UNDERGROUND NUCLEAR POWER PLANTS CONSIDERING ROCK ENGINEERING PRECEDENT

By Nick Barton

ABSTRACT

Various rock engineering developments over the last decades have made the use of very large engineered rock caverns feasible as a method for developing underground nuclear power plants. Early site investigations in Norway in 1971 for potential UNPP, were followed by pre-UDEC physical models in 1976 and 1977 with tens of thousands of blocks formed by joint-simulating sets of intersecting tension fractures. The objective was the simulation of 50m spans in jointed rock. This was followed a decade later by Norwegian construction of the 62m span cavern for the winter Olympics in 1994. The Gjøvik cavern measures 62 x 24 x 90m and was constructed in 7 months. It is supported with systematic rock bolts and just 10cm of fiber-reinforced shotcrete. This cavern, despite its moderate rock quality Q from 2 to 30, RQD from 60 to 90, remains by far the largest engineered span for public use. However, the large span is dwarfed in another direction by the 80 to 90 m heights of a very few of the world's hydropower caverns. These are all located in China. Underground siting of nuclear power plants of a variety of potential sizes, presents obvious safety enhancement in relation to the earthquake, terrorist, and tsunami risks of surface plants. Rock engineering is clearly not one of the limitations for UNPP.

Keywords: caverns, rock quality, deformation, numerical modelling, nuclear power,

INTRODUCTION

By chance the author's first on-site job in a 50 years career was related to *the planned underground siting of a full-scale nuclear power plant.* This job was assigned a few months after arriving in Norway to work in the Norwegian Geotechnical Institute (NGI, Oslo). The year was 1971, and the first potential site identified by the Norwegian State Power Board (today called Statkraft) was at Brenntangen, in good quality gneiss, on the east side of the Oslo fjord. Some of the field testing performed was described by Di Biagio and Myrvoll, 1972 and more briefly by Barton, 1972, in an international conference in Stuttgart: *Percolation through Fissured Rock*. Norway already had two small underground research plants in Kjeller, and in Halden. These had been used in European reactor research and in medical research studies. They have since been decommissioned after approximately 60 years of operation.

The first task at Brenntangen was to extract details from borehole permeability measurements in an inclined hole that was part of the initial site characterization of this potential location. This was followed by other investigations, including tracer tests. The hope was to find less jointing and lower permeability as depth increased, which would help with decisions of how deep to site the largest caverns, including the need for a 50m span reactor cavern, if this site option was to be chosen.

The interpreted hydraulic apertures of the tested rock joints (i.e. natural fractures) was approx. 90, 70 and 60μm in three depth zones of 1-20m, 20-40m and 40-120m, based on flow tests down this first 45° inclined borehole. (Since 100 μm = 0.1mm, satisfactorily small water-conducting joint apertures were indicated). Their estimated mean spacings within each depth zone were respectively 1.5m, 2.6m and 3.1m, reinforcing the image of lower permeability with depth. Significantly, from 5 to 20% of the 5m long permeability test sections had shown zero flow. The zero-flow percentage increased at the greater depths, therefore contributing to the improved distributions of estimated apertures and spacings. A metric version of Snow, 1968 was used in these early analyses. This method was later amplified so that physical apertures could be estimated using the small-scale joint roughness JRC, for the purpose of grout-take estimation, and choice of micro-cements and/or injection pressures.

Rock quality improvement at depth will be revisited later in this paper, as it is fundamental for cavern (and tunnel) siting: for example how far into a hillside will be optimal in minimising tunnel and cavern support costs, in relation to distance from the necessary cooling water source (lake, sea, reservoir) in post-Fukushima planning *for logical and secure siting of nuclear power plants underground* (UNPP). A planned sequence of differently sized rock caverns and tunnels is assumed, and small-scale powerplant units are not a pre-requisite for such siting, but might be in countries with young, less stable rock mass conditions. Proximity to water-cooling facilities

PR-INJECTION BENEFITS FOR A ROCK MASS

An important development made in recent decades is that the permeability of the rock mass needed to house important sub-surface facilities can now be controlled during excavation. This is where pre-injection of stable, non-shrinking fine cements and additives comes into the picture. It is known from experiences of high-pressure pre-injection in the last two decades that water can be successfully and permanently displace from rock tunnel excavations, using ultra-fine or micro-cements. When the rock is already of good quality with tight joints as in the above case, we must use high pre-injection pressures (5 to 10 MPa) and ultra-fine cement and micro-silica. The latter has particles in the nano-range, as fine as cigarette-smoke, and helps to ensure stable, non-shrinking grout mixes with low initial water/cement ratios.

The grout effectively seals and strengthens successive joint sets and improves the quality of the chosen rock mass. For example, besides permeability reduction, the seismic velocity and the deformation modulus are each improved. A recent review of the problem of achieving dry tunnels in jointed rock, and a discussion and demonstration of the measured rock mass property improvements is given in Barton and Quadros, 2019. The ability to physically improve rock masses means that a *larger number of potential sites* for UNPP can be found in any given country.

The author knows from decades of characterizing rock that is recovered from cored boreholes, and from tunnel and cavern logging, that there are countless sub-surface locations in many countries that have rock masses of suitable rock quality for the economic construction of tunnels and large caverns. This means construction using just moderate amounts of reinforcing rock bolts (of e.g. 4 to 6m length) and fiber

reinforced S(fr) sprayed concrete of 10-20cm thickness. When rock is of fair to good quality 5 to 10cm thickness of S(fr) is sufficient according to the Q-system recommendations (Grimstad and Barton, 1993, Barton and Grimstad, 2014). Eminently suitable rock bolts (CT-type) have been used for the last 30 years that have five layers of corrosion protection which is centred around a double-annulus of grout, together with galvanizing and epoxy painting. These are 'thousand-year' bolts, much superior to the *nominal* '100-120 years' of many current projects.

It should be noted that there are now some 800 underground hydroelectric power plants in the world. Some decades ago Norway led with 200 of these, and 3,500km of related headrace and pressure tunnels. Updated numbers are 230 and 4,300km respectively. Since 99% of Norway's electricity supply is derived from hydropower, the reliability (and of course stability) of these underground caverns is essential.

For those of us who are engaged in major rock engineering projects, it is actually hard to believe that the *advantages of underground siting in rock* have not yet been used to any extent for nuclear power plants. Cooling water is readily accessed via inflow-and-outflow tunnels, so plant-access tunnel length is hardly an issue. Security against natural disasters such as tsunami and earthquakes (and against 'unthinkable' terrorist attacks using hijacked aeroplanes) is eliminated with sufficiently deep siting. The effects of earthquakes are significantly reduced underground, due principally to the high modulus of deformation of rock at depth, and the removal of the risk of liquefaction. We are well below the zone of dynamic wave-amplitude enhancement typical in the near-surface.

PHYSICAL MODELS, NUMERICAL MODELS, LARGE CAVERNS

Back in the late nineteen seventies, physical models were utilized in joint Norwegian (Statkraft) and Swedish (BeFo) studies of large caverns, using tens of thousands of tension-fracture developed 'rock blocks' in which model 'rock caverns' were excavated at different simulated depths (mostly 50 to 100m) using different joint or fracture patterns. Rock stress was varied in the horizontal direction, while vertical stress was generated by gravity loading. The focus at this time was 50m span *'reactor caverns'*. Norway eventually constructed an Olympic ice-hockey cavern (for the 1994 games).

The illustrations assembled in Figures 1 and 2 summarize the progression: from physical modelling research, to construction, with pre-construction design checks using numerical models shown in Figures 3 and 4. Politicians had rejected the need for nuclear power and a UNPP in Norway, so this was the opportunity to actually exceed 50m span, and nearly double prior (pre-1990's) cavern spans. More recently (in China) we have seen cavern spans for power houses reaching 34m, and much larger vertical dimensions. A striking example is shown later which actually exceeds the record nuclear power production (approx. 8,000MW) by a factor of two: namely 16,000MW at the Baihetan (twin-) hydroelectric power plants.

Results from the early physical models, showing the effect of simulated joint orientations, anisotropy, and horizontal stress magnitudes were given in Barton and Hansteen, 1979. The test frame and a twin-cavern model are illustrated in Figure 1a

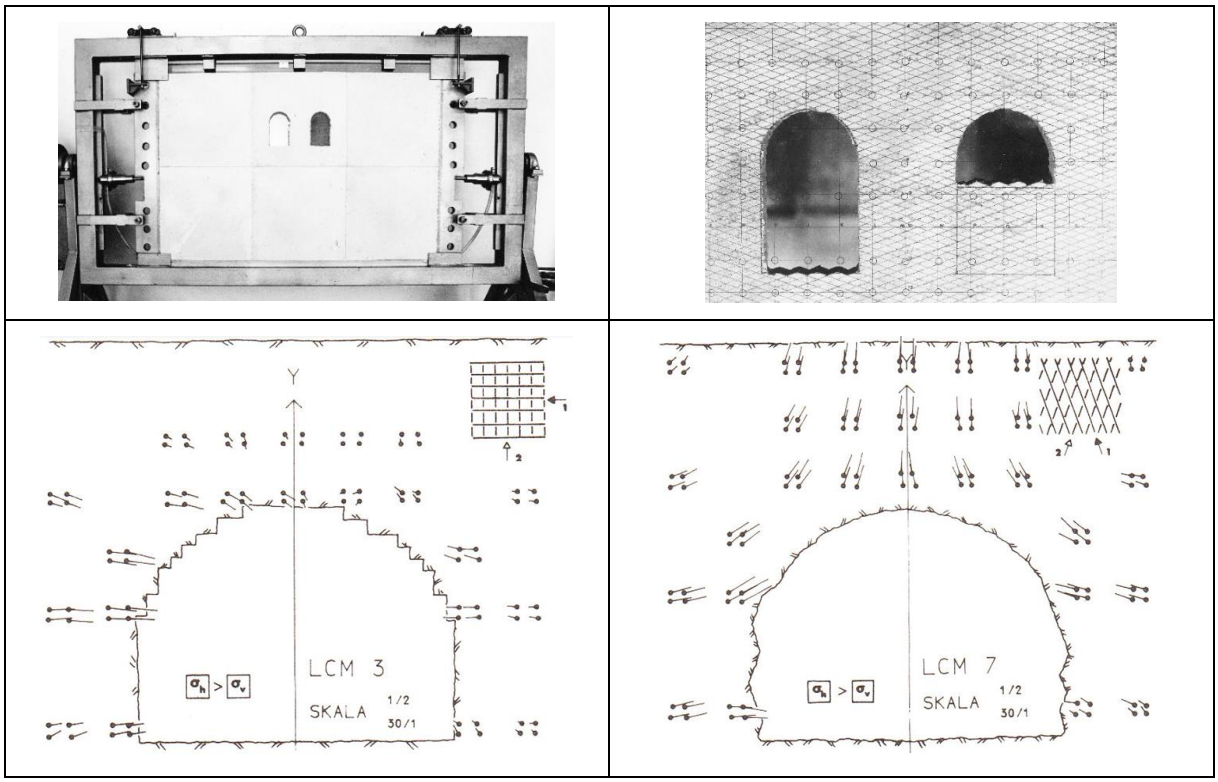


Figure 1. a, b, c, d. Physical models of large caverns (1976-1977) prior to Cundall's UDEC model. Photogrammetry was utilized for measurement of deformation. Two examples of single '50m' span caverns are shown. These had variable 'joint' anisotropy and orientation due to the tension fractures created by double-bladed guillotine. Variable horizontal stress could be applied: isotropic or $\sigma_h > \sigma_v$. as here. Barton and Hansteen,1979. (Numbers 1 and 2 next to small arrows depict the primary continuous and secondary 'joint' sets. The latter has steps therefore exhibiting real cohesion).

and 1b. Deformations were recorded using photogrammetry, and two model cavern examples are given in Figure 1c and 1d. High horizontal stress was demonstrated as a positive component of stability, and some 'joint' orientations were seen to be more favourable than others due to joint interlock effects. Horizontal 'bedding' was clearly unfavourable. Comparison with continuum (unjointed) FEM models were also made at this time. Differences could be considerable, depending on fracture orientation.

Subsequently, from the mid 1980's, we could use sophisticated numerical models with the capability of modelling the effects of *variable* joint set spacings and variable shear strengths, thanks to the remarkable developments of Peter Cundall with UDEC in 1980. Mark Christianson of Itasca was subsequently responsible for UDEC-BB in 1985.

This model incorporates the non-linear, scale-dependent Barton-Bandis joint model. The different joint sets to be represented (if possible in two-dimensional models, if not possible then in a simpler manner in three-dimensional 3DEC models) are represented by JRC and JCS roughness and wall-strength parameters and estimates of φ_r , based on Schmidt hammer tests using a simple empirical equation. The effective block size L_n for each joint set determines the scaling of shear strength, dilation and shear stiffness. (Barton and Choubey, 1977, Barton and Bandis, 2017).

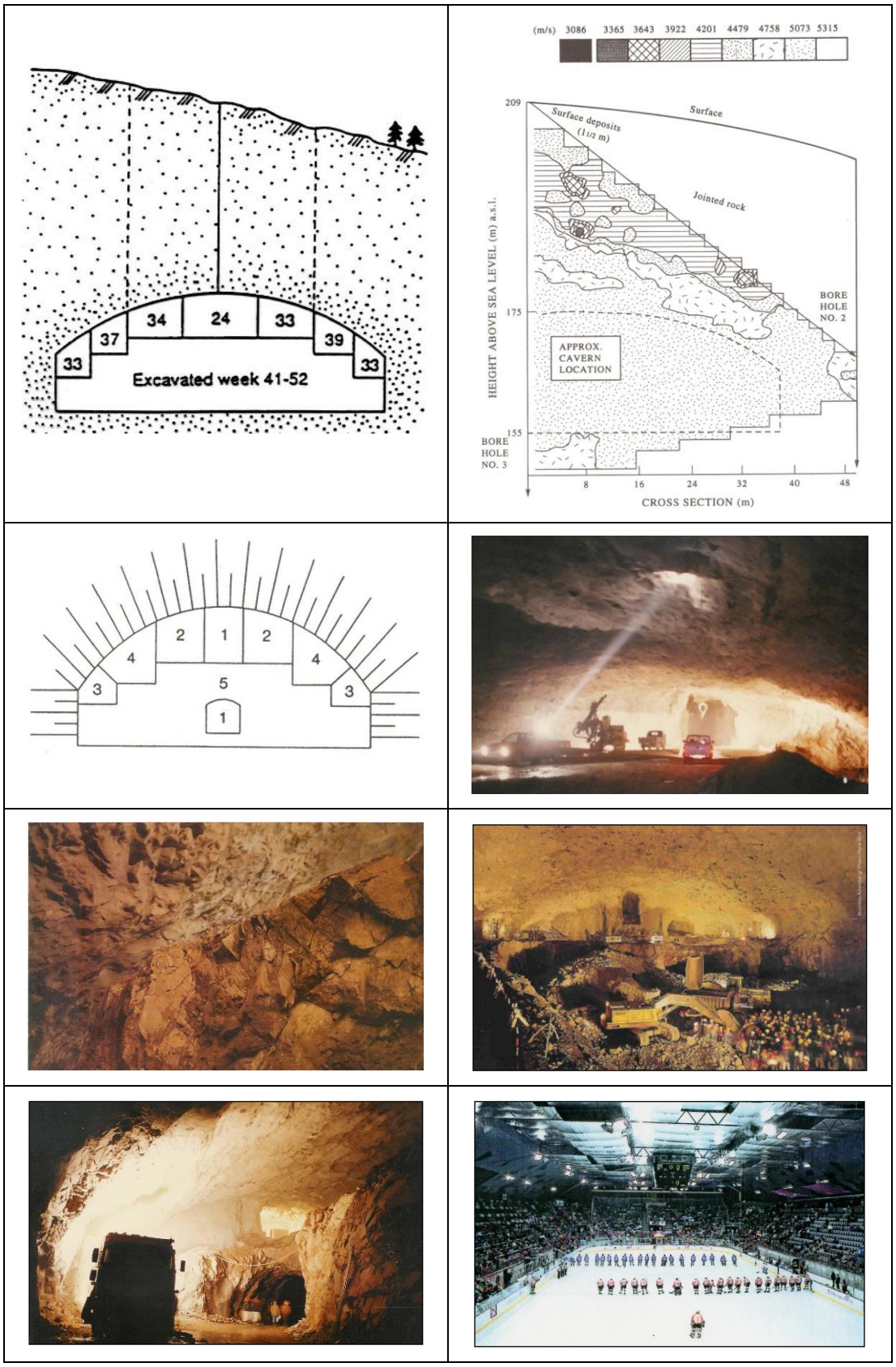


Figure 2. a. Excavation sequence (week numbers) for the 62m span Gjøvik cavern. b. An example of cross-hole Vp tomography. (A longitudinal section showing velocity is given later). c. Rock bolt and anchor support. d. Photograph of the wide top-heading. e. Detail of 10cm shotcrete in arch, in moderate quality rock with up to 1m of over-break. f. Contractor: Selmer-Veidekke construction photo. g. robotic S(fr) on spring-line. h. 1st round Olympic ice hockey match, 1994.

This unique cavern was built in jointed gneiss of medium to good quality, with favourable horizontal stress. Some details of construction are given in Figure 2. Details of UDEC-BB modelling and comparison with monitoring results are given in Figures 3 and 4. The 7 to 8 mm deformation was very close to the numerical model prediction. Technical details are given in a multi-author multi-participant NGI paper from 1994: Barton, N., By, T.L., Chryssanthakis, P., Tunbridge, L., Kristiansen, J., Løset, F., Bhasin, R.K., Westerdahl, H. & Vik, G. 1994. *Predicted and measured performance of the 62m span Norwegian Olympic Ice Hockey Cavern at Gjøvik*. Int. J. Rock Mech, Min. Sci. & Geomech. Abstr. 31:6: 617-641. Pergamon.

Up to 5,400 spectators have sat in this cavern on numerous occasions to listen to concerts. The acoustics are excellent. Smaller numbers of spectators have watched numerous sporting events, usually ice-hockey, during the last 25 years. The permanent rock support for this large cavern, situated at 30 to 50m depth, is corrosion-protected 6m long steel rock bolts at 2.5m spacing, and just 10cm of steel-fiber reinforced shotcrete. In case of deep rock wedges, twin-strand cables of 12m length were used as well as the bolting. There was no concrete lining.

ROCK CAVERN STABILITY PRINCIPLES

The over-riding principal with cavern construction in jointed rock is that we rely on the quality of the rock mass. This bears most of the load when rock quality is *fair to good to very good.* (Numbers: 'Q-values' will be given in place of these adjectives later). We do not rely on any concrete lining 'to bear load'. Nevertheless, nuclear engineers will probably insist on additional reinforced concrete in the domed arches of their reactor caverns. This of course is a cost-driver. It is not used in conventional rock cavern construction, even in huge hydropower projects, because it is unnecessary.

The main load-bearing structure is the tens of meters thick rock arch surrounding any cavern construction. We generally choose to avoid cavern construction in weak or faulted rock, in which thick reinforced concrete *might actually be needed*. These conditions are avoided because cavern construction would be several times more expensive, taking several more years. There are usually alternative sites, perhaps deeper locations, for ensuring that 'single-shell' rock mass support and reinforcement is sufficient. We symbolise this by 'B+S(fr)' meaning systematically spaced rock bolts and steel-fiber reinforced shotcrete. If we need 'double-shell' construction, we can write this in short-hand as B+S(fr)+CCA – the latter referring to *cast concrete arches*.

ROCK MASS SUITABILITY FOR LARGE CAVERNS USING Q-VALUE

For describing the rock mass quality with something more than adjectives (medium, good, very good) we can us the Q-system or RMR (rock mass rating). These were independently developed rock mass classification methods from 1973 (Bieniawski) and 1974 (Barton et al.) They are now widely used in civil and mining engineering in many countries. A cruder sketch-recognition method favoured by some is GSI (geological strength index). The objective is to describe rock mass quality for comparison of sites, and to derive input data for tunnel and cavern reinforcement decisions, or input for numerical modelling, or all the above. GSI is assumed to provide input data for simplified numerical modelling, but is not linked to rock support.

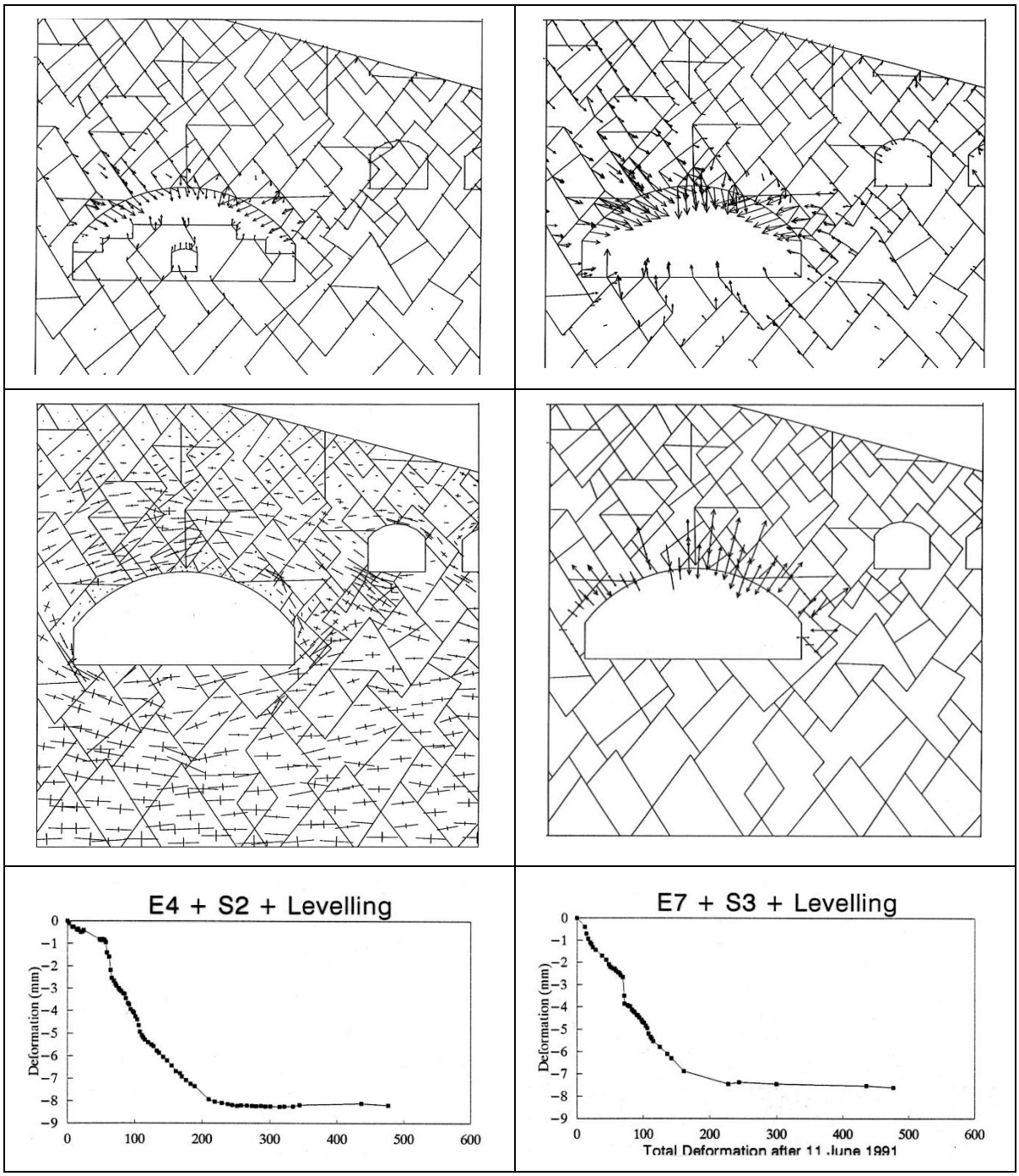


Figure 3. a, b: Some results of UDEC-BB modelling of the planned excavation: maximum deformations were 6 to 8mm. c, d: Stress distribution and bolt-and-anchor forces at joint crossings. e, f: Monitoring of cavern arch deformations, using surface-installed and top-heading-installed MPBX extensometers, and surface monitoring. The 7 to 8 mm deformation was very close to the numerical model prediction.

Figure 5 illustrates where the Q-system is used, for the benefit of civil engineers who may not be familiar with rock engineering, but who have general interest in ‘use of the underground for possible UNPP siting. The rock mass quality Q-value is composed of six parameters, which can be combined as ‘block size’ x ‘inter-block friction’ x ‘active stress’, based on empirically derived ratings. The resulting six orders of magnitude scale of Q is illustrated by two extremes in Figure 6, with ratings for individual parameters shown for each case: Sugar Loaf in Rio Q>1000, and a fault zone.

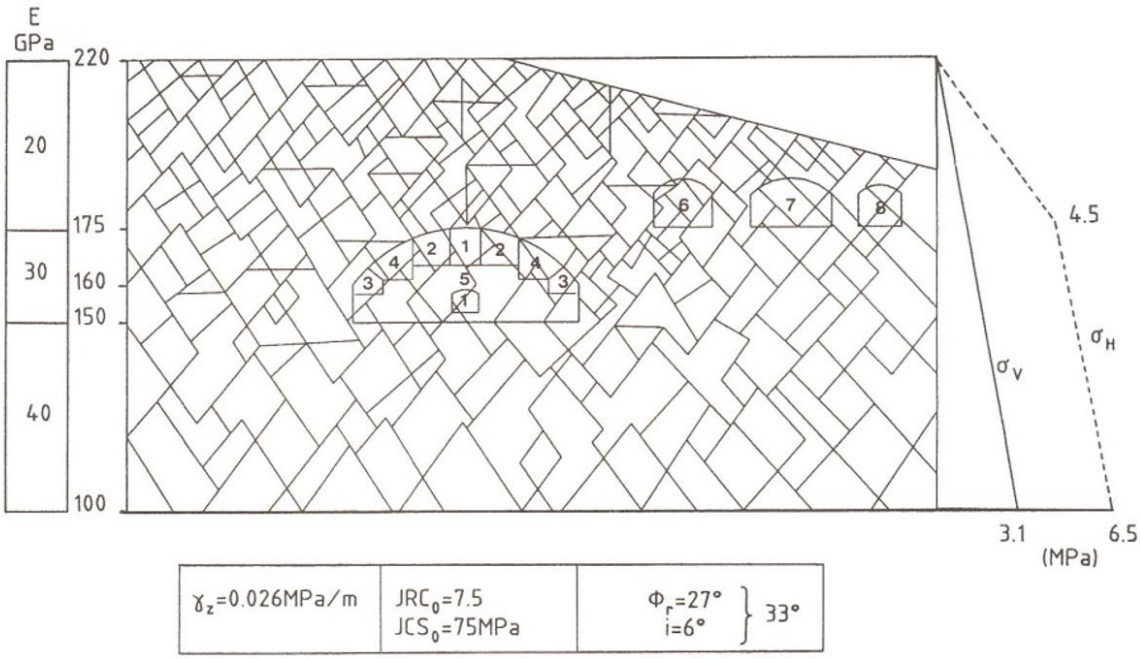


Figure 4. The UDEC-BB model geometry showing the stages of numerical excavation (1 to 8) which included three additional tunnels/caverns to one side of the 60m span main cavern. These caused some small modifications to the stress 'arch' shown in Figure 3. Numbers down the left side show height above sea level and three depth-dependent moduli of deformation (E in GPa). Below to the right side are vertical stress (gravity-based) σ_V and estimated horizontal stress (σ_H). The input data for the numerical modelling of the joint strength and stiffness using joint roughness and wall strength is shown in the boxes. A 'Patton i-value' was used to better represent the roughness of the undulating joints.

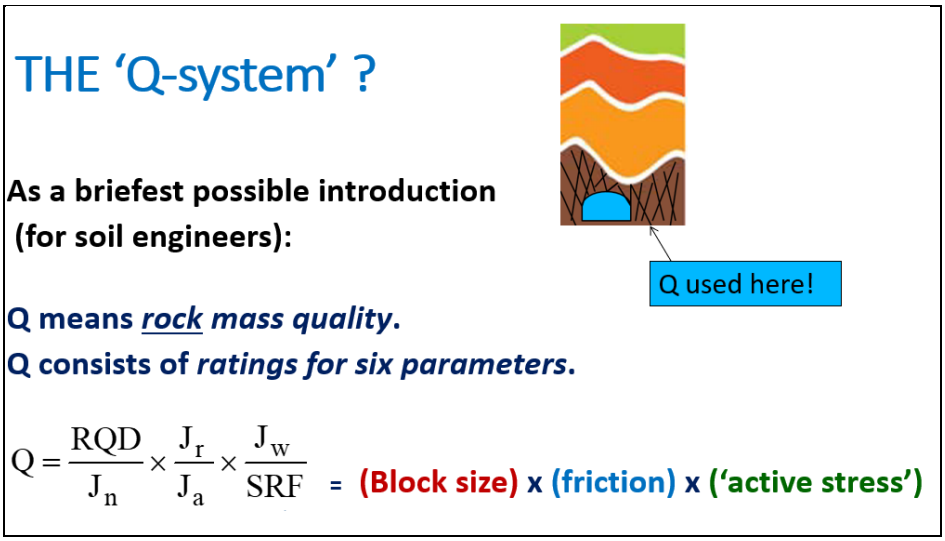


Figure 5. Basic elements of the Q-system. Six parameters are used in civil engineering applications for tunnel and cavern design and for construction follow-up. The first four parameters, which basically describe relative block-size and inter-block friction are widely used in the mining industry for the dimensioning of large stopes, and for stability prediction.

Application of the first four of these Q-parameters is demonstrated in Figure 7, with six-parameter Q-values for the Gjøvik cavern arch given in a 'fold-out' plan view in Figure 8. Extensometer locations (E1-7: external, S1-3: internal) are also shown.

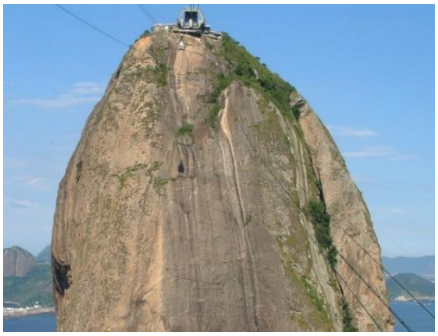
	BRAZILIAN HYDROPOWER PROJECT COLLAPSE IN FAULTED ZONE *LOWEST END OF THE ROCK MASS QUALITY SCALE. (Inadequate support).* Q ≈ 10/20 x 1/8 x 0.5/20 i.e. **< 0.001**
	SUGAR LOAF MOUNTAIN, RIO DE JANEIRO *TOP END OF ROCK MASS QUALITY SCALE. (No support would be needed).* Q ≈ 100/0.5 x 4/0.75 x 1/1 i.e. **>1000**

Figure 6. Examples of the 'end-members' of rock mass quality according to Q. The six orders of magnitude range correlates in a simple way with P-wave velocity, deformation modulus, permeability (the latter needing a modification for clay-filled joints, which reduce quality and permeability).

Q-values have several potential roles in tunnel and cavern engineering. We can log the quality of surface exposures such as rock cuttings, and log the quality of exploratory drill-core, thereby assisting in the decisions about cavern layouts in term of depth, such as depth into a hillside next to a cooling water source, such as the sea, a lake or multi-lagoon reservoir, as used at the Trawsfynydd NPP in N. Wales.

As shown in Figures 7 and 8, Q-values are used for logging during construction, so that suitable permanent support can be chosen for the arch (domed-roof, as relevant) and walls of a large cavern, and associated tunnels. Figure 9 shows correlation with P-wave velocity and with deformation modulus, both of which are useful during site investigation, for deciding on cavern location (Figure 8), and for numerical modelling (e.g. Figures 3 and 4). The details of shotcrete thickness decisions and rock bolt spacing decisions can be decided by reference to Figure 10.

The record 62m span of the Gjøvik cavern is coincident with the upper boundary of the tunnel support 'Q-chart' since at present we do not have larger man made caverns in terms of span width, and the Q-support diagram is specifically an *empirical* (*'a posteriori'*) method. It is not an '*a priori'* method as with some commercial numerical modelling routines that are based on opaque, non-empirical equations. In Figure 11a, some of the numerous deformation measurements recorded by ten multiple-position extensometers (MPBX) are plotted, using log-log scales of Q/Span versus deformation. Based on additional data from Taiwan tunnels (Figure 11b) a very simple central trend for the data was discovered: Δ(mm) = SPAN(m)/Q.

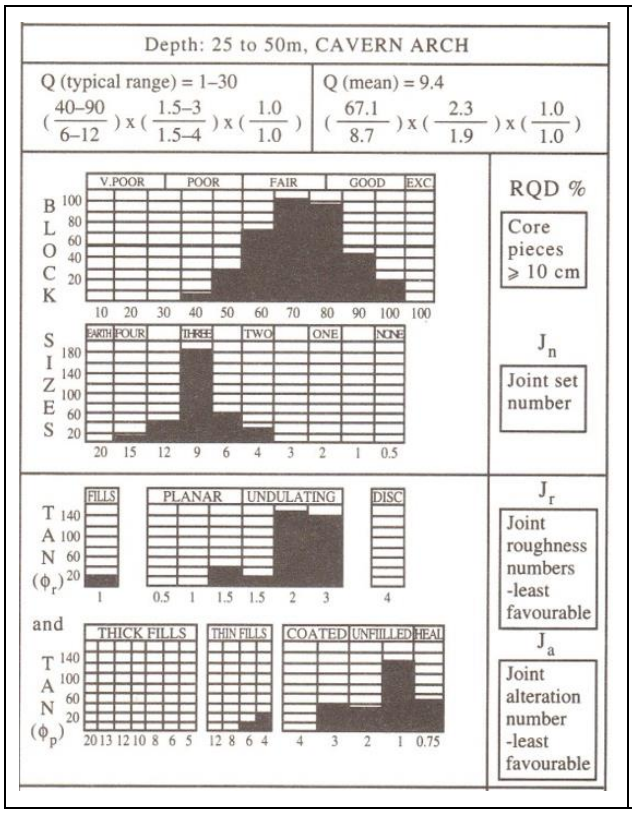


Figure 7. The rock mass quality statistics for the arch of the Gjøvik cavern are shown by means of the first four Q-parameters. Jw for water and SRF for the stress/strength conditions were each logged as 1.0 as there were dry conditions and stresses were moderate in relation to the approx. 90MPa compression strength of the grey and red jointed gneiss.

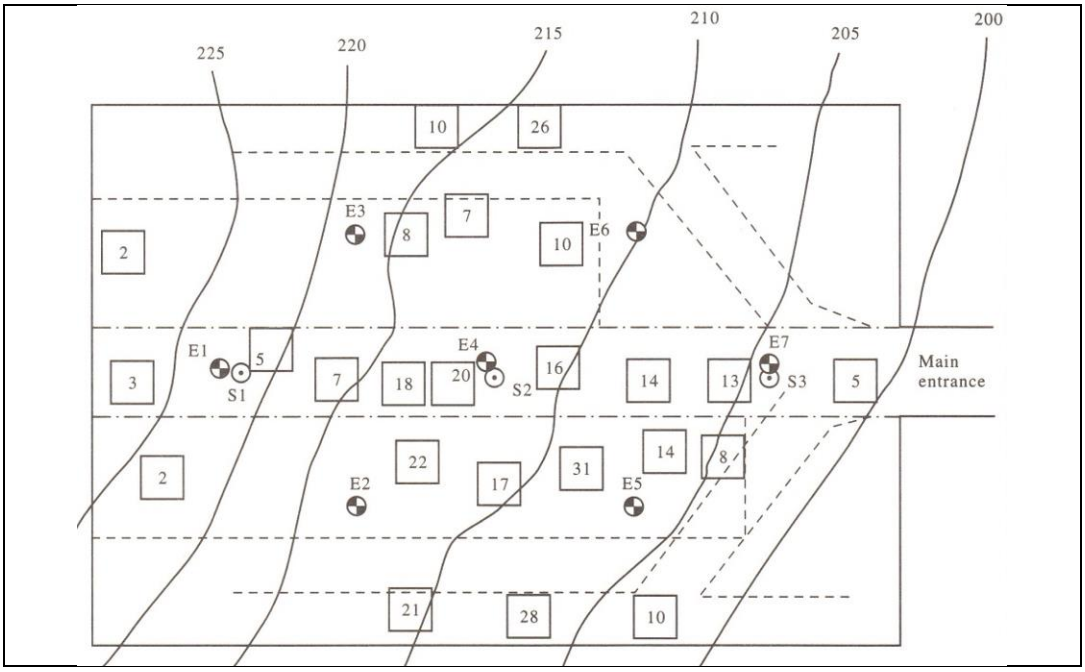


Figure 8. The values given in the squares/boxes, give the estimated Q-values (mostly 'poor' to 'good', or approx. 2 to 30) for the Gjøvik cavern arch. These are quite similar to the range of Q-values that were independently logged using drill-core from four boreholes, and Q-values logged in existing caverns at the same site. The three engineering geologists had similar opinions about the rock quality, but made their independent assessments during a time-span of several years, before and after 1992 (i.e. pre-project, site-investigation-boreholes, during construction). The range of Q-values suggests the blue-region in the V_P – Q_c – depth plot, Figure 9.

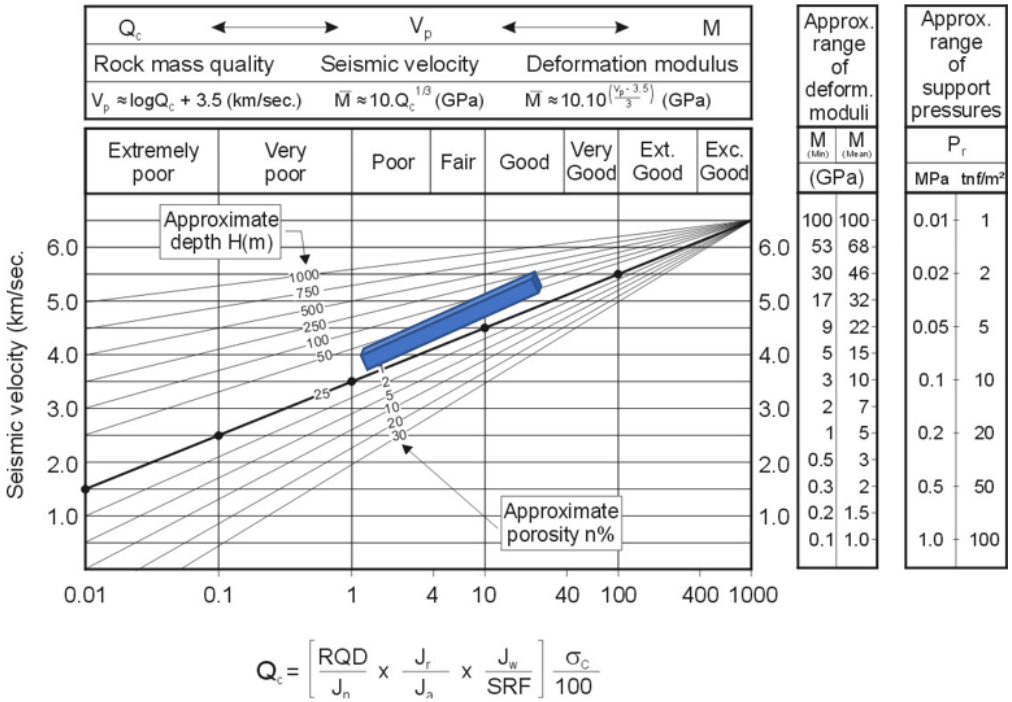


Figure 9. Empirical correlations of Q-values (modified by normalized rock compressive strength $\sigma_c/100$), seismic P-wave velocity, deformation modulus, and support pressure needs for bolting or anchoring design. The Gjøvik cavern lies roughly in the blue area (Q = 2 to 30, V_p = 3.5 to 5.5km/s, M = rock mass static deformation modulus = 15-45GPa). Both velocities and deformation moduli are depth-dependent, a 'detail' ignored by many modellers and rock mechanics software sales companies.

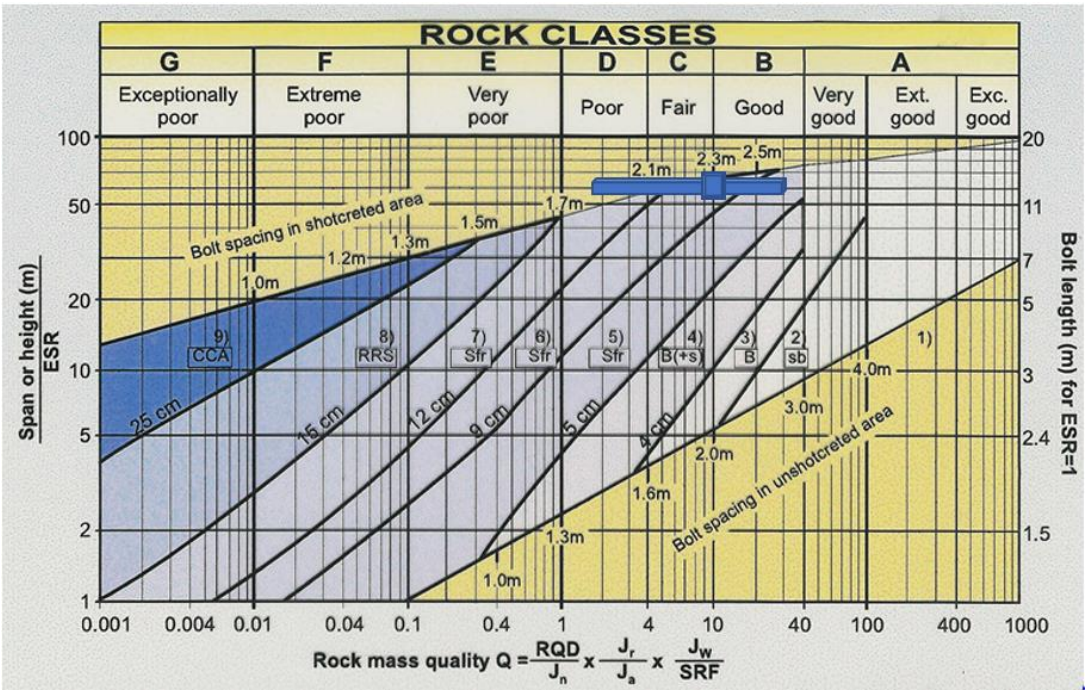


Figure 10. This figure, published in 1993 by Grimstad and Barton, and therefore relevant to a cavern design of the early nineties, shows shotcrete thickness and bolt spacing as a function of a tunnel or cavern span (expressed in meters) and the rock mass quality Q-value. The range of Q-values at the Gjøvik cavern are as indicated: 2 to 30. Mean Q approx. 10.

The 62m Gjøvik cavern span, with assumed ESR = 1 (quality-of-support number) is indicated by the blue ‘box’. It shows the approximate range of Q-values (2 to 30) with a mean of 9 to 12 (see ‘raised’ central ‘box’). The latter is based on the results of core-logging and arch-logging. The permanent support for this cavern arch, excavated in moderate quality grey and pink gneiss, was 10cm of S(fr) and 2.5m c/c bolting of 6m length. The additional 12m c/c anchors shown in Figure 2c were for temporary support during construction (in case large rock wedges were exposed in the arch).

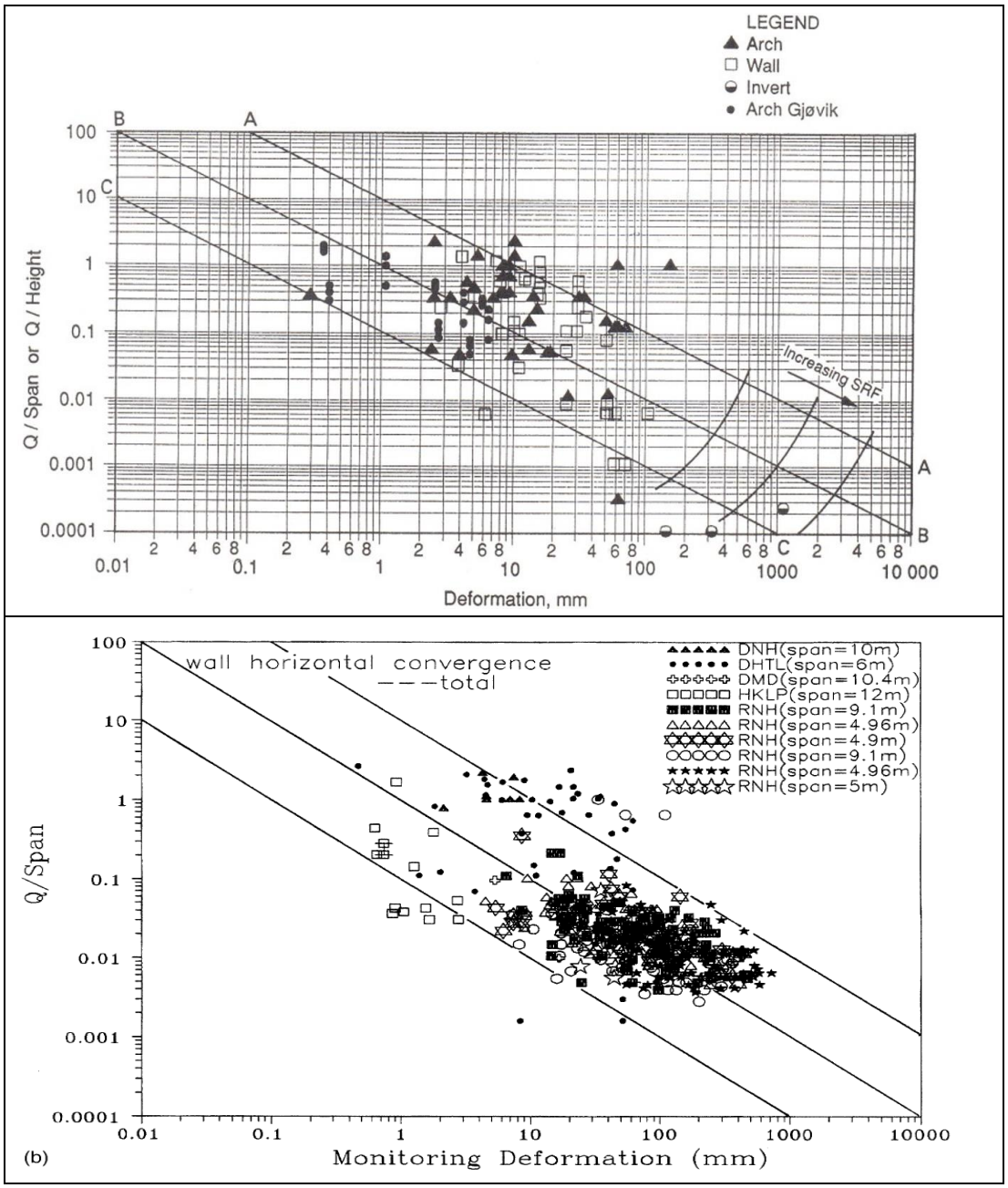


Figure 11. a. Gjøvik cavern deformation data plotted as a function of Q/Span (from Barton et al. 1994). b. The same plotting format was used by Chen and Guo in Taiwan (priv. comm.) for hundreds of tunnels in Taiwan. This prompted the writer to derive the very simple formula for the central trend: Δ(mm) = SPAN(m)/Q. Gjøvik cavern: 6mm = 60/10. (See Table 1).

Greater accuracy in deformation estimation compared to the simple central trend (Figure 11 caption) is obtained by using three empirically improved formulations, as shown in Table1. Two examples of application are given. It has been found that the use of these equations is advised when checking numerical modelling. For instance, there have been occasions when numerical modellers have made joint continuity too pronounced in UDEC-MC and UDEC-BB modelling (MC means simplified linear Mohr-Coulomb based representation of joint behavior). Subsequent monitoring and Q-logging of the finally constructed cavern(s) indicated the value of these equations, in relation to the exaggerated modelled deformations. Joint continuity was concluded to have been exaggerated in the modelling.

Table 1. Improved empirical equations for estimating the likely magnitudes of tunnel or cavern deformation. (Barton, 2002). When using the last three equations, SPAN is expressed in mm, deformations are in mm, and stress and strength are in e.g. MPa.

$\Delta = \frac{SPAN}{Q}$	$\Delta_v = \frac{SPAN}{100Q}\sqrt{\frac{\sigma_v}{\sigma_c}}$	$\Delta_h = \frac{HEIGHT}{100Q}\sqrt{\frac{\sigma_h}{\sigma_c}}$	$k_o = \left(\frac{SPAN}{HEIGHT}\right)^2\left(\frac{\Delta_h}{\Delta_v}\right)^2$
Nathpa Jakri power station cavern		**Gjøvik Olympic cavern**	
$\Delta_v = \frac{20{,}000}{100 \times 3} \times (6/35)^{1/2} = 28$ mm $\Delta_h = \frac{50{,}000}{100 \times 3} \times (4/35)^{1/2} = 56$ mm (SPAN = 20m, HEIGHT = 50m, Q = 3, σ_V=4 MPa, σ_h = 6 MPa, σ_C = 35 MPa). (In the middle of the range of MPBX measurements for the arch and walls).		$\Delta_v = \frac{60{,}000}{100 \times 10} \times (1/75)^{1/2} = 6.9$ mm (SPAN = 60m, Q_{mean} = 10, σ_V = 1 MPa at 40 m depth, σ_C = 75 MPa) (Almost identical to that measured with nine MPBX, and almost identical to UDEC-BB modelling results).	

Figure 12 illustrates the pre-benching upper-walls and arch of the Gjøvik cavern. The robotically applied shotcrete rig (AMV type with elevated operator) is approached from 90m away, 30m away, and finally shows the boom and operator cabin close to the nozzle, as is desirable. In the left column we see another cross-hole seismic tomography result, and a plot of RQD and joint frequency down one of the 60m deep boreholes. There were just four pre-investigation boreholes.

Despite the ‘stable’ values of rock quality (RQD and joints/meter, and also Q) with increasing depth, the P-wave velocity is seen to increase strongly with depth (as also the related deformation modulus), due to the satisfactory horizontal stress development with depth, as seen in Figure 4 (refer to σ_H on the right-hand side of the model). Joints are partially closed with increased depth (and therefore increased horizontal stress), and this can be expected to cause a general increase in velocity and deformation modulus, and a reduction in permeability (as at the Brenntangen ‘nuclear’ site described briefly in the Introduction).

USE OF ROCK FOR LARGE HYDROPOWER CAVERNS

This introductory text on the use of rock for underground siting of important facilities, including the present focus on post-Fukushima nuclear power plants, would be

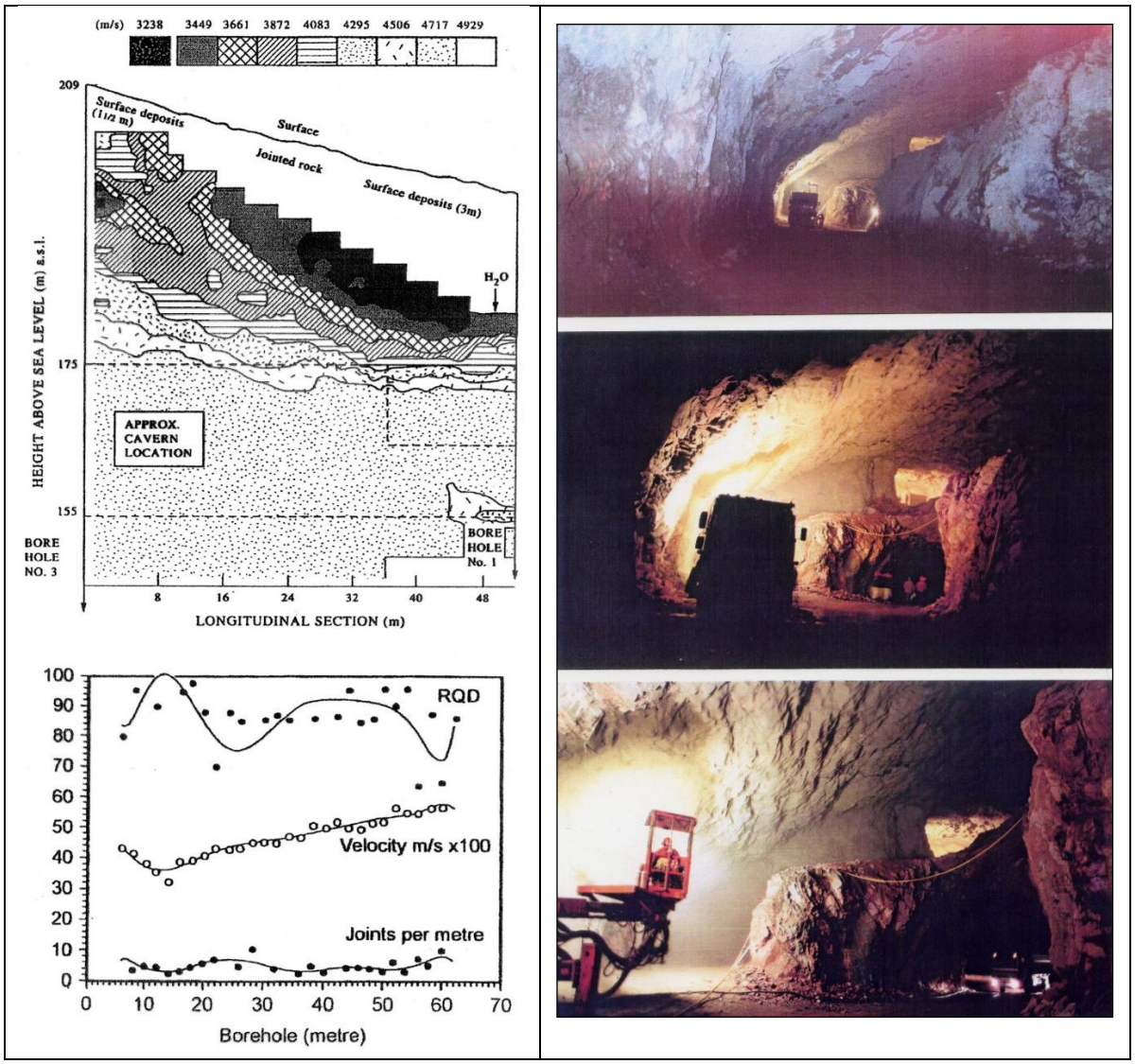


Figure 12. Left: cross-hole seismic tomography showing the reduced velocity (and rock quality) in the 10 to 30m nearest to the surface. This shallow cavern location was slightly adjusted (longitudinally) as a result of the P-wave velocity distribution. At this cavern site there were limited rock quality component improvements at greater depth. The reader is referred (bottom-left diagram) to the trends of RQD (% of core-pieces of >10cm length), joints per meter, Q-values (ranging from 2 to 31), which did not increase at greater depth in this case. Right: views along the 90m long cavern, finally reaching the shotcreting robot, with the operator in a half-way-up-the-boom cabin so as to be close to the shotcreting work. This is a quality-improving feature seen in the last 30 years in AMV robots in Norway.

largely incomplete if it did not include past and recent hydropower caverns. These house expensive machinery, in some cases with greater generating power than even the world's largest (surface) nuclear power plants. For example: the Kashiwazaki-Kariwa plant in Japan is currently the world's largest nuclear power plant, with a net capacity of 7,965MW. Nevertheless, it is dwarfed by the world's largest underground hydroelectric (twin) power plant: Baihetan in China can generate 16,000MW, using 8 units of 1,000MW in twin powerhouses under opposite banks of the powerful river,

Figure 13. Internet pictures of the Baihetan hydroelectric project, China, showing the twin 2 x 8,000MW layout and a photograph from inside one of the caverns (with a height of > 85m). Note the shotcrete lining and protected anchor and rock bolt heads, of variable spacing. The power caverns are coloured yellow in the '3D' exposed view.

Figure 14. Right-bank of the Baihetan project prior to large-scale construction. The cross-jointed and actually 'macro-cracked' columnar basalt had a tendency to degrade rapidly when unloaded. Immediate B+S(fr) was the solution, but this was delayed in some major excavations, and resulted in reported problems with over-break and rock failure. Due to the high rock stresses under the canyon walls, the stress/strength ratio (σ_θ / σ_c) was clearly unfavourable, and elevated SRF values would have been recommended if using the Q-system for tunnel and cavern support selection. Nevertheless, cavern wall heights greatly exceeded empirical data, thus requiring careful numerical modelling.

which runs between canyon walls. The rock for both caverns (the inside of one is shown in Figure 13) is columnar basalt, which proved to be a challenge due to a tendency to loosen if not supported and reinforced in a timely manner. The rock exposed along the canyon walls is shown in Figure 14. The walls of these foot-paths disintegrated rapidly since without S(fr) protection, as only temporary paths.

Conclusions

1. From the point of view of rock engineering experiences from around the world, it is difficult to imagine that there could be insurmountable underground construction challenges for the siting of *nuclear power plants in rock caverns*. The heavy underground construction industry, and of course numerous reputable and highly experienced consulting companies have designed and built complex projects in rock thousands of times. Foremost in complexity are probably the 800 or more underground hydroelectric stations, which often require three parallel caverns of large volume. The machine halls housing a typical line of multiple generators have reached several hundreds of meters length, spans of more than 30m and heights in excess of 80m.

2. In the case of the Baihetan project in China, built across a fast-flowing tributary of the Yangtsi River, there are twin power plants beneath each valley (canyon) side, each with the generating capacity equal to the largest nuclear power plant in the world (TEPCO's Kashiwazaki-Kariwa plant in Japan with a net capacity of 7,965MW). Baihetan generates 2 x 8,000MW, using 16 x 1,000MW turbines. The two powerhouse caverns each measure 438m length x34m span x89m height. These are much larger in volume than will be needed for nuclear power, and they were built in rock masses of somewhat unfavourable character (columnar basalts).

3. A practical glimpse of the design checks, modelling, monitoring and construction of a cavern of even larger span (62m) but of limited height and length is given as an illustration of some of the practical issues that need to be addressed. Before the Norwegian Olympic cavern project (from the early nineties), a project for potential underground siting of a nuclear power plant was initiated in Norway in the early 1970's. In fact, for the last 60 years Norway had two small underground plants: Kjeller and Halden, which have now been decommissioned: in 2018 and 2019.

4. There was initial Norwegian focus on a potential UNPP site at Brenntangen on the east side of the Oslo fjord. Various site characterization tasks were performed by NGI. Subsequently both Norway (Statkraft) and Sweden (BeFo) supported a physical model-based research project for investigating the challenges associated with 50m span reactor caverns in jointed rock. Surface construction of nuclear power plants was in the meantime initiated in Sweden, mostly in the mid-seventies and mid-eighties (Barsebäck, Ringhals, Oscarshavn, Forsmark). *No nuclear power* generation was the development option eventually chosen in Norway, due mainly to the rich sources of hydropower in this country.

5. The opportunity to build a large span cavern in rock, actually for use by the sporting (or concert-going) public did eventually arise in Norway, and it was to be ready for the 1994 Winter Olympics. The initial idea was sketched on a proverbial restaurant napkin by Mr. Jan Rygg for the benefit of a town planner, and final design was by Fortifikasjon/Noteby, with site investigation and design checking assistance by NGI. The rock engineering construction of the 62 x 25 x 90m cavern took just seven months, by the Selmer-Veidekke JV rock group. The cavern is next to an underground swimming pool, and various civil defence caverns, on the outskirts of Gjøvik, a community in southern Norway, not far from Lillehammer – the official site of the 1994 Winter Olympic games.

6. With suitable siting the civil construction costs of 10m, 20m, 30m, 50m (or even larger) excavations can be reliably estimated and their stability guaranteed by application of modern rock design and construction techniques.

7. As opposed to the typical surface plant, one sited underground is secure from physical damage caused by hurricanes, tsunamis, earthquakes, and missile attacks or aeroplane accidents.

8. Although less likely, due to the need for longer investigation times, and more stringent siting, it is entirely likely that a chosen UNPP site of major proportions could eventually be in the same general location as a deep final 'geologic' repository for HLW. Forsmark in Sweden, and Onkalo in Finland are presently linking existing surface nuclear plants with deep HLW repositories. The granitic rock masses at both sites have quite high Q-values, and would be eminently suited for large-scale cavern construction (i.e. UNPP) if this was a future requirement.

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