Understanding and utilizing in-situ rock stresses in design and building large rock caverns

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Abstract

The tunnelling industry has the means to make any "hole in the ground" almost to any size and shape that is requested. The main challenge is to integrate such solutions in the long-term development and urban planning. The underground solutions must be cost-effective to compete with surface alternatives and they must be safe and felt to be safe by the users.Only by ensuring that we are capable of providing safe underground structures will underground infrastructure be perceived to provide a sound internal environment. There is public confidence that the tunnelling industry is capable of producing tunnels and caverns to the satisfaction of the clients.

This paper discusses the challenges of designing and building large underground caverns needed for a further developing of the underground infrastructure. Underground caverns excavated into the rock mass require a strict control on a number of design parameters describing the rock mass quality and its ability to host large caverns. Many of these are typical rock mechanical parameters that are identified for any underground excavation. However, one particular rock mechanics parameters is of utmost importance when designing large underground caverns but to the surprise of the authors of this paper it is very often ignored and neglected by many, namely the in-situ stress conditions. This paper will particularly shed light the importance of including in-situ stress conditions in the design tasks. Therefore a particular focus of the article is directed to the understanding and utilizing of in-situ rock stresses to materialize such caverns.The paper will also present some examples to visualise the beauty of underground rock caverns and their use to serve the public and thus constituting a major asset to the society including the world largest man made rock caverns, the Gjøvik hall in Norway.

Keywords: In-situ Stress,Underground Space, Large Caverns, Design, Construction

1. Introduction

The application of large underground caverns has seen a wide range of use in Norway during the last 30 to 40 years. It began with a demand arising from the development of Hydroelectric Power schemes in the post-war period. The first major caverns in Norway were built for such purpose. Later, the technology evolved and it became evident for other businesses that this technology could be utilized for other applications too. Particularly for the civil defense it became obvious that such solutions could realize combined shelters and sports facilities. During a few decades of heavily development of this field a large number of combined underground shelters and swimming pools and sports halls were built. It culminated with the construction of the most famous one, likely to be the Gjøvik mountain hall, which was large enough too room the ice-hockey rank for the Winter Olympics in Norway in 1994. The oil and gas industry realized the possibilities of storing hydrocarbons underground and began to take this concept into use. However, the use of large underground caverns has also been materialized for use as storage of waste water, sewage cleaning facilities, cold store, storage of potable water etc., as a result of the knowledge of in-situ rock stress conditions.

A major parameter in the realization of these large span caverns is the presence of sufficient in-situ rock stress conditions, and the ability of taking advantage of these stresses. The in-situ rock stress conditions enable a confined arch to be formed above the cavern roof and thus stabilize the caverns and assisting in the realization of large span caverns to be built for a number of purposes in urban areas as well as rural areas. With no doubt the in-situ rock stress has indeed a great importance in design and construction of sub-surface opening.

This article presents some of the wide applications that large underground cavern has received in Norway as well as describing the fundamental philosophies of the design principles and the construction of these caverns.It is interesting to quote Tunnel Engineering Handbook by Bickel

&uesel [1992]: "The in-situ stresses present in a rock mass have an important effect on the physical properties and behaviour of rock as en engineering material. The in-situ stress is a confining pressure which affects the viscoelastic and strength properties of the rock. The intensity and direction of these stresses control the strain patterns and redistribution of stress in the rock mass remaining after an excavation has been made. Knowledge of the in-situ stresses is essential to the sound, logical design of rock reinforcement systems, excavation procedures and opening layouts, and to the interpretation of expected rock strength and deformational properties."

Fig. 1 TheBahtu Cave in Kuala Lumpur in Malaysia

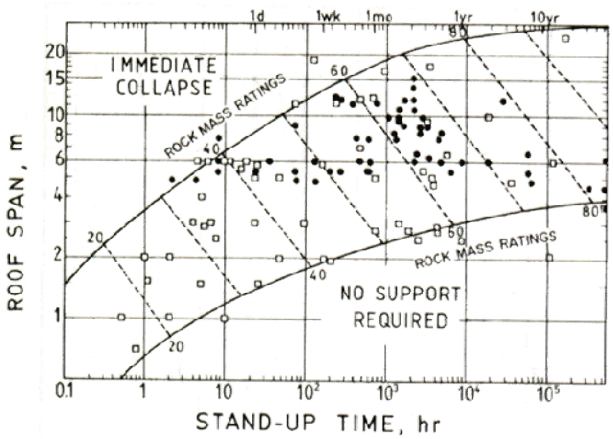
Taking these aspects into account it has been possible to utilize the rock mass for sustainable and environmental friendly solutions underground for the benefit of the larger public.Not least, Mother Nature is also a major 'contractor' with respect to building large underground caverns, and to keep them stable in an unsupported state of mode. A major tourist attraction in Kuala Lumpur in Malaysia is the Bahtu Cave. Thousands of people visit the cave every day, still it seems unsupported to the author with naked eye.

2. Some general characteristics of the rock mass

For the development of large underground caverns in Norway the following capacities of the rock mass has been the most important. Secondly, for unlined gas storages, cold storage, and storages with functional requirements not limited to stability and cavern size the following capacities are important:

- stress induced confinement,
- self-standing capacity.
- thermal capacity,
- impermeable nature.

Any rock mass has a certain self-supporting capacity, although this capacity may vary within a wide range [BIENAWSKI, 1984]. The fact that there is some "stand-up" time implies that the rock mass for a certain time is not a dead load, thus it shall not be treated as if it was (Fig. 2). An appropriate engineering approach is to take this capacity into account when designing permanent support. Rock strengthening may, however, be needed to secure certain properties/specified capacities, in the same way as is the case for any other construction material. The fact that, the rock mass is not a homogenous material shall not disqualify the utilization of its self-supporting and load bearing capacity.



As large span excavations became common, particularly in connection with near surface underground sport halls, the need for more accurate stability control measures became more apparent. Also, numerical analysis is an essential tool in the design of such caverns, and thus it turned important to develop methods and test procedures for the purpose of obtaining information on the in-situ stresses in the rock mass.

Fig. 2. Typical Stand-up Time [BIENAWSKI, 1984]

Also, experiences from the mining industry showed numerous cases where excavations of underground openings with extremely large span (60 – 80 m and more) were seemingly stable without any rock support measures at all. Research showed that a major reason for this was the existence of sufficient tectonic, horizontal stresses even at shallow depths, thus providing favourable compressive

stresses in the excavation roofs, creating stable self supporting underground structures. This called for in-situ rock stress measurements to be done preferably before the excavation started, followed by rock stress control and deformation measurements during and after excavation. Examples from the mining industry were carefully documented as the response in the surrounding rock mass was monitored following the extension of the mining openings and excavation of the pillars. Stress measurements have been carried out in connection with a number of large underground excavations to document the stability of large spans and rock pillars.

Horizontal stresses of geological origin (tectonic stresses) are quite common in Norway, and in many cases the horizontal stresses are higher than the vertical stresses, even at depths greater than 1,000 m. The majority of rock stress related problems in Norway actually originates from high horizontal stresses, rather than vertical stress due to the rock overburden. This has been the case in a number of road tunnels and tunnels connected to hydropower development, and high stresses have also caused considerable stability problems in power house caverns. This has again called for rock stress and displacement measurements.However, the pure existence of high, or sufficient in-situ rock stresses, particularly horizontal stresses is an important condition that enable large underground caverns to maintain stability, particularly when the width of the caverns exceed 12 - 15 m which is the size of an oversized road tunnel. The majority of such rock caverns in Norway has been built in accordance with the term “unlined tunneling”. This will be described in the following.

3. Some general characteristics of the rock mass

Using the term “unlined” tunnelling must not be misunderstood in such respect that the tunnels are constructed without any rock support at all. In general, permanent rock support consists of rock bolts and sprayed concrete, and only extremely good rock mass conditions may constitute an exception with no support measures. Water and frost protection may be taken care of by installing free-standing systems which are not load bearing, except for traffic loads, and do not contribute to the rock support. [GRØV, 2001a]The capability of the initial, primary lining structures are utilised as permanent to the extent the quality of the rock mass requires. Such an initial primary lining is often sprayed concrete with a thickness ranging from 50 to 200 mm in combination with rock bolts, commonly fully grouted and corrosion protected, some times installed in a systematic pattern. The latest sprayed concrete technology, with alkali-free accelerators, allows greater thickness to be applied. Sprayed concrete based support structures may in many cases adequately replace massive, cast-in-place concrete structures or pre-cast segments. Such applications provide a long term use of sprayed concrete support compared to a temporary use as primary lining before a later, final concrete liner is installed.

The sprayed concrete mix design can be determined based on the actual project specific requirements that exist for the properties of the concrete. Today, the technology associated with sprayed concrete allows the designer to select amongst a number of properties of sprayed concrete: bond strength, strength, stiffness, ductility and permeability.This indicates that the sprayed concrete holds a great variety of properties that can be customised for any specific project. The challenge for the designer is to identify the relevant and applicable quality of the sprayed concrete. Furthermore, the development of spraying robots allows efficient application, with accurate dosage of admixtures and additives to cope with situations that require particular demands. At the same time additives have been developed that allow for example; a fast setting concrete with high early strength without influencing its final strength, the possibility ofapplying up to a 200 mm thick layer in one spraying operation etc.Rock mass impermeabilisation is taken care of by pre-grouting ahead of the tunnel face. The effect of improved stability in this respect is also an important momentum.

4. A Summary of the Importance of Rock Stress for Design

Table 1 summarizes the importance of rock stress for design and construction of various types of underground facilities. The table has been established based on the authors personal experience and may only be used as a guide. It indicates clearly that the presence of in situ stress in the rock mass can be used for the benefit of enabling various underground solutions. At the same time it is important to point out that the ignorance, or lack of such stress characteristic features may lead to unacceptable stability situations or situations where the underground facility may no longer fulfill its function.Table 1 is based on Norwegian typical rock mass conditions with a moderate to high E-moduli and UCS-values. Weaknesses occur as discontinuities in the rock mass and they can exhibit rather varying characteristics and capabilities as far as being a construction material and may transfer stresses in

different ways.The in-situ stresses are normally exposing a nature which reflects the gravitational component in the vertical direction, whilst in the upper 500m typically the horizontal component is much higher than the could be derived at by a pure theoretical approach.

Type of facility	σ_1	σ_3	$K = \sigma_{hor} / \sigma_{vert}$
Rock caverns	Moderate to high level can enable an optimized geometry. Too high may lead to stability problems.	Low level may produce a too small arch building in the roof/lack of confinement thus imposing stability problems.	K = 1 - 2 is OK. K > 3 is not OK. K < 0.5 is not OK.
Pressurised tunnels / cavens	Minor influence, high level may give stability problems.	Minor principal stress component must be higher than the water pressure (or the pressure from any other confined material) as an ultimate requirement.	No particular requirement.
Facilities with particular requirements to tightness	Moderate to high level can provide good confinement and stability and improved tightness.	Critical low level gives poor safety against leakage. Groutpressure<σ_3 Storage pressure<σ_3	No particularrequirement.
Transport-Tunnels	Moderate to high level is good for the confinement and stability. High stresses may give stability problems	No particular requirement, but should not be too low as that would reduce the confinement and reduce stability.	K = appr. 1 is OK.

Table 1 Overview of Requirements to Rock Stresses for Design of Various Underground Facilities

5. Construction of Large Underground Caverns

The construction of large underground caverns is normally split in various sections, such as top heading and benching. The number of benches may vary based on the height of the caverns and the equipment chosen for the job. The top heading is normally done by horizontal blast holes, forming a full horseshoe shaped “tunnel”. In wide caverns a set up of equipment may include two drill jumbos working side by side at the same face, as one single jumbo may not cover the full face alone.It is important to completely install all necessary support at the top heading before benching may start as it is difficult to later gain access to the roof and the upper parts of the walls for later support work, unless involving the use of large cranes, significant working platforms or scaffolding.During the rock mass classification it is therefore required to map properly the jointing as to identify potential large wedges that may occur later during the excavation of the lower parts of the walls. In figures 3 and 4, a typical sequence of an oil and gas storage is shown. In this case it is also shown that the excavation of the cavern shall take place in a fully saturated environment, the artificial water curtain has been installed prior to allowing the tunnel face to advance.It has been typically experienced that such large underground excavations have influenced the ground water conditions lowering temporarily the ground water level in the vicinity of the caverns.

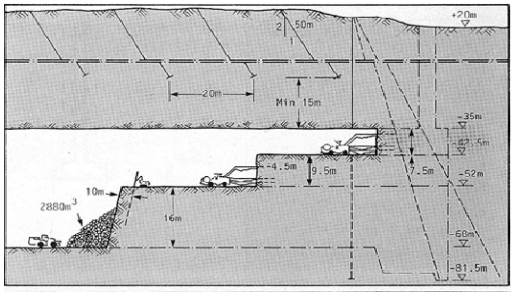


Fig. 3 Excavation Sequence Gjøvik Hall (Photo: NGI)Fig. 4 Typical Excavation Sequence (Top heading and lower benches and with artificial groundwater infiltration from surface) (Ill. unknown)

6. Stress utilization in Hydroelectric power plants

During and after the Second World War, the use of the underground was given preference out of considerations to wartime security. But with the rapid advances in rock excavation methods and equipment after the war, and consequent lowering of the costs, underground location came to be the most economic solution (Fig. 5). This also tied in with the development of concrete lined, and later unlined pressure shafts and tunnels, to give the design of hydroelectric power schemes a freedom of layout quite independent of the surface topography (Fig. 6).

Underground location of powerhouses in such schemes is now chosen whenever sufficient rock cover is available. Frequently the overall project layout requires the powerhouse to be placed under very deep rock cover where rock stresses may be substantial. This requires an investigation of the stress condition in advance for finding the most favourable orientation of the cavern and the optimum location, orientation and shape of ancillary tunnels and caverns.

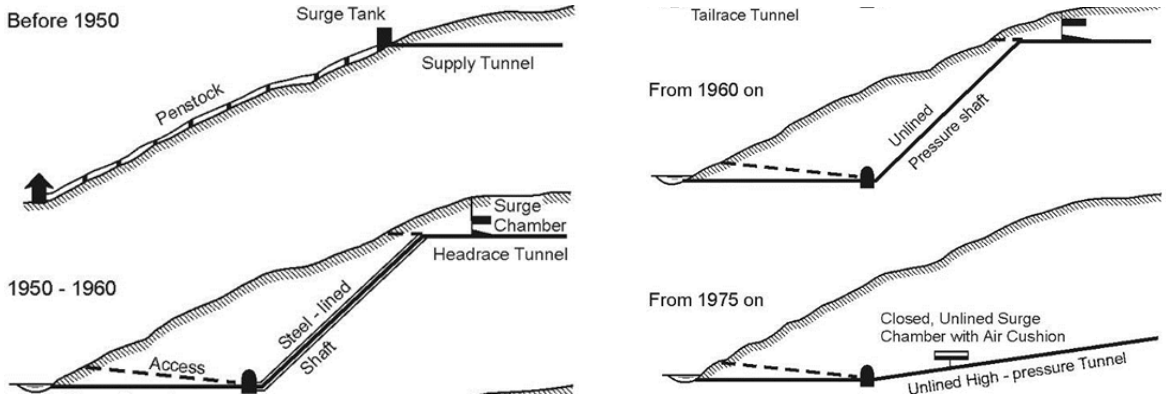


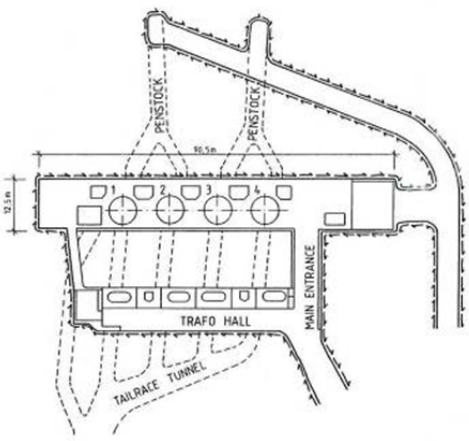
Fig. 5 and 6 Pre-1960 Development of Hydro Electric (left) and Post-1960 Development of Hydro Electric Power Stations (right) [BROCH, 2006]

The layout of these underground power stations and their adjacent facilities would typically include caverns that are 12 - 25 m wide and up to 35 m high. The width and height are depending on the size and type of the installations. The length of these caverns might be several tens of meters long, typically some 40 to 60 m long, but even up to a length of 100 m has been produced. The excavated volume would typically range from 10,000 m3 of solid rock and to 50,000 m3. In Norway some 200 such underground hydro electric power stations has been constructed.The cost of the underground structures are tightly associated with the distance between neighbouring caverns, their size as well as the length and dimensions of the connecting tunnels. Therefore it is a must to try and optimize the layout in such a way that the caverns are placed with as short distance s possible. A typical rule of thumb would be a distance of 1 – 1.5 times the cavern width, however the local geometry of joints must be considered.An interesting aspect of these hydropower projects is also the fact that some underground hydropower stations may as well include a transformer station being located right next to the power station itself, and with a large number of cross cuts, adits, exits etc. at various elevations surrounding the main cavern. These would of course influence the stress re-distribution in the rock mass and causing stress concentrations in the vicinity of the openings. It is therefore important to always work with a 3D-view and consider the influence and consequences of tunnel and cavern openings at various levels in the layout. Pillars may be subject to high stresses due to such stress redistribution and particular attention should be taken to the pillars themselves, their dimensions, layout, geology and support. This consideration is valid for horizontal pillars as well as vertical pillars. All in all these projects comprise complicated geometry (Fig. 7 & 8). The location of the rock caverns are normally fixed in the design concept based on information gathered during a comprehensive pre-investigation phase, however, pending on the actual rock mass conditions as encountered during tunnelling in the approach to the designed and planned location, changes may of course take place. Several underground hydro power stations in Norway have experienced changed locations and local optimisation to better fit the rock mass conditions. It is common to take into account such information as related to the following:

- rock types and mechanical properties;
- characteristics and frequency, spacing of rock mass discontinuities;
- in-situ rock stress;
- groundwater conditions.

During the approach to the planned location of the cavern(s) the rock mass is thoroughly mapped, joint systems are observed and characterised, weakness zones are interpreted, in-situ rock stress conditions are measured, ground water is monitored. If these conditions are not in accordance with the expected and required quality of the rock mass, it may be conclusively decided to shift the location of

the hydro power station, and other adjacent caverns, or make some layout adjustments. Typically, the final layout of the caverns, their location, geometry, alignment, lay-out of the tunnel system and rock support design may only be decided upon when the above information is obtained from the excavation of the approaching tunnels. Numerical analyses as well as analytical calculations are useful tools for the design and planning of the caverns. These must of course be verified during the construction phase by adequate monitoring and follow-up of the stability of the underground caverns. The present state-of-the-art prescribes systematic bolting of the rock ceiling immediately after excavation of the top heading, followed by fibre-reinforced shotcrete from 70 to 200 mm in thickness, according to rock quality.



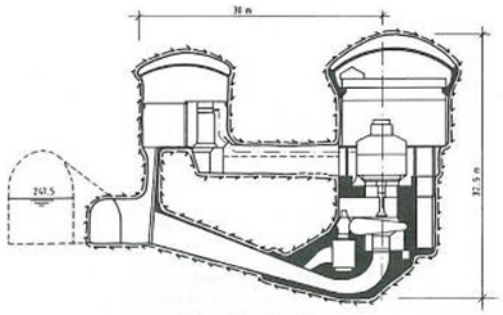


Fig. 7 and 8.Lay-out Solution, Planand section for Underground Hydro Electric Power Schemes [BROCH, 2006]

In a 20 years period starting from 1974 and ending in 1995 a total of some 20 combined underground defense shelters were built. These shelters serve as sports halls in peace time. Typically they were built to serve the purpose of hand ball arenas and swimming pools in accordance with international standards relevant for such arenas.It was a particular functionality of these facilities to try and place the caverns at a shallow depth, to reduce the length of the accesses. Several of these caverns were constructed with a width of 25 m and only 15 m rock cover. The common size of these projects include in the range of 55,000 m3 of rock excavation. Figure 9 shows the interior of such caverns.The main conclusion of these projects was that even at shallow depth, there is in-situ stresses in the rock mass that could be utilized for the stability of the caverns, although the gravity component of the stress situation was very small. The knowledge of this stress situation enabled the possibility of constructing such caverns.

This application of underground caverns culminated with the design, construction and realization of the Gjøvik Mountain Hall. This underground facility utilized the knowledge gained during a long period of underground construction and a rock cavern with a width of 61 m was built. The project will be further detailed below.The utilisation of the underground openings had seen a significant growth in Norway, particularly due to the development in the hydroelectric power sector, where an increased number of projects utilised underground alternatives for waterways, pressurised tunnels and location of hydropower stations and transformer rooms. The Norwegian tunnelling industry developed techniques and methods to improve the efficiency and quality of underground works. A comprehensive experience base was established which became important when the underground storage of hydrocarbons was introduced.

The tunnelling industry shifted from hydropower projects to oil and gas storage and then towards the current use for infrastructure purposes. In the 1970's Norway grew to be a major oil and gas producing nation with the corresponding need for larger storage facilities. It also became evident that the use of surface structures needed to be reconsidered. The solution in Norway was to excavate large rock caverns, utilising the availability of suitable rock mass conditions and the tunnelling experience obtained through the hydropower development. In brief the main advantages of underground rock storage are:

- Utilizing variety of rock capabilities
- Environmentally friendly and preserving
- Protection during war
- Cost aspects
- Operation and maintenance
- Protected from natural catastrophes

Fig. 9 Combined Underground Sports Facility and Civil Defence Shelter (Photo unknown)

The typical size of the rock storage caverns in most recent projects indicates a cross-section of appr. 500m2. The caverns are close to 20m wide and 33m high, and this has become a typical cross sectional area for such caverns (Fig. 10).

7. Underground Caverns for Oil and Gas Industry

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

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

As mentioned above, in the Norwegian concept lining as a barrier had been abandoned due to the significant costs associated with such solutions. Also the techniques of pre-grouting of the rock mass to stem or reduce water leakage started to be developed during this period.Table 2 presents a summary of the Norwegian crude oil storage facilities and refinery caverns for hydrocarbon products. Table 3 shows a summary of the major petroleum gas storage facilities. It should be noted that all these gas storage facilities are for propane, except Mongstad commissioned in 1989 that also stores butane and Sture in 1999 that stores a propane/butane mixture.

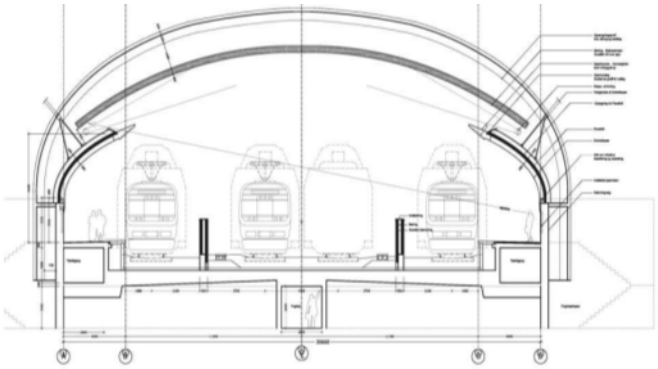
Fig. 10 Typical Underground Oil and Gas Storage Cavern with Straight Walls (Photo unknown)

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

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

8. An Underground Railway Station in a Large Span Cavern

The Norwegian Rail Road Authorities has just recently commenced the construction work with a new high speed rail way on the west coast of the Oslofjord connecting the cities south and south-west of Drammen with a new link to Oslo, the Vestfoldbanen. The city of Holmestrand is located along this particular rail link. Holmestrand is a city which has the sea to the east and a mountainous hill to the west. Due to the stiff curvature of the high speed rail link it was decided that the rail way should be placed in a tunnel to pass Holmestrand. This means that a railway station in open air was abandoned. However, a railway station was considered as an important link part of this railway link. Consequently, the station was replaced by an underground facility being placed in the mountainous ridge west of Holmestrand.



The speed for this rail way link is 250km/h. The rock cavern which is designed to cater for the station area at Holmestrand is planned with a cross section of theoretically 350 m2 and with a length of almost 900 m. The width of the cavern will be 36 m and with a height of 15 m (Figure 8). This is sufficient to accommodate a total of 4 parallel tracks and platforms. The cavern is placed with a horizontal distance to the surface of 100m.

Fig. 11. Cross Section of the Underground Station (Ill. Norwegian Rail Road Authorities)

The site location has been subject to a typical pre-investigation scheme based on Norwegian standard including surface geological mapping, core drilling, structural geological analysis to mention some. The rock mass consists of basalt which has a typical UCS of approx. 240 MPa. The area is cut by several joint sets which have been identified at the surface and also by one significant weakness zone.More important though, and the key to such an excavation is the stress measurements being conducted. These consist both of hydraulic fracturing and 3-D overcoring. It was been found that the major stress component acts perpendicular to the cavern axis and the following magnitudes were found:3D-overcoring yielded 4-11MPa, and hydraulic fracturing yielded 2-6MPa

As a comparison, the rock cover is about 70m which would produce a vertical stress component gravitationally of approx. 2 MPa and thus a horizontal component in the range of 0.5 MPa. This example demonstrates the need of performing stress measurements and utilizing these measurements to optimize the cavern design.

9. A futuristic solution: cargo terminal under-ground

In Trondheim, the Public Railway Authorities were looking at the possibility of locating a new, major cargo terminal. The purpose of a railway cargo terminal is to receive, reload and distribute cargo and a typical layout is comprised of the following main areas: Loading area; dedicated to the handling of goods between rail wagons and other types of transport. Shunting area; dedicated to the assembly of the various wagons to make up individual freight trains. Receiving and departing area; where each freight train is recorded at the cargo terminal. A cargo terminal would necessarily occupy a large surface space, and it may therefore come into conflict with other interests and use of land. Although inter-linked, it is possible to spatially separate some of the different areas without disrupting the

operational and functional efficiency of the terminal. Furthermore, the operation of the cargo terminal will inevitably interfere with and disturb the surrounding environment; not only in terms of noise and traffic from continuous operation, but also due to the aesthetic adverse impact on the terminal surroundings (Figure 9).

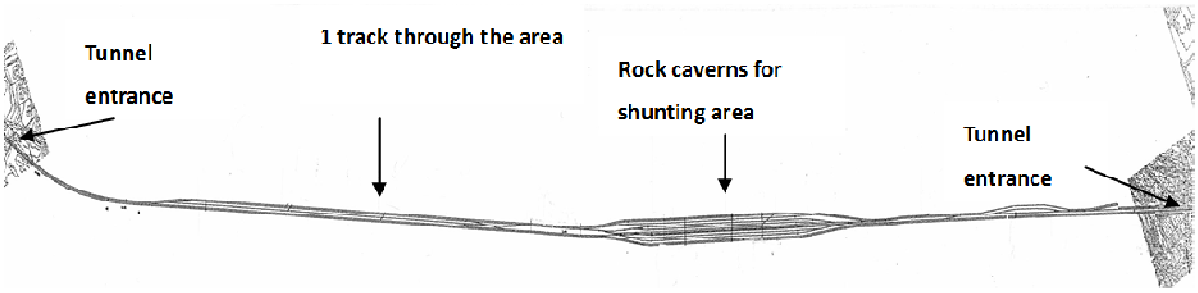
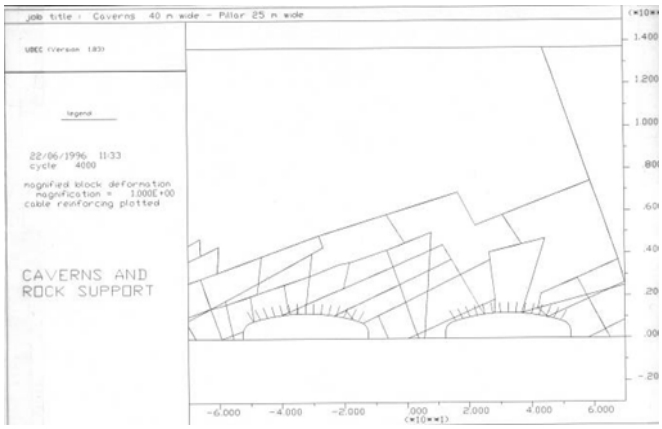


Fig. 12. Schematic layout of an underground cargo terminal in Trondheim

Cargo terminals are traditionally considered to be surface installations. The location proposed by the authorities in Trondheim is dominated by residential, cultural and recreational areas, which has aroused hostile public opinion. The proposed site location was chosen due to its proximity to other important freight carriers. In an effort to mitigate the adverse impact on the existing surface structures, a study was prepared to present an alternative solution; to relocate certain portions of the cargo terminal to underground rock caverns. In discussions with the operator of the future cargo-terminal in Trondheim, the shunting area was identified as a candidate for relocation from the reminder of the cargo terminal.The reasons for selecting the shunting area as a candidate for an underground location are the following:

- It is noisy; operating the shunting area where wagons are hooked on/off to the trains, generates a metallic, high frequency click-clacking noise together with the intense screaming from the mechanical braking of the wagons.
- It is aesthetically adverse; the shunting area comprises many parallel tracks and switches, several hundred meters in length and several tens of meters wide.
- It is an around the clock operation; the assembly of freight trains is a time consuming operation and to cope with the traffic at the terminal, an around the clock operation is required causing a continuous disturbance to the surroundings.
- It is space demanding; the total surface area required for the cargo terminal is equivalent to 25 football fields, with the shunting area including its service facilities occupying approximately 1/3 of this space.



It allows valuable surface space to be utilised and developed for other purposes, or to be reinstated to its original use. When the land is occupied for residential, recreational and cultural use, it is important to reduce the impact that such change would have. A surface installation would require many houses to be demolished whilst a subsurface solution would reduce the impact on the current use of the area. The visual impact alone created by a 40m wide band of railway tracks with noise reduction barriers along its entire boundary constitutes a significant impetus to go underground.

Fig. 13 Caverns and Rock Mass Structures [GRØV, 2001b]

It improves the physical environment and limits awareness of and disturbance to the public. Most people prefer to keep their daily residential and recreational life apart from industrial areas. The necessity of such areas is generally accepted, however, most people would prefer not to have them as neighbours. In this example, the most positive impact achieved by an underground installation is the reduced noise. A surface installation would probably disturb the community within a distance of several hundred meters. The airborne noise can to some extent be mitigated, but only partially. Ground vibrations created by the moving of heavy equipment will also be significantly reduced for a subsurface installation.

It eases flexibility for future expansion. Construction of such a large, complex structure as a cargo terminal, within an area which is already extensively developed, implies a significant limitation and flexibility as regards to any future expansion. A well planned, subsurface structure could provide greater flexibility for future expansion.

It reduces the need of construction activity at the surface and provides a surplus on the mass balance. A cargo terminal requires a large, flat surface area and could lead to a major earthmoving exercise. A subsurface solution produces a surplus on the mass balance, and the blasted rock constitutes a value in itself, which could be sold or used for other projects.

It has a constant climate around the year that ensures reliable and regular operation. A shunting area is sensitive to weather conditions, as icy switches etc. may cause adverse effects and operational hazards, such as derailing. Particularly during cold winters in Norway the operational regularity of the shunting area would increase if located “in-doors”.

It preserves the environment. For a subsurface solution dedicated systems for collection and handling of various types of spill was planned. With current tunnelling technology and methods, the cost estimate showed an obvious disadvantage. The subsurface alternative involved an increased investment ofapproximately 20% over the surface solution, when taking the cost savings on surface preparations and land acquisition into account. The installations in the rock caverns as well as the rock support measures were reasonably cost effective. However, in the bottom line the subsurface alternative has a higher cost/benefit ratio, when no price tag was put on the environmental aspects.

A railway track constructed in the early 20th century, is currently in use as a ring connection around the city. This includes a 2.5 km long rock tunnel excavated in greenschist and greenstone, mostly occurring as a quite massive rock. The rock mass conditions are fairly well known with respect to its ability to host subsurface structures, as a number of smaller caverns and tunnels have been constructed in this geological formation.

The rock mass has proved to be of fairly good quality, which has resulted in a minimum of rock support and mostly dry conditions. A joint set parallel to the foliation of the schist, with a 30□ dip is the most dominant. The feasibility study concluded that these conditions were favourable for large rock caverns. The fact that the existing tunnel is available for inspection increases the level of confidence with respect to the quality of the rock mass. Along the 2.5 km long tunnel, cast-in-place concrete lining was applied on 3 sections with significant weakness zones, constituting a total length of 110 m. Rock support is in general rock bolts. Minor spots with water dripping or moisture were observed. The rock cover above the proposed caverns is a maximum of 125 m. The regional, in-situ stress regime indicates that the maximum horizontal stress component could be expected to be influenced by remnant stresses, producing a K-factor (horizontal stress component/ vertical stress component) in the range of 1.0. The in-situ stress situation was modelled with vertical and major horizontal stress components in the range of 3.5 to 4 MPa.

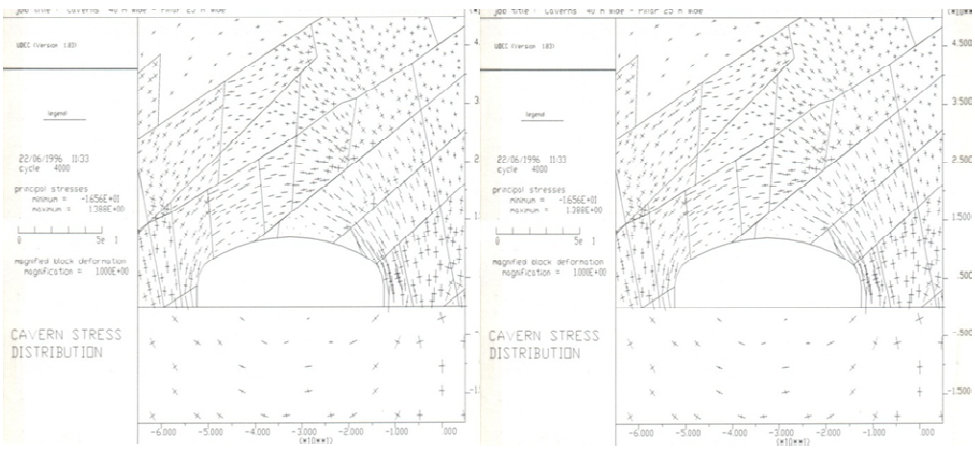


Fig. 14 Stress distribution and pillar stress [GRØV, 2001b

The proposed solution utilised the existing railway tunnel with a major enlargement to reach the required cavern width of 42 m was studied (Figure 9). Discussions with the operator of the cargo

terminal indicated that this width was needed to allow for the required number of nine parallel tracks. Based on both analytical and numerical calculations, the feasibility study concluded that the 42 m wide cavern with a cross section of almost 400 m2 and length of almost 700 m, was feasible in the actual rock mass conditions. An alternative layout including two parallel caverns, each 28 m wide and separated by a 25 m thick pillar, was also studied and found feasible (Fig. 15 & 16). By 28m width, each cavern could hold 6 parallel tracks, or a total of 12 tracks, thus allowing a greater capacity than the option with a single 42 m wide cavern.

The subsurface proposal allows for the shunting area itself, and the various service facilities required for its operation. In addition, one separate track allows for the possibility of trains passing through the tunnel without interference with the ordinary operation of the shunting area. Consequently, together with the main cavern(s), the concept proposed a number of small enlargements of the existing tunnel, which utilise the entire length of the 2.5 km tunnel to provide the various service facilities. The whole design included excavation of some 400,000 m3 of solid rock, which together with the existing tunnel wouldmake a total underground volume of almost 500,000 m3. The numerical analysis, which actually modelled the two parallel caverns with a width of 40 m and a 25 m pillar, indicated that a satisfactory level of safety could be achieved by the systematic application of bolting, spacing at 2 m x 2 m, 25 mm diameter and 5 m long. The numerical model did not include support measures such as sprayed concrete or cast-in-place concrete. However, the design included sprayed concrete as a surface sealing. Until this date, however, neither the caverns have been excavated nor has the above ground facilities been built. A major political decision is lacking, until then we have to wait and see before any such caverns can be built, but its feasibility and advantages for the society are obvious.

10. What Made the Gjøvik Mountain Hall Feasible?

The Gjøvik Olympic Mountain Hall was constructed during the years from 1991 to 93 as one of the ice-hockey venues for the 1994 Lillehammer Winter Olympic Games. With a span of 61 m, a length of 95 m, and a height of 25 m it is the largest man-made rock cavern for public use in the world (Figures 12&13). The total excavated volume was 140,000 m3. The width of the cavern is not the only peculiar feature of this project, as interesting is the fact that the rock cover varied between 25 m and 55 m, i.e. the overburden throughout the underground cavern is far less than its span.

The rock type at the site is Precambrian gneiss. The rock has developed a network of tectonic micro-joints, which often were filled with or coated with calcite or epidote. These rock mass conditions resulted in a well jointed rock mass with an average RQD of about 70. The typical joint character is one of persistence, moderate to marked roughness, and normally without clay filling, i.e. positive characteristics when considering large spans. The Q-value is typically 30 for the best and 1 for the poorest quality rock mass, with 12 as an average value.

Experiences from large span mining chambers in Norway indicated that a major prerequisite to obtain stable large span caverns without heavy rock support was sufficiently high horizontal rock stresses. At a very early stage of planning, in situ rock stress measurements were therefore carried out from an existing tunnel. The results showed dominating horizontal stresses in the order of 3 – 5 MPa. At a depth of 25 – 55 m the vertical stress due to gravity is less than 1 MPa, indicating that the horizontal stresses are stresses generated by geological processes (tectonic stresses). This was verified later by hydraulic fracturing tests, conducted in several rounds, including both over-coring and hydraulic fracturing in vertical boreholes drilled from the surface above the hall. Based on these findings, it was decided to proceed with the investigations.

With reliable in-situ stress values from the stress measurements, numerical modelling was carried out, using various BEM, FEM, UDEC and FLAC codes. The final conclusion was that a stable and virtually self supporting 62 m span could be constructed under the given geological and rock mechanics conditions. Maximum roof deflection could be expected to be in the 5 – 10 mm range. A key element of this entire process is related to the in-situ stress measurements, which were Adjacent to the main cavern hall area, an underground swimming pool was constructed in 1974, thus the geology of the area was well known, and later this facility was included in the new construction. In addition, the gentle slope of the hill hosted an underground telecommunication centre in the close vicinity of the Gjøvik hall.To monitor roof deformations a number of multiple position borehole extensometers (MPBX) were installed.

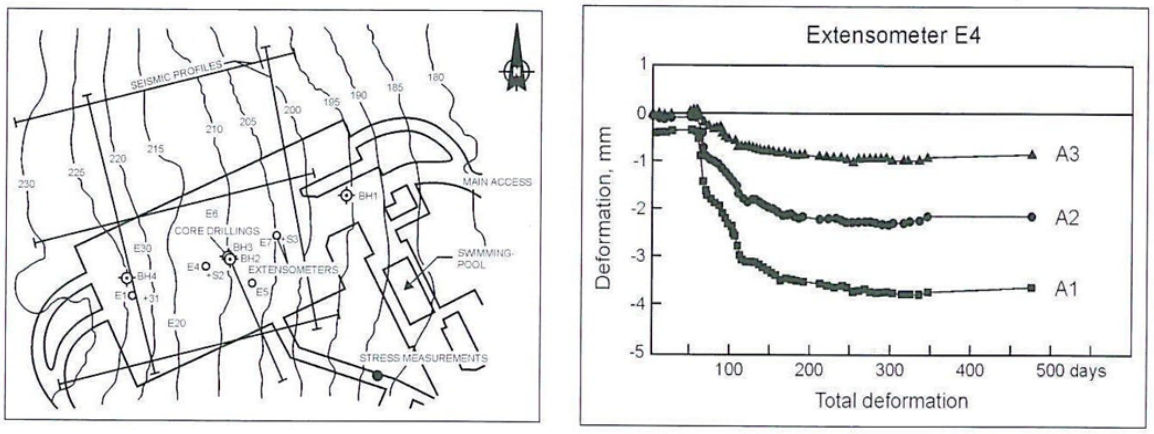


Fig. 15 and 16 Cross Section of the Gjøvik Mountain Hall [MYRVANG, 2004] and Cavern construction sequence (Photo unknown)

Figure 15 shows the cavern layout and the position of seven, 3-anchor (position) MPBXs placed in boreholes drilled from the surface (marked E1-E7), and three placed in boreholes drilled vertically upwards in the cavern roof (marked S1 – S3). In addition, surface precision levelling was carried out on top of the three centre-line extensometers.

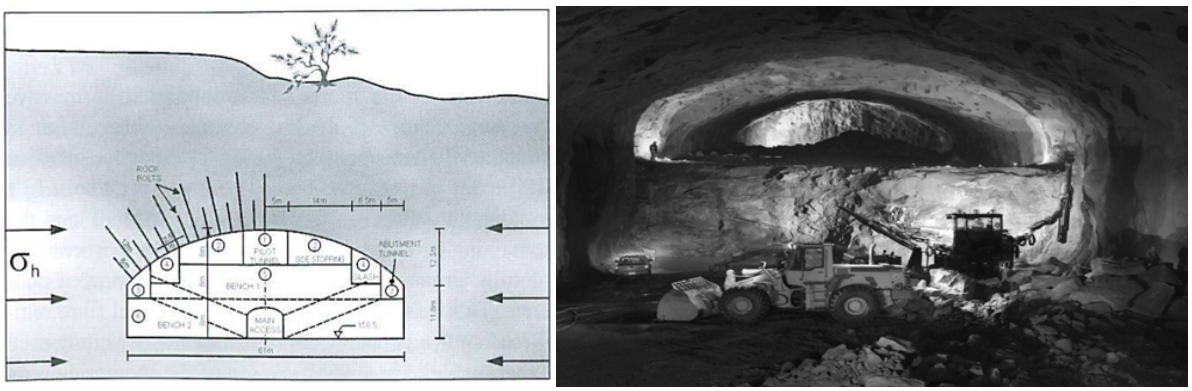


Fig.17. Cavern Lay-out and Borehole extensometer Locations, Typical Reading of Surface Extensometer (right hand side) [MYRVANG, 2004]

Readings were taken regularly throughout the construction period. Figure 17 shows typical readings for the central extensometer E4, with A1 being the anchor close to the cavern roof. After the full span was blasted after about 100 days, the deformations show a decreasing trend until they stabilise completely after some 300 days. The maximum deformation is less than 4 mm. By adding the readings from surface and roofextensometer and the surface levelling, the maximum deflection was estimated at about 7 mm. This is well within the predicted values from the different numerical models. To check the roof stresses, 2D in-situ rock stress measurements were carried out mid-span close to the S1, S2 and S3 extensometer locations. They all showed compressive roof stresses in the range of 2 – 5 MPa, which are good indicators of a stable, immediate roof.

The investigations clearly indicate that the roof "globally" is a self-supporting structure. The roof is, however, systematically supported by 6 m fully grouted 25 mm rebar bolts in a 2.5 m x 2.5 m grid, where every four bolt is substituted by a 12 m cable bolt. This is followed by 100mm fibre reinforced shotcrete. A series of eight rebar bolts were instrumented by strain gauges, and the load change was monitored as the span was increased from 10m, through 37m to the final span of 61m. Only three bolts showed any indication of load take up. This happened close to the roof surface, with very moderate load in two cases (10 kN and 15 kN), while the third one showed 87 kN, which is about 40 % of the yield load. To check the performance of the fibre reinforced shotcrete, strain gauge rosettes specially made for concrete were installed in four locations. The readings only indicated very low tensile stresses, which probably are due to shrinkage of the shotcrete. The rock/shotcrete bond

was tested by a direct pulling test on drill cores containing the bond. The average tensile strength of the bond was 0.85 MPa, which was regarded satisfactory. The main purpose of the shotcrete seems to be to bond the rock surface together, preventing smaller rock volumes to fall. Based on this, it may be concluded that the need of the systematic pattern of 6 m and 12 m bolts and cables is questionable. A systematic 2.5 m x 2.5 m pattern with 3 - 4 m fully grouted rock bolts, combined with 75 mm to 100 mm shotcrete will do the job even for 50 m to 60 m span. However, horizontal, tectonic stresses of a certain magnitude are necessary to establish the global stability of the roof.

11. Rock pressure measurements

The planning basis generally available with regard to the knowledge of rock stresses is as follows:

- Experience and the database from a number of completed tunnel and underground constructions in Norway with various levels of rock stress (high, moderate and low).
- Results of reported rock pressure measurements at various construction sites.
- Analysis tools based on correlation between calculated stresses and measured rock properties.
- General industry knowledge of the phenomenon.

However, the above-mentioned factors should all be regarded as general knowledge regarding rock stresses and are not necessarily applicable to all construction projects under all framework conditions.Nevertheless, it is generally true that knowledge of local stresses in a rock mass creates important, fundamental premises for the way in which a planning project can be handled. Thorough knowledge of general as well as local conditions facilitates the optimal planning of the construction project. However, to obtain knowledge of the local conditions, investigations must be carried out to supplement the existing foundation, which is often of a general nature.Disregarding the local rock stresses may result in poor optimisation with regard to geometric conditions and layout, problems with unexpected, unfavourable stress conditions, and so on.

So, is it always necessary to know the exact values of the maximum and minimum principal stresses and their orientations? No, in fact it is not. The need will naturally depend on the type of construction being planned, the degree of technical difficulty and the geographical framework conditions.There must also be co-operation with the client and an understanding of the importance of obtaining adequate basic knowledge. However, the planner, in a technical capacity, must be responsible for advising of the need for, and the value of obtaining, access to better basic data. In other words, the degree of cost-effectiveness must be clarified and documented when deciding to obtain more detailed information about rock stresses. In some cases, general knowledge will be adequate for the needs of the construction project.Stress measurements can in principle be performed in a number of different ways. In Norwegian projects the three most commonly used methods are as described below:

- Hydraulic methods (hydraulic splitting/hydraulic fracturing)
- Three-dimensional overcoring
- Two-dimensional overcoring

Hydraulic methods

Hydraulic fracturing or splitting is carried out in a borehole which is drilled from the outset so that one of the principal stresses coincides with the orientation of the hole. Traditionally, such orientation of the borehole has been a requirement. The test procedure follows international standards. When performing the measurements, a 0.5-1.0 metre long test section of the borehole is closed off using a double hydraulic packer consisting of two rubber seals. The packer is located at the depth at which a measurement is required and water is pumped into the seals to isolate the test section from the rest of the hole. During the test, the pressure in the seals is adjusted so that it is always higher than that in the test section. This prevents leakage between the borehole wall and the rubber seal and thereby uncontrolled leakage from the test section. The seal pressure must however not be too high relative to the pressure in the test section, as this would increase the danger of the seals themselves fracturing the rock formation.The borehole may be isolated by sealing the hole with cement, with a pipe penetrating into the test section. The disadvantage of this method is that it is only possible to pressurise one section of each borehole. Another possibility is to use an impression packer. This type of packer is

coated with a softer layer of rubber and is pressurised in the same way as the test packer. Pressurisation causes the rubber to be pressed out against the borehole wall, creating an impression of any cracks.

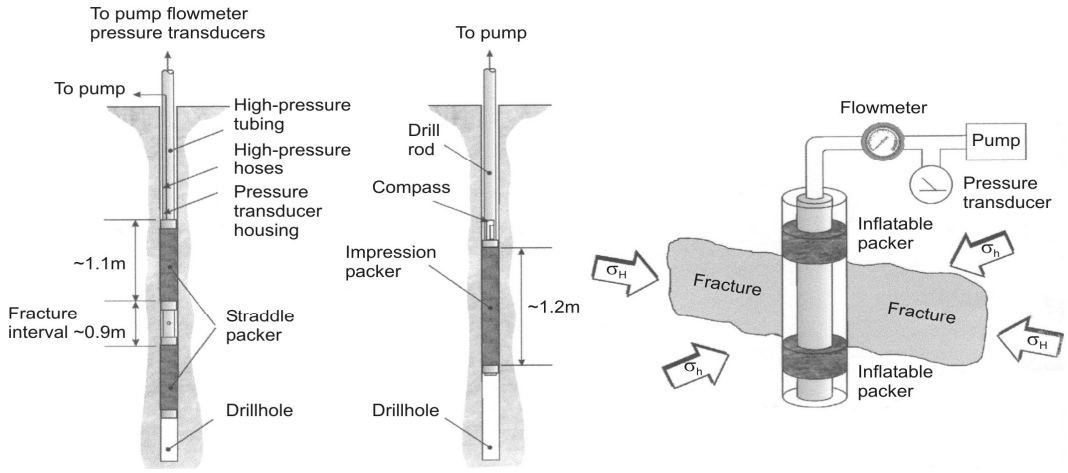


Fig. 18. Principles and equipment for hydraulic fracturing, impression measurement

The pressure in the test section is increased at a rate of 0.1 to 0.5 MPa/sec. When the water pressure in the test section reaches a critical level, cracks will be initiated in the borehole wall and water will flow into the crack system. It is recommended that after fracturing has occurred, enough water is pumped in to create a crack approximately three times as long as the borehole diameter. The water flow into the test section is then shut off (the shut-in point). During the test, the pressure and water flow into the test section are recorded as a function of time and an idealised pressure/time plot is obtained. After the pressure has been returned to approaching the pore-pressure level, the test section is again pressurised and the characteristic pressures are recorded. At least three cycles are carried out in each test section.

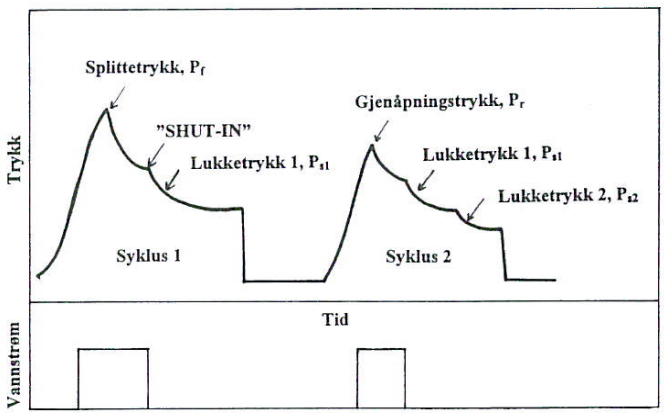


Fig. 19. Idealised pressure-time diagram for fracturing tests in which the vertical stress is not equal to the minimum principal stress (Ref. 6)

The characteristic pressures recorded in connection with hydraulic fracturing tests are:

P_f = fracture/breakdown pressure recorded in the first cycle
P_s/P_{isi} = instantaneous shut-in pressure recorded after shutting off the water flow
P_r = re-opening pressure, on re-opening of fractures in subsequent cycles

The most important parameter determined by hydraulic fracturing is the shut-in pressure. The physical significance of this phenomenon will not be described in detail here. At shut-in, the water flow into the system is stopped, the flow into the crack ceases and the pressure gradient disappears. This causes an immediate increase in pressure at the end of the crack, which will be elongated until static equilibrium is reached. Because of leakage of water to the pore spaces around the borehole, the pressure will gradually decrease. When the pressure has sunk to a certain level, the crack will begin to close. On the pressure/time diagram, this corresponds to the point at which the gradient of the curve is

greatest following shut-in. The pressure drop in the crack will be uniform since the leakage to the pore spaces is constant. As closing of the crack occurs, the system can be assumed to be in a static state. The shut-in pressure thus represents the stress perpendicular to the fracture plane, which is the minimum principal stress.

$$\sigma_3 = P_{s2}$$

The overcoring method

The selection of a measuring location will depend entirely on what type of measurements are to be carried out (2D or 3D). Three-dimensional measurements are performed to determine the original stress state and the test location must be selected with this in mind. This means that the measurements must be performed in such a way that they are affected as little as possible by existing rock cavities, fault zones or other discontinuities in the rock formation.

The measurements are performed underground from a diamond borehole drilled to a distance of at least 1to1.5 times the diameter of the rock cavity from which it is drilled (see Figure 16).

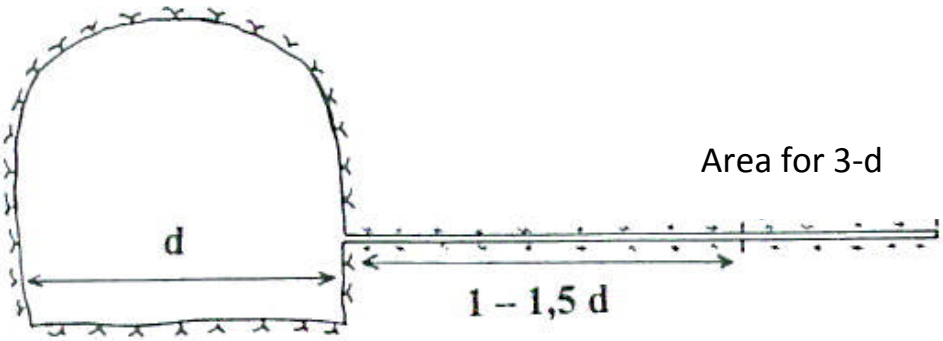


Fig. 20. Schematic diagram of selection of a test location for 3D measurements

Two-dimensional measurements are performed when one wishes to determine the stress situation in one plane, for example stresses in horizontal or vertical pillars and internal tension in the roofs and walls of large rock excavations, or to study high tangential stresses, for example in zones where there is danger of sloughing. The hole orientation is determined depending on the direction in which the stresses are to be measured, since when using two-dimensional measuring cells the stresses are recorded in a plane perpendicular to the borehole direction. The selection of a test location for two-dimensional measurements is shown in Figure 20.

In 1966 a measuring cell was developed which made it possible to determine the three-dimensional stress state based on measurements in a single hole. The method was refined and used by the Rock Mechanics Laboratory at the Norwegian Institute of Technology from 1968 and from 1985 by SINTEF Rock Engineering. Other versions have subsequently been used by, among others, Vattenfall and the Luleå University of Technology in Sweden, and by the Commonwealth Scientific and Industrial Research Organisation (CSIRO) in Australia. This is the method currently used by SINTEF Rock Engineering.

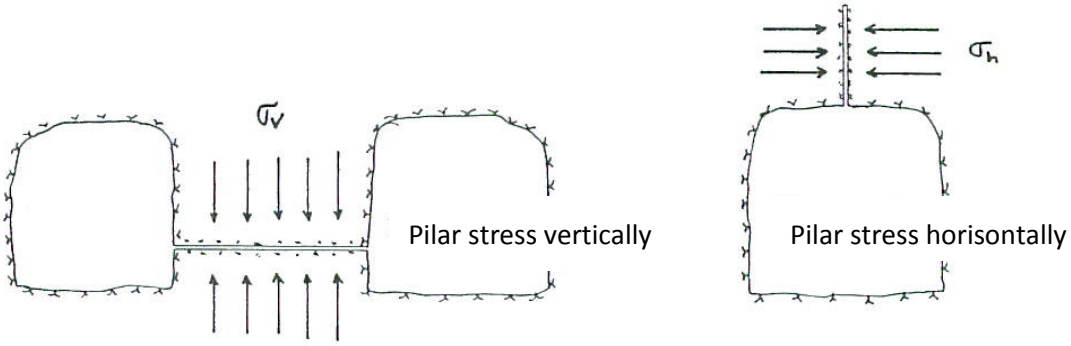


Fig. 21. Schematic diagram for selection of a test location for 2D measurements

12.Conclusions

In Norway a large number of underground caverns have been built during the last 30 - 40 years. Most of these caverns are related to the development of the hydro electric power supply. It is said that worldwide there is about 500 underground power stations, out of which 200 are located in Norway. This is a rough figure but it indicates how the underground has been utilized for this particular purpose. Furthermore, several underground caverns have been constructed for the purpose of storing oil and gas, whilst a large number of underground large caverns are related to sports complexes and civil defense facilities.

The majority of these caverns is in the range of 15 to 25 m wide, some tens of meters high and up to several hundreds meter long. They are all built in accordance with a concept of unlined caverns, mainly supported by rock bolts and sprayed concrete. One major common feature of these caverns is the presence of sufficient horizontal in-situ stresses. Another common feature is the field testing of in-situ stresses of the selected site location. Dedicated testing methods, both hydraulic fracturing and over-coring have seen great developments in Norway. Indeed, the presence of in-situ rock stress allows the possibility of opening large underground caverns. This is an important aspect that needs to be taken into account when designing underground caverns and wide tunnels in urban areas. The technology was developed in rural areas when the hydro electric power development took place in Norway, and is now available for various purposes when utilizing the capability of the rock mass. It is required to add that the experience and knowledge gained from the mining industry with respect to the understanding of the in-situ stress conditions has been invaluable for the development of large caverns for civil application.

The examples presented in the above shows that the utilization of the underground can benefit the development of urban areas by using large underground caverns to host various activities. The use of the underground in urban areas may not be restricted to various types of transport tunnels, and even small or normal sized transport tunnels would indeed benefit from the presence of sufficient in-situ stress conditions.

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