An Approach for Selection and Design of Rock Bolting Systems

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ABSTRACT: This paper considers rock bolting characteristics resulting from different roof bolting principles and their influence on rock mass stability in underground excavations. The effectiveness of each bolting principle in regard to geotechnical data is analyzed. An approach is developed for designing frictional rock bolting systems, taking into account the rock mass properties.

! INTRODUCTION

In the present paper, an attempt is made, on the basis of the differences between the characteristics of conventional and frictional rock bolts, to identify an appropriate method for designing a rock bolt system with frictional pipe anchors. Different design methods are known for conventional rock bolts, whereas for frictional rock bolts, rational methods expressing the characteristics of this new class of anchors have not yet been developed. Current practice in this area is based mainly on known schemata for designing rock bolting systems with conventional anchors. Therefore, the authors focused mainly on rock masses susceptible to developing plastic zones, where frictional pipe anchors, in contrast to conventional ones, also perform well.

2 ROCK BOLT CHARACTERISTICS

Resistance, which exercises certain anchor types to the rock displacement, can be defined best by means of its working characteristics. Most scientists have replaced the working characteristic of the rock bolt with the initial pull test characteristic obtained immediately after rock bolt installation. This type of characteristic usually occurs with increased stiffness and because of this it is called the initial pull test characteristic. For the purpose of defining the characteristics of several types of expansion rock bolts in different mines and geological conditions in Bulgaria, numerous long-term studies have feeen carried out, both to determine the reaction of the anchor in time with magnetostriction dynamometers and to measure the displacement of me anchor end.

The type of the characteristic is defined only after statistical processing of a large quantity of data. In investigations carried out by Nikolaev (1964, 1989), a relation was sought between the initial pull test and working characteristics of expansion rock bolts with retensioning. The results processed show that for most cases, the curve of these rock bolts is a square parabola, but this is according to observations started after the initial anchor tension, which is linear. The parabola begins from one certain value $P_0 = C_0$. In Figure 1 two example curves of pull test characteristics are illustrated as follows:

$$P_i = a_0 u + b_0 u + c_0 \tag{1}$$

where Pi = the tension force of the anchor; u = displacement of the anchor end during tensioning; a_0, b_0, C_0 = coefficients of the pull test characteristic, differing for the individual expansion types of rock bolts.
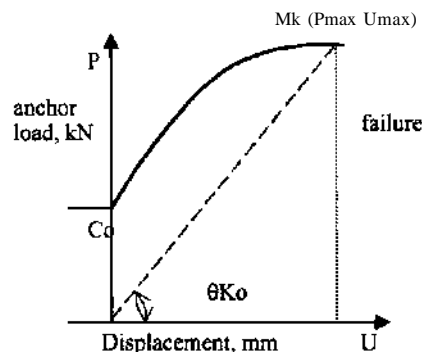

Figure 1. Initial pull test charac(eristic of expansion shell rock bolt.

In Figure 1, a typical pull test characteristic of an expansion rock bolt is given, ending with anchor rod failure or with slipping.

The observations show that in rock bolts with a wedge expansion device, the initial anchor tension is not maintained for a long time. After a relatively short time (from 2 to 10 h), the force P, begins to decrease to a certain value. After this, by appropriate density and length of the rock bolts for each individual case, an increase in the rock bolt reaction can be observed, followed by another decrease in the tension force and another increase until a certain value of anchor reaction is reached. In this case, the rock mass displacement is stopped and force P_t remains almost constant if the mine and geological conditions are not changed. After translation of the beginning of the coordinate system P- U in point Co, the equation of the working characteristic is simplified as follows:

$$P_k = a_k u_k^2 + b_k u_k^2 \qquad (2)$$

This is illustrated graphically in Figure 2. The pull test characteristic of the rock bolt is shown by Pk, and a broken line, while P_{pi} and a continuous line show the working characteristic.

We must emphasise that the decrease from the pull test characteristic to the working characteristic of the rock bolts with the wedge expansion device in each step is in the same proportion, namely:
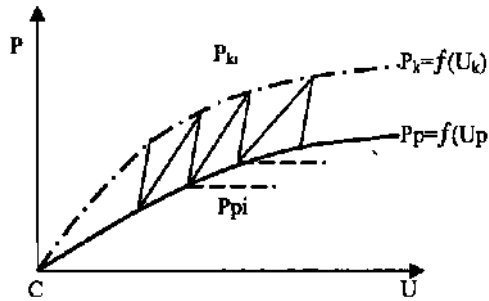

Figure 2 Relationship between initial pull test and working characteristics of expansion shell rock bolts.

$$(P_{ki} - P_{pi})/P_{ki} = \Delta_h \qquad (3)$$

With some approximation we assume that the decrease from Pt, to P_p, is at right angles with abscise u. The maximum derivation of 100 observations carried out was 8%.

With some margin of safety, we can assume for the working characteristic that the parabola passes through the down points of the steps (curve P_{pi}). From Equation 3, we can express the working char-

acteristic by multiplying the pull test characteristic by the constant q:

$$P_p = q P_k = q a_k u_k^2 + b_k u_k^2 = a_p u_p^2 + b_p u_p^2 \qquad (4)$$

where q depends on the ratio between the plastic and Theological components in the strength characteristic of the rock masses.

The characteristic of the ordinary wedge anchor is that it has great stiffness and it exhibits almost a straight line. The initial expansion force of the wedge head is constant and cannot be increased during its lifespan. In Figure 3, the characteristic of this type of rock bolt is illustrated graphically. The stiffness of this bolt is exceptionally high, and the values of Pk and P_p are two to four times lower than those for the expansion anchors, with increasing expansion of the anchor head. The value of P'mа* increased in the range of 60 to 80% only in the massive with uni-axial compressive strength of about 30 to 40 MPa, but with substantial increase in the stiffness.
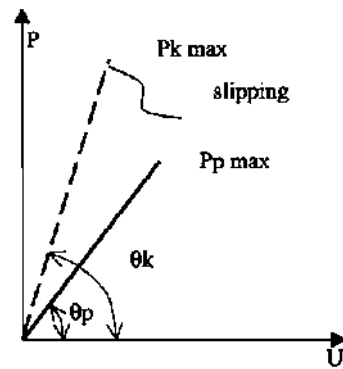

Figure 3. Pull test and working characteristics of expansion wedge rock boh

The characteristic of the cement-grouted rock bolt is that it has infinitely high stiffness, i.e., with an angle of inclination of almost 90°. In this case, the pull test characteristic differs from the working characteristic in that tension acts directly on the anchor rod. With good design and implementation, P_p mх has a high value.

For the frictional pipe anchor TFA, the pull test characteristic consisted of two lines connected by a small curvilinear section. This section expresses the relation between the first part with increasing resistance and the second part with constant resistance. The stiffness is lower than in the other types of anchor (Fig. 4), which ensures a certain amount of yield without decreasing the initial resistance. The upper branch of the curve illustrates the working characteristic of TFA. The increase of the installing

load-bearing capacity of the anchor marked by AP_P depends on physicomechanical and structural characteristics. The greater the necessity of rock mass anchoring is, the bigger the force is that clamps and presses the rock bolt, i.e. the bigger AP_P is.

In Figure 5, a characteristic curve of interaction for anchor support-rock mass is shown. The stiffness and the time of anchor installation are of particular importance in competent rock structures in the high-stress conditions existing in deep mines and tunnels. The installation of bolt support with rigid characteristics before the relaxation of stresses and passing of the initial deformations might be dangerous because of the risk of overloading this type of anchor.
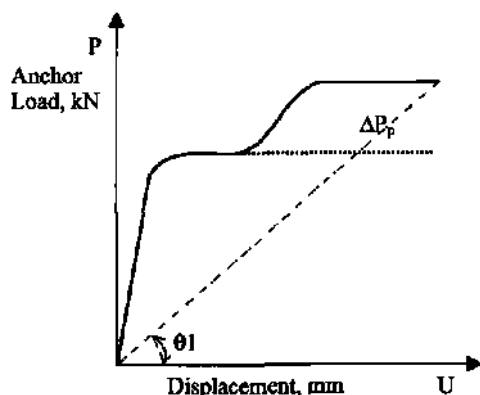

Figure 4. Initial pull test and wn<V-»» characteristics of factional pipe anchor TFA.

3 THEORETICAL MODEL FOR INCREASING THE BEARING CAPACITY OF TFA

During installation of TFA in an anchor hole with a diameter smaller than its elliptical cross section, the elastic deformation causes normal stresses toward the hole wall. The friction strength T induced determines the initial bearing capacity of the rock bolt Po as follows:

$$P_0 = T_0 = N_0\,\mu \qquad (5)$$

where μ = coefficient of the friction rock-metal; and No = normal stresses caused by elastic deformation.

Immediately after installation or after a short rime, the pull test shows identical results for Po, while for a longer period it is increased, which in any case can reach values which are twice as high as the initial installing stress. This increase can be explained by the process of deformation in the rock mass caused by plasticity or plastic flow. This effect can be described approximately by assuming that the rock mass is an elasto-plastic medium.
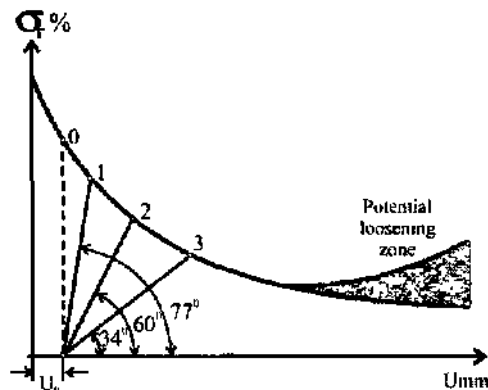

Figure.5. Interaction curve: anchor support-rock mass, where 1 = stiffness of expansion wedge rock bolt; 2 = stiffness of expansion shell bolt, 3 = stiffness of fractional anchor TFA.

A circular mining excavation driven to a certain depth is considered, ensuring the development of a plastic ring around the excavation. The plastic area causes additional stresses around the rock bolt and it is assumed that the compressive stresses have a positive sign.

The increased bearing capacity can be approximately described by Equation 6:

$$P_a = P_0 + \sigma_{\theta p}\,\mu S_k\,K_p \qquad (6)$$

where P_B = increased bearing capacity; $\sigma_{\theta p}$ = tangential stress in the plastic area around the <u>mining</u> excavation; S_t = contact zone of the anchor with the rock on 1 meter length; μ = coefficient of the friction rock-metal; $Kp = Lp/Ro$ - relative thickness of the plastic area; and Lp, Ro = thickness of the plastic area and radius of the excavation.

The tangential stress $\sigma_{\theta p}$ can be expressed as shown in Equation 7.

$$y = \sigma_{\theta p}/p = (1 + K_p)^\alpha\,\beta + W_k\,ctg\varphi\,[(1 + K_p)^\alpha\,\beta - 1] \quad (7)$$

$$W_k = C_{tp}/p;\ \ \alpha = 2\sin\varphi/1\text{-}\sin\varphi\ ;\ \ \beta = 1 + \sin\varphi/1\text{-}\sin\varphi$$

where Ctp = cohesion in the plastic area.

Equation 7 İs worked out for the stresses in the plastic area with cohesion and angle of internal friction. Using a solution proposed by Bulichev (1994), which takes into account the influence of the support resistance p, the resistance of the support on the contour of the excavation can be expressed by the equation:

$$p = \delta\,n_a\ ;\ p_0 = 0.4ba\,n_a\,p_0/\,Ro \qquad (8)$$

325

where n_a = the number of rock bolts per square meter; S = coefficient of correction, which expresses the irregular loading of the anchors; and b_a = dimension of die square anchor plate;

If we substitute $CTap = yp$ İn Equation 6 and Equation 8, after the transformations we obtain die expression for increasing the bearing capacity as a percentage:

$$C\% = 100 \ (P_a / P_o - 1) = 2.55 \ 10^{-2} \ n_a \ y \ K_p \qquad (9)$$

Using die conditions for plasticity, we can express the relations between die radius of die plastic area, hydrostatic stress, cohesion and die angle of internal friction. In a manner appropriate to our task, we transform dial Equation 9 as follows:

$$\gamma H / C_{kp} = [(1+K_p)^\alpha \cdot (1-\sin\varphi)] / [tg\varphi(1-\sin\varphi)] \qquad (10)$$

4 DESIGN OF ROCK BOLTING SYSTEM WITH TFA

The interaction of the frictional pipe anchor (TFA) with the rock mass is fundamentally different from mat of other anchor types.

The strength elastic contact, which occurs along die whole length of die rock mass, allows influence on the deformation processes in the rock mass.

Over twenty years' experience with TFA shows that it plays a successful part not only in the deformation processes along the axis, but also in those at die cross-section of the axis, sometimes even with increased bearing capacity. These advantages of TFA even allow rocks to be reinforced in mining excavations driven İn mining and geological conditions susceptible to developing well-formed plastic areas.

This advantage of TFA allows the replacement of other types of support with anchor supports in old mining excavations subject to great deformations, or it allows the support to be reinforced by anchoring of the area between the usual standing supports.

It is well known that conventional rock bolts are used for suspending the plastic area to the solid rock.

The metfiod suggested here for designing the rock bolting system is based on die idea of taking into account the increase in die cohesion value in the plastic area; first, from the direct action of die side strength contact, and second, from the component acting İn the cross-section of the axis (between four rock bolts).

The increased cohesion in the plastic area $Cj^\wedge$ is expressed as follows:

$$C^*_{kp} = (l_p / l_a) \ C_{kp} \ K_{1p} + (l_e / l_a) \ C_{kp} \ K_{2p} \qquad (11)$$

where l_a, l_p and I_e are the total (active) length of the anchor, the length of the anchor İn the plastic area, and the length of the anchor in the elastic area respectively. All these lengths are relative because they are expressed in the ratio to radius of the excavation. C_{kp} = cohesion in the plastic area; K_{1p}, K_{2p} = coefficients of reinforcement resulting from die direct action of the cross-section contact stresses (from the mounting of the anchor and deformation process in the rock mass), and the reinforcement coefficient from the action of the side component of a pyramidal body weight, and also the coefficient taking into account the reaction of the anchoring forces.

The investigations carried out by Parushev (1986) with TFA frictional rock bolts İn a non-fractured medium by modeling in an equivalent material defined the coefficient K_{1p} by the equation:

$$K_{1p} = [(n_a + 6,3)^2 / 24] - 0,87 \qquad (12)$$

The coefficient K_{2p} by S.Nikolaev (1994) is expressed by the formula:

$$K_{2p} = [(1+\sin\varphi) \ p_m \ n_a] / (1-\sin\varphi)\gamma \qquad (13)$$

where p_m is the reaction of the rock bolt for the linear meter length, MN/m.

After replacing l_p and l_e with $l_p = Kp$ and $l_e = I_a - K_p$ and dividing the expression to C_{kp}, we obtain the coefficient of the reinforcement n, which after transformations is:

$$n = \delta \ K_{1p} + l_a \ (1-\delta) \ K_{2p} \qquad (14)$$

where $\delta = K_p / l_a$

As we know, the strength indices of the rocks at a certain point of the massive have the character of random values with a definite law of distribution. The great experimental experience of both Bulgarian and other researchers show that with enough accuracy for practical use, we can assume the normal Gauss distribution.

With this precondition, the probability of failure can be described by a function of Laplace as follows:

$$V = 0.5 - 1/\sqrt{2\pi} \int_0^z e^{-x^2/2} dx \qquad (15)$$

The upper limit of the integral is:

$$z = (1-1/n)/v \qquad (16)$$

where v is the coefficient of variation, which in most cases exhibits the values: $20\% \leq v \leq 30\%$.

Assuming an upper limit of v = 30%, for z we obtain: z = 3.33(1-1/n)

The coefficient of failure probability in mining excavations must not exceed V<0.02

Replacing these requirements in (15), we found the nearest solution for z = 2.06

For this value, V = 0.0197 < 0.02

For the found z, the requested value of n = 2.618

Replacing n with this value İn (14), and finding the solution of the equation for l_a we obtain:

$$U = (2.618 - \delta \, K_{1p})/(1-\delta) \, K_{2p} \qquad (17)$$

where practical values for S and 1 a are:

$0 < S \ll 1$ i.e. $5 > 1$
$RoU > 1$ and $l_a < 1.3$

The calculation data for design are obtained by laboratory-strength indices transformed by methods taking into account structural disturbances.

Using the method of Hoek-Brown described by Hoek (1996) for compressive strength, we obtain the following:

$$\sigma_{CM} = \sigma_{cl} \, S^a \qquad (18)$$

and using the strength theory of Mohr-Coloumb, the analogical equation for cohesion İs

$$C_{KM} = C_{kl} \, S^a \qquad (19)$$

where $a^\wedge$ and Ck_M = transformed compressive strength and cohesion in rock mass respectively; and Gci and Citi = laboratory compressive strength and cohesion respectively.

Example: circular mining excavation at depth H = 350m; radius Ro= 2m; density of rock $y = 0,022MN/m^3$; laboratory strength indices: uniaxial compression strength a_ci - 48Mpa; cohesion $C^\wedge$ = 16.8 Mpa; angle of internal friction (p = 20°.

By Hoek (1996) we obtain S = 0.015 and a = 0.5. From Equations (18) and (19) we can calculate the strength indices in the rock mass c_{CM} = 5.88 MPa and Cii_M = 2.06 MPa. Using relation (10) expressed toward Kp and replacing the indices above, it İs calculated that Kp= 0.53. The distance between the rock bolts is determined by the Equation 20:

$$a = 2hf + 2.5b_a = 2hcos\varphi \, / \, (1-sin\varphi) + 2.5b_a = 2.856h + 0.375 \qquad 20$$

where h = the height of the equilibrium volume between the anchors. For the mining excavations we can assume h = 0.2m; f = coefficient of hardness by Protodiakonov. For masses with f < 6, the coeffi-

cient can be calculated by the trigonometric expression f = costp / (l-sinif>); b_a = dimension of the square anchor plate. In this case, b_a = 0.15m

Then we calculate a = 0.946m = 1.00m.

From the illustrations in Figure 6, we can read the value of l_a. If Kp= 0.53 and n_a= 1 piece/m² then l_a= 1.025 or for the active length of the rock bolt we have L,,= 1.025, Ro = 2 05m, and the total length of the rock bolt is: $L_{101a}i = L_a + 0.10 = 2.15m$
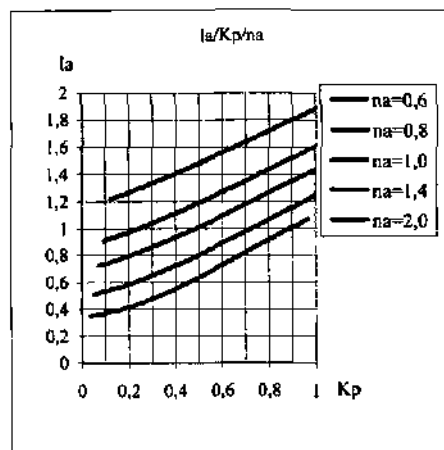

Figure 6 Relationships between thickness of the plastic zone Kp, anchor length 1. and density of anchor distribution $n,$.

5 CONCLUSION

The theoretical, laboratory and in-situ investigations, as well as the analyses of the results found allow us to draw the following conclusions:

1. The working - long-lasting characteristic of a relevant type of rock bolt should be taken into account when a rock bolnng system is designed.

2. The real increase in the load-beanng capacity of TFA is proved both by pull tests and theoretically.

3. Frictional pipe anchors including TFA are also effective in mining excavations surrounded by plastic zones.

4. An approach developed allows frictional rock bolting systems to be designed by taking into account the specific properties of this class of anchors by the availability of plastic zones

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