Rock-mass hydrojacking risk related to pressurized water tunnels

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1 Introduction

The uncontrolled flow of water from a tunnel under pressure can cause serious problems, in particular if the flow can reach the ground surface. The water outflow can mobilize new landslides or activate ancient movements with possible severe consequences for the on-surface infrastructures. If the tunnel is located in an urbanized area, the consequences of such accidents might be extremely serious, like for example in the case of the accident occurred to the Grande Dixence Power Plant which caused 3 victims ([1], [2]).

The decision to adopt a completely water tight lining in a tunnel, such as for example a steel one, usually leads to a considerable time and cost increase in the project and also to a relevant technical effort to be faced. For these reasons the adoption of such systems must be carefully evaluated by the Designers.

In a rock-mass the risk of hydrojacking or hydrofracturing phenomena is essentially controlled by three main aspects: the natural stress state in the rock-mass surrounding the tunnel, the orientation of the main natural joints and the quality of the rock-mass itself. In literature there are some design rules, like the so called Norwegian Rule to determine the risk of uncontrolled water leakage from the tunnel, related to the hydrojacking-hydrofracturing phenomena.

The reliability of such rules and their physical meaning has been recently reviewed in the framework of some hydraulic projects in South America and Africa. The aim of the current paper is to illustrate the state of the art related to the available design rules concerning this specific item and then to describe the gained experiences on this regard.

2 The concept of hydrojacking and hydrofracturing

As illustrated by Deere and Lombardi [3] any rock-mass has to be considered as a pervious material. Its permeability is related to the existing cracks, fissures and joints while the intact rock can be considered as impervious. Any water pressure gradient will therefore cause a flow through the discontinuities. 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 rock-mass permeability increases and consequently also the flow rate does. Under certain conditions the water leakage might become unacceptable.

In order to observe hydrofracturing, i.e. development of new cracks within the rock-mass caused by the acting water pressure, the tensile strength of the rock must first be overcome. If the applied water pressure is not sufficient to overcome the natural tensile strength of the rock-mass, a hydrojacking phenomenon, i.e. opening of existing cracks, might take place. The minimum pressure to cause hydrojacking is generally lower than the one required for hydrofracturing. Hydrofracturing test according to ASTM D4645 (Standard Test Method for Determination of In-Situ Stress in Rock Using Hydraulic Fracturing Method) or ISRM 2003-part 3 (Suggested methods for rock stress estimation. Part 3: hydraulic fracturing (HF)) should therefore always be combined with hydrojacking test (ISRM 2003-part 3: Suggested methods for rock stress estimation. Part 3: hydraulic testing of pre-existing fractures (HTPF)) to identify the determining mechanism.

The limit for hydrojacking is determined by the in situ stress normal to the existing joint set under consideration. ***Fig. 1*** shows the permeability of a rock-mass (expressed in Lugeon) as a function of the ratio of internal water pressure p to the total stress σ normal to the joint according to the F.E.S. theory (Fissured Elastic Saturated Rock-Mass, Lombardi [4]). In case the water pressure reaches the total stress, hydrojacking occurs and the permeability increases indefinitely.

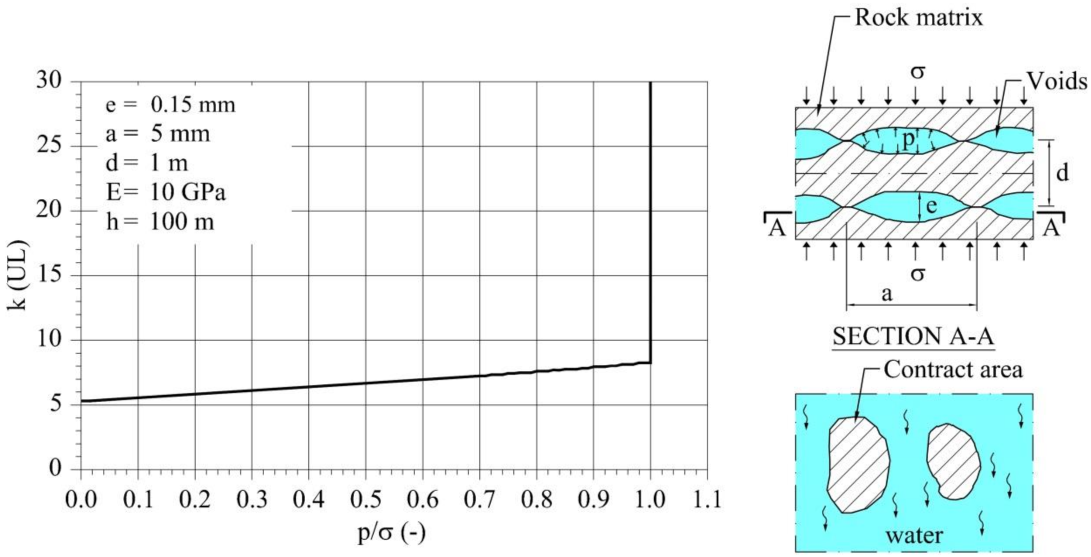

Fig. 1. Permeability of a rock mass as a function of the water pressure to stress ratio according to FES-Model [1].

It is interesting to observe how, for low overburden, the crack permeability does not change significantly with the variation of the water pressure. An increase in crack opening and permeability is effectively only observed during hydrojacking, i.e. at $p/\sigma=1.0$.

3 Design rules and international regulations suggestions

In technical literature there are many design rules to define the suitability of the pressure conditions within a hydraulic tunnel with respect to its minimum overburden. The two most common ones are the so called Norwegian rule, first defined by R. Selmer-Olsen in 1969 [5], and the Don Deere rule defined by the Author in 1988 [3]. In the following some comments will be proposed on the physical meaning of these design rules.

As reported by Broch (1982, [6]) the Norwegian Rule, referred also as a "rule of thumb", was developed and then widely used in the design of unlined pressure shafts in valley sides in Norway and thus form this took its usual name.

The first attempts to define a design rule before the late '60 were merely connected to the geometry of the hydraulic plant as illustrated in **Fig. 2** and expressed by the following correlation (1).

$$h > c \cdot H \tag{1}$$

where: c = 0.6 for valley sides inclined less than 35° and then progressively increasing up to 1.0 at $\beta = 60°$

In 1968 the unlined pressure shaft at Byrte, in Norway, with a maximum static water head of 300 m and an uncommon shaft inclination of 60° failed. In 1970 Selmer-Olsen [5] presented therefore a revised rule in which the inclination of the tunnel with respect to the slope was indicated (2):

$$h > \frac{\gamma_w \cdot H}{\gamma_R \cdot \cos\theta} \tag{2}$$

where: γ_w = density of water, γ_R = density of rock-mass. For H and θ refer to Fig. 2.

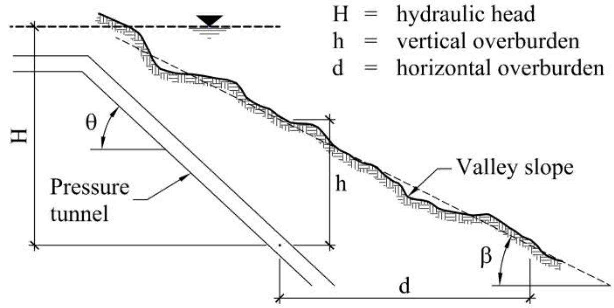

Fig. 2. Adopted symbols

The expression (3) known today for the Norwegian rule was introduced by Bergh-Christiansen and Dannevig (1971, ref. [7]) and Bergh-Christiansen (1974, ref. [8]) after the failure occurred at Askora, in Norway, where an unlined tunnel in sandstone, with a water head of 200 m, was hydraulically split.

$$h > \frac{\gamma_w \cdot H}{\gamma_R \cdot \cos^2 \beta} \tag{3}$$

Another design rule commonly used in the design practice is the so-called Don Deere rule [3]. This rule is a merely geometrical rule expressed by:

$$\begin{aligned} h &> 0.8 \cdot H \\ d &> 2.0 \cdot H \end{aligned} \tag{4}$$

Also this kind of rule is a preliminary design rule based on the Authors' experience on different design cases and is just a geometrical condition to be fulfilled during a tunnel/shaft alignment definition.

To evaluate the reliability of the Norwegian rule many cases have been presented in literature (Broch, 1982, [6]) **Fig. 3** shows a selection of existing unlined pressure shafts/tunnels, where major leakages or damages occurred or not. In the figure the lower curve represents the limit defined by the Norwegian rule assuming an average unit weight of 26.5 kN/m^3 for the rock-mass. The same figure also indicates the limit according to the Don Deere rule, where the horizontal overburden is converted to a limit for the vertical overburden by simply assuming a slope inclined by the angle β.

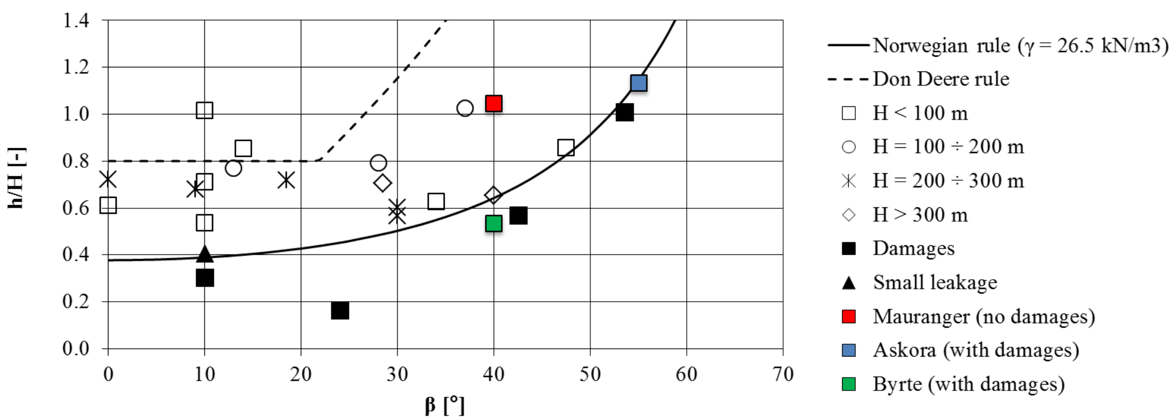

Fig. 3. Pressure shafts/tunnels in valley slopes and limit defined by the design rules to avoid hydrofracturing.

According to the results presented in Fig. 3, the Norwegian rule appears to be quite reliable: only the cases below the limit curve suffered damages, but not those above this limit. The Don Deere rule has some larger safety margins.

In terms of safety factors to be considered, according to the Authors the Norwegian rule basically allows to evaluate if a pressure tunnel or a shaft can be left unlined into rock or not. In other words the factor of safety can be assumed equal to 1. In the international technical literature some safety values to be respected are nonetheless indicated:

FS_{min} = 1.10: ASCE/PERI Guides, 1989 ([9]);

FS_{min} = 1.15: Palmstrom, Berdal, 1987 ([10]);

FS_{min} = 1.20: Broch, 1982 ([6]).

According to *Fig. 1* the behaviour of the cracks basically does not requires a safety factor. This might confirm the approach of the Authors of the Norwegian rule as well as the generally low safety values exposed above.

4 Assessment of the stress state in the rock-mass

The estimation of the stress tensor near a slope might be quite complex. Therefore, different assumptions must be considered.

The simplest way is to make the assumption of the infinite slope theory. The starting point is the equilibrium of the element (body) presented in ***Fig. 4*** (left side).

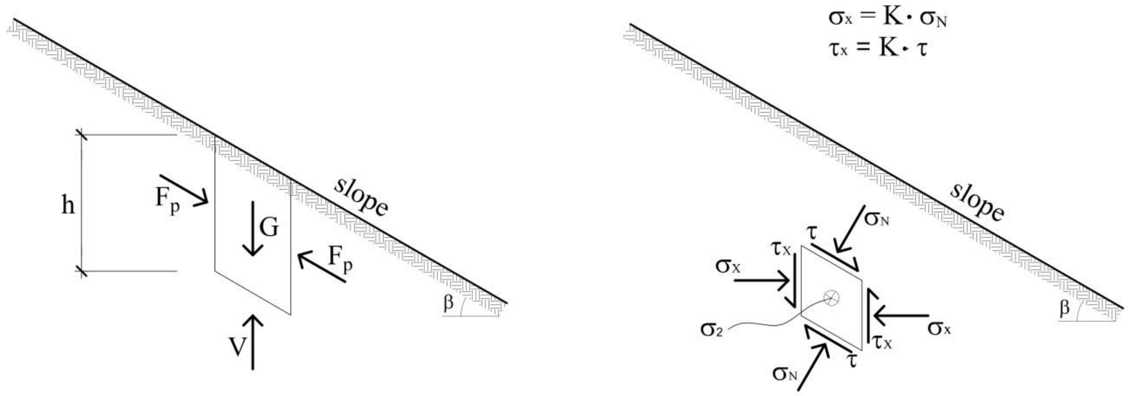

Fig. 4. Stress state near an infinite slope.

For an infinite slope the forces parallel to the slope F_p are constant along the slope and the reaction V at the bottom equals the dead load G of the considered element. The stress state on a plane parallel to the slope at a vertical depth h can be defined simply by:

$$\sigma_N = \gamma_R \cdot h \cdot cos^2\beta \quad \text{and} \quad \tau = \gamma_R \cdot h \cdot cos\beta \cdot sin\beta \tag{5}$$

The stress state on a vertical plane (σ_x and τ_x) cannot be determined by means of equilibrium equations but can be related to the stress at the bottom of the element (σ, τ) by a coefficient (*K*) as follow.

$$\sigma_x = K \cdot \sigma_N \quad \text{and} \quad \tau_x = K \cdot \tau \tag{6}$$

This is justified since τ/σ = tanβ according to equations (5) and also τ_x/σ_x = tanβ because the resultant must be parallel to the slope.

Finally the complete stress state is defined by the out of plane stress, assumed to be a principal stress and defined as:

$$\sigma_2 = K_2 \cdot \gamma_R \cdot h \tag{7}$$

According to this stress state the principal stresses in plane can be defined by the following equations:

$$\sigma_1 = \left(\frac{1+K}{2} + R\right) \cdot \gamma_R \cdot h \quad \text{and} \quad \sigma_3 = \left(\frac{1+K}{2} - R\right) \cdot \gamma_R \cdot h \tag{8}$$

$$\text{with} \quad R = \sqrt{\frac{(1+K)^2}{4} - K \cdot \cos^2 \beta} \tag{9}$$

And the angle ξ giving the direction of major principal stresses to the horizontal is defined by:

$$\tan \xi = \frac{K \cdot \sin \beta \cdot \cos \beta}{\frac{1+K}{2} + R - (1 + K \cdot \sin^2 \beta)} \tag{10}$$

A particular case is obtained with K = 1 as exposed in **Fig. 5**:

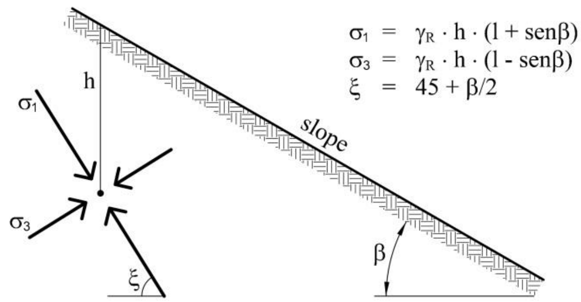

Fig. 5. Orientation of principal stresses near a slope (for K=1)

This very last case represents the equilibrium situation that requires the minimum rock-mass strength, since the deviatoric tensor is at its minimum value. This equilibrium is therefore the most probable, but not the only possible one.

In conclusion for an infinite slope, multiple stress states can be defined all respecting the equilibrium condition, varying the parameter K. Other equilibrium conditions, can also be defined. An example is the case A shown in ***Fig. 6***, for which the shear stress parallel to the slope is zero.

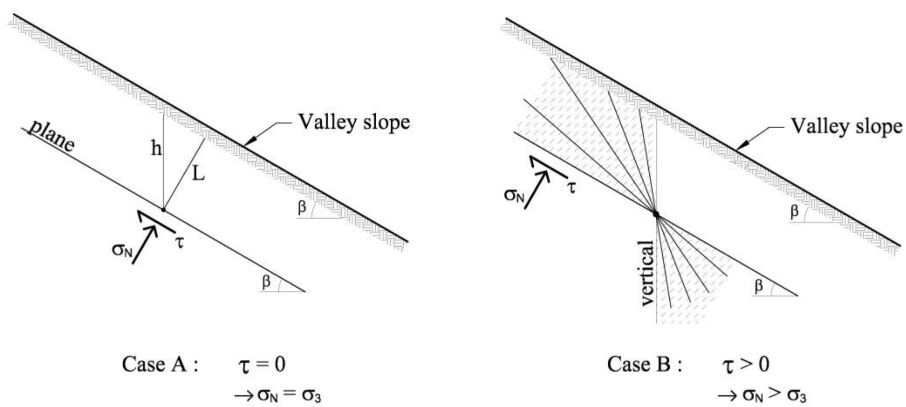

Fig. 6. Stress state near a valley slope.

Independent of the chosen stress state, the stress σ_N perpendicular to the slope is univocally defined by the equation (5). In fact the Norwegian rule given by equation (3) simply indicates that the stress σ_N must be higher than the water pressure within the tunnel.

In many case histories the necessity for a lining was based on the fact that the internal pressure in the tunnel or the shaft must be lower than the in situ acting minimum principal stress. In fact the stress normal to the slope only corresponds to the minimum principal stress for case A in *Fig. 6*, where the shear stress τ on a plane parallel to the valley slope is zero. This case, which can be considered as a quite particular case, may occur when the rock-mass stiffness in the slope direction is greater than the rock-mass stiffness normal to the surface, for example due to the presence of a schistosity or a stratification parallel to the slope. In case A the stress parallel to the surface varies along the slope by increasing toward the foot.

For case B the stress normal to the slope does not correspond to the minimum principal stress, since the shear stress on the plane parallel to the slope is not zero. Therefore, the Norwegian rule cannot guarantee that the water pressure is less than the minimum principal stress.

It is important to consider that the minimum principal stresses are not relevant unless hydrofracturing occurs. In the generally more probable case of hydrojacking, the stresses normal to the various discontinuities are relevant. The importance of the normal stress acting on joints was clearly exposed by Bergh-Cristensen, 1982 (ref. [11]) with the case history of Mauranger Power Plant (Norway). This Power Plant includes an unlined pressure shaft with a maximum static head of 450 m. With an average slope angle of 40° and a minimum rock cover of 360 m, the bottom of the shaft respects the criteria defined by the Norwegian rule (minimum required rock cover of 220 m with a γ_R = 2.5 t/m^3) (see *Fig. 3*). In situ rock stress measurements (hydrofracturing tests) however indicated the existence of a minimum principal stress (σ_3) in the point studied of only 0.5 MPa. Even with an internal water pressure of 4.5 MPa, i.e. 9 times larger than the minimum principal stress, the shaft was built unlined. The plant has been in continuous operation since 1974 without any problem of leakage or stability of the surrounding slopes. The reason determining that the shaft could be built unlined is related to the fact that all joint sets naturally present in the rock-mass have an orientation such that the in situ normal (σ_α) stress acting on them is higher than the acting water pressure.

It is therefore determinant to assess the stresses normal to the various joints. According to the infinite slope theory, the normal stress acting on a joint as a function of its orientation angle α is shown in **Fig. 7** and it can be expressed as follows (11). All the parameters have been defined previously.

$$\sigma_\alpha = \left(\frac{1+K}{2} - R\cdot\cos(2\xi - 2\alpha)\right)\cdot\gamma_R\cdot h \qquad (11)$$

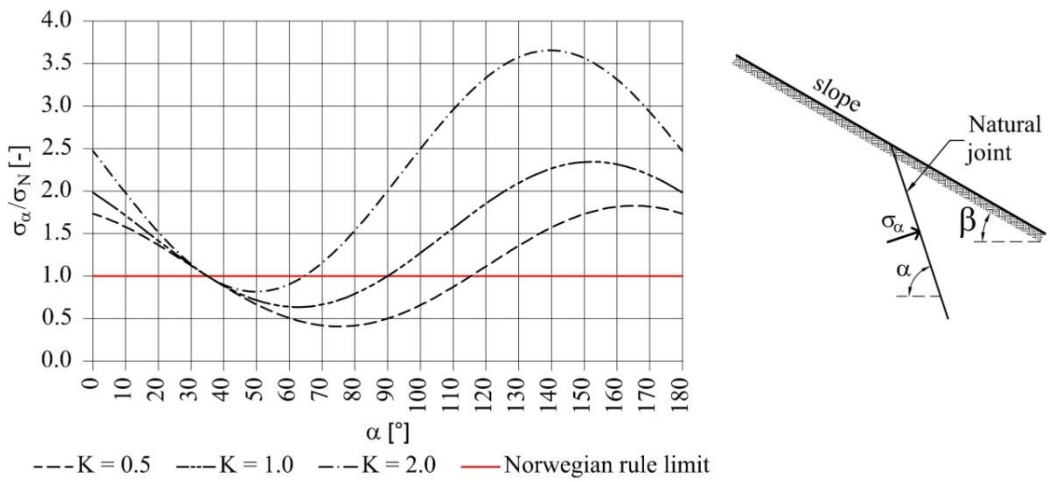

Fig. 7. Variation of the stress normal to a randomly oriented joint with respect to the stress σ_N normal to the slope (β = 35°).

According to Fig. 7, the stress normal to natural joint inclined less than the slope (β) will be higher than the value of σ_N. Stable conditions against uncontrolled water leakage are thus assured when the Norwegian rule is respected. It must be considered that these joints represent potential sliding planes and that therefore hydrojacking would directly cause the slope instability.

On the other hand, normal stresses lower than σ_N are obtained for the planes that are more inclined than the natural slope. In these cases, there is a potential of hydrojacking, but the situation is more favourable than having a hydrojacking in a joint less inclined than the slope. The slope instability is less probable and the risk of hydrojacking tends to reduce as the joint opening proceeds upward.

Moreover, in case of a minimum principal stress oriented out of the plane of the section as shown in *Fig. 6*, i.e. horizontal minimum stress parallel to the valley, possible sub-vertical natural joints might also represent a direct connection with the slope surface. In this case the problem of hydrojacking is limited by confinement unless the shaft is located in a promontory. Generally, this case may not cause global stability problems and can be handled with grouting of the rock-mass to limit the leakages.

5 Application to two case histories

5.1 Assessment by the Norwegian rule

In the following, two recent cases are presented in order to practically illustrate some of the previously exposed concepts. Here in ***Fig. 8*** a chart identical to *Fig. 3* is presented introducing the values related to these two design cases.

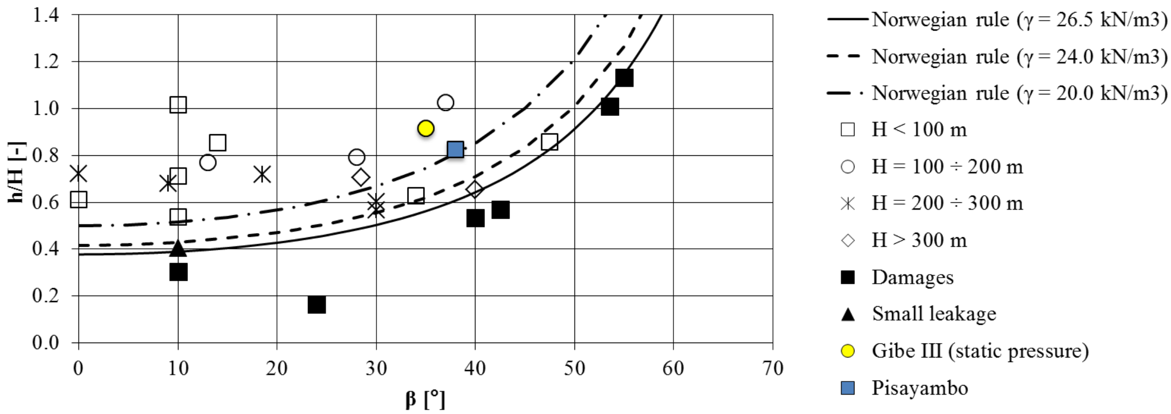

Fig. 8. Application of the Norwegian rule to some relevant case histories.

5.2 Gilgel Gibe III hydropower Plant – Ethiopia

The Gibe III Power Plant on the Omo River in Ethiopia consists of a 243 m high RCC gravity dam and has a total installed power of 1870 MW. The power house is equipped with 10 Francis turbines and is located at the dam toe on the left river bank. The power house is connected to the reservoir by 2 power tunnels with a length of about 650 and 850 m, respectively, and with an internal diameter of 11 m. The maximum water pressures expected within the tunnels are 10.65 bar in static and 13.15 bar in dynamic conditions. In some stretches along the power tunnel alignment, the overburden is rather low reaching a value of about 80 m.

This situation required an assessment of the possible hydrojacking risk along the stretches of the power tunnels with low overburden. In the case of the Gibe III power plant, the most critical situation is represented by the position of the right power tunnel (RPT) in the stretch with minimum overburden ($h \approx 100$ m and $\beta \approx 35°$). In this case (see the yellow circle in *Fig. 8*) the right power tunnel of the Gibe III Project fully respects the discussed Norwegian rule, with a factor of safety of 1.5 considering the curve for a unit weight for the rock-mass of 24 kN/m^3.

A detailed analysis on the orientation of the natural joints with respect to the stress acting on them was performed. The analysis was divided into the following steps:

- identification of main joint sets orientations: a total of 751 joint were considered. The available data were processed in order to identify the main joint sets;
- estimation of stress tensors (principal stresses and their orientation);
- detection of the joints on which the normal stress is lower than the water pressure.

To analyse the structural situation, different options for the field stress were considered. Assuming the minimum vertical overburden on the right power tunnel equal to 100 m and an average slope inclination equal to 35° different values of K (equation (6)) were considered. The chosen values of K were 1.0, 1.4, 1.8, 1/1.4 and1/1.8. The variation of these values was chosen to perform a sensitivity analysis for the influence of this parameter on the potentially critical situation. The K_2 (equation (7)) value was assumed constant and equal to 1.0. Because of the morphology of the lateral valley insisting on the power tunnels, in correspondence of the minimum overburden, a consistent lateral confinement was expected. In addition to this, the grouting realized around the tunnel aims to increase the lateral confinement, especially in presence of joint sets perpendicular to the tunnel axis.

Since the number of joints might typically be excessively high to be individually processed, it is useful to implement an efficient and practical method that allows directly the detection of the relevant planes. The basic idea is to represent on a stereonet the principal stresses and the eventual zone around the minimum one where the stress is lower than the water pressure. By adding on the same stereonet also the poles of the joint sets, one can graphically detect the joints which are potentially concerned by hydrojacking. Relevant are all the joint poles lying within the identified zones.

The stereonet representing the worst case of K=1/1.8 for Gibe III is illustrated in ***Fig. 9***. The poles represent the natural joints orientations, while the coloured zones represents the orientations of the possible joints on which the static (blue area) or the dynamic water pressure (red area) acting in the power tunnels exceeds the normal stress, thus potentially causing a natural joint opening and a consequent uncontrolled leakage.

Principal stresses

	σ_I	σ_{II}	σ_{III}
Magnitude [MPa]	3.18	2.52	0.75
Orientation [°]	303	213	123
Dip [°]	73.5	0.0	16.5

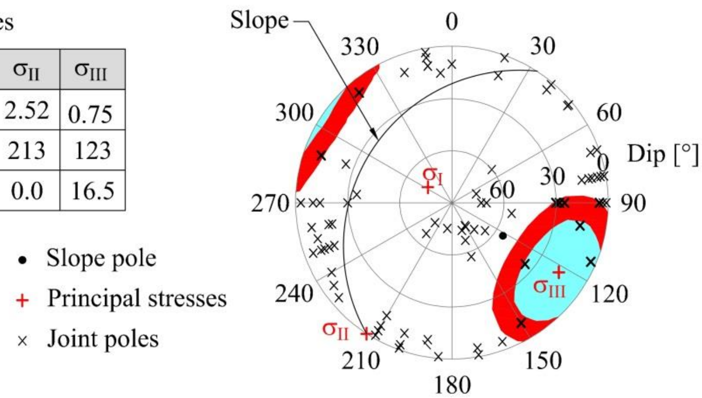

Fig. 9. Stereonet with zones on which the static (blue area) and the dynamic (red area) water pressures exceed the natural stress.

A minimum level of risk was attributed to the case in which some joint orientations might fall into the area representing the exceedance of the normal stress by dynamic pressure. It has to be underlined that this kind of approach can be considered as very conservative due to the fact that the maximum dynamic pressure equal to 13.15 bar will be reached in the Gibe III plant for a short period of time during the plant operation and that it will decay in minutes.

In this case, 3 singular joints remain within the area representing a risk for the case of the static pressure. It must be underlined that this case corresponds to a stress state characterised by a K value of 1/1.8, thus very far from the stress state represented by K = 1, which represents the equilibrium state requiring the lowest stresses values. In addition the 3 poles within the static critical area represents the 3.3% of the global amount of the collected data on surface (90) and the 0.004% of the total amount of data considered in the analysis (750). Neglecting the dynamic conditions, the results of the performed analysis were positive, confirming the absence of the risk of natural joints opening under the effects of the static pressure level in the power tunnels.

Despite these positive results it was decided, also considering the relevance of the power plant, to provide both power tunnels with a concrete lining, heavily reinforced with steel bars, and, in addition to this, an internal steel lining have been installed in the shallower tunnel (right one) over a length of about one third, in order to avoid any kind of risk of leakage from the water way.

5.3 Pisayambo Hydropower Plant – Ecuador

The Pisayambo Hydropower plant in Ecuador, operating since 1977, is located approximately 160 km South-East of Quito. The reservoir is located at about 3540 m asl and is connected to the Pucarà power house (73 MW) with a 5475 m long headrace tunnel with an internal diameter of 2.6 m. The average slope of the headrace tunnel is 0.669% and the nominal capacity is 18.6 m^3/s.

On September 2011, after 34 years of normal operation, the plant had to be shut down, after being affected by a large landslide on the slope located adjacent and parallel to the final stretch of the headrace tunnel (***Fig. 10***) [12]. The Power Plant was stopped due to the presence of concrete debris in the turbines and an inspection of the headrace tunnel showed that between chainage 4940 and 5140 a series of cracks and damages occurred in the concrete lining of the tunnel reinforced with steel bars.

The geology in the area of the landslide is described as 3-6 m of organic soils, 10-75 m of fractured volcanic rocks and then, ahead of this, the rock–mass is formed by sound volcanic rocks. The stretch affected by damages was crossed by a geological fault

In the case of the Pisayambo power plant the accident occurred in a stretch with low overburden (h ≈ 50 m and β ≈ 38°). The water head is 62 m in static and 92 m in dynamic conditions. For the static situation, the application of the Norwegian rule shows basically that no risk would occur for the slope (see the blue square in Fig. 8). Only when reducing the unit weight of the ground at 20 kN/m^3, considering the actual slope material as more soil-like (disturbed rocks and faulted zones), then the safety margins reduce significantly. On the other hand, in maximum dynamic conditions the risk of hydrojacking is temporarily high.

In the Pisayambo case probably other factors than the mere water pressure, such as the presence of tectonic structures (faults), or ancient slides over the tunnel stretch which was affected by damages, actually influenced the slope stability. Despite this, it is evident that the safety margin, with respect to the Norwegian rule, was already quite reduced thus probably creating the precondition for the observed accidents. The situation "at the edge" might probably also justify the quite long operating period (34 yrs.) of the plant before the accident occurred. In this case, the bad conditions of the rock-mass did not allow to detect clearly defined joint sets on which to perform a detailed acting stress vs. orientation study as the ones performed for the Mauranger and the Gibe III cases.

The damaged stretch of the headrace tunnel was rehabilitated with a by-pass tunnel, allowing to increase the minimum overburden up to 110 m and placing the new tunnel in a rock-mass of better quality.

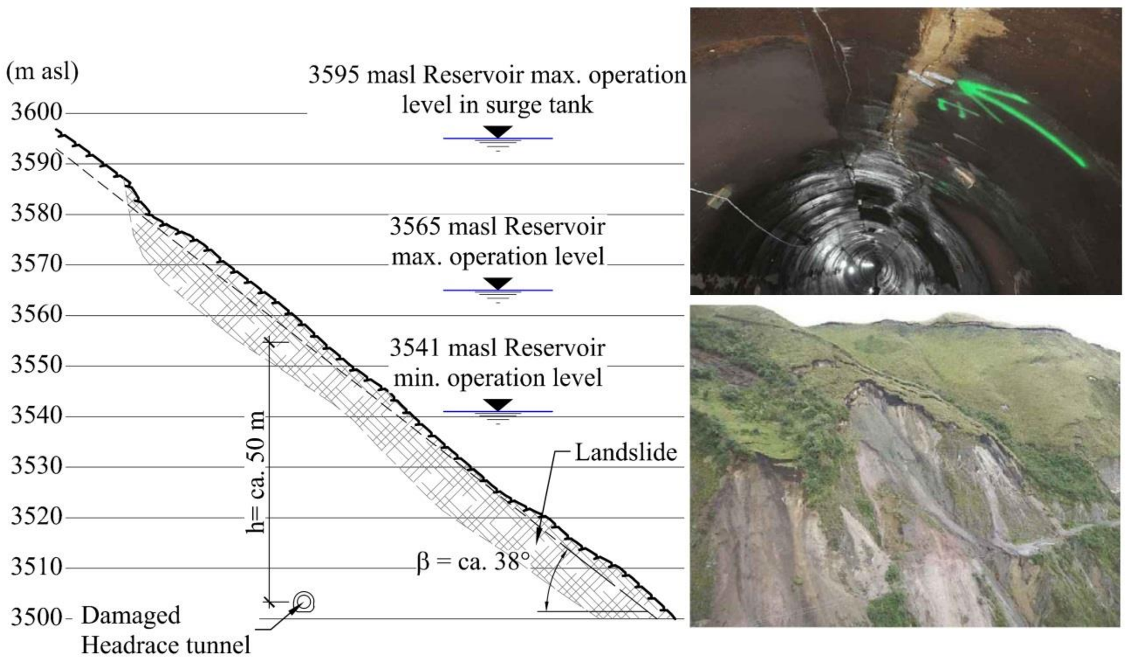

Fig. 10. Left: Situation of the Pisayambo headrace tunnel in the damaged stretch. Right: Image of the crack inside the tunnel and of the main landslide.

6 Conclusions

The design of pressurized underground water ways and the assessment of the related risk of hydrojacking and uncontrolled water leakage imply a very careful evaluation of the geological and stress conditions existing in the rock-mass to be excavated, already from the very early stage.

The paper briefly reviews the commonly used empirical rules to assess the risk of hydrojacking underlining the noticeable reliability of the Norwegian rule. The stress state near a slope is systematic exposed allowing to highlight the justification behind the Norwegian rule. This rule is therefore not only based on a statistical analysis of case histories, but it has a physical background.

Nevertheless, these rules should not be uncritically applied, but their use should be carefully coupled with a detailed analysis of the geological conditions and of the stress state acting within the rock mass. Hydrojacking occurs in case the stress normal to natural joints is less than the water pressure. The assessment of hydrojacking risk should by based on the identification of these joints sets.

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