the Norwegian design has been further adopted to Chinese hydropower caverns. This paper presents the applications that integrated the Norwegian Tunneling Method with numerical modeling to assist the construction of two large hydropower caverns.

1 INTRODUCTION

From 2007 to 2010 Norconsult AS had been involved in the construction of large powerhouse caverns for the Chinese hydropower projects: Nuozhadu and Guandi, with install capacity of 5580MW and 2400MW, respectively. Professor Einar Broch, Professor Bjorn Nilsen joining the Norconsult team of Dr Arild Palmstrom and Dr Ziping Huang have been visiting the dam sites and providing practical advices during the excavation of the caverns. Dr Ziping Huang carried out the FE numerical modeling.

The Norwegian rock engineering design employs the observation method including instrumentation of the excavation, the "support as you go" philosophy, in order to make use of the rock to support itself. This methodology is supplemented with numerical modeling of the excavation to predict the rock of the rock and support elements by integration with the monitoring data in a dynamic process. It has been successfully applied in the construction of the underground caverns for Guandi and Nuozhadu hydropower projects. The modeling looks inside the failure mechanism of support elements, verify the engineering judgment, which is essential important to make rational remedial support design to the instability problems. Design advices were provided and partly implemented especially with regard to the prestressed cable anchors, the concrete lining of side walls in the surge chambers.

2 GUANDI HYDROPOWER CAVERN

The Guandi Hydropower Project is located on the downstream part of Yalong River, in Sichuan province, Southwest China. It is about 58km from the Jinping II hydropower plant (completed in November 2014), 80km driving distance to Xichang city. The major components of the project are the 168m high RRC dam, the reservoir with storage of 760 million m^3 and the underground powerplant with install capacity of 2400MW (4 x 600MW). The underground powerhouse complex consists of machine hall (L x W x H: 243.4m x 31.1m x 76.8m), transformer chamber (193.7m x 18.8m x 25.2m) and surge chamber (205.0m x 21.5m x 72.5m), as illustrated in 3D in Figure 1.

The construction of the project commenced in September, 2007. The first unit commissioned in March, 2012 and the project completed in March, 2013. The excavation of the underground powerhouse complex began in November 2007 and ended in September 2010. The underground powerhouse complex is located in a solid basalt rock formation with the longitudinal axes of the three caverns parallel in a direction of N5°E. The layout of the powerhouse complex was designed to be favorable to the stability of the surrounding rock masses with respect to the in-situ rock stress and the orientation of the geological structures. A complementary in-situ rock stress test had caused the adjustment of the caverns orientation at the feasibility design stage.

During the excavation unfavorable shear zones were revealed, instability problems were encountered such as rock blocks fall, slabbing, long cracks in the shotcrete in the surge chamber roof, large deformation in the rock masses and high stress in bolts in the vicinity of the shear zones intersecting the caverns. Additional rock supports were applied in the surge chamber. Monitoring data were analyzed in addition to site observation. The

numerical modeling were carried out to study the shotcrete cracking failure mechanism and the effect of rock support to the roof arch, the effect of concrete lining of the surge chamber wall.

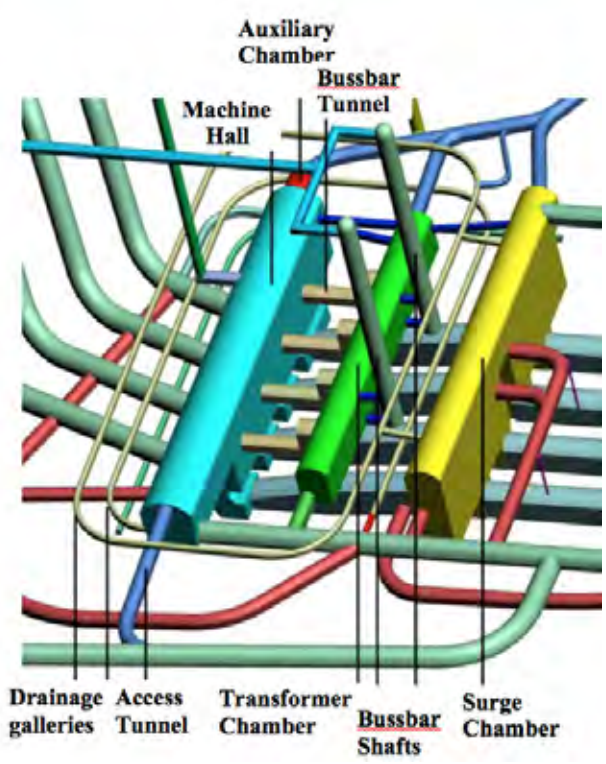


Figure 1: Layout of the Guandi underground powerhouse complex

The powerhouse complex is located in the right bank, downstream of the dam and in the rock layer P2β15 of fresh hard Permian basalt, seen in Figure 2. The lava layer has a thickness of about 288~392m and orientation of SN/W∠75º~85º, with geological structures dipping moderate to steep and striking in NWW, NNE and NEE directions. The basalt rock is competent with UCS values in a range of 133~291MPa and the elastic modulus 73~95GPa. The major principal stress, σ1, is measured in a range of 16~35MPa with directions in N14°~55°W. The north end wall of the machine hall has a horizontal and vertical distance to the ground surface of about 205m and 186m, respectively. The maximum overburden is about 426m.

In the powerhouse area, no regional fault or major weakness zone has been found. The basalt rock mass was classified as Class II (Q = 8.3~16.7, while Class V, Q < 1.5). The rock formation is supposed featured by many shear zones 0.3m to 1.0m wide with elongation of 50~100m, caused by tectonic movements and well developed joint system. The excavation revealed in general that joints were less developed than the assumed, as well as the shear zones. However, a 180m long outcrop of a shear zone fxt01 of 10~50cm wide was found flat, intersecting almost the whole roof and downstream top side wall of the surge chamber, as shown in the numerical model in Figure 3. It consists of crushed rock, mylonite, breccias, quartz loose fragments and clay. The shear zone fxt05 of 10~20cm wide with outcrop in the south end of the surge chamber roof likely extending towards north above the roof, has similar composition material and orientation with fxt01. Numerical models were set up as shown in Figure 3. The results of the stress distribution as indicated in Figure 4 verified the observation that the geometry and location of the shear zone played a critical important role in combination with steeply oriented small scale shear zones, fractures and joints. It degraded the quality of the rock mass in a sensitive location such as the roof arch, and caused the major instability problems including slabbing of the roof rock (Huang and Wu, 2013). And thus unusual high compressive stress built up in the shotcrete in the roof. Where the stress exceeds the strength of the shotcrete, shear failure took place which resulted in continuous long cracks in the shotcrete along the roof . The type of the shotcrete crack verified the shear failure mechanism.

The rock support design of the caverns was made following case study, experience and the numerical modeling analysis at the design stage. Enhanced rock supports in the surge chamber roof and springline area were applied during the excavation when ft01 was revealed. After the shotcrete cracking was observed along the surge chamber roof more than 200 pre-stressed cable anchors of 1000~1750kN capacity, 15~20m long were installed where the cracking occurred. Huang and Wu (2013) commented the enhanced rock support with non uniform and asymmetric reinforcement along the roof arch perimeter consequently at the vault area not enough support pressure may also lead to the shotcrete cracking taking place. With regard to the additional pre-stressed cable anchors in the roof the asymmetric distribution of the anchors probably not function effectively as expected. According to the numerical modeling results the effect of cable anchors is not significant. Efforts shall be made to enhance the reinforcement of the roof with uniform and symmetric rock support along the roof arch perimeter to ensure the roof arching effect (Huang et al, 2002).

In the initial design, concrete lining of the surge chamber walls is considered. Both Norwegian experience and the numerical modeling analysis suggested to omit

the concrete lining to make use of the rock mass to support itself. It is noted that the design recommendation was adopted by the designer and implemented.
Systematic instruments of extensometers, bolt/cable dynamometers were installed in the powerhouse caverns. The displacement and stress monitored in all caverns showed small magnitude increment of displacement and stress with benching and trend stable except some points exceeding the measurement limit, or failure where local instability took place such as rock block fall, rock slabbing and cracking in shotcrete. The maximum displacement is measured about 47mm in the upstream wall of the machine hall. While the roof settlement of about 41mm in the surge chamber indicated the shotcrete cracking. Large stresses in the bolts were mostly at 2m deep in the rock. One cable dynamometer in the surge chamber roof showed load slight decrease after locked.

The data provided useful information for the numerical analysis to back analysis the input rock parameters and to predict the rock mass behavior at next step. Further monitoring data were compared with the modeling results to tune the modeling and thus the prediction.

The modeled rock mass deformation and stress in support elements are in general accordance with the monitoring data and the observations, indicating that the caverns were overall stable. The simulated concrete lining on the wall and the pre-stressed cable anchors in the roof of the surge chamber had little effect to the deformation of the rock masses. The high compressive stresses in the rock mass and the shotcrete in the surge chamber roof revealed the key geological structure that mainly contributed the shotcrete compressive cracking: the flat shear zone extends long and close over the roof.

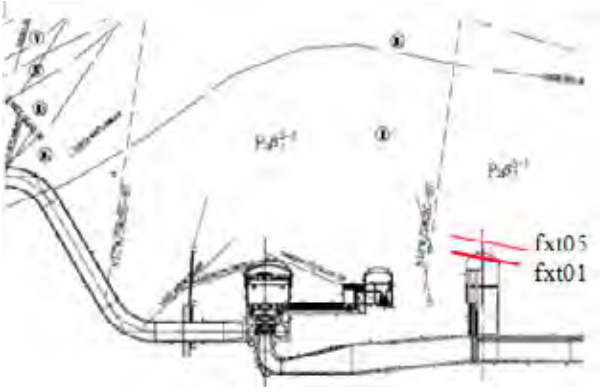


Figure 2: Geological profile of the powerhouse complex along the waterway

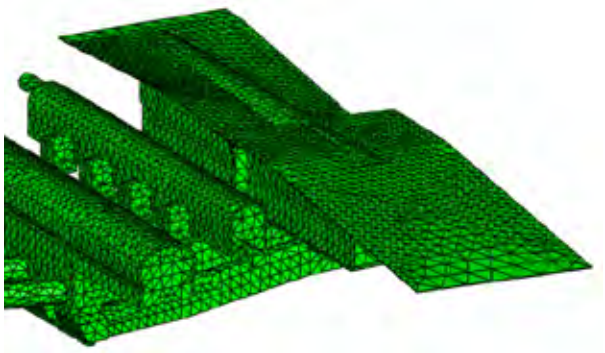


Figure 3: Model geometry and mesh of the powerhouse caverns with the shear zones fxt01 (lower) and fxt05 (upper) intersecting the surge chamber

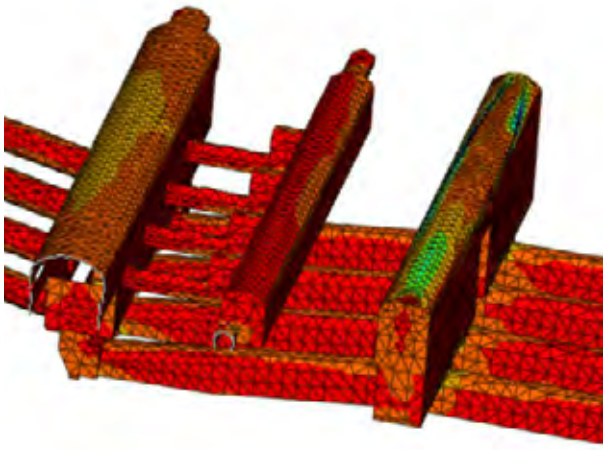


Figure 4: Major principal stress contour in shotcrete after excavation of Step 5(a). In the surge chamber roof, The yellow and green colors in the roof indicate high stress in a magnitude of about 70MPa, failure of shotcrete, while red color indicates stress of less than 20MPa, brown 20~30MPa.

3 NUOZHADU HYDROPOWER CAVERN

Introduction
When a Norconsult team visited the Nuozhadu Hydropower project site the first time in June 2007, the excavation of powerhouse caverns was at the beginning. The consulting service covered the entire excavation period and ended in July 2009. The project was completed in June 2014. The advisory service applied the same design methodology as describe above to combine sound engineering judgment and experiences, monitoring data analyses with numerical modelling to advise the excavation and support of the powerhouse complex in a dynamic way following the construction..

The project is located in Lancang River, Yunnan Province, China. The power plant utilizes 187m head

in 9 units with a total install capacity of 5850 MW. The underground powerhouse caverns are located in massive granite rock with the longitudinal axes orientation of N76°E, including the machine hall (418m×31m×81.6m; length×width×height), the transformer hall (346m×19m×23.6m), tailrace gate chamber and three tailrace surge chambers with diameter up to 38m and height of 92m, as illustrated in Figure 5.

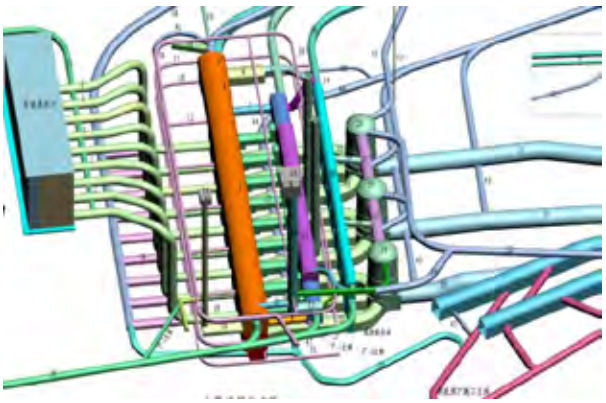


Figure 5: 3D illustration of the Nuozhadu underground powerhouse complex

The geological conditions, design and excavation

The geological conditions are in favor of the large caverns in terms of good rock quality, very lower permeability, no major discontinuities. The slightly weathered to fresh granite rock strata are thick and oriented in N20°~50°W/NE∠10°~25°. The granitic rocks are hard, massive only slightly jointed with intact rock average UCS of 143MPa. The cracks in the rock mass are assumed to be partly inherent in the rock, partly formed by the blasting. The RQD is measured as 90 – 100. In-situ stress measurements were carried out in Adits using both overcoring and hydraulic fracturing methods. The results indicate low to moderate stress condition. The lateral stress ratio of the machine hall cross section is about 0.84. The rock mass can be characterized as good to very good with Q = 26, RMR = 83, and estimated deformation modulus of about 35GPa. Figure 6 shows the excellent contour drilling and blasting in the top heading. Only one significant fault, F22 (N0°~22°W/SW∠42°~58°) was revealed about 3 m thick containing two clay filled seams, each of some 5-15cm, on the upstream wall of the machine hall, intersecting all other caverns. It is considered as a shear zone, with Q = 0.08, RMR = 38. The deformation modulus is estimated about 2GPa. The numerical model included the fault F22 to study the impact to the cavern stability. In general the revealed rock conditions during the excavation are better than estimated at the design stages.

The caverns were benched down started with the adit in the machine hall roof. The typically rock support is presented in Figure 7 for the machine hall. In the walls of the machine hall, transformer chamber and three tailrace surge chambers pre-stressed cable anchors were designed, typically 1000kN, 20m @5m x 5m. For the surge chamber, concrete lining was designed. Three layers drainage galleries were designed in addition to the drainage holes to reduce the groundwater pore pressure. A well designed monitoring system was implemented including extensometers, rockbolt, cable anchor dynamometers. Extensometers were installed from the drainage gallery before the heading excavation started in the machine hall.

After benching down to the turbine elevation in the machine hall (till 1st of May, 2009), the maximum roof displacement is about 15.6mm in the vicinity of the fault F22, the maximum wall displacement is about 14.3mm. Among total 72 extensometer measurement points at surface and 2m deep, 68% of the points, the displacement is less than 5mm and stable, 10% with displacement between 10mm to 20mm. Lower deformation was monitored in other caverns. Stress in rockbolt installed in the machine hall is general low (88% measurement points show stress less than 50MPa) except few bolts intersecting faults or shear zones, joints. Among 48 cable dynamometers, 35 cable anchors show deduction of the locked load. The maximum increase of the load is about 9% in the cable anchor intersecting the Fault F22.



Figure 6 Excellent contour drilling and blasting in the 22.5m wide top heading showing very little overbreak

The numerical modeling results

The modeling results as shown in Figures 8 to 10 indicate the distribution of the stress, deformation. The change of rock stress is small. Consequently the cavern deformation is small, as well as the stress increment in rockbolt and force increment in pre-stressed cable anchors. These are in accordance with the monitoring data. The surrounding rock mass is self stable. The effect of the prestressed cable anchors is negligible except those intersected the fault zone, so does the concrete liner of the surge chambers. Relative larger displacement takes place in the vicinity of the fault zone, the intersection places of the tunnels and the caverns.

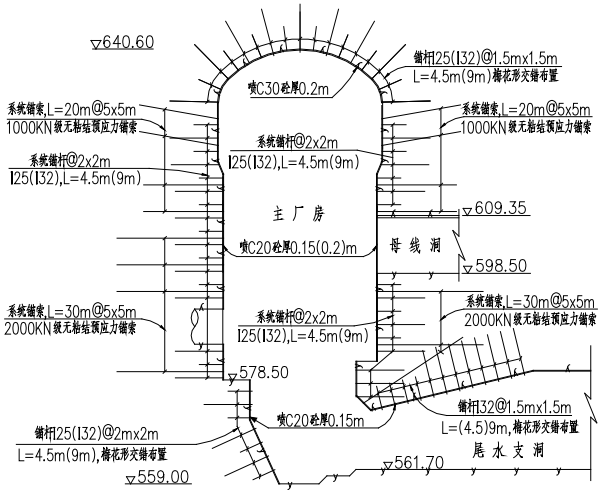


Figure 7: Rock support design of the machine hall.

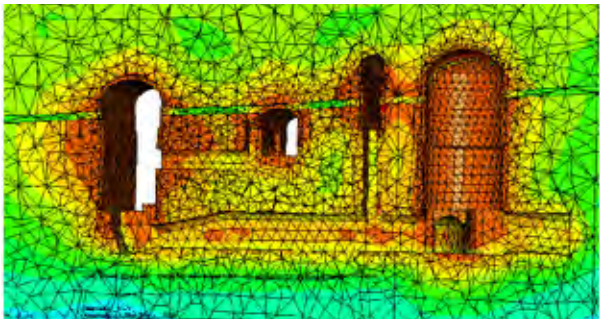


Figure 8 Minor principal stresses contour in the rock mass surrounding the rock caverns, red color indicates low compressive stress while green indicates high. Along and in the vicinity of the Fault F22, tensile stresses occurred. The destressed range is thinner in the surge chamber wall than that in the machine hall wall.

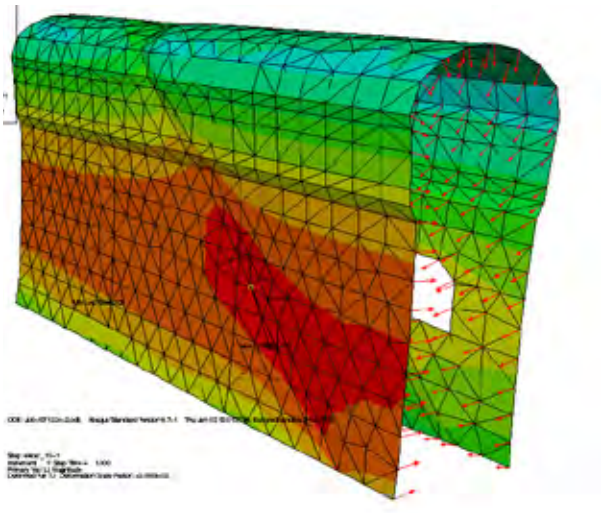


Figure 9: Displacement magnitude contour after step 10 in the upstream wall of the machine hall showing effect of Fault F22.

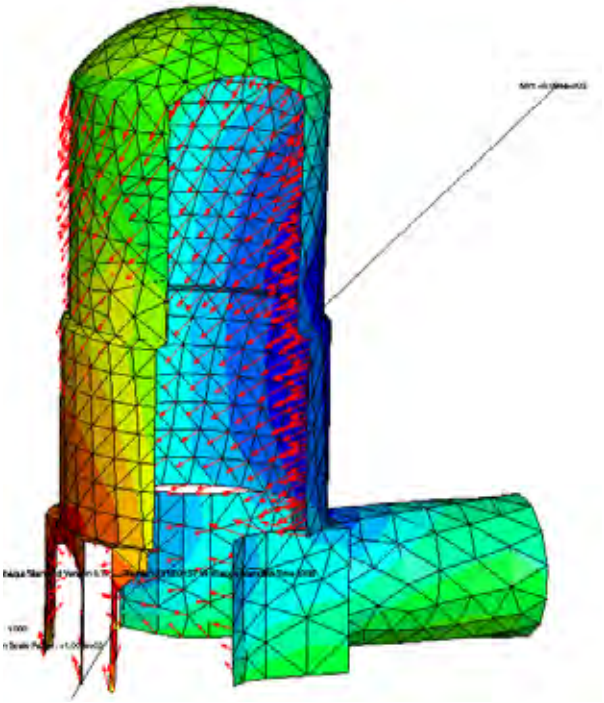


Figure 10: Horizontal displacement contour with maximum value of -2.63mm above the fault F22. Wall displacement is about half of that in the machine hall walls.

Consulting recommendations
Since the properties of the rock masses are better than what was used in the previous design. It was recommended the designer to reconsider the use of tensioned cable anchors in all caverns and the concrete lining for rock support in the surge chambers. In the intersection of the surge chamber with the tailrace tunnels, concrete lining is necessary as the rock mass in the corners to a large extent will be distressed.

It is believed that the systematic rock bolting with the use of up to 9m long rock bolts as shown on the drawings is a sufficient support in the major parts of the straight high walls when combined with reinforced shotcrete. Locally prestressed cable anchors shall be installed in the vicinity of the faults. The original design of the excavation and rock support is in general rational at a certain conservative estimate.

4 ACKNOWLEDGEMENT

It is acknowledged that Norconsult allow the author to use the relevant Norconsult reports in this paper. And the author would like to take this opportunity to thank Dr Arild Palmstrom, Professor Eina Broch and Professor Bjorn Nilsen for their constructive work and the team work during the consulting service to the Chinese Clients on the hydropower caverns construction.

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Portal at Holmestrand
Courtesy: Norwegian National Rail Administration



Storage caverns, Boliden Odda AS
Courtesy: Anne Hommefoss

15. NEELUM JHELUM HYDROPOWER PROJECT

HALVORSEN, Asbjørn
LU, Hui

Overcoming design challenges and achieving a successful outcome during the design and construction of an underground powerhouse cavern in variable rock units in northern pakistan.

ABSTRACT

The 969 MW Neelum Jhelum Hydropower Project, in Azad Kashmir, is located within a challenging geological setting. The excavation and support of the project's underground powerhouse cavern (137m long x 25m wide x 47m high) has been successfully completed. This article describes the design philosophy of the powerhouse complex and the approaches taken to deliver a functional solution. Particular focus is given to the siting of the powerhouse, the design of support system and the performance of the powerhouse complex to date.

1 INTRODUCTION

Construction commenced on the 969 MW Neelum Jhelum Hydropower Project in 2007. The project is located near to Muzaffarabad, the capital of Azad Kashmir. Neelum Jhelum Consultants, a Joint Venture of MWH International, Norplan (Multiconsult), NESPAK, ACE and NDC has been appointed to provide design and supervision services during construction.

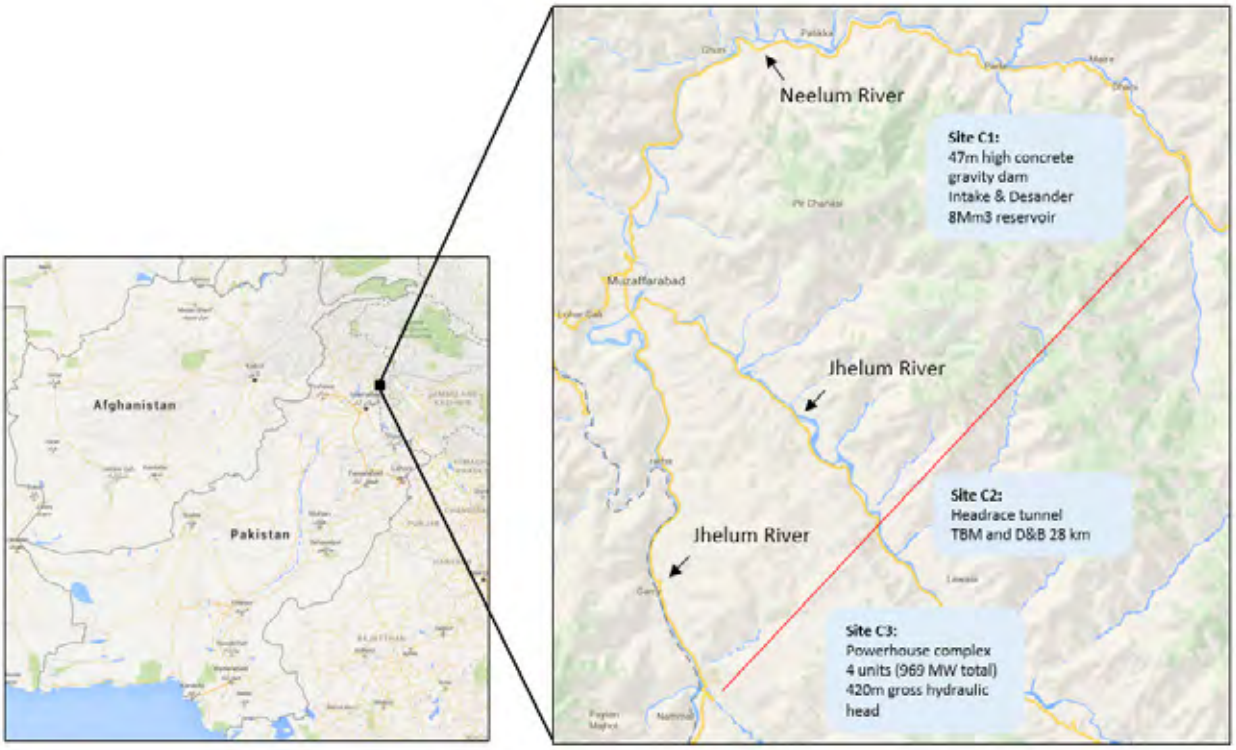


Figure 1: Neelum Jhelum Hydropower Project – Project Location (Images Google Maps)

1.1 Project Overview

The Neelum Jhelum Hydropower Project diverts water from the Neelum River near Nauseri. The 47m high mass concrete gravity dam and intake works are designed to divert up to 280m^3/s from the Neelum River. From the intake, a 28.5km long headrace tunnel conveys the water to the underground powerhouse. The first 15.1km of the headrace tunnel is comprised of twin tunnels, each 7m in diameter while the remainder of the alignment is comprised of a single tunnel 9.5m in diameter. The primary reason for the change in the tunnel profile relates to the substantial cover along sections of the alignment (Up to 1900m) and potential problems related to in-situ stresses stemming from this. The flow passes through the four 242MW Francis turbines prior to discharging back into the Jhelum River through a 3.5km tailrace tunnel. The scheme develops a gross head of 420m.

1.2 Geological Context

The geological setting for the project is within the Murree formation, which is comprised of molassic sedimentary rocks of Palaeocene age, partly weak. The Muzaffarabad Fault, located northeast of the Jhelum River and running almost parallel to the river, defines the boundary between Upper and Lower Murree formations. The 2005 Muzaffarabad earthquake (Moment magnitude 7.6), was caused by rupture along a 113 km long segment of the Muzaffarabad Fault, with the trace crossing the headrace tunnel alignment.

The powerhouse cavern, at a depth of approximately 400m, is located in the Upper Murree formation, dominated by arenaceous rocks. The terrain above the powerhouse is partly soil covered and partly exposed rock. Therefore, prediction of rock conditions encountered at the powerhouse on basis of geological surface mapping was concluded to be highly uncertain. Core drilling undertaken during the feasibility study showed alternating sandstone, siltstone and mudstone beds, with weak zones due to faulting. These rock types encountered during the excavation of the powerhouse cavern are described in Table 1.

2 POWERHOUSE COMPLEX SITING

The optimum positioning of the powerhouse and detailed design of the rock support is strongly dependent on actual ground conditions. At the time of tender design, limited information on ground conditions was available to the designers. This is typical in the design of deep underground structures where detailed information only becomes available during excavation. As is the case with many similar underground works where initially only limited information is available, this leads to a dilemma as to the optimum timing for finalizing critical aspects of the design. A decision too early may lead to a design based on insufficient information resulting in the need for excessive conservatism and / or later redesign. On the other hand delaying the decision may result in contractual claims and / or may cause significant delays to the delivery of the project.

2.1 Powerhouse complex siting philosophy – Tender design

The Tender location/ layout of the powerhouse complex was determined on basis of following concerns:

- The large caverns (powerhouse and transformer cavern) should be located in the best available rock masses and oriented as favourably as possible with respect to geological structure and assumed orientation of principle rock stresses. At tender design, these stresses were defined according to the World Stress Map [1], which shows the principal stress in the area of concern to be NE-SW with an average direction of N50°E. This was used during the initial optimization due to the lack of reliable rock stress

Rock type	Uniaxial Compressive strength (MPa)*
Grey, clean and competent sandstones SS-1	86
Brownish and less competent sandstones SS-2	47
Brown to reddish less competent Siltstone	66
Grayish brown inferior Mudstone	42
Dark brown to reddish incompetent Shale	24

*Table 1 - Rock descriptions and uniaxial compressive strength ranges. *Typical values based on laboratory testing of core samples*

information available. The orientation was set such that the dip direction of the sedimentary units was approximately parallel to the longitudinal axis of the powerhouse cavern. The dip of these units is approximately 45° - 60°. The orientation of these beds relative to the powerhouse are shown below.

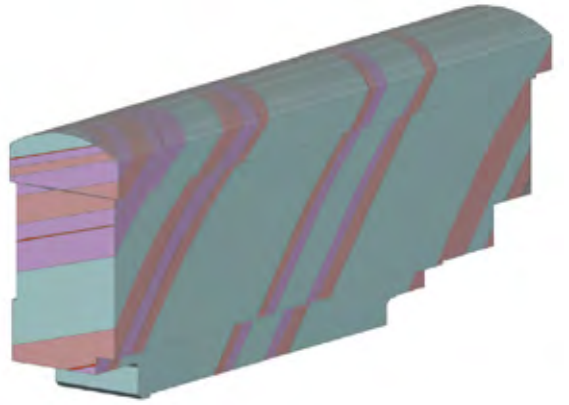


Figure 2: 3D Geologic model of the powerhouse cavern

- The penstock inlet should be located with sufficient rock cover and confining rock stresses to ensure a safe margin against hydraulic splitting in the adjacent pressurized headrace tunnel.

The tender design dimensions of the powerhouse cavern were 130.6m long x 23.7m wide x40m high accommodating four Francis turbines with a total nominal installed capacity of 969 MW.

2.2 Powerhouse complex siting philosophy

The tunnel system around the powerhouse cavern and the transformer hall is complex. Information on geological conditions obtained by logging of the tunnels approaching the planned cavern location gave a good basis for optimization of the cavern location.

Optimization of the powerhouse cavern utilized information obtained from a 270m long exploratory core hole drilled from the cable tunnel and through the powerhouse cavern (logging from the exploratory hole presented on Figure 3), as well as updated rockmass parameters and in-situ stress information obtained during the excavation of the adjacent access tunnels.

Analyses of this information showed that it would be favourable to shift the powerhouse location eastwards by approximately 90m (The revised location of the Powerhouse Cavern is shown below in yellow and superimposed on the tender design location). The basis for this decision is described in figure 3.

2.3 Consideration of geological conditions and rock quality

The primary purpose of relocating the position of the powerhouse cavern was to locate the structure within more competent rock units. Following adjustment of the location, the cavern was largely located within the competent SS-1 sandstone. This is shown clearly on Figure 3 and Figure 4.

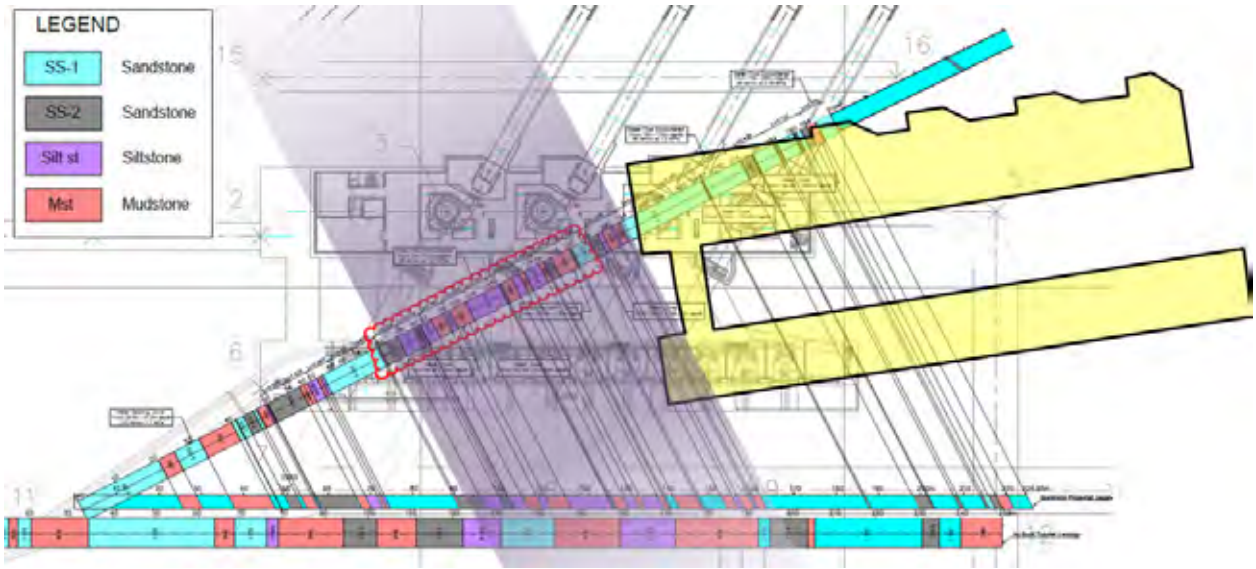


Figure 3: Tender vs Detailed Design Powerhouse Locations (Plan)

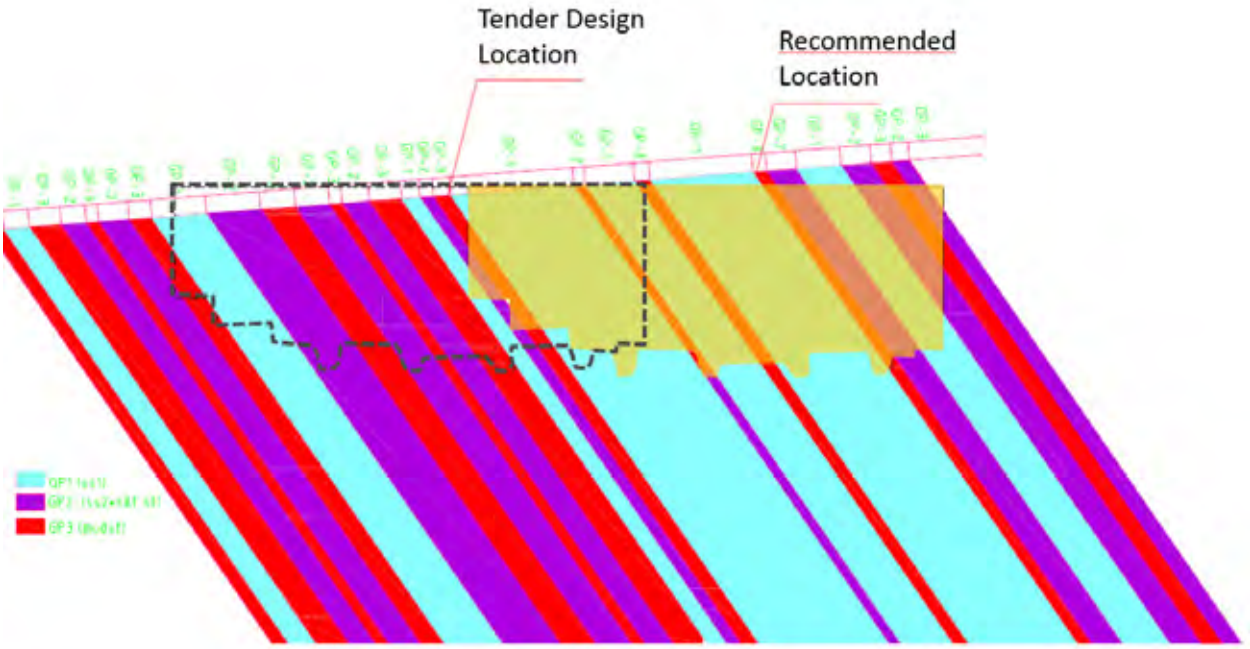


Figure 4: Tender vs Detailed Design Powerhouse Locations (Section)

Unquestionably, the most competent rock encountered is the SS-1 sandstone. Likewise, the least competent units are the weak mudstones and shale. SS-2 sandstone and siltstone are considered to be of intermediate quality. Due to the alternating sedimentary beds, and thus varying competence of the rocks encountered, it was not possible to place the full length of the cavern in competent rocks. However, Figure 4 shows that the revised powerhouse location makes best use of the SS-1 sandstone units encountered during the investigations.

2.4 Consideration of sufficient confining rock stresses
The final decision on the location of the powerhouse cavern also considered the in-situ rock stresses. Sufficient confining stresses were required at the transition between the unlined headrace tunnel and the steel penstock, in order to prevent hydraulic jacking of the pressurized tunnel. The location of this transition also influences the decision on optimal powerhouse cavern location.

To determine the general location, both overburden criteria as well as information from rock stress measurements were considered. In addition to hydrofracturing tests carried out in the manifold by a specialist contractor, simple but effective hydrojacking tests, consisting of high pressure lugeon tests employing grouting equipment, were performed in the headrace manifold to confirm that concrete lining rather than steel lining could be employed.

3 DESIGN AND MODELLING

3.1 Design and modelling philosophy
Numerical modelling was used to obtain an understanding of the possible behaviour of the powerhouse cavern excavation and support system.

3.2 Numerical Analysis
Phase^2 FEM analysis software by Rocscience was the primary analysis tool used during numerical modelling of the Neelum Jhelum powerhouse cavern.

Various alternatives were modelled before the support system design was finalised. These alternatives including studies of various support systems, excavation sequence, and the models' sensitivity to the rockmass strength parameters and in-situ stress ratio are not discussed in this article.

In addition to the empirical and numerical analyses, Plaxis 3D was used to model some of the more complex geometries in the powerhouse cavern.

During construction, observations and measurements of the deformations were used both to assess the effectiveness of the installed support and to calibrate the numerical model to better assess the remaining excavation.

3.3 Input Parameters and model assumptions
The generalized Hoek-Brown[2] and Hoek-Diederichs methods[3] were used to estimate the strength and stiff-

ness parameters of the rockmass. The rockmass strength and stiffness were based on various of laboratory tests, geological mapping and back-calculation from measurements taken in existing tunnels.

The average overburden was calculated as 430m. The in-situ ground stress ratio was input as 0.8 in-plane and 1.5 out-of-plan direction based on the actual ground stress measurements.

In total 134 UCS (Uniaxial Compressive Tests), 21 triaxial tests, 44 Hoek direct shear tests were done. The intact strength & stiffness and corresponding parameters (GSI (Geological Strength Index) & D (Blasting influence factor)) to calculate rock mass strength and stiffness [2][3] are presented in Table 2.

Several simplifications are required to transform the complex 3D nature of a powerhouse complex into a usable 2D Phase2 model. The model used during the analysis is shown in Figure 5.

3.4 Calibration

Calibration of the numerical model was undertaken based on deformation measurements (convergence and extensometer readings) taken during the early phases of the excavation. The model was calibrated to better represent the anticipated performance of the support system both during and after completion of the excavation and support works.

3.5 Results

The numerical model developed in Phase^2 was used to estimate the response of the ground and the installed

Rock type*	Intact rock properties			Parameters to obtain rock mass strength and stiffness	
	UCS (MPa)	mi	Ei (GPa)	GSI	D (reference)
SS-1	86	17	32	65	D=0.7 for the rock 2m deep
Siltstone	66	7	23,1	50	
Mudstone	42	9	12,6	50	

*Table 2: Assumed rock properties. *SS-2 and Shale were not required for the modelling after the relocation of the powerhouse*

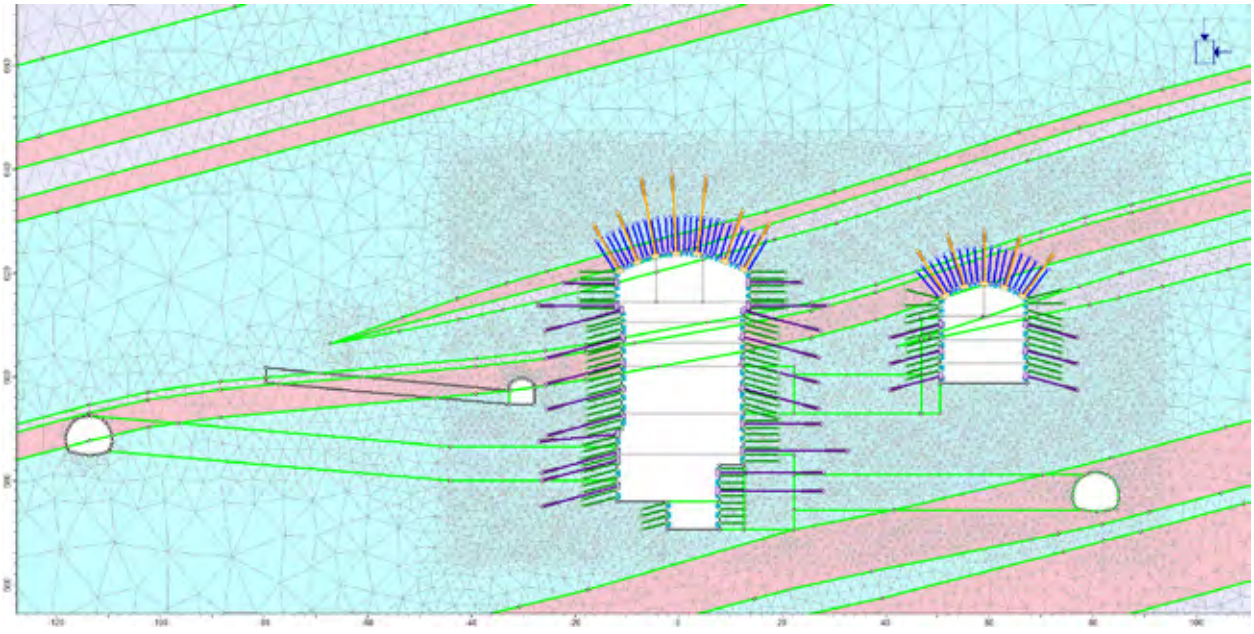


Figure 5: Idealised Phase2 model geometry and material stratification

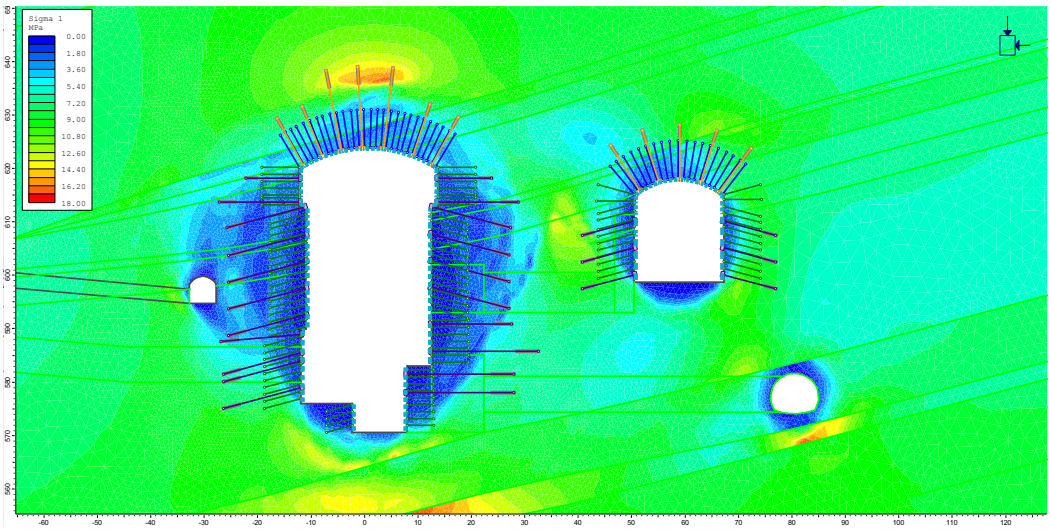


Figure 6: Powerhouse cavern stress distribution

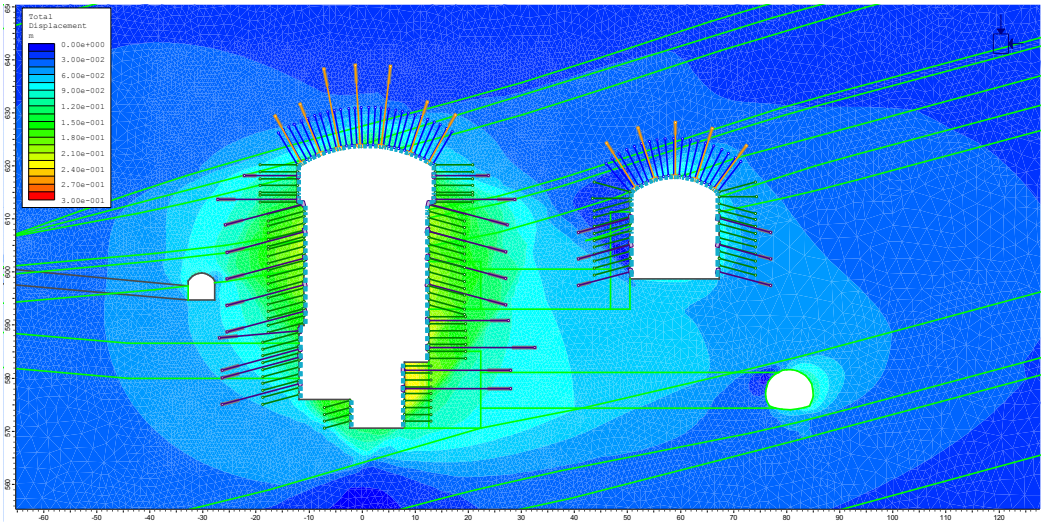


Figure 7: Powerhouse cavern displacement distribution

support during the various excavation stages of the powerhouse cavern excavation. The results showing the distribution of stresses and displacements for one of the scenarios modelled (In this case when the powerhouse has been excavated to its final level) are shown in Figure 6 and Figure 7.

A comparison between the modelled displacements and the actual displacements measured during the excavation of the powerhouse cavern is presented in Section 5.

4 DESIGN SOLUTION

The support design for the Neelum Jhelum Powerhouse Cavern determined by the Phase2 analysis undertaken consisted of shotcrete, fully grouted rock bolts and post-tensioned anchors.

- Rock bolts: 7m long, 25mm diameter, 1.0m×1.0m c-c spacing were specified to be installed in the powerhouse crown;
- Rock bolts 7m long, 25mm diameter rock bolts, 1.3m×1.3m c-c spacing, horizontal or 15° inclined were specified to be installed in the powerhouse walls.
- Post-tensioned strand anchors 15m or bar anchors 10m long, 5× 15.24mm diameter, 4.0m×4.0m spacing were specified to be installed on the crown;
- Post-tensioned strand anchors 20m, 15m or 10m long, 5× 15.24mm diameter, 5.0m×5.0m or 4.0m×4.0m spacing were specified to be installed on the walls.

- Shotcrete: Three layers (With a total thickness of between 200mm-300mm) including two layers of wire mesh with 5mm diameter and 10cm ×10cm apperture were specified to be installed on both the crown and walls.

5 PERFORMANCE TO DATE

The performance of the powerhouse cavern support system is monitored through 29 convergence arrays and 45 extensometers. In addition to these, vibrating wire piezometers will be installed prior to the project being 'watered up' to monitor any increase in pore pressure that is not dissipated by the drainage curtain.

This monitoring will continue during watering up and operation of the scheme to assess the response of the structure to the varying conditions.

To date, the performance of the support system has been in line with expectations. Figure 8 shows the predicted convergences compared to the measured convergences on the downstream wall of the powerhouse cavern.

It is acknowledged that numerical models are simplifications from the in-situ conditions, utilizing input parameters estimated from a relatively small sample from the population. Therefore, it is not expected that the modelling results will exactly match those measured in the field, but rather to provide a good basis for the how the structure will react.



Figure 8: Excavated powerhouse cavern

The relocation of the powerhouse cavern is believed to have significantly reduced the challenges encountered during construction resulting in a quicker excavation and support period and reduced support costs.

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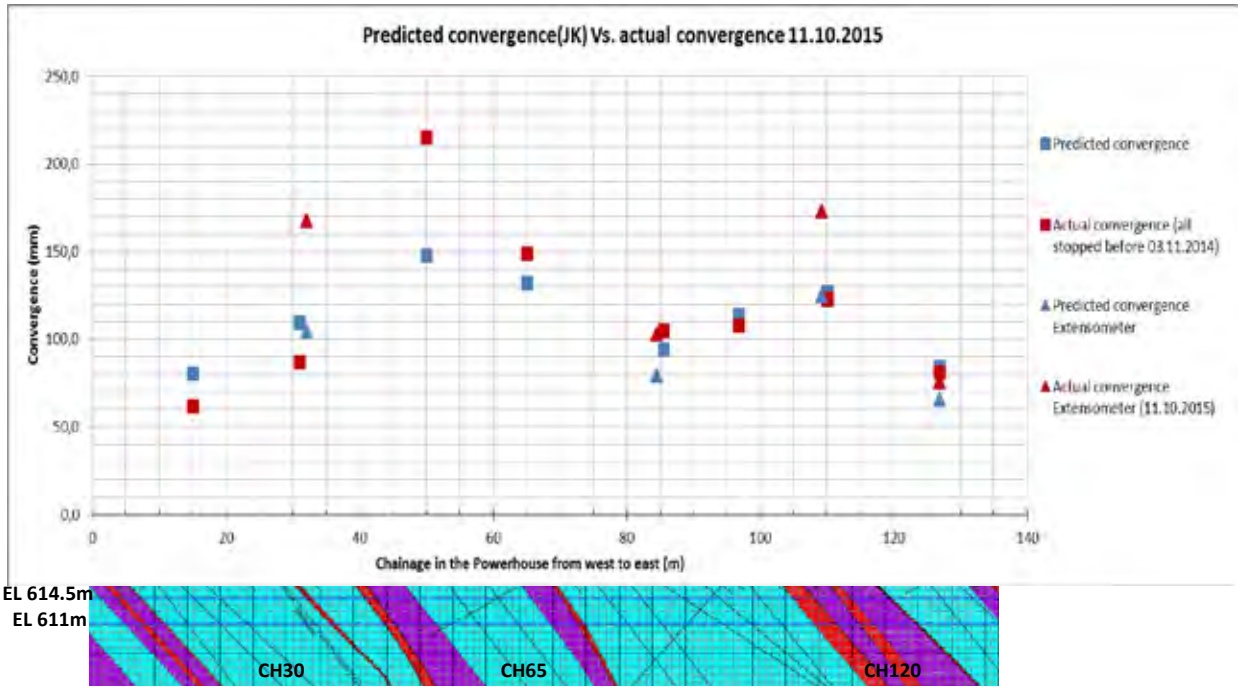


Figure 9: Neelum Jhelum Powerhouse Cavern, Predicted vs Actual Convergence

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