Lining of pressure tunnels and hydrofracturing potential.

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Summary

Pressure tunnels as elements of a hydropower plant represent an important share of the total investment.

A trend to cheaper solutions is therefore developing. It consists in simplifying, reducing or eliminating the lining of the tunnels. In doing so, important design rules were disregarded in some occasions leading to unpleasant events.

In the following, these rules are recalled, thus putting special emphasis on the question of hydrofracturing.

Introduction

A critical element of hydroelectric projects with remotely located powerhouses is the pressure tunnel that conducts the water from the intake structure at the reservoir to the turbines in the powerhouse. The construction of the tunnel and its lining are major cost items of the project and any delays in the construction can place the tunnel lining on the critical path for power generation.

Thus, for economic and time reasons there are incentives to reduce, or to eliminate altogether, the lining from some lengths of the tunnel. Such economy may be false, however, because the cost of providing originally and adequate lining may be much less than the cost of repairing a lining that has failed during operation,considering the value of the lost power and the social and political costs (1). Moreover, available sites in recent years often have presented poor geological conditions and, in some cases, with increasingly higher hydraulic heads in the tunnel.

A number of failures have occurred over the past decade with severe cracking of concrete linings and with escape of tunnel water (1,2). Some of these appear to have been associated with hydrofracturing of the adjacent rock (hydrosplitting or hydrojacking along a bedding plane or joint).

In the present paper this phenomenon is examined both from the theoretical and practical viewpoints. Preceding that discussion is a brief review of types of linings, and their roles in pressure tunnels.

Types of Linings

Classical Solutions

No lining. Many existing tunnels have no lining at all although they may be locally supported by rock bolts. Weak zones will be supported by bolts, mesh, and shotcrete, or by steel ribs and concrete. Deere (1) notes the desirable geotechnical characteristics for an unlined tunnel:

— Massive, good quality rock
— Absence of erodible or soluble material
— Substantial rock cover
— High groundwater level
— Low rockmass permeability

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Buen and Palmstrom (3) in their 1982 paper state, "The solution with unlined pressure shafts and tunnels is both cost — and timesaving. Unexpected adverse geological conditions have, however, in the past been the cause of extensive repairs and loss in production in several hydro power schemes. This proves the necessity of thorough planning and control during all phases of construction."

Reinforced shotcrete lining. Shotcrete with or without steel mesh reinforcement, and with or without rock bolt support, are commonly employed as initial support during tunnel excavation. For use as the lining of a pressure tunnel, however, both the rock bolts and the mesh reinforcement of the shotcrete should be mandatory. Shotcrete is a semi-permeable lining with the permeability dependent upon the number of shotcrete layers, the quality of the mix, and the workmanship of the application.

Plain concrete lining. Without doubt the most common type of lining for pressure tunnels is the unreinforced concrete lining. This does not imply that it is necessarily the best type for all conditions. Such lining is subject to *longitudinal cracking* (usually a single crack along either the horizontal or vertical axis) due to radial displacement when the tunnel is first pressurized if the rock modulus is too low for the applied internal head. Concrete linings may also develop *transverse* (circumferential) cracks on 3-m to 9-m spacing, and occasionally longitudinal cracking as well, due to thermal shrinkage as the freshly placed concrete cools down to ambient from its maximum setting temperature. Thermal cracking can be reduced or eliminated by using a cement with low heat of hydration, a low cement content (225-275 kg per m^3), cool aggregates and cool mixing water (or ice), and reasonable lining thickness (30-35 cm). Backfill grouting of the crown voids and high pressure consolidation grouting are both required. All honeycombed concrete, cold-joints, shrinkage cracks, and construction joints should be repaired with mortar or epoxy resin before first filling. While a concrete lining is considered a semi-permeable lining, its permeability can range from low to high depending on the care taken in design and construction. The tensile strength of a lining should not be counted on, in taking part of the internal pressure, because much of it has already been used in the thermal cooling; the concrete lining may contain many incipient longitudinal cracks (e.g. along construction joints) even before the tunnel is pressurized. Also, cold reservoir water can induce some additional thermal shrinkage. Plain concrete is in fact a brittle material.

Reinforced concrete lining. A reinforced concrete lining is more ductile than one of plain concrete, the ductility increasing with the percentage of steel. The steel reinforcing distributes the crack and they are much thinner than the single crack of a plain concrete lining. Because the water loss is proportional to the second or third power of the crack width, the leakage through numerous thin cracks in the reinforced concrete lining (of perhaps 0.1-0.3 mm widths) could be one to two orders of magnitude less than through a single 5-mm crack in a plain concrete lining. All of the cautions given for design and construction of the plain concrete lining also apply for the reinforced lining. With these precautions, the reinforced concrete lining can be considered a slightly permeable lining, appropriate for use in soil and weak rock tunnels, in zones of low rock cover, and as transitions between sections of steel lining and plain concrete lining. For high heads in low quality, permeable rock, very heavy reinforcement could be required.

Steel lining. A steel inner-lining with 30 cm or so of backfill concrete between the steel lining and the rock walls of the tunnel can be considered an impermeable lining (providing that the grout-hole caps are sealed tightly). Steel thicknesses can vary greatly (commonly 12 mm-30 mm) depending upon internal head, rock modulus, tunnel diameter, and reinforcing bands. Steel liners are used at the powerhouse end of pressure tunnels where hydraulic pressures are high and the rock cover often low.

Steel linings are frequently required along the tunnel alignment in areas of permeable ground, water-sensitive rocks or low cover where water losses are unacceptable. Freestanding steel pipes may be used in some cases instead of a steel lining.

Novel Solutions

Prestressed concrete lining. While not common, prestressed concrete linings are gaining some popularity, especially in Europe, as they are only slightly permeable and can be thirty percent cheaper than steel. The tangential prestressing develops compressive stresses in the lining that offsets the thermal stresses and the tendency for longitudinal cracking induced by the radial expansion of the lining under internal tunnel water pressure. Various prestressing techniques have been used: tensioning by hydraulic jacks of embedded, anchored cables such as the VSL method (4); high-pressure grouting, simultaneously, of several adjacent rings of radial grout holes (5); and gap grouting of perforated tubes uniformly placed around the circumference between rock wall and concrete lining (TIWAG) (9). The latter method was employed in the Drakensberg pumped storage scheme recently constructed in South Africa (10,11).

Composite membrane linings. Seeber (9) describes the use on several projects of impermeable seals (sheets) attached to the rock or to the shotcreted surface, that are held in place by a cast-in-place concrete lining. The impermeable seals may be of plastic (2-5 mm PVC) or polyethylene with a fabric or a perforated steel sheet backing) or of thin steel lining (4-5 mm) for higher pressures. The smooth walls of machine-bored tunnels are ideal for this procedure.

Prestressing of the concrete lining is done by high-pressure grouting through circumferential grout tubes between the rock and the sealing material (TIWAG gap grouting). Seeber (9) believes this system can replace part of the thick-walled steel lining and serve as a transition to the concrete-lined section. Also, precast concrete pipes with an external thin steel sheet and a backfilling of plain concrete have been used.

Role of the Lining of a Pressure Tunnel

Hydraulic Requirements

The hydraulic requirements for the lining are well understood:

1) To conduct the water from the reservoir to the powerhouse with a reduced head loss;
2) To prevent, or strongly limit, seepage loss of the water being conducted.

Both concerns involve loss of energy and revenue. Engineering studies involving the geological conditions, groundwater level, construction costs, and energy losses determine the optimal diameter, type of lining, and required rock treatment. The possible consequences of water seepage on potential piping, saturation, and landsliding of adjacent slopes must be evaluated.

Structural Requirements

The structural requirements are to support, to protect, and to resist. The lining must aid the initial rock reinforcement in supporting the longterm rock loads and external water pressure. The lining must protect the initial support from corrosion and the rock from erosion, piping, and solution. And, when the tunnel is pressurized, the lining must help the rock in resisting the differential pressure between the pressurized tunnel water and the external groundwater without appreciable cracking and leakage.

Design Philosophy

The design must take care of all aspects of the hydraulic requirements and the structural requirements. There are a variety of lining types to be considered. Each solution has proper methods of computation, design, and construction to be followed. Key geotechnical aspects are the position of the groundwater level (which may have become temporarily lowered during tunnel excavation); the modulus of the rock and the distressed zone following grouting; and the permeability of the rock mass.

Each solution has, therefore, its own field of applicability and sometimes various solutions may be equivalently well suited to a given situation.

In the next section attention is given to a particular type of failure mechanism that has not always been considered in the past.

The Problem of Hydrofracturing

General

As already mentioned, a number of accidents have occurred to pressure tunnels that may be related to a hydrofracturing of the rock mass. The escape of large water flow has caused secondary damages in addition to the economical loss of water or energy. To eliminate the risk of hydrofracturing has to be considered a very fundamental task for the engineer involved in hydropower design.

According to some authors, hydrofracturing should be the event that produces fractures in a sound rock while hydrojacking should be just the opening of existing crack or joints due to high pressure water. As a conservative rule, one should consider that cracks will or can exist in any rock mass so that only the case of the opening of existing discontinuities may in fact be of concern in the construction practice. Herein, this phenomenon is also simply called "hydrofracturing".

Jointed Rock Mass

As a rule any rock mass has to be considered, at least from a practical viewpoint, as more or less pervious. The permeability is obviously due to the existing cracks, fissures and joints while the rock itself often can be considered impervious. Any water pressure gradient will therefore produce a flow through the rock mass. This flow is harmless as long as the water pressure is relatively low, and no opening of the joints is produced. But as soon as the pressure is high enough to open the joints, the permeability and therefore the flow of water increases. Depending on the configuration of the problem, the increased flow may lead to an increase of the interstitial water pressure.

In this case the process is self-feeding and leads to a hydrofracturing or a hydrojacking of the rock mass with a large unacceptable escape of water. Theoretically, in the case of unfissured rock, the rock's tensile strength could resist the

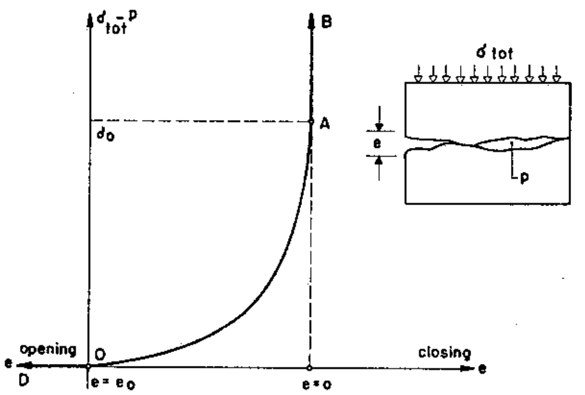

Figure 1 — Non-linear elastic opening of the joints due to internal water pressure according to F.E.S. - Model (Fissured, Elastic, Saturated Rock Mass-Model).

σ_{tot} = total stress in the rock mass
p = internal water pressure
e = opening of the joints
e_o = opening at last contact between the two rock blocks

water pressure or part of it. The question is how wise it is to trust the tensile strength of a large rock mass, in the case of an important structure like a pressure tunnel or shaft.

We will therefore in the following consider only the conditions of a naturally jointed (or fissured) rock mass.

Figure 1 shows the working line of a joint in a rock mass submitted to a total stress and to an internal water pressure accordingly to the results of the F.E.S. (Fissured, Elastic, Saturated Rock Mass) -Model.(6,7,8).

From this figure, we may conclude that:

— any increase of the water pressure in the joint will open it more or less according to its working line OA (unless obviously the rock mass is absolutely tight due to extremely high total stresses as at section AB);
— as soon as the water pressure equals the total stress no limit exists any more to the widening of the joints, section DO);
— obviously, the water pressure can never be higher than the total stress acting in the rock mass at the same point.

Should the water pressure tend to overpass the existing total stress — as in the case of a hydrojacking — a rearrangement of the stress pattern would take place in the rock mass and a new equilibrium will build up where the total stress in the rock will be at any point at least equal to the water pressure. This new equilibrium, as far as it can exist, may be understood as a kind of "overbridging" of the exceeding water pressure.

Natural Stress Field

Before the excavation of the tunnel, a natural stress field exists in the mountain. The knowledge of the same is obviously a basic necessity to properly design a pressure tunnel so as to avoid any risk of hydrofracturing. At least a prudent estimate of the minimum stress component at any point along the tunnel is mandatory. It is evident that the fracturing will likely occur along the weakest surface, that is along the joints which are submitted to the smallest total stress. These would be oriented more or less perpendicular to the axis of the minimum stress component.

An older criterion used in designing pressure tunnels referred only to the overburden on the tunnel. Implicitly only a hydrofracturing along a horizontal plane through the tunnel was taken into consideration at that time, as if the vertical stress component were automatically the minimal one.

As later on cases of hydrofracturing were also observed along planes parallel to the valley slope, the stress component perpendicular to it was then considered as the probable minimum and used as criterion for the dimensioning of the tunnel.

Additionally an Australian criterion was used which requires the lateral extension of the mountain to be double of the overburden needed to resist the internal pressure.

All these criteria, shown at Figure 2, may furnish in the best hypothesis only a first rough estimate for the minimum stress component. To improve them, but just slightly, they are based not simply on the actual topography, but on a topography from which protruding ridges were removed.

Obviously one must take into consideration planes of weakness which may cross the tunnel axis in any direction in space.

A further step in improving the estimate of the prevailing stresses consists in computing the stress pattern in the mountain due to the dead weight of the rock on the basis of an elastic model using any kind of F.E.-computations. This method overlooks however the fact that, due to tectonic stresses (or strains), the actual stress pattern needs not to correspond to the assumption made of an elastic stress law. In fact very important deviations were observed in the field from the stresses computed in this way.

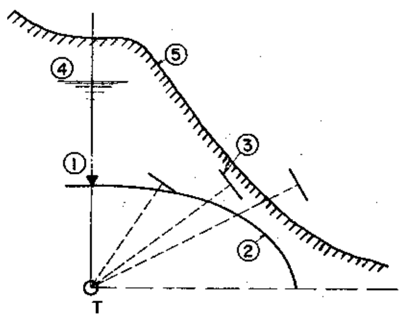

Figure 2 — Simple criteria to define the risk of hydrofracturing.
- T Tunnel
- 1 Overburden
- 2 Australian criterion
- 3 Stress component perpendicular to slope
- 4 Natural ground water
- 5 Terrain line

In any case an adequate factor of safety must be used in accepting the computed stresses on the base of these criteria.

In certain circumstances the observation of the natural ground water table may be of great help. Should, e.g., the ground water table be higher than the future piezometric level in the tunnel, obviously no risk of hydrofracturing can exist.

The most reliable method to define the minimum stress component, when the risk of hydrofracturing cannot be excluded by means of other considerations, is the use of hydrofracturing tests. The many questions related to such tests can of course not be discussed here.

Stress Field around the Tunnel

The knowledge of the natural or primary stress field in the rock mass is however just a first step in the evaluation of the risk of hydrofracturing. The next step is the definition of the secondary stress pattern around the tunnel after the excavation of the tunnel and also on a long term basis, especially before the first filling of the pressure tunnel. At this moment the rock mass around the tunnel has to be considered as being drained.

Figure 3 shows the well known secondary stress distribution due to the cavity. For the sake of simplicity of the representation we will consider the horizontal plane to be the critical one even if this is generally not the case.

Grouting of the rock mass around the tunnel will induce a favourable change of the stress distribution. An example of such a change may also be seen on figure 3.

Water Pressure in the Rock Mass after Filling

Unless the lining is absolutely tight, filling the tunnel will cause a certain water flow from the tunnel to the exterior. We may consider that after a certain transient state, which may be of long duration, a steady flow will take place, an example of which being shown on Figure 4.

Even this steady flow is the result of a quite complex combination of hydraulics and rock mechanics phenomena in a partially natural and partially artificial environment.

Starting from the internal pressure in the tunnel (where dynamic variations of short dura-

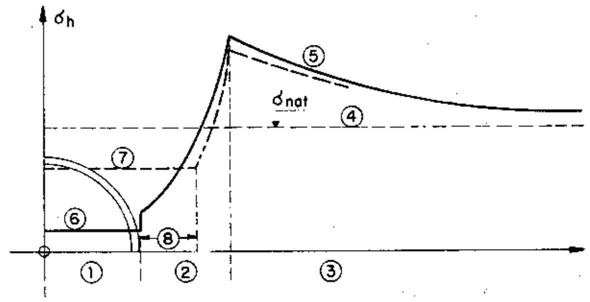

Figure 3 — Empty tunnel in the drained rock mass; stresses in horizontal plane (assumed to be the critical one).

1 Excavated and lined tunnel
2 Plastic zone around the tunnel
3 Elastic zone
4 Natural compressive stress before excavation
5 Vertical (tangential) stress, component after excavation
6 Vertical component of the rock pressure on the lining
7 Stresses induced by a grouting around the tunnel
8 Grouted zone

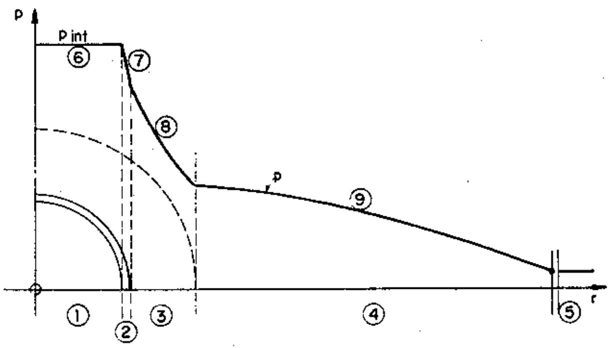

Figure 4 — Example of distribution of water pressure around the tunnel.

1 Excavated tunnel
2 Lining
3 Grouted zone
4 Ungrouted zone
5 Drains
6 Internal water pressure
7 Pressure drop through the lining
8 Pressure drop in the grouted zone
9 Pressure drop in the ungrouted zone (decreasing permeability with increasing distance due to the closing of the joints by decreasing water pressure)

tion may be disregarded), there will be a pressure drop due to the more or less pervious lining. The eventually grouted zone around the tunnel will be highly beneficial in reducing the permeability of the rock (which may have been loosened by blasting and by its inward displacement) and in producing an additional pressure drop.

From this point on, the permeability of the rock mass will define the further drop in pressure. The boundary conditions may be defined by natural factors like the groundwater level or the terrain surface, or by artificial means like drains.

In the determination of the flow pattern we must consider that the water pressure will open the joints and thus change the permeability of the rock mass which in turn will modify the pressure distribution in a somewhat complex way.

Furthermore it has to be considered that the problem is not simply a one-dimensional, as shown in the picture, but a tri-dimensional one.

Most likely a longitudinal flow along the tunnel will take place which will tend to equalize the pressures in the various cross sections. Of course, the results will be strongly influenced by anisotropy of the rock mass and its permeability.

It is immediately clear that the ratio of the permeabilities of the lining to that of the rock mass at various places will play a determinant role in the distribution of the water pressures around the tunnel.

Equilibrium around the Tunnel

To avoid a hydrofracturing of the rock mass an equilibrium between the existing stresses and the pressures due to the water flowing out from the tunnel must be obtained.

In Figure 5 the stresses from figure 3 are compared with the pressures from Figure 4.

As a rule it may happen that the equilibrium between the total stress and the water pressure will not be obtained at every point around the tunnel, especially close-in.

As the internal pressure can never be higher than the total stress, a rearrangement of the stress field must take place which may be interpreted as an overbridging of the excess water pressure in the vicinity of the tunnel. In fact, a kind of local hydrofracturing will take place almost in any case in a small zone near the lining. Its extension will be limited by the decrease of the water pressure and by the increase of the rock stresses with distance from the tunnel.

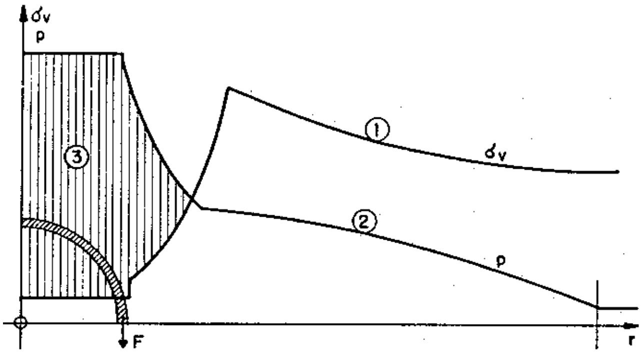

Figure 5 — Equilibrium between water pressure and vertical stresses in the horizontal plane as an example.

1 Stress in the rock mass (after tunnel excavation) from figure 3
2 Water pressure around the tunnel, from figure 4
3 Excess of water pressure to be supported (overbridged) by stress rearrangement in the rock mass
Note: stress changes will modify permeability of the rock joints and thus the water pressure distribution and the excess of water pressure to be taken by the rock, leading possibly to a hydrofracturing
F Tensile force in the steel

An actual hydrofracturing will be considered to occur only if the described extension will exceed an acceptable limit, and therefore a large flow will escape from the tunnel.

In fact the conditions of equilibrium need not be satisfied at any point, but the total forces have to comply with this condition. In a simple manner one may say that the weight of the overburden must be higher than the total hydraulic uplift.

However it must be kept in mind that with the opening of fissures the uplift force can increase dramatically, leading to a kind of elastic instability: a hydrofracturing.

Practical Procedure

The just described procedure to define the actual stresses in the rock masses, the flow pattern and the complex manifold interaction between stresses, pressures, joint opening and permeability is considered correct from a theoretical viewpoint.

However, sufficiently exact information on the actual permeability of the lining and of the rock mass, as well as on the stress distribution around the tunnel, will seldom be availabe. Also in many cases such complicated computations are not required, e.g. if the conditions of equilibrium can be satisfied with a high enough factor of safety. In such cases a simplified procedure can be followed (even if it is not as simple as the criteria mentioned at the beginning).

In many cases it could be sufficient to prove that the minimum stress measured by means of a number of hydrofracturing tests is higher than the future internal water pressure in any section of the tunnel, using however an adequate factor of safety.

It is believed anyhow, e.g. when drains are necessary to ensure an equilibrium, that investigations of the nature of the above described could be very useful.

Conclusions

The problem of hydrofracturing of a pressure tunnel can be of great concern when the internal pressure approaches or overpasses the value of the minimum natural stress component at any section of the tunnel stretch.

A number of measures can be taken to improve the situation and to increase the factor of safety. Among others the following procedures can be mentioned.

- make the lining completely tight using a sheet of metal or plastic;
- make the lining less pervious in avoiding cracks and defects in the concrete;
- reinforce the lining to get more, but finer, cracks instead of few wide ones;
- pre-stress the lining using one of the many available technical solutions;
- grout the rock mass around the tunnel to reduce its permeability and also to increase its effective stresses;
- use drains which may additionally contribute to reduce the water flow escaping from the tunnel, as the reduction of the water pressure will permit the fissures to close and will reduce the permeability of the rock mass;
- set the tunnel at a lower elevation, the simplest solution if possible.

In many cases a more detailed analysis of the stress and pressure distribution around the tunnel can be useful. A good precaution is to fill the tunnel slowly and to monitor the procedure with leakage tests and observation for surface leakage.

In any case, a good knowledge of the geological and geomechanical conditions and of the natural stress field as well as the rig' : choice of the lining type, are essential requisites for a successful design of a pressure tunnel. Where hydrofracturing potential exists, the lining rather than the rock must be counted on to limit the leakage.

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A tribute
to Prof. Dr. Victor F. B. de Mello

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