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ROCK MECHANICS INVESTIGATIONS FOR UNLINED PRESSURE TUNNELS AND AIR CUSHION SURGE CHAMBERS

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ABSTRACT

NGI's investigations for unlined excavations designed for high internal pressures include geological mapping, rock mass classification, minifrac rock stress measurements and joint permeability measurements. In special cases numerical modelling may be performed, using the discrete element code UDEC and the Barton-Bandis joint sub-routine which allows joint conducting apertures to be tracked. Conducting apertures are affected by effective stress levels, shear, dilation and possible gouge production. Each of these effects can be measured on large diameter jointed drill core in a special biaxial deformation-flow apparatus. Practical problems caused by leakage at the transition from the unlined pressure tunnel to the steel lined section are addressed. A water curtain solution to leakages from air cushion surge chambers is described, using an operating example.

INTRODUCTION

A convenient starting point for a discussion of internal pressure effects on unlined rock excavations is the minifrac test. Four possible scenarios can be envisaged in such a test. These are illustrated in Figure 1. The four behaviour modes illustrated may occur in minifrac tests conducted from boreholes in heavily jointed rock; clearly an unfavourable starting point for such tests.

By changing the scale of the diagram, we can envisage an unlined pressure tunnel in a quite massive rock mass with two sets of widely spaced joints. Hydraulic joint jacking (A), hydraulic fracturing (B), hydraulic shearing (C), or combined modes (D) are each possible depending on the magnitude of the following factors:

1.	Internal water pressure	P_b
2.	Minimum rock stress	σ_h (total stress)
3.	Joint permeability	K_j
4.	Stress ratio	σ_H/σ_h
5.	Joint shear strength	ϕ'_{mob}
6.	Joint orientation	β^o

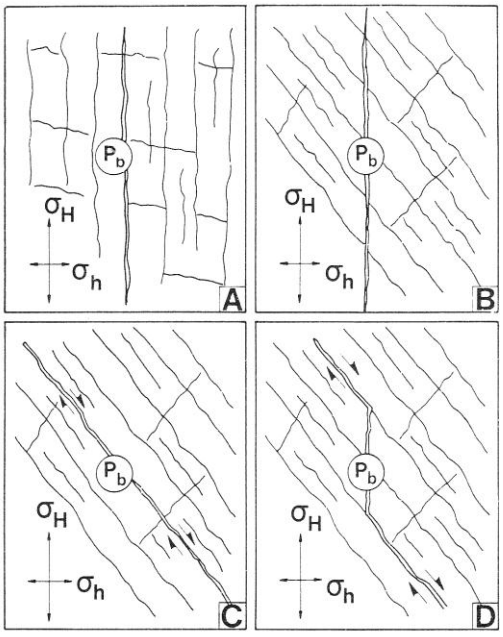


Figure 1. Hydraulic joint jacking, rock fracturing and joint shearing caused by interaction of jointing, differential stress and over-pressured excavations.

SIMPLIFIED MINIFRAC TEST

Vik and Tunbridge (1986) have described a simple, inexpensive minifrac test method used at NGI to determine the risk of hydraulic fracturing or excessive leakage from unlined pressure tunnels, or from air cushion surge chambers. The principle of the test is illustrated in Figure 2. The grouted steel tube is of sufficient length to ensure that the hydraulic fracture occurs in rock that is far enough away from the tunnel to avoid tangential stress concentrations, generally two excavation diameters. In hard rocks this is adequate; in soft heavily stressed porous rocks this may

need to be increased according to recent experimental results (Bandis et.al. 1987).

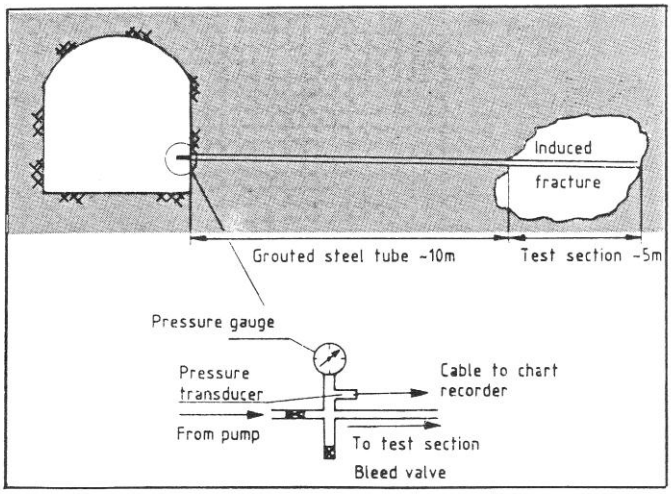


Figure 2. Simplified minifrac test set-up for evaluating risk of leakage from unlined pressure tunnels.

The results plotted in Figure 3, illustrate minimum stress magnitudes derived by NGI's minifrac tests in four Norwegian hydropower projects. The hypothetical overburden total stress line, based on a rock density of 2.6 kN/m^3, illustrates that K_o (σ_{min}/σ_v) may sometimes be as low as 0.4, but averages approximately 0.6. These Norwegian projects are generally located in granitic gneiss bedrock. The average value of ($\sigma_{max}/\sigma_{min}$) is 2.0, but much larger values have been measured (Vik and Tunbridge, 1986).

Experiences have indicated that minimum stresses measured by over-co ring methods at these same locations give consistently lower values of K_o (approx. 0.5 instead of 0.6). Since minifrac tests are of large scale and are not affected adversely by rock anistropy, it is suspected that residual stresses may prevent complete sample relaxation in the case of the overcovering tests.

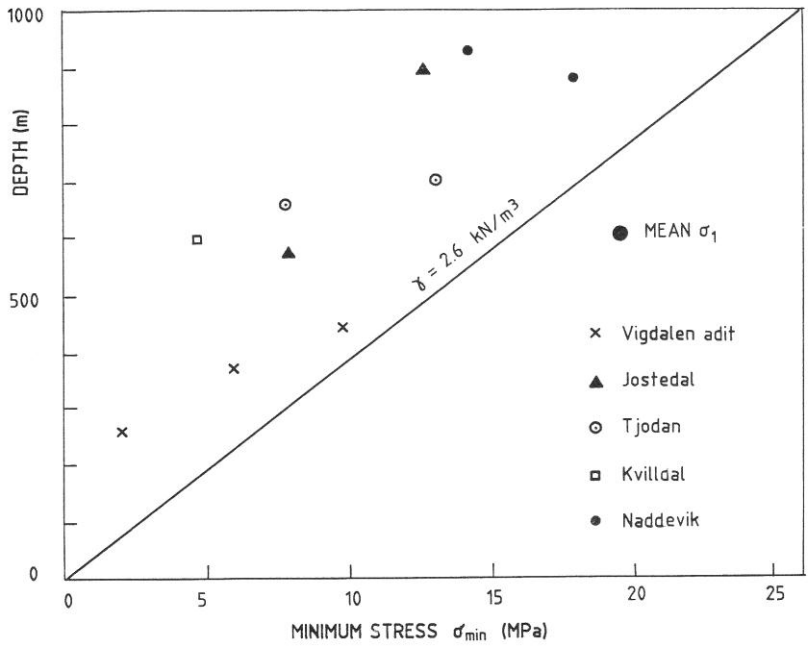


Figure 3. Results of minifrac rock stress measurements from four of NGI's hydropower consulting projects (from Vik and Tunbridge, 1986).

OVERBURDEN CRITERIA

Design criteria or numerical models that actually match minimum overburden and rock density with the maximum water pressure may be unsafe, and large leakages have in fact occurred that demonstrate that overburden considerations can be inadequate. When even these overburden criteria are ignored, leakages are almost inevitable.

Minifrac stress measurements performed in sedimentary rocks using surface boreholes for access to pressure tunnel levels also demonstrate problems with overburden design criteria. Figure 4 illustrates that the lowest value of K_o may appear in the softest interbeds (shale). (This is the opposite of experiences in deep petroleum reservoirs as discussed by Barton, 1986).

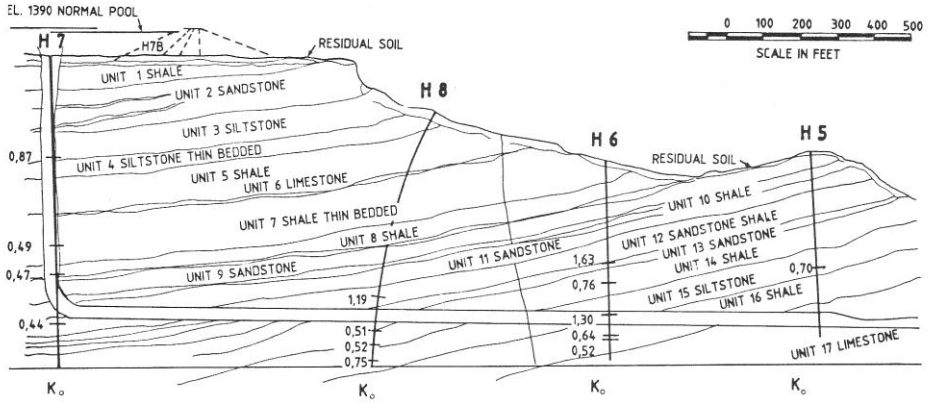


Figure 4. Measurements of K_0 using the minifrac technique in sedimentary sequences resulted in modified pressure tunnel design (Barton, 1986).

The low values of minimum rock stress in the case illustrated in Figure 4 resulted in expensive increases in steel reinforcement for the concrete lined pressure tunnel, since large scale rock deformation moduli were also low in the same shale formation. It is of interest that the mean value of K_0 for these interbedded sedimentary rocks was approx. 0.77, compared to approx. 0.58 for the Norwegian bedrock at greater depth.

ROCK MASS DEFORMATION PHENOMENON

The unlined pressure tunnels typical of Norwegian hydropower schemes have reached maximum heads in the region of 1000 m. Minimum rock stresses in excess of 10 MPa are clearly required to prevent the type of failures sketched in Figure 1 (cases A and B).

However, failure modes that are governed by shear modes C or D (Figure 1) have been registered in hydrothermal projects using deep boreholes (Pine and Cundall, 1985). A similar mechanism could develop at larger scale from a pressure tunnel if factors 3, 4, 5 and 6 listed in the introduction were unfavourable.

A water pressure less than the minimum principal stress would appear to remove the threat of hydraulic fracturing of the rock mass. However, a special leakage phenomenon has been observed in several pressure tunnels in Norway and elsewhere. The locations of these leakages are usually immediately upstream of the concrete plug separating the unlined section of pressure tunnel from the steel-lined section.

Leakages have occurred even when conventional overburden design criteria are satisfied with large factors of safety. Failure can also occur when the water pressure is less than the minimum rock stress.

It has been suggested by Andersen (1970) that increased effective normal stress causing joint closure downstream of the plug is a plausible mechanism for the development of these leakages. Figure 5 illustrates schematically the process that may be involved. Before construction of the pressure tunnel the rock joints are under a certain level of effective normal stress. Construction will cause some local groundwater drawdown, but it is reasonable to expect that the groundwater level will be re-established, perhaps exceeded, when the unlined section of the pressure tunnel is under full head.

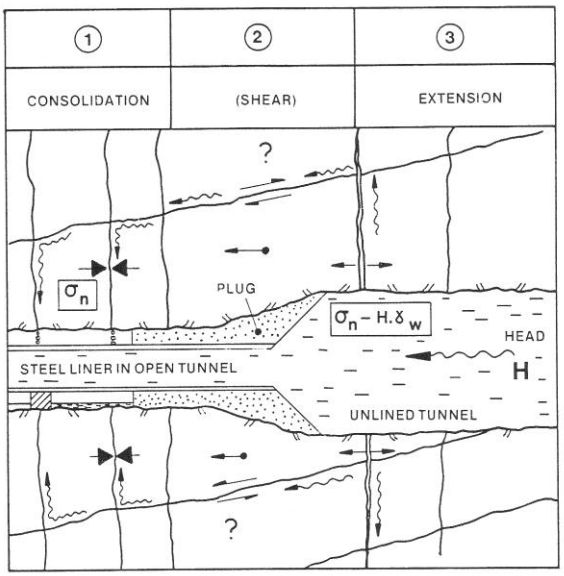


Figure 5. Potential leakage mechanism around a concrete plug separating an unlined pressure tunnel from the steel lined section.

Downstream of the plug, if the steel liner is not cast in concrete and contact grouted, there will be an open annulus around the liner that is under atmospheric pressure. Drainage towards this annulus, as illustrated in Figure 5, will result in increased effective nor-

mal stress and consolidation (joint closure) in this zone, all the way around the tunnel.

This consolidation mechanism will probably cause extension in the zone immediately upstream of the plug where the effective normal stess is already lower. Any possible shearing mechanism due to infavourable jointing might further facilitate such a transfer of strain in the rock mass. On occasions joints have opened several centimeters with leakage all the way to the valley side, in additon to leakage around the plug (Lien & Valstad, 1971). Conditions are worsened if the vertical joints depicted in Figure 5 are filled with deformable material, such as silt or sand, that can also be washed out of the joints.

NUMERICAL MODELLING FOR THE PRESSURE TUNNEL PROBLEM

Pressure tunnels for conventional and pumped hydro projects are frequently concrete lined. Efforts are usually made to estimate or measure the deformation modulus of the surrounding rock so that load sharing can be designed. However, it is not infrequently that heavy steel reinforcement needs to be used to assist the concrete-rock interaction in resisting the water load. This measure is taken when the rock is of low modulus, and also when the depth to overburden is inadequate. The steel reinforcement is designed to distribute any cracking that might occur around the periphery, so that cracks are very fine and result in marked head losses.

A special case of the above design problem is encountered when a membrane is designed to seal potential cracks in the concrete. The design problem is then to predict the rock mass deformation and the degree of concrete cracking that occurs. Can the membrane stretch over a crack of x mm in width?

A special modelling effort to tackle a related problem for the oil industry was recently performed by NGI (Makurat and Barton 1986). The UDEC discrete element code developed by Cundall (1980) was used with a new joint sub-routine based on the Barton-Bandis joint model (Christianson 1986). This describes joint behaviour by means of the two readily measured index parameters:

JRC joint roughness coefficient
JCS joint wall compression strength

Both joint closure and shear-dilation coupling are modelled, and a constant track of joint conducting aperture (e) is kept, by continuous tracking of the physical joint aperture (E). The latter is affected by shear, dilation and closure. The non-linear behaviour illustrated in Figure 6 is fully modelled by this technique, with the additional sophistication that joint apertures are also tracked.

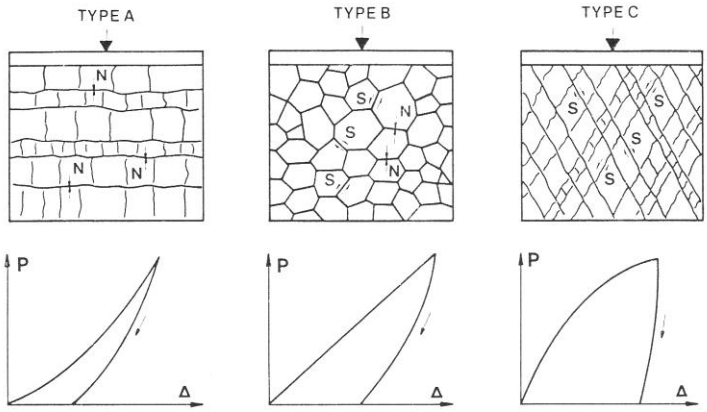
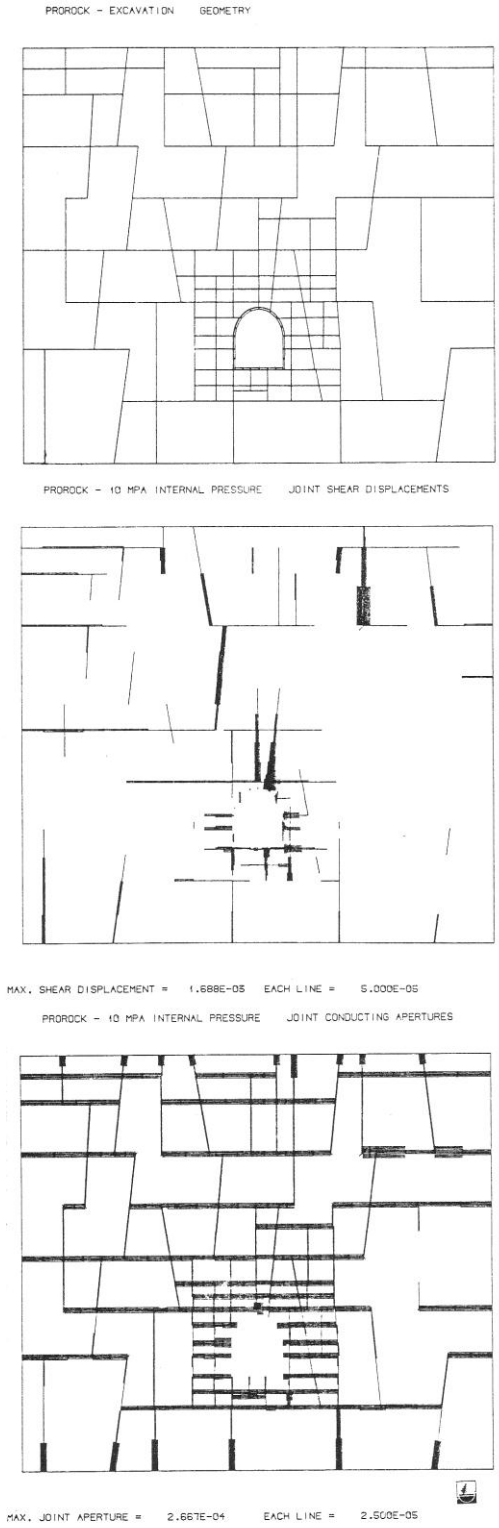


Figure 6. Non-linear rock mass deformation phenomena due to joint closure and shear components. After Barton, 1986.

Figure 7 illustrates the numerical modelling capabilities with this new version of UDEC. The upper diagram shows the lined tunnel in jointed rock. The middle diagram illustrates which joints are sheared (max 1.7 mm) as a result of the internal pressure of 10 MPa. The lower diagram shows the distribution of joint apertures (max 0.267 mm).

Input data for the joint sub-routine that tracks joint apertures can be obtained from simple index testing of jointed drill core. (Barton et.al. 1985). Actual physical evidence of closure-conductivity and shear-dilation-conductivity coupling can also be obtained from recently developed NGI testing equipment, as illustrated in Figure 8.



It will be noted in Figure 7 that the joints that show most shear (thickest lines; 1.69 mm) mostly do not show corresponding increases in conducting apertures. This is because the widely spaced jointing defines such large block sizes that significant dilation has not yet occurred. Small blocks dilate after smaller shear displacements as shown by numerous tests conducted by Makurat (1985) and co-workers in the apparatus depicted in Figure 8. Increases in conductivity of two to three orders of magnitude are possible in the first 1 to 2 mm of shear.

Figure 7.

An example of the potential for discontinuum modelling of the pressure tunnel problem, assuming a membrane lined tunnel. Makurat and Barton, 1986.

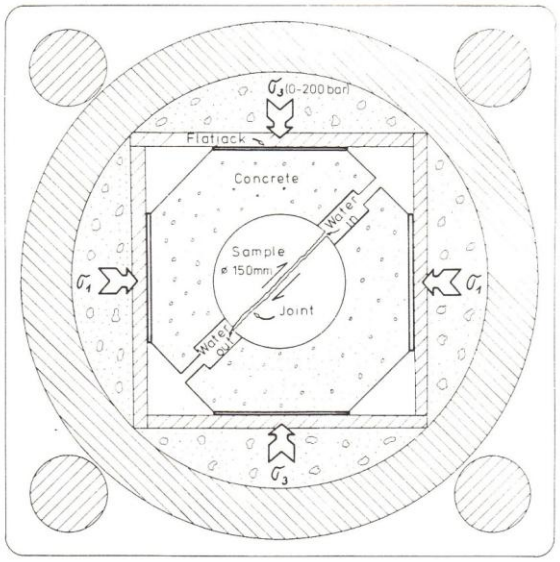


Figure 8. Schematic cross-section of biaxial apparatus for shear and normal loading of jointed core, with simultaneous measurement of fluid flow. (Makurat, 1985).

COMPRESSED AIR SURGE CHAMBERS

The unlined compressed air surge chamber in rock is a Norwegian speciality that is used to replace the conventional surge shaft. The air cushion is designed to dampen the pressure oscillations caused by changing generator demand. The required rock cavern, partly filled with compressed air, is located upstream from the power house in the best quality rock that can be found within operating distance restraints.

NGI has performed the geological mapping, permeability testing, stress measurement and air leakage calculations for several of the surge chambers operating at Norwegian State Power Board (Statkraft) hydropower projects.

Johansen and Vik (1982) have described air leakage calculations and operating experiences from Lang-Sima (6,200 m^3), Oksla (17,000 m^3) and Nye Osa (12,500 m^3); the latter operated by the Hedmark Energy Authority. Their calculations for the 120,000 m^3 surge chamber at Kvilldal are also mentioned, while final operating experiences are described by Pleym and Stokkebø (1985). In this section some of

the rock mechanics design considerations for Kvilldal are described in view of the special water curtain that was adopted to prevent leakages.

Figure 9 indicates the layout of the unusual four sided chamber in relation to the power house. The volume of compressed air varied from 75,000 to 90,000 m^3, with an air/water interface of 5,200 m^2. The chamber is situated in massive granitic gneiss with an overburden of approximately 550 m.

The final chamber location was based on engineering geological mapping and some 1600 m of investigation core drilling from the headrace tunnel, together with systematic permeability measurements. Three of the seven investigation boreholes were drilled in the final area chosen for the surge chamber. No grouting was carried out in view of the very low permeability.

At the maximum operating pressure of 42 atmospheres an air leakage rate of 1.0 m^3/min was calculated, based on the Tokheim and Janbu (1982) theory. Difficulties in applying their formulation for a round -ended cylindrical cavern to the real four-sided chamber were inevitable. However, early leakage rates during the first months of filling were close to those predicted, ranging from 1 to 2 m^3/min. Piezometer installations at chamber depth had indicated high pore pressures, so a small gradient was expected.

Gradual filling of the chamber with air, to keep pace with the installation of successive generating units, took longer than expected and calculations showed that a leakage rate of some 4 m^3/min was occurring during the autumn of 1982. Various solutions to this problem were discussed.

During a mandatory drainage of the tunnel system in January 1983 air could be heard leaking out of joints which intersect the chamber and access tunnel. A water filtration "umbrella" was chosen as the solution to the excessive air leakage. The system was designed and constructed during a three month' period before the May thaw.

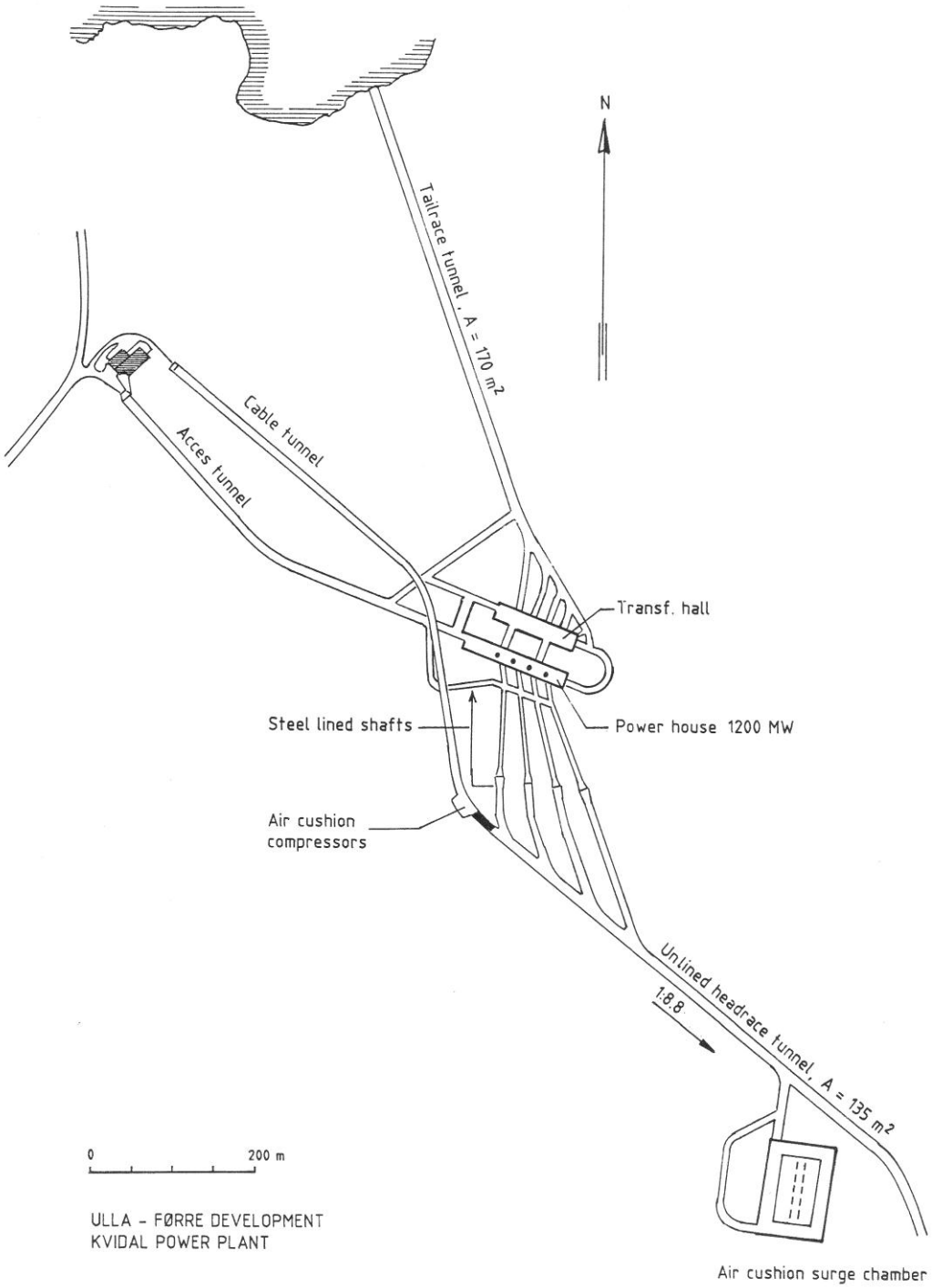


Figure 9. Statkraft's 1200 MW Kvilldal power station showing the four sided compressed air surge chamber (Pleym and Stokkebø, 1985).

Figure 10 illustrates the shape of the infiltration umbrella over this unusual surge chamber. Some 2,500 m of boreholes were required. The holes ranged in length from about 50 to 80 m and had a minimum 7.5 m clearance from the roof of the chamber.

The decision concerning the water infiltration pressure was influenced by the desire to achieve a high water flow gradient towards the chamber while wishing to avoid hydraulic fracturing of the rock mass. Minifrac stress measurements performed by Vik in four holes indicated minimum rock stress levels of 5.2, 4.5, 4.3 and 4.7 MPa.

A water pressure of 5.2 MPa was chosen, relying on the logarithmic or linear decay of pressure that occurs around an infiltration borehole when intersecting joints at right angles or axially. This rapid pressure decay was expected to just prevent hydraulic fracturing, but provide the best possible flow gradients to prevent air leakage.

The water infiltration system was put into operation before air-filling began to ensure water-filling of all the larger joints. Water consumption was 32 litres/minute, and air leakage was "reduced to almost zero" (Pleym and Stokkebø, 1985).

CONCLUSIONS

1. Overburden design criteria and numerical models that rely on minimum rock cover, rock density and considerations of surface topography may not eliminate all potential leakages from unlined pressure tunnels. Minimum rock stress levels measured by minifrac tests indicate K_o values as low as 0.4 in both igneous and sedimentary rocks ($K_o = \sigma_{min}/\sigma_{vert}$).

2. Rock mass deformation phenomena in the transition zone between the unlined and steel lined section of a pressure tunnel can cause serious leakages. Problems can occur even when the water pressure is less than the minimum rock stress. Location of the plug in massive rock, or injection of any deformable joints is needed to avoid such problems.

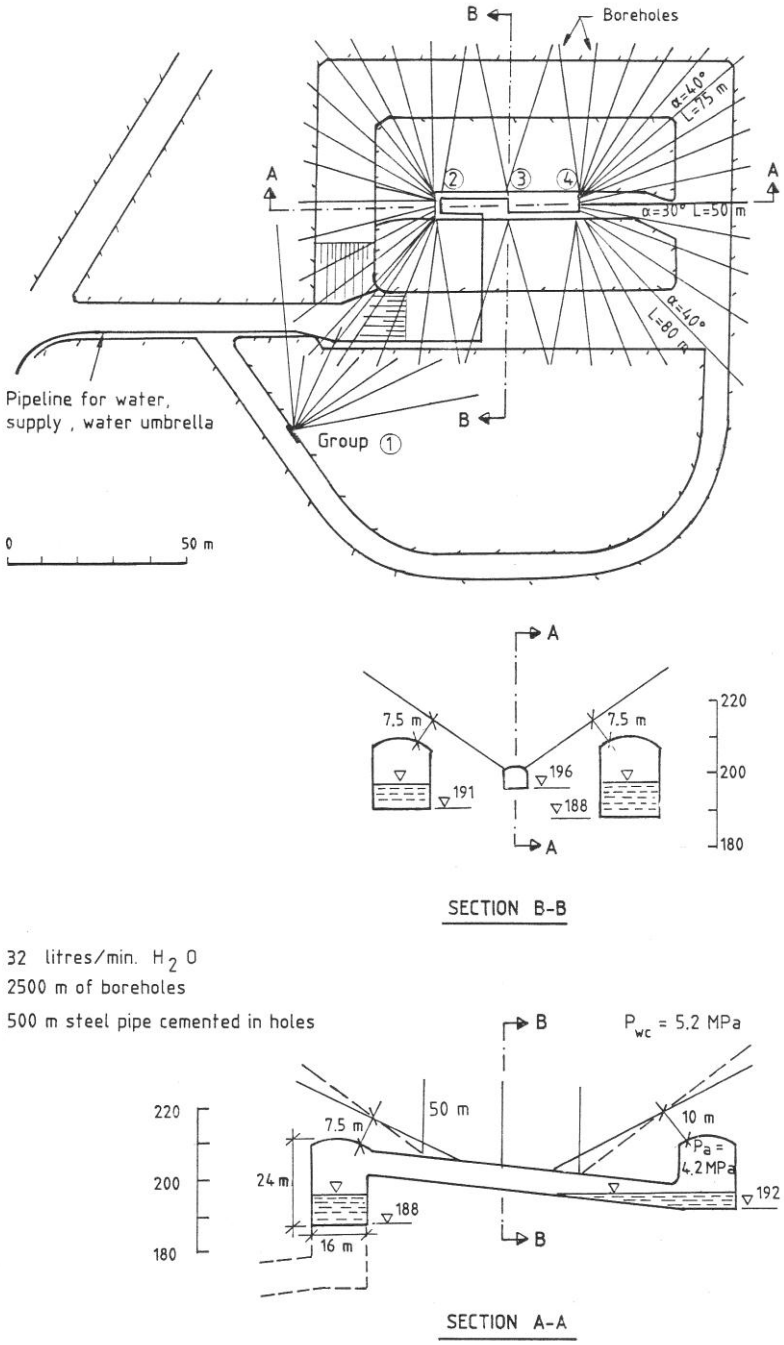


Figure 10. Details of the water infiltration umbrella used to combat air leakages. (Depth. of cover is 540 to 560 meters).

3. Numerical models are now available that allow joint deformation phenomena and liner cracking to be modelled in two dimensions. Changes in joint or crack apertures can be tracked for different levels of internal pressure.

4. Water infiltration umbrellas for air cushion surge chambers (and pressurized gas storage caverns) should be designed with minimum rock stress levels in mind. Gradients can then be maximized without risking hydraulic fracturing.

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