A simplified approach of the rock-bolting design based on the principles of NATM

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ABSTRACT: A simplified approach is presented in this paper in order to evaluate the efficiency of rock-bolting in underground excavations. The need of simplified approaches for the design of the rock bolting, especially for tunnel sections in multiple geomechanical conditions is an practical evidence. For these cases, where the geology varies considerably through the tunnel length, the use of 2D numerical analysis is usually non pertinent. The rock bolting design is mainly based on the limit equilibrium method in blocky rock mass (as the well-known Un-wedge software) or on empirical approaches (support pressure in Q-method for exemple).

The basic principles of the approach presented below, are based on the NATM, as described by Rabcewizc in the 60's. The bearing capacity of the reinforced rock arch depends on the development of shear lines through the support system (rock-bolts and shotcrete) and the appearance of a sliding body that under the influence of the geostatic stresses is extruded towards the interior of the tunnel. The basic aim of this paper is to present a critical review on the Rabcewizc method. According to NATM, but with more rigorous and modern formalism, the design of rock-bolting is based on the limit equilibrium, where the shear strength of the reinforced arch is performed to determine the global stability in terms of a safety factor. The equivalent simplified approach is a typical 2D computation, realised with GEOSTAB, slope stability software, where the rock-bolting should verify the stability of the sliding body, subjected to an overload. The overloading varies from the residual strength of the rock at the tunnel wall, to the initial stress at the infinite without considering a fictitious internal pressure as in the convergence-confinement method but a real state of stresses at a certain distance of the face.

The initial development of this simplified approach came out during the feasibility study of deep underground works, into argillaceous rock, with multiple sections. The main limitation of the approach is the assumption that the rock mass can be seen as a continuous media. Nevertheless, for the main of the sections, the design of the primary support was made according to this design method.

Keywords: rock-bolting, simplified approach, limit equilibrium, GEOSTAB

1 Introduction

Formulated at the early 60s, the New Austrian Tunnelling Method (NATM) is based on the mobilization and conservation of rock's intrinsic strength, allowing him to participate with the temporary support to the overall stability of the structure.

The NATM is applied since several decades to the design of underground structures under high geostatic stresses, where the objective of the temporary lining is to pursue the strains of the rock mass in order to solicit the least possible the final lining.

The design method, proposed by L. Rabcewizc at 1964 (Rabcewizc, 1964), which justified the concept of the NATM, verifies the "bearing capacity of the reinforced rock" as the sum of the resistances of each available element, rock, bolts and sprayed concrete.

However, the inaccuracies of the method, especially as far as the definition of the safety factor of the composite rock- temporary support is concerned, confirm the need for the development of a more rigorous approach according to a modern formalism.

The basic aim of this paper is to present a critical review on the Rabcewizc method but also to present a simple approach for the design of the temporary support constituted by bolts and shotcrete, based on the stability analysis of a reinforced rock mass, by using the common software of stability analysis, GEOSTAB ©, developed by GEOS Ingénieurs Conseils. This method concerns mainly the design of the primary support for preliminary studies, where the use of more sophisticated means is not usually demanded.

2 Rabcewizc method

2.1 NATM concept

The concept of the NATM is based on the hypothesis that the rock around the excavation is a part of the support system and his resistance should be preserved and mobilised at the maximum point. It is supposed that the mobilization of the resistance is provided to a certain level of strain and beyond that a significant loss is observed.

The recommended support system consists of a systematic rock-bolting and a flexible shell of shotcrete. Once a significant part of the convergences is achieved, the final lining is placed in a subsequent phase.

2.1.1 Failure mechanism

The failure mechanism described by Rabcewizc is based on observations made to bolted thin shotcreted walls. According to these observations, mainly in cases where perfect adherence between the thin concrete shell and the wall of the excavation is admitted, the failure of the support arises when the shear resistance of the shotcrete is exceeded. No failure is observed when the flexural or the compressive resistance are exceeded.

According to the direction of the principal geostatic stress, the failure of the thin concrete shell is observed either at the tunnel vault (sh/sv>1.0) or at the tunnel walls (sh/sv<1.0). This failure pattern would correspond to the formation of a "sliding body" behind the excavation's wall that is pushed to the inwards of the tunnel under the geostatic stresses (cherry stone[1] effect).

[1] cherry stone : the sliding volume formed between the shear failure lines

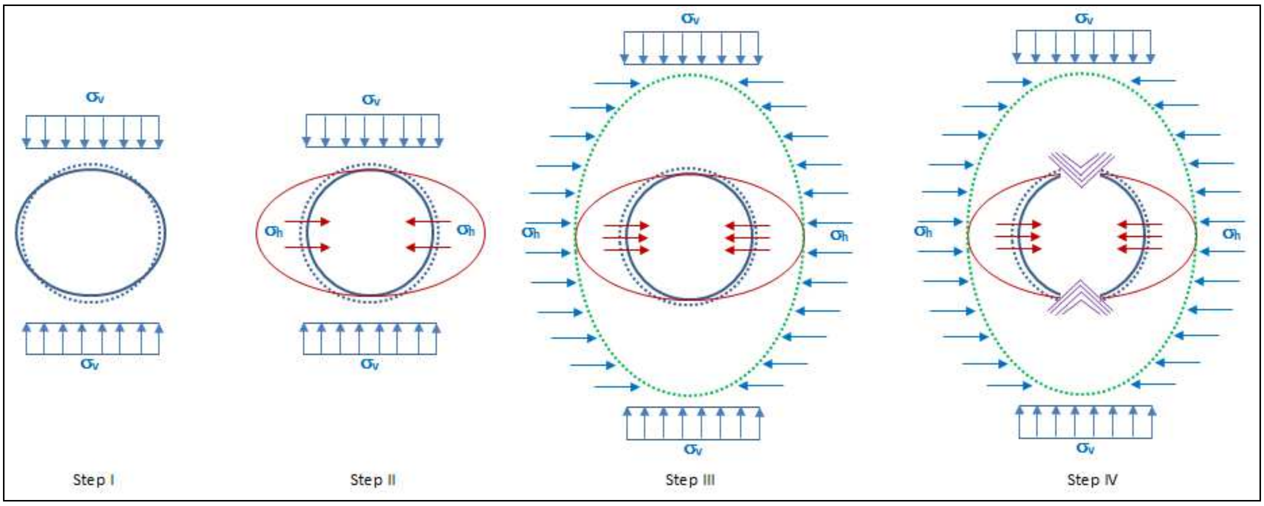

Figure 1. Evolution of the failure mecanism of a non supported excavation, the given exemple is for sh/sv<1.0

According to Rabcewizc, the evolution of the failure mechanism is summarized in four steps:

- Step 1 : Response of the rock mass in the elastic field, weak convergence between the vault and the invert of the tunnel;
- Step 2: Local failure (rock burst) at the lateral walls of the excavation under high geostatic stresses.
- Step 3: Behind the local failures (rock burst), the sliding bodies ("cherry pit") are expelled towards the interior of the excavation. In case that the excavation remains non supported, the displacements of the vault and of the invert as well, increase with time; These large deformations are supposed be the post-failure behaviour even in brittle rock;
- Step 4: During a subsequent phase, the vault and the invert are bending under the influence of the vertical geostatic stresses before buckling under the horizontal ones.

Similar mechanisms had been observed to several alpine tunnels where the initial in situ stress is important and the strength of the rock mass as well (Mont Blanc tunnel, Frejus tunnel in the French-Italian alps, Lötschberg tunnel in the swiss alps…).

2.1.2 Design and justification of the support system

The justification of the "mixed" support system, composed of radial bolts and shotcrete, is made by verifying the "bearing capacity" of the reinforced crown, which corresponds to the available resistance that prevents an internal failure from sliding of the "cherry pit".

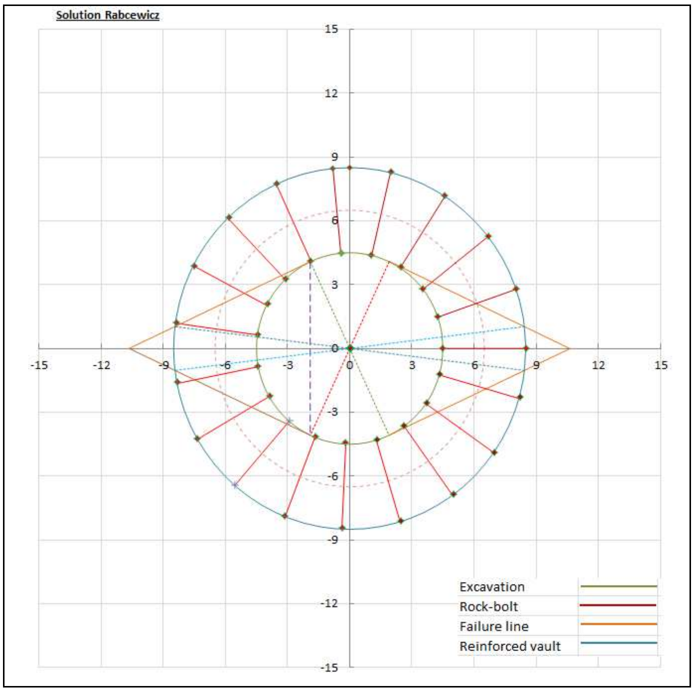

Figure 2. Failure lines and reinforced crown

The shear resistance of the reinforced crown is calculated from the Mohr-Coulomb criterion, for a principal minor stress equal to the confinement pressure provided by the support system of rock bolts and shotcrete.

The bearing capacity of the reinforced crown is calculated as the sum of the available capacities of each support element.

2.1.3 Objective and approximation of the method

The final objective of the method is to calculate the available safety factor of the proposed support system. This safety factor is calculated as the fraction between the bearing capacity of the support system and the geostatic stress that is applied on it.

The calculation of the safety factor depends on the stiffness that the support system induces to the plastic zone around the excavation. The increase of the bearing capacity leads to the increase of the stiffness of the reinforced crown, assimilated to a thick elastic tube. As a result, the reinforced crown finds his equilibrium under less important displacements and therefore subjected to higher ground stresses according to the principles of the Convergence-Confinement method. The research for higher safety factors does not seem to be attainable.

Moreover, following the adopted method, the confinement provided by the shotcrete seems to be preponderant, which is an optimist hypothesis next to the small thickness of the shotcrete layer that does not after all work as a continuum shell, mainly under high coverage, when longitudinal undercuts are designed to absorb some convergences.

3 The proposed new method

The proposed analysis is conducted with the use of GEOSTAB ©, software allowing the calculation of safety factors for reinforced slopes by bolts, anchors, frames, pins. The support should verify the stability of the sliding volume subjected to a load.

The parameters used for the example are summarized to the following table. No gravity is considered for the rock mass, and the geostatic stress field around the tunnel is isotropic, with the stress value at 16.5 MPa. The stress value corresponds to a deep tunnel at 550m.

Table1. Parameters of the example

	Unit	Value
Radius	m	4.5
Geostatic stress σ_o	MPa	16.5
Cohesion C	MPa	5.0
Friction angle ϕ	°	17.5
Compressive strength of the rock Rc	MPa	13.7
Maximum tension of bolts Tmax	KN	250
Thickness of the shotcrete layer e	m	0.15
Rock-bolting mesh $e_L x e_T$	m²	1.5x1.5

As far as the bolting is concerned, the software takes into account the adhesion between the grout and the surrounding rock but the pull-out limit of bolts in rock is generally far over the yield limit. Two different bolt lengths have been tested for this example - 4.5 and 6.0 m long. No influence of the bolting is observed once the active bolt length exceeds the length of the failure line.

3.1 Principles of the method

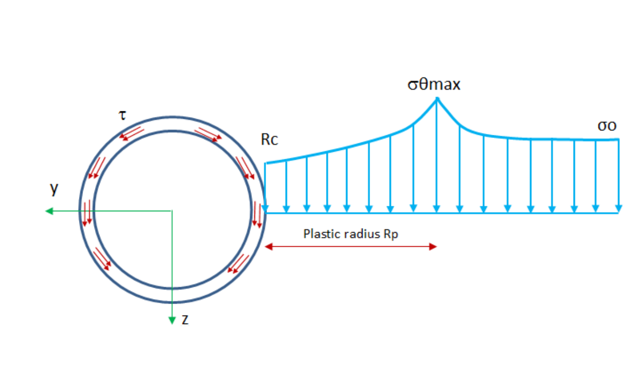

Figure 3. Tangential stress distribution in the rock mass around the tunnel

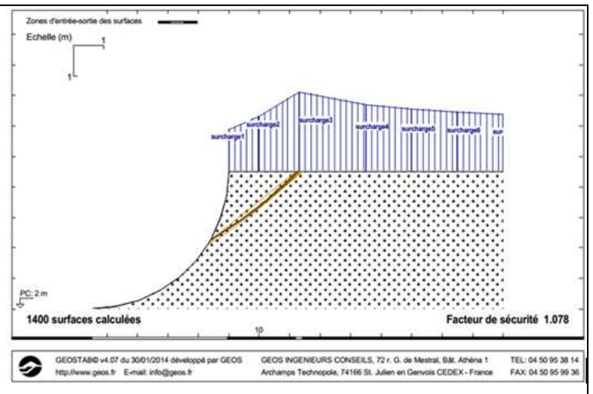

Figure 4. Static equilibrium approach inside the unreinforced plastic crown

The method is based on the internal equilibrium of the plastic crown resulting from the excavation of the tunnel. Inside the plastic crown, the tangential stress that increases around the tunnel, is balanced by the shear resistance developed along the failure lines.

This type of internal mechanism can be assimilated to a sliding volume subjected to a load into the tangential direction that varies from the compressive strength of the rock mass to the excavation wall at the in situ initial stress. The maximum value of the overload at the interface between the plastic and the elastic zone (or "plastic radius"), according to the elastoplastic distribution into the crown, is given by the following equations:

Minimal plastic radius:

$$Rp = \left[\frac{2 \cdot \lambda e}{(Kp+1)\cdot \lambda e - (Kp-1)}\right]^{\frac{1}{Kp-1}} .R \qquad (1)$$

Maximum tangential stress at r=Rp:

$$\sigma\theta \max = (1-\lambda e).\sigma 0 \qquad (2)$$

With:

$$\lambda e = \left[\frac{(Kp-1).\sigma o + Rc}{(Kp+1).\sigma o} \right] \quad (3)$$

The approach differs from the Convergence-Confinement method because no internal fictitious pressure is considered, even though the distribution of the tangential stress results of a three-dimensional equilibrium around the head of the tunnel. It is a real 2D state of equilibrium at a distance of the face. The distribution of the radial stress into the plastic crown, which otherwise tends to zero at the wall, is neglected to the equilibrium equations because of its low contribution.

The global stability of the sliding volume is verified by taking into account the rock-bolts system that induces a resistant force to the sliding surface. The support provided by the shotcrete shell with the undercuts is not taken into account because in this configuration the shotcrete does not work as a rigid shell capable to resist in compression. The contribution of the shotcrete shell will be developed in the § 3.2.

3.2 Application at a rock-bolted tunnel wall

- Global stability

For the given example, far behind of the tunnel head, the extension of the plastic radius is about 1 ½ of the tunnel radius. The global stability of the excavated section is verified by a radial rock-bolting. The effect of the shotcrete shell is not taken into account. The obtained safety factor is approximately 1.0, with or without rock-bolting. The bolting provides a gain of only 2%.

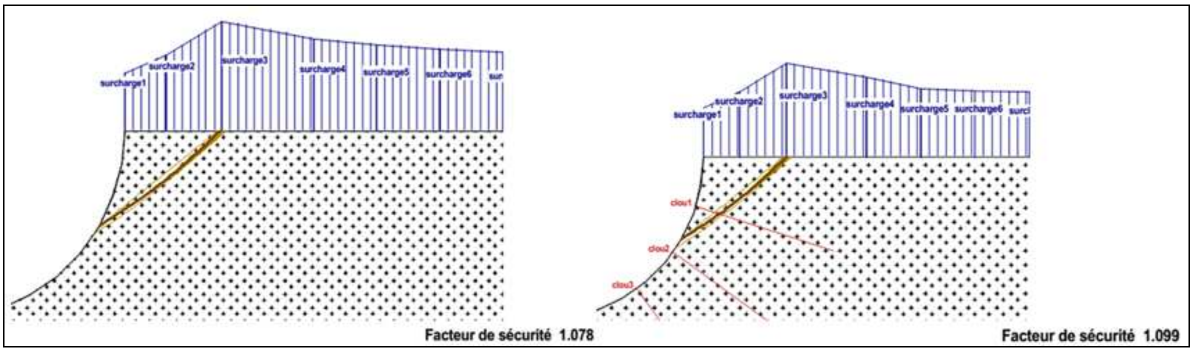

Figure 5. Global stability of a non reinforced and a rock-bolted section away from the tunnel head, GEOSTAB ©

This gain in stability is equivalent to an increase of the rock mass cohesion at about 150 KPa.

- Local stability

However, as it is shown in figure 6, the safety factor of the local stability, close to the tunnel wall, tends to 1.0 regardless to the pattern of the bolting, once the overload tends to the simple compressive strength Rc: at this point, it is the layer of the shotcrete between the bolts that should verify the local stability of the section.

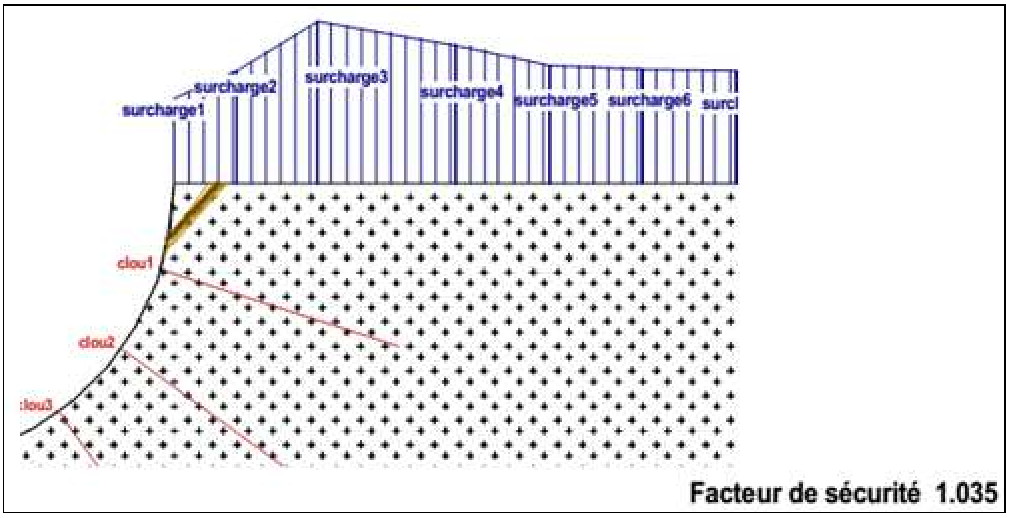

Figure 6. Local stability of the section between the rock-bolts, GEOSTAB ©

3.3 Contribution of the shotcrete layer

The shotcrete layer should be verified towards:

- shear failure
- bending failure

Shear failure

The shotcrete is verified towards the shear resistance that is mobilized by adherence to the rock.

According to the France Association of Tunnel & Underground Structures (AFTES), the shear resistance mobilized from the shotcrete layer is:

$$R_{cisaillementmobilisée} = u.e.\tau_s \qquad (4)$$

With:

u: perimeter of the shear zone

e: thickness of the shotcrete layer

Ts: shear resistance of concrete, estimated at 20% of its compressive strength

The maximum pressure applied from the rock-bolts system of a mesh $e_L x e_T$ is:

$Pb = \frac{T_{\max}}{e_L.e_T}$, with Tmax, the maximum tension mobilized to the bolts.

The safety factor resulting from the design of the shotcrete in shear failure is given to the table 2.

Table2. Design of the shotcrete layer in shear failure

Design of sprayed concrete in shear failure		Sprayed concrete (young age)
Shotcrete layer thickness	e [m]	0,15
Characteristic strength	fck [MPa]	30
Compressive resistance of concrete, young age	fcm(t) [MPa]	5
Shear resistance	τs [MPa]	1
Longitudinal spacing of bolts	eL [m]	1,5
Transversal spacing of bolts	eT [m]	1,5

Design of sprayed concrete in shear failure		Sprayed concrete (young age)
Perimeter of the shear zone	u [m]	3.0
Surface of the shear zone	Sc [m²]	2.25
Shear resistance mobilized from the shotcrete layer.	R [MN]	0.45
Equivalent pressure mobilized in shear mechanism	Peq. [MPa]	0.20
Equivalent pressure induced from the rock-bolting system	Pb [MPa]	0.11
Safety factor	FS	1.80

The equivalent pressure Ps1 that the shotcrete layer can take is about 200 KPa.

Bending failure

Without taking into account the bolts and towards the global stability of the excavated section, (according to the failure mechanism proposed from Rabcewizc), the verification of the shotcrete layer should be made not only versus a shear failure mechanism but versus a bending failure mechanism as well:

Normal force: $$N = P_s.R \tag{5}$$

Bending moment: $$M = P_s.R.\frac{R}{2} \tag{6}$$

Stress: $$\sigma_b = \frac{P_s.R}{e}.\left(1 \pm \frac{3.R}{e}\right) \tag{7}$$

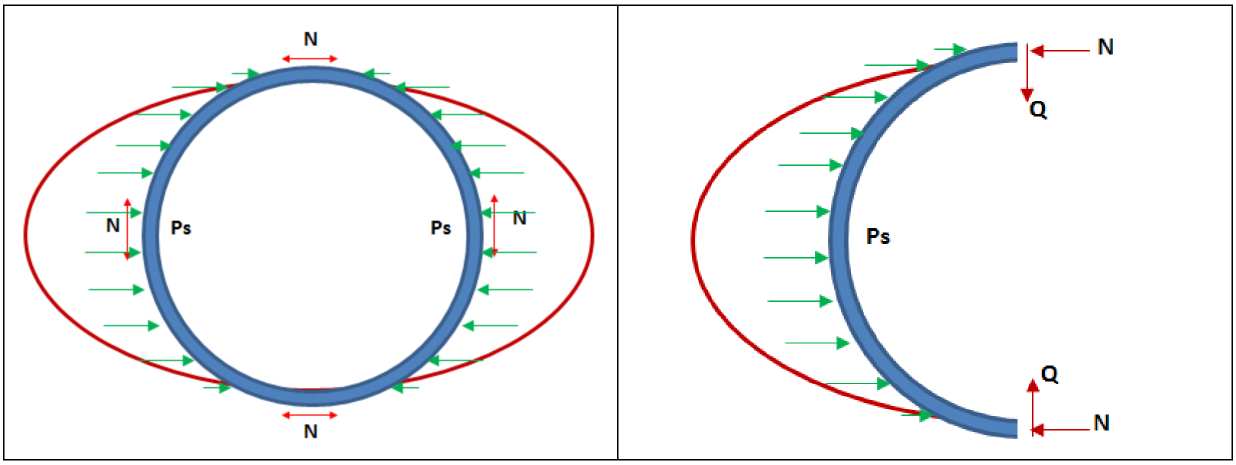

Figure 7. Design of the shotcrete layer in flexion

The equivalent pressure Ps2, that the shotcrete shell can resume, is only about 10 KPa for a tunnel radius of 4.5m. The pressure is significantly lower to the equivalent mobilized pressure in the shear failure mechanism.

This observation guides to a second verification of the shotcrete layer this time in flexion between two bolts spaced to 1.5m. In this case, the bearing bolt plates are considered as embedded points (no rotation allowed). As a consequent, the equivalent height used for the calculation of the normal force and the bending moment is limited to the spacing between two bolts. The stress σb is given from the equation (7).

The equivalent pressure Ps3 that the shotcrete layer can take is about 100 KPa, equivalent to the provided pressure from the rock-bolts system but always inferior to the one mobilized from the shear failure mechanism of the shotcrete.

The previous computations show that the effect of the shotcrete layer to the global stability of the section is not preponderant. The confinement that provides becomes significant to the local stability, between two rock-bolts, observation that confirms his supporting role to the rock-bolting, as it has already been proven from several approaches.

4 Perspectives of the development

The results presented to the previous paragraphs do not allow concluding to a significant increase of the tunnel stability because of the rock-bolting. Its contribution is similar to the one of the other approaches simulating rock-bolting with an equivalent increase of the cohesion or as an internal pressure of confinement.

However, it is possible that a development taking into account a softening behavior of the rock mass at the vicinity of the excavation's wall would be more promising.

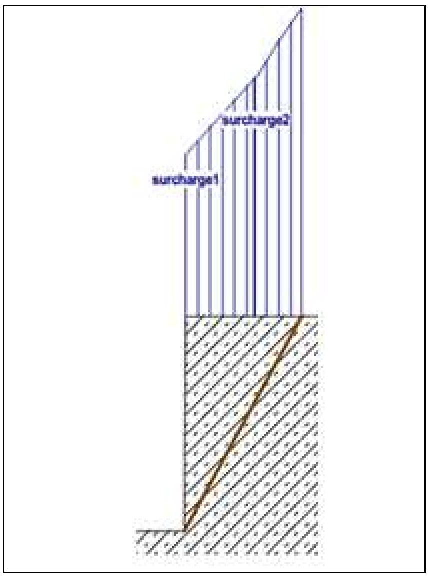

Figure 8. Simplified approach

Moreover, it is also possible to reduce the problem to that of a simple volume, defined from the geometric parameters r and Rp, subjected to the "tangential" overload, according to a plane failure mechanism.

The objective of the future developments would be the analytical study of the influence of the basic parameters –rock mass resistance, initial stress, inclination of the wall, slope of the failure surface- to the general stability of the excavated section as their incidence to the justification of rocks-bolting efficiency.

5 Conclusions

The present article develops a safety factor approach for the justification of the rock-bolting in underground structures. The origin of this method derives from the initial approach of Rabcewizc, but the development is conducted according to a more coherent procedure, based on the principle of the stability of reinforced slopes in the software GEOSTAB ©. The approach aims to better quantify the interest of rock-bolting and shotcreting to the stability of an excavated section towards local and global failure phenomena.

As resulting from the review by the french tunnelling association (AFTES working group 32) on the rock bolting design, this sort of simplified approach remains relevant especially for tunnels with a variable section constructed in a variable geology, as shown by the recent developments proposed by Jean Launay.

Acknowledgements

We are grateful to J. Launay for his pragmatic presentation of the method initially developed by Rabcewizc, quite difficult to access due to its formalism.

References

Rabcewizc, L., 1964. The New Austrian Tunnelling Method. Water Power, Nov.-Dec. 1964 & Jan 1965.

AFTES GT20, 2001. Recommandations “Conception et dimensionnement du béton projeté utilise en travaux souterrains”. Travaux et Ouvrages Souterrains n°164.

Launay, J., 2010. Approche du fonctionnement et du dimensionnement d’une voûte de tunnel renforcée par boulons scellés et béton projeté. Tunnels et Espace Souterrain n°219, pp.

Loew S., Barla G., Diederichs 2010, M. Engineering geology of Alpine Tunnels,Taylor & Francis Group, London, ISBN 978-0-415-60034-7