STABILNOST POVRŠINE IZKOPA PLITVIH GRADBENIH IN RUDARSKIH PREDOROV

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o avtorju

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izvleček

Stabilnost površine izkopa plitvih predorov, ki so bili izkopani na problematičnih tleh, je danes relevanten problem na področju gradnje tunelov in rudarstva. Čeprav je ojačitev s palicami iz steklenih vlaken učinkovita, še vedno ni zanesljivih analiz in obsežnih metod. V tem prispevku je prikazan nov računski postopek, ki analizira statične pogoje površine v plitvih predorih, tudi če so ojačani s steklenimi vlakni. Postopek temelji na omejeni ravnotežni metodi, uporabljeni na zemljišču pod površino. Najpomembnejši rezultat izračuna je, da lahko varnostni faktor izkopavanja površine izračunamo tudi, če je predor ojačan, iz česar potem lahko nadaljujemo z dimenzioniranjem posega. P. Oreste je izvedel postopek na dveh primerih in je dosegel zadovoljujoče rezultate.

ključne besede

palice iz steklenih vlaken, ojačitev površine, površinski predor, omejena ravnotežna metoda, varnostni faktor

THE STABILITY OF THE EXCAVATION FACE OF SHALLOW CIVIL AND MINING TUNNELS

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Abstract

The stability of the excavation face of shallow tunnels excavated in difficult ground conditions is currently a relevant problem in the sector for tunnelling and mining. Even though face-reinforcement interventions with fibreglass dowels have proved to be efficient, there is still no reliable analysis and dimensioning method available. A new calculation procedure is illustrated in this paper for the analysis of the face static condition in shallow tunnels, also when reinforcement interventions with fibreglass dowels are used. The procedure is based on the limit-equilibrium method applied to the ground core ahead of the face. The main result of the calculation is that the safety factor of the excavation face is also obtained in the presence of reinforcements and from this it is then possible to proceed with the dimensioning of the intervention. The procedure has been applied to two real cases and satisfactory results have been obtained.

Keywords

fibreglass dowels, face reinforcement, surface tunnel, limit equilibrium method, factor of safety

1 INTRODUCTION

Full-face excavation in tunnels, even in poor grounds, is currently being used increasingly often and when used advantage is taken of the potentiality of the machines, the equipment and the large dimensioned plants to reduce the construction times, limit the costs and guarantee better safety conditions [1, 2]. An excavation face, however, can be instable for medium-large excavation sections when the ground has poor geotechnical characteristics. In order to guarantee the stability of the excavation face, it is necessary to intervene at the excavation face with fibreglass reinforcements. They allow an increase in the stabilising forces on the core and a reduction in the destabilizing ones. These result in a very efficient intervention that can even render the excavation face stable in very difficult conditions and which is at the same time flexible (easy to change in the function of the geotechnical characteristics of the ground and of the stress conditions in the site), easy to use and reliable.

In spite of the successes that have been obtained when using fibreglass reinforcements at the excavation face, no adequate calculation instruments have yet been developed that are able to proceed quickly with the analysis of their behaviour and therefore with their dimensioning. In particular, it is currently problematic to define the number and type of reinforcements that must be used: simplified analysis methods introduce such large approximations that the results of the calculations are no longer reliable, while numerical methods, as it is necessary to use tri-dimensional ones, require very long calculation times and complex operations in order to be able to set up a model and to correctly interpret the obtained results.

This paper illustrates a new calculation procedure for the analysis and dimensioning of fibreglass reinforcements at the excavation face of surface mining and civil tunnels. This procedure is based on the limit-equilibrium method applied to the ground core ahead of the face. By evaluating the interaction between each reinforcement element and the surrounding ground in detail, it has been possible to determine the maximum static contribution that the reinforcements are able to develop.

2 STABILITY ANALYSIS OF THE EXCAVATION FACE USING THE LIMIT-EQUILIBRIUM METHOD (LEM)

The stability of an excavation face in a surface tunnel can be studied using the limit-equilibrium method (LEM), dividing the ground ahead of the face into two portions that are considered infinitely rigid and which can present relative displacements both between each other and with respect to the remaining part of the ground (the Horn mechanism) (Figure 1). For the sake of simplicity, the face section is approximated as being rectangular; the prism opposite the face, which is free to slide, can allow the upper parallelepiped to move vertically and produce the so-called “rise” effect, which has obvious repercussions on the ground surface. Indeed, in the case of shallow tunnels, excavation-face instability can lead to the formation of a subsidence basin on the ground surface. There have been many cases in which accidents have occurred, sometimes rather serious ones, concerning existing buildings on the surface, due to the impossibility of contrasting the face-instability mechanism.

The LEM is based on the following main hypotheses:

– the potentially unstable mass is represented by one or more monoliths that are considered infinitely rigid (undeformable), inside which failure cannot occur;
– the kinematics of the block occurs due to sliding on simple surfaces known a priori as far as the shape and dimensions are concerned;
– it is a static-type analysis, in that only the possibility of the initial displacement of the ground blocks is proved and the evolution of the potential instability phenomenon is not considered in any way whatsoever;
– the possibility of ‘progressive failure’ occurring along the sliding surfaces is not taken into consideration.

It is evident that the LEM is based on particular simplifications of the hypothesised instability mechanisms and therefore requires that the results should be considered in a particularly critical manner. The use of the LEM in the analysis of many instability mechanisms is, however, very common in the geotechnical and geomechanical fields, thanks to its simplicity, the intuitive nature of the approach and the possibility of evaluating the degree of stability through the safety factors.

In order to evaluate the stability condition of the face, it is necessary to define the resisting (limit equilibrium condition) and the active forces on the instable ground zone so that their ratio can be computed along the potentially feasible displacement direction. This implies a set of logical operations:

1) identification of the geometry of the possible unstable ground zones, varying the slope angle ϑ;
2) evaluation of their geometry (vertexes, volume and areas of the unstable ground zones);
3) computation of the resultants of the volume and surface forces acting on the unstable zones;
4) evaluation of the resisting forces;
5) static analysis, that is, computation of the safety factor or of the force that induces the limiting equilibrium condition.

In the specific case under examination (Figure 2), block 1 (a triangular prism) is enclosed by the planes 1, 2 and 3 and by the parallel planes a and d, on which the triangular bases rest. Plane 2 represents the sliding surface. Block 2 is instead enclosed by planes 3, 4 and 5, by the parallel planes b and e, and by the ground surface.

The vertical force V that block 2 applies to block 1 is given by the weight of block 2, by the possible pressure applied to the surface and by the strength that

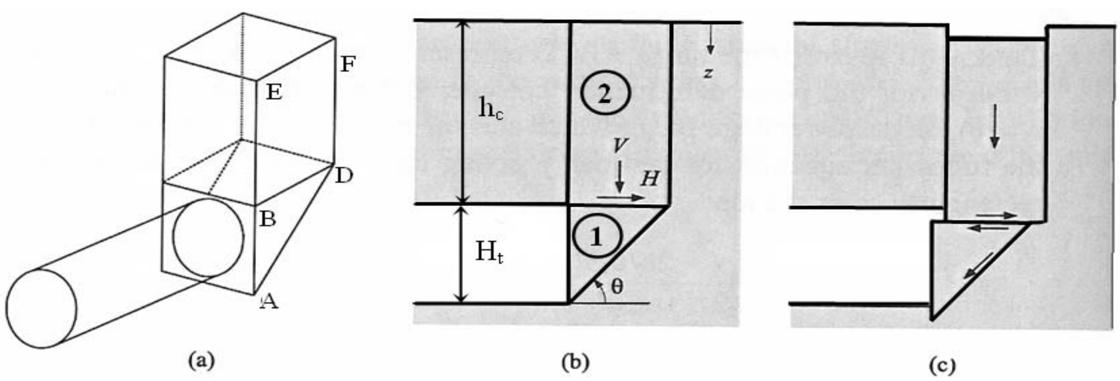

Figure 1. a) The Horn mechanism used for the analysis of the stability of the excavation face [3];b) longitudinal section with the two blocks of ground (block 1 is prismatic and block 2 parallelepiped) under the hypothesis of failure of the face in a surface tunnel; c) admissible kinematics, for the two identified blocks, during failure of the excavation face. Key: V and H: forces that block 2 applies to block 1; ϑ: angle that is formed between the sliding surface of block 1 and the horizontal plain; z: depth from the ground surface; H_t: height of the tunnel; h_c: depth of the tunnel crown from the ground surface.

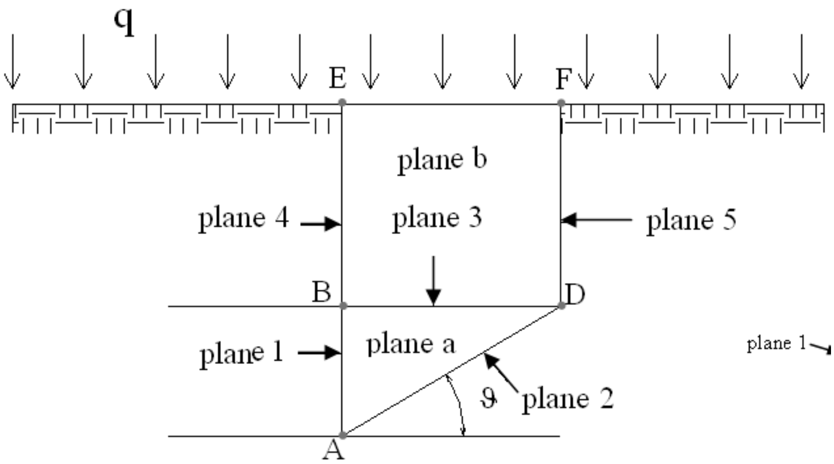

Figure 2. Longitudinal section of the zone close to the excavation face, with the scheme of the geometry adopted in the simplified analysis of the face stability.

can develop on the lateral surfaces of this block at the moment in which it tends to move downwards:

$$V = W_2 + q \cdot \left(\frac{B \cdot H_t}{\tan\vartheta}\right) - \left(c + \sigma'_{n,m,2} \cdot \tan\varphi\right) \cdot 2 \cdot \left(B + \frac{H_t}{\tan\vartheta}\right) \cdot h_c ;$$

$$V \geq 0 \qquad (1)$$

where: W_2: the weight of block 2: $W_2 = \gamma \cdot \left(\frac{B \cdot H_t}{\tan\vartheta}\right) \cdot h_c$

γ: the specific weight of the ground;
q: the load applied to the ground surface;
B and H_t: the width and height of the excavation face;
ϑ: the inclination of plane 2 with respect to the horizontal;
c and φ: cohesion and friction angle inside the ground, adopting the Mohr-Coulomb failure criterion;
$\sigma_{n,m,2}$: the mean normal effective stress on the lateral surfaces of block 2.

V is only considered when positive; if it is negative, it is made equal to 0 so as not to consider the possibility that block 1 is hanging from block 2.

The horizontal force H that block 2 applies to block 1 in correspondence to plane 3 is produced from the shear strength of the ground that can develop when a relative horizontal displacement between block 1 and block 2 occurs:

$$H = \left[V - u_3 \cdot \left(\frac{B \cdot H_t}{\tan\vartheta}\right)\right] \cdot \tan\varphi + c \cdot \left(\frac{B \cdot H_t}{\tan\vartheta}\right); \quad H \geq 0 \qquad (2)$$

where:

u_3: the mean pore pressure of the underground water in correspondence to plane 3.

$H = 0$ is also given for $H < 0$.

Block 1, in incipient movement conditions, is also subject to two other forces, due to the shear strength of the ground, which act in the direction of the displacement vector, but in an opposite versus to it: force R_2 on the sliding surface (eq.3) and force R_a on the lateral planes a (eq.3):

$$R_2 = \left[\left(W_1 + V\right) \cdot \cos\vartheta + \left(H - X_1\right) \cdot \sin\vartheta - U\right] \cdot \tan\varphi + c \cdot \left(\frac{B \cdot H_t}{\sin\vartheta}\right);$$

$$R_2 \geq 0 \qquad (3)$$

where:

W_1: the weight of block 1: $W_1 = \gamma \cdot \left(\frac{H_t^2}{2 \cdot \tan\vartheta}\right) \cdot B$

X_1: the horizontal filtration force in block 1, due to the movement of the underground water, if present, towards the excavation face;

U: the hydraulic under thrust force on the sliding surface:

$$U = u_2 \cdot \left(\frac{B \cdot H_t}{\sin\vartheta}\right); \qquad (4)$$

u_2: mean pressure of the underground water on plane 2.

$$R_a = \left(c + \sigma'_{n,m,ad} \cdot \tan\varphi\right) \cdot 2 \cdot \left(\frac{H_t^2}{2 \cdot \tan\vartheta}\right); \; R_a \geq 0 \qquad (5)$$

where:

$\sigma_{n,m,ad}$: the mean normal effective stress on the lateral surfaces a of block 1.

A more detailed study of the effect of the groundwater filtration on the static of the excavation face was developed by Oreste [4].

Once the forces acting on block 1 have been determined (W_1, V, H, R_2 , R_{ad} , and X_1), it is possible to determine the safety factor in the function of the angle ϑ:

$$F_{s,\vartheta} = \frac{R_2 + R_{ad} + H \cdot \cos\vartheta}{(W_1 + V) \cdot \sin\vartheta + X_1 \cdot \cos\vartheta} \qquad (6)$$

where the forces that oppose the sliding of block 1 appear in the numerator, i.e., the forces mobilized by the ground strength on surfaces 2 (sliding surface) and on planes a, and the component H parallel to plane 2; while the forces that tend to induce sliding, i.e., the components parallel to surface 2 of the forces W_1, V and X_1, appear in the denominator.

As the angle ϑ of the potential sliding surface is not known a priori, the minimum value of $F_{s,\vartheta}$ is assumed for ϑ variables from 0 to 90° as the safety factor F_s :

$$F_s = \min\left[F_{s,\vartheta}\right]_{\vartheta=0\div 90^\circ} \qquad (7)$$

3 FIBREGLASS-REINFORCEMENT SYSTEM AT THE EXCAVATION FACE

The reinforcement of the ground core at the excavation face with fibreglass elements in shallow tunnels has the main purpose of increasing the ground strength; this, as a consequence, leads to the stability of both the excavation face itself and of the ground surface, even when excavating large tunnels in poor or very poor ground (Figures 3 and 4).

The technique consists of inserting sub-horizontal fibreglass dowels into the core ahead of the excavation face. These dowels are connected in a continuous way to the surrounding ground through the injection of mortar in the boreholes; they therefore act in a traction and shear dominion and have no external constraint system. The reinforcement elements are then demolished during excavation.

The use of this reinforcement system has become very common in recent years with a tendency of advancing using a full face excavation even in difficult geotechnical conditions. Fibreglass elements (generally hollow bars with an external diameter of 60 mm and a thickness of about 20 mm or filled bars of various types) are characterised by high degrees of strength and low levels of specific weight, but also by a remarkable fragility that makes it possible to carry out the excavation with traditional ground-excavation machines, without any particular problems for the tools. The fibreglass pipes are produced with thermo-hardening polyester resin, reinforced with glass fibres, whose content in weight is higher than 50 %.

Figure 3. Example of face reinforcement in a shallow tunnel using longitudinal fibreglass pipes (Avigliana Tunnel, Turin, Italy): a) view of the face during the drilling stage; b) details of the reinforcement intervention with the pipes already in place.

The reinforcement elements are usually arranged on the excavation face in concentric circles, with a certain regularity, trying to maintain a constant density as the distance from the centre of the section changes.

The intervention is marked by a high flexibility (its characteristics can easily change during advancement without the necessity of having to change machines) and an operative simplicity. However, this intervention requires that the excavation operations should be stopped for a few days in large tunnels and in the presence of a high density of reinforcements.

The dimensioning of the intervention should be able to define the most important geometric parameters: the number, the length and the section of the elements that it is necessary to use to make the excavation face stable,

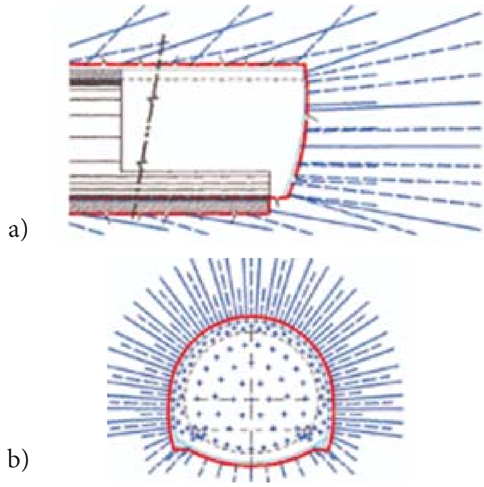

Figure 4. Typical scheme of a face-reinforcement intervention using longitudinal fibreglass elements: a) longitudinal section; b) transversal section [5]. A large number of elements, which are placed divergent from the tunnel axis, are often concentrated around the perimeter of the face to allow a crown of ground around the tunnel to be reinforced.

with an adequate safety margin. It is just as important to define the minimum residual length that the elements can have at the end of the advancement stage, before proceeding with the reinforcement of a subsequent tract.

4 SIMPLIFIED ANALYSIS METHODS TO EVALUATE THE ACTION OF REINFORCEMENTS

The reinforcement of the ground at the excavation face in surface tunnels is at present often dimensioned through simplified approaches. The number and the type of the reinforcement elements is defined in an empirical manner, in an analogous way to similar cases in which the intervention was performed successfully, or using simplified analytical formulas. Recourse to numerical calculations that are able to simulate in detail each reinforcement element and each construction stage of the tunnel is still rare outside research practice; a tri-dimensional numerical modelling with very small model elements in order to be able to precisely capture the reinforcement-ground interaction, and also the simulation of each single excavation and support stage of the tunnel are in fact required. All these aspects make an adequate numerical calculation complex and not suitable for the parametric analyses that are necessary to obtain a correct dimensioning of a reinforcement intervention.

Empirical methods are mainly based on acquired experience in the construction of structures with characteristics similar to those of the project or on experimental models set up in the laboratory, which reproduce the phenomenon under examination, though on a reduced scale.

An application of these methods, which is very common and has been well tested for structures both in rock and in ground, is that of the use of technical classifications which, however, consider the overall stability of the void and do not generally deal with the problem of the stability of the excavation face in a specific way.

Broms and Bennermark [6], on the basis of observations of real cases and laboratory tests that consisted in extruding clayey materials through a circular hole, revealed that the stability conditions of the excavation face are guaranteed if the stability index N_f is lower than 6-7, where N_f is equal to:

$$N_f = \frac{\sigma_v - p_f}{c_u} \qquad (8)$$

where:

σ_v: the vertical stress at the depth of the centre of the excavation face;
p_f: the pressure applied to the excavation face;
c_u: the undrained shear strength.

Kimura and Mair [7], on the basis of centrifuge tests on clay reconsolidated in the laboratory, obtained values of the N_f index between 5 and 10 for the condition of stability of the excavation face and were able to demonstrate the marked dependence of N_f on the depth.

The reinforcement elements should therefore be able to guarantee the development of a fictitious pressure p_f (and therefore of a force S that is equal to such a pressure multiplied by the excavation face area A_f) so as to obtain an index N_f that is lower than 6-7, with a certain margin of safety.

It is also possible to obtain the horizontal force S that is necessary to apply to the excavation face to obtain the required safety factor using the LEM. Once the desired value of F_s has been decided, the force H is back calculated from equations 3, 6 and 7 (H^*). S is then derived by the difference (H^*- H), that is the difference between the value of H obtained from the back analysis and the value of H calculated from eq. 2.

Once the force S is known through empirical laws or using the LEM, the reinforcement elements can be simply dimensioned, hypothesising that they perform their static function only by developing an axial force on their inside (a hypothesis of flexural stiffness nil of the reinforcement system).

This hypothesis makes it possible to limit the verification of the behaviour of the reinforcement elements to the following three inequalities:

$$\frac{S}{n} \leq \sigma_{adm} \cdot A_{bar} \qquad (9)$$

$$\frac{S}{n} \leq \tau_{adm} \cdot \left(\pi \cdot \phi_{hole} \cdot L_a\right) \qquad (10)$$

$$\frac{S}{n} \leq \tau_{adm} \cdot \left(\pi \cdot \phi_{hole} \cdot L_p\right) \qquad (11)$$

where:

σ_{adm}: the maximum allowed traction stress in the fibreglass;
τ_{adm}: the maximum allowed shear stress at the mortar-ground interface;
Φ_{hole}: the diameter of the hole;
L_a: the length of the dowel in prismatic block 1;
L_p: the length of the dowel in the stable portion of the ground;
n: the number of dowels foreseen at the excavation face;
A_{bar}: section area of each single element.

It is possible to define A_{bar}, L_a, L_p and n from equations 9-11, though not in a univocal manner.

5 DETAILED ANALYSIS OF THE GROUND-DOWEL INTERACTION

In order to perform an accurate analysis of the ground-dowels interaction, a new analytical formulation is presented in the following; it is able to provide a reasonable evaluation of the behaviour of a single dowel and it also allows a quick dimensioning of the reinforcement system.

The unknown factors are the global forces (axial N, shear T and bending M stresses) that develop along the dowels and which are functions of a small dislocation displacement of the potentially unstable block.

The design of the dowels can take place through a sequence of trials, assuming different reinforcement schemes, until the safety factor of the potentially unstable block at the excavation face is higher than the minimum allowed value.

After having defined the safety factor of the excavation face in its natural state (i.e., without reinforcement), according to the criteria reported in section 2, and having verified the needs of reinforcement to increase the safety factor, the main stages of the design can be summarised as follows:

a) definition of the chosen reinforcement scheme (number, position and dimensions of the dowels);
b) assignment of a value to the angle ϑ;
c) assignment of an arbitrary displacement δ to unstable block 1 along the sliding direction; evaluation of the components of such a displacement-vector acting in the normal δ_t and axial δ_n dowel directions (Figure 5);
d) based on δ_t and δ_n, evaluation of the shear force T, of the bending moment M, of the axial tensile force N and of the relative dowel-rock displacement v_r, induced along each dowel. In order to design the dowels and analyse the stability of the face, the values of T, N and M at the sliding surface (plan 2) (where the maximum values of the shear force and the traction axial force develop) are of particular importance;
e) calculation of the "local" safety factors for failure of the bar and of the dowel-ground connection, for each dowel, on the basis of the values of T, N and M and of the displacements v_r evaluated in d);
f) evaluation of the ratio η between each calculated local safety factor and the corresponding previously imposed minimum allowable value;
g) the minimum value of η for each local safety factor and each dowel at the face, multiplied by the arbitrary displacement δ applied to the block, represents the maximum displacement δ_{max} that the unstable block can sustain before at least one of the local safety factors drops below its minimum allowable value;
h) evaluation of the N and T forces in each single dowel at the sliding surface, for a displacement of the block equal to δ_{max}; such forces represent the maximum contribution that each dowel can offer to the stability of the block;
i) determination of the "global" safety factor of the unstable block 1, considering the contribution of the dowels, for the assigned value of ϑ;
j) repetition of steps b)-i), increasing the value of ϑ at each cycle until $\vartheta = 90°$; the "global" safety factor of the unstable block is obtained at each cycle for the assigned value of ϑ;
k) the minimum value of the obtained global safety factors is then considered as the safety factor of the reinforced excavation face; if this value is lower or much higher than the minimum allowable value, return to point a), change the reinforcement scheme and repeat the whole procedure until the reinforcement scheme that is suitable for stabilising the face is obtained.

The detail of the proposed mathematical procedure is developed in Oreste [4]. The global safety factor $F_{s,\vartheta}$ of the unstable block 1 must now be evaluated, taking into consideration the stabilising forces produced by each dowel (i=1÷n)

$$F_{s,\vartheta} = \frac{R_2 + R_{ad} + H \cdot \cos\vartheta + \left[\left(\sum_{i=1 \div n} N_{0,\delta_{max},i} \right) \cdot \cos\vartheta + \left(\sum_{i=1 \div n} T_{0,\delta_{max},i} \right) \cdot \sin\vartheta \right]}{(W_1 + V) \cdot \sin\vartheta + X_1 \cdot \cos\vartheta} \qquad (12)$$

where R_2 is now given by the following expression, which substitutes eq. 3:

$$R_2 = \left[\left(W_1 + V - \sum_{i=1 \div n} T_{0,\delta_{max},i} \right) \cdot \cos\vartheta + \left(H - X_1 + \sum_{i=1 \div n} N_{0,\delta_{max},i} \right) \cdot \sin\vartheta - U \right] \cdot \tan\varphi + c \cdot \left(\frac{B \cdot H_t}{\sin\vartheta} \right); \; R_2 \geq 0 \qquad (13)$$

where $N_{0,\delta max}$ and $T_{0,\delta max}$ are the axial force and shear force in the dowel at the dowel-sliding surface intersection when $\delta = \delta_{max}$.

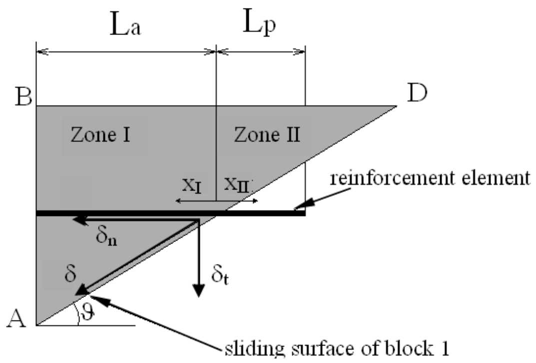

Figure 5. Breakdown of the displacement δ imposed on block 1. Key: δ displacement of block 1 along the sliding direction; δ_n and δ_t components of the displacement in the axial and normal directions of the dowel; L_a length of the dowel in the ground block 1 (zone I); L_p length of the remaining part of the dowel in the stable ground (zone II); x axial coordinate; ϑ: angle between the sliding direction and the horizontal plane; A, B, and D vertices of ground block 1.

The presence of the dowels obviously induces an increase in the safety-factor value in relation to the characteristics of the chosen reinforcement system.

The calculations developed in steps b)-i) are repeated for different values of ϑ, increasing it at each cycle, for example by 5°, until ϑ=90° is reached; at each cycle the "global" safety factor of the unstable block 1 is obtained for the assigned value of ϑ. The safety factor of the reinforced excavation face is therefore the minimum value of the obtained $F_{s,\vartheta}$.

If the increase in the safety factor of the face due to the reinforcement system is still not sufficient, or when it is considered excessive, it is necessary to modify the reinforcement scheme on the basis of the obtained results and to repeat the procedure from stage a) to stage k).

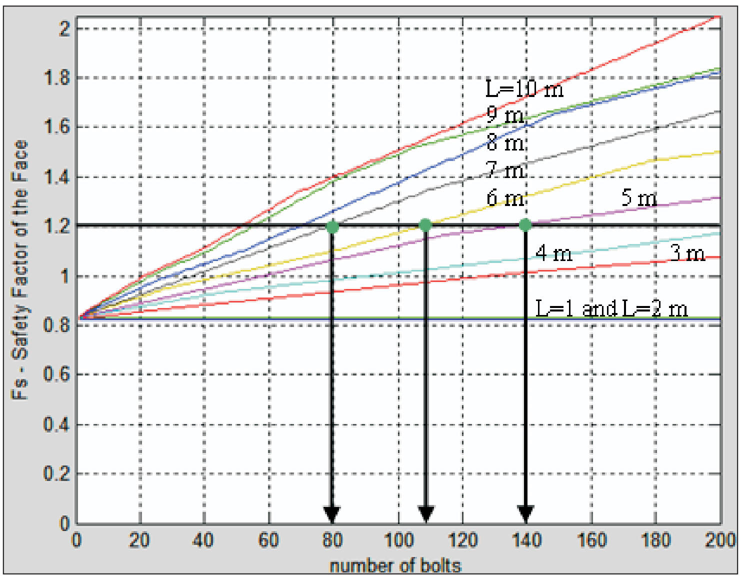

Figure 6. Example of the procedure for the determination of the number and the minimum length of the reinforcements at the face. Each line of the graph refers to a different length of the dowels at the excavation face

The dimensioning of the intervention is generally limited to a definition of the number of reinforcements and their minimal residual length. The total length of each reinforcement is subsequently identified by the maximum length that can be obtained without deviations of the hole. From the proposed procedure it is possible to obtain a summarizing graph (the results of a calculation example are shown in Figure 6) in which the safety factors of the face are reported with a variation in the number of reinforcements and their length (which should be intended as the minimum residual length).

After having fixed the desired safety factor (1.2 in the example of Figure 6), it is possible to define the necessary intervention scheme: 80 dowels with a minimum residual length of 7 m, or 110 dowels with a minimum residual length of 6 m, or even 140 dowels with a minimum residual length of 5 m. The total length of the reinforcements does not usually exceed 22 m.

Two real cases, both of which occurred in the North-West of Italy, in which failure of the excavation face occurred in sandy quartzite grounds of medium to coarse size, were studied by applying the proposed calculation method. The examined tunnels, of polycentric shape and areas of about 104 m^2, belong to the same zone in the Province of Biella (Italy). In both cases, loose sand formations (arkose sands) derived from the decay of the rocky granite substratum were being crossed. The reinforcement scheme at the excavation face in both cases foresaw the use of 40 injected fibreglass pipes with a total length of 14 m, 5 m of which were overlapping. The hole diameter (Φ_{hole}) was 150 mm. The tunnels were full section excavated.

In the first case a sliding of the face occurred during the excavation in correspondence to the second steel set after having realised the reinforcement phase (the residual length of the dowels at the face was therefore 12 m), when the overburden was about 10 m.

In the second case the collapse occurred when the residual length of the dowels at the face was 10 m. The overburden on the crown, at the moment of the failure, was about 5 m.

From the results obtained using the proposed calculation method it was possible to see how the safety factor of the excavation face in both cases is slightly lower than unity and this justifies the failures that occurred. These results make it possible to confirm the causes of the previously identified events (insufficient drainage system and an imperfect construction of the pre-support structure) and also to validate the proposed calculation procedure.

6 CONCLUSIONS

The problem of excavation-face stability in surface tunnels in poor ground currently represents one of the most interesting challenges in the tunnel sector. Cases in which the excavation faces collapse in spite of the fact that they have previously been reinforced with fibreglass dowels are in fact not so rare. A new calculation procedure for the analysis of reinforcement interventions using fibreglass dowels at the excavation face in surface tunnels has been illustrated in this paper. The procedure, which is based on the limit-equilibrium method applied to the ground core ahead of the excavation face, is able to evaluate, in detail, the interaction between each reinforcement element and the surrounding ground and it permits the maximum static contribution that each reinforcement element is able to give to the stability of the face to be determined.

The calculation procedure has been applied to two real cases of tunnel-face collapse in the presence of fibreglass reinforcement intervention and the results show a safety factor just below unity for both cases, which is in agreement with the events that have occurred. This has made it possible, on the one hand, to consider the presented procedure reliable, and on the other, to confirm the hypotheses initially advanced concerning the causes of the collapses.

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