Estimation of shaft radial displacement beyond the excavation bottom before installation of permanent lining in nondilatant weak rocks with a Novel formulation

Estimation of shaft radial displacement beyond the excavation bottom prior to the installation of permanent lining in non-dilatant weak rocks with a novel formulation

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Abstract

The Convergence-Confinement Method (CCM) applies to circular tunnels in an in situ stress field in which all three principal stresses are equal and where the rock mass exhibits elasto-perfectly plastic shear failure. As the radial wall displacement cannot be easily obtained by using analytical methods, an extensive parametric analysis of the bi-dimensional numerical modelling in order to investigate the strain of the shaft wall close to the excavation bottom was performed. 81 cases were derived from the combination of the geometrical parameters and three weak rock categories. By processing the data relating to u_{R0} (radial displacement of the shaft wall at the excavation bottom) values obtained by numerical calculation in the different cases studied, it was possible to calculate the u_{R0}/R ratio as a function of the lithostatic stress p_0, the lining thickness s, and the shaft radius R. Novel equations were obtained for quickly estimating the value of u_{R0} knowing the lining concrete thickness, the shaft depth and the shaft radius, for the different qualities of rock considered.

Key words: non-dilatant weak rocks; shaft; numerical modelling; Convergence-Confinement Method

Introduction

The choice of the lining system depends on the costs, ground conditions, contractor's preference and the construction method. Shaft linings can be installed by using the underpinning method (if the rock can stand unsupported, see Fig. 1), by using the caisson-sinking method (in the case where the vertical excavated face is difficult to achieve), by using diaphragm walls, bored piles, or raise-boring (British Tunnelling Society 2004).

Typical shaft lining materials are steel, concrete, fiberglass and corrugated metal (Henn 2003). With regard to the shaft installation, weak rocks (such as schists, shales, tuff, marl, for instance) are often the biggest challenge for site investigations, design, and construction (Peck and Lee 2007; Spagnoli et al. 2016) because of their poor mechanical properties. Deep shafts excavated in rocks with poor geomechanical properties need to have a suitable lining design in order to guarantee the stability of the walls (e.g. Jia et al. 2013). The calculation of the lining thickness for circular shafts is based on the assumption that the pressure on the contact rock-lining is known (Öztürk and Ünal 2001). This pressure can be calculated analytically assuming a state of hydrostatic stress, considering a failure criterion and determining the pressure of internal support that will prevent the rupture zone, which develops around the shaft (Öztürk and Ünal 2001). There are several methods to design shaft linings:

- Analytical methods are the deterministic solutions of closed form, such as the Convergence-Confinement Method (CCM) described by Wong and Kaiser (1988), Hoek and Brown (1980), Panet (1995), which is also the most used analytical method.
- Empirical methods use equations based on different mines with respect to the laboratory and in-situ test results. All systems have quantitative estimation of the rock mass quality linked with empirical design rules to estimate adequate rock support measures, such as rock bolt, shotcrete and steel set (e.g. Barton, et al. 1974; Bieniawski 1989; Hoek and Brown 1980).
- Numerical methods which consider nonlinear analysis, anisotropy and discontinuities of the rock mass, complicated geometry of the problem and troublesome geological profile (e.g. Fabich et al. 2015).

The chance of using the bi-dimensional axisymmetric numerical modelling further simplifies the implementation of the model and reduces the calculation time. However, the simulation of the excavation process and the installation of the support for a certain shaft section (about 10 times the diameter), is a time-consuming process. The stress and strain states both in the rock and in the support structure, when the shaft reaches the central part of the model, are analyzed. As the variation of the excavation step barely influences the final result (Oreste et al. 2016), for the sake of simplicity, exaggerated excavation step (higher than those observed in the reality) can be considered, in order to reduce the calculation time. Due to the complexity of the lining/rock interaction, the design procedure needs iterative steps: a support structure is assumed (with certain dimensions and rigidity of the material) and the numerical calculation is performed. After having analyzed the results, it is possible to see whether the support structure previously hypothesized is correct or a further design step is needed. If the results give an excessive applied pressure to the shaft wall, or the support structure undergoes less stresses than the lining was designed for, the successive hypothesis must consider a less robust support structure, and vice versa. The progressive fine-tuning of the support structure can require several numerical models and the process can be time-consuming. For this reason, analytical methods are often used. They allow a simplified and quick analysis of the interaction between the shaft lining and the surrounding rocks, and they can give rather precise results regarding the interaction problem. Numerical modelling is, therefore, only used to verify the results in detail. The most used analytical method is the CCM. This research considers the CCM as a tool to predict the radial ground displacements and the formation pressure on a shaft. As the radial wall displacement, u_{R0}, cannot be easily obtained by using analytical methods, an extensive parametric analysis of the bi-dimensional numerical modelling developed in this research is able to detect the support structure influence from the radial displacement at the lateral shaft contour. This allows to correctly position the reaction line on the convergence-confinement curve (CCC) of the circular cavity as a function of the lithostatic stress p_0, the lining thickness s and shaft radius R, for (non-dilatant) weak rocks categorized as with poor, medium and good qualities.

The convergence-confinement method for designing support structures

The CCM, usually applied to tunnels (e.g. Duncan-Fama 1993; Nguyen-Minh and Guo 1996; Oreste 2003, 2014; Carranza-Torres and Fairhurst 2000), has been proposed by Wong and Kaiser (1988) as a tool to predict the radial ground displacements and the formation pressure on a shaft (Fig. 2). The link between the radial stresses and the displacements of the lateral shaft surface is represented by the convergence-confinement curve, which is very important for analyzing the interaction between the rock and the lining (Spagnoli et al. 2016). The CCM has also been validated for real underground structures (e.g. Kitagawa et al. 1991; Mariee et al. 2009; Svoboda and Masin 2010). This method is based on the assumption that the rock at the shaft bottom provides an initial support pressure equal to the in situ stress p_o. As the shaft excavation advances and the bottom moves away from the section under consideration, the support pressure gradually decreases until it reaches zero at some distance behind the shaft (Hoek et al. 2008). The extent of the plastic zone can also be estimated by this method at each stage of the process (Wong and Kaiser 1988; Vlachopoulos and Diederichs 2009) by controlling the internal support pressure p_i, applied by the linings (Hoek et al. 2008). The CCM allows an analysis considering the interaction between the pressure applied to the circular shaft wall and the corresponding radial displacements. The method considers the following hypotheses:

1. Circular and deep shaft;
2. In situ homogeneous and isotropic rock around the shaft with ideal (or brittle) post-failure behavior;
3. Constant and isotropic lithostatic stress p_0 (K_0 = 1) around the shaft (e.g. Oreste 2009).

The assumptions considered above, are common during the construction of large circular shafts at great depths. This simplifies the derivation of stress and strain developing at the shaft rock contour. In the case the Mohr-Coulomb failure criterion is applied, the CCC is obtained as a closed-formed solution, and it is a function of:

- Elastic parameters (elastic modulus and Poisson’s ratio);

- Strength parameters (cohesion and friction angle);
- Strain parameters in plastic field (dilatancy angle);
- Geometric parameters (shaft radius and depth, the latter giving the lithostatic stress state).

In the case a curvilinear failure criterion (typical of rocks) is employed, such as the one of Hoek and Brown (1980), the solution is no longer a closed-form one. It is rather a finite difference numerical solution necessary (Oreste 2014). In this case, instead of cohesion and friction angle, Hoek and Brown's m and s parameters as well as the compressive strength of the intact rock, σ_c, have to be considered. The final result is nonetheless the CCC, generally representing a first linear trend (for internal pressure between the so-called critical value, p_{cr}, and the virgin in situ lithostatic stress, p_0), which changes to a curvilinear path (for internal pressure between 0 and p_{cr}) with a downwards concavity (Carranza-Torres and Fairhurst 2000). In order to consider an interaction mechanism between the rock and the lining, it is necessary to intersect the CCC of the shaft and the reaction line of the support. The reaction line represents the relation between the applied pressure to the support and the corresponding radial displacement of the tunnel wall. Assuming an elastic lining behavior, the relation is a linear one. The reaction line has a slope depending on the lining stiffness.

Another parameter needed to correctly position the reaction line on the CCC is the radial wall displacement, u_{R0}, at the point where the lining is installed. Generally speaking, support structures are installed in proximity of the excavation face (i.e. the temporary shaft bottom). Therefore, u_{R0} coincides with the radial shaft wall displacement in correspondence with the temporary excavation bottom. This parameter cannot be easily obtained by using analytical methods. Vlachoupulos and Diederichs (2009) could estimate, by means of the numerical modelling, the variation of the radial wall displacement of the circular cavity by varying the distance from the excavation face, in the absence of support lining and for different rock types. The equations provided by Vlachoupulos and Diederichs (2009) allow to preliminary estimate the radial wall displacement at the excavation face, as a function of the final wall displacement. Therefore, in order to use the CCC for designing the support structure, the equations of Vlachoupulos and Diederichs (2009) require an iterative procedure, which quickly converges. The final wall displacement depends on the radial displacement at the excavation face and, therefore, only after having defined the latter, it is possible to determine the wall displacement.

Through this procedure, it is possible to obtain an estimation of the radial shaft wall displacement at the excavation face, and therefore to correctly position the reaction line on the CCC. In this way, the intersection point, which gives the final applied pressure on the rock by the lining (p_{eq}), is obtained. By knowing p_{eq}, it is possible to verify the suitability of the hypothesized support structure. By means of the CCC, the design iterative procedure can be quicker. Based on recent numerical analysis (Oreste et al.2016), the use of the CCC method combined the equations of Vlachoupulos and Diederichs (2009), in order to define the starting point of the reaction line, leads to a non-exact evaluation of the final pressure acting on the lining (Oreste et al. 2016). The equations of Vlachoupulos and Diederichs (2009), originally obtained in absence of supporting structure, cannot be used for positioning in a reliable way the reaction line on the CCC of the shaft. For this reason, this research shows an extensive parametric analysis of the bi-dimensional numerical modelling in order to investigate the displacement of the shaft wall close to the excavation bottom. The findings of this parametric analysis may be useful for a proper design by using the CCC procedure.

Numerical modelling

The numerical modelling for this research was developed with the bi-dimensional explicit finite difference program Flac 2D v.6.0 (Itasca 2008) used in the axisymmetric configuration. The model analyzed the stress and strain state developing in the rock and in the linings during the construction phases. The study considered a circular vertical shaft with a support structure of concrete. The lithostatic stress state was hypothesized, for the sake of simplicity, as homogeneous, i.e. the horizontal stress is constant independent of the direction. This simplified assumption permits to accelerate the numerical calculation with a bi-dimensional method in the axial-symmetric configuration. The following assumptions were also made (already considered by Spagnoli et al. 2016):

- The failure criterion adopted for the rock is the linear Mohr-Coulomb, generally adopted for weak rocks, for which the curvature of the failure criterion is less accentuated;
- The residual conditions were considered equal to those of the peak, assuming, therefore, an ideal elasto-plastic behavior of the rock in the phase of post-failure;
- The elastic modulus was considered for simplicity constant both in the elastic and plastic phase;
- The dilatancy angle, ψ, which describes the strains behavior in the plastic range, has been considered as equal to zero, assuming plastic strains at constant volume, as described by Hoek and Brown (1997), Alejano and Alonso (2005) and Alejano et al. (2010) in the case of deep rocks with poor mechanical properties;
- The lining was considered to be composed of concrete, with typical values of elastic modulus (30,000MPa) and Poisson's ratio (0.15);
- The horizontal lithostatic stress was considered equal in the two different directions: this condition is generally taken to the great depths to which the variability of the lithostatic stress in the three directions of the space is drastically reduced.

The parametric analysis considered, within the weak rock types, three different categories: rock with poor mechanical properties (type A with cohesion 0.3MPa, friction angle 25° and elastic modulus of 4,000MPa), medium mechanical properties (type B with cohesion 0.9MPa, friction angle 31° and elastic modulus of 8,000MPa) and good mechanical properties (type C with cohesion 1.5MPa, friction angle 35° and elastic modulus of 12,000MPa). For each of these categories, 27 numerical models were developed, considering different combinations, which are possible to obtain by changing the following geometrical parameters:

- Shaft radius, R: 1, 3 and 5m;
- Concrete lining thicknesses: 0.1, 0.2 and 0.3m;
- Virgin horizontal in situ stress state (p_0): 15, 30 and 45MPa.

The purpose of the geomechanical parameters described above is to simulate rock with poor mechanical properties such as shale, coal, rock salt, to name a few (Waltham 2009), where shafts are constructed, as for instance the 600m deep shaft to be used for in situ retorting at the Occidental Petroleum and Tenneco oil shale mine in Rifle (Colorado) or for the Lake Huron mine (Canada) in rock salt.

The numerical model considers about 36,000 quadrilateral elements, employed for representing both the rock and the concrete constituting the support structure.

For each of these numerical models, the excavation process and the support installation were simulated. The simulation started from the upper edge of the model until reaching the central position, for a section corresponding to 8-10 times the shaft radius. Fig. 3 shows a detail of the model for the case with shaft radius R=5m and lining thickness s=0.2m.

The excavation step was chosen as 1.2m, since it was possible to observe its marginal influence on the final results (see Oreste et al. 2016). The excavation phase and support installation were considered as simultaneous, i.e. during the excavation from the shaft bottom the installation of the support was simulated. The excavation has been modeled through a simple elimination of the elements belonging to the excavated rock, whereas the shaft installation was simulated through the reactivation of the elements in the zone occupied by the support structure. These reactivated elements were considered as having zero stress state. The stresses within the lining grow with the successive lowering of the temporary excavation bottom. Although it has been reported that some rocks present a nonlinear stress-strain behavior (e.g. Nawrocki et. al. 1998), this paper assumes a single constant elastic modulus for the depth in the model. The elastic modulus has been considered, for simplicity, isotropic and constant around the shaft at the investigated depth (Spagnoli et al. 2016). By considering the mechanical properties described above and using the well-known Mohr-Coulomb failure criterion equation the unconfined compressive strength (UCS) values for the weak, medium and good states are 0.9, 3.2 and 6MPa respectively. The equation considers the relation between the cohesion, c, and internal friction angle of the soil/rock, φ_i.

$$UCS = \frac{2c\ cos\varphi_i}{(1-sin\varphi_i)} \qquad (1)$$

According to Santi (2006) rocks with UCS values less than 20MPa are empirically classified as weak rocks. 81 cases were analyzed (i.e. 3^4) by combining the geometrical parameters (R, s, p_0) and the three rock categories previously introduced. This analysis is able to cover as many cases as possible that may be encountered in the construction of medium-to-large diameter deep shafts in weak rocks.

For each of the 81 cases investigated it was possible to obtain the trend of the radial displacements at the lateral shaft contour, by varying the distance from the temporary shaft bottom obtained from the numerical calculation (Fig. 4).

This trend, with the typical S shape, is very important as it represents the strain evolution of the rock, with the presence of the support structures, both above and below the temporary shaft bottom. From this trend, it is also possible to observe the interaction mechanism between the lining and the shaft, during the construction phase. A similar trend regarding the radial displacement of the circular cavity contour was obtained by Vlachoupulos and Diederichs (2009) for the case without support structures. The authors were able to describe with analytical equations the radial displacement in the excavated section and ahead of the excavation face, as a function of the ratio maximum plastic radius (in the rock) and shaft radius. The numerical model developed in this research is able to detect the support structure effect from the radial displacement at the lateral shaft contour. It is very important for the CCM to know a particular radial displacement value (u_{R0}) at the temporary excavation bottom, where the lining is installed (see Fig. 4). From the u_{R0} value it is possible to correctly position the reaction line on the CCC of the circular cavity.

Results and discussion

By processing the data relating to u_{R0} values obtained by numerical calculations in the different studied cases, it was possible to observe that the u_{R0}/R ratio is a linear function of both the lithostatic stress p_0 and the lining thickness s (i.e. of its stiffness); whereas it depends on the quadratic form of the shaft radius, R:

$$\frac{u_{R0}}{R} \cdot 1000 \cong (c \cdot s + d) \cdot p_0 - (e \cdot s + f) \qquad (2)$$

Where: p_0 is the horizontal lithostatic stress at the considered depth (in MPa);

s: is the lining concrete thickness (in m).

The parameters c, d, e and f, depend only on the radius R and they vary for the three different rock categories considered as:

Weak rock with poor quality:

$$c = 0.0167 \cdot R^2 + 0.0185 \cdot R - 1.7687 \quad (3)$$

$$d = -0.0507 \cdot R^2 + 0.4854 \cdot R + 0.3368 \quad (4)$$

$$e = 0.1395 \cdot R^2 + 0.461 \cdot R - 14.518 \quad (5)$$

$$f = -0.6103 \cdot R^2 + 4.9151 \cdot R + 2.0596 \quad (6)$$

Weak rock with medium quality:

$$c = 0.0071 \cdot R^2 - 0.031 \cdot R - 0.3226 \quad (7)$$

$$d = -0.0067 \cdot R^2 + 0.086 \cdot R + 0.0892 \quad (8)$$

$$e = 0.0941 \cdot R - 3.3904 \quad (9)$$

$$f = -0.1013 \cdot R^2 + 0.9468 \cdot R + 0.4928 \quad (10)$$

Weak rock with good quality:

$$c = 0.0067 \cdot R^2 - 0.0367 \cdot R - 0.0869 \quad (11)$$

$$d = -0.0018 \cdot R^2 + 0.0309 \cdot R + 0.0392 \quad (12)$$

$$e = 0.0759 \cdot R^2 - 0.4762 \cdot R - 0.7117 \quad (13)$$

$$f = -0.0591 \cdot R^2 + 0.5068 \cdot R - 0.0182 \quad (14)$$

By inserting in a graph the linear trends observed by the ratio u_{R0}/R (given by numerical modelling by varying p_0) and the values u_{R0}/R, obtained with an iterative procedure using the equations of Vlachoupulos and Diederichs (Oreste, 2015), it is possible to observe differences for estimating u_{R0} (see Fig. 5). The equation of Vlachoupulos and Diederichs can describe the radial displacement of the shaft wall by changing the distance from the excavation bottom (shaft bottom). This equation is however obtained in the case of absence of linings, but it is usually employed when support structures are present, as in the literature there are no other equations able to consider the presence of the linings within the shafts. Besides, the parametric analysis developed in this study by means of the numerical model considered the presence of the concrete support, and, therefore, can be considered as a rigorous solution of the problem. The differences for estimating u_{R0} between the equations of Vlachoupulos and Diederichs (2009) and the model shown in this research can give errors regarding the estimation of the radial loads on the lining (p_{eq}). The equation 2 allows correctly evaluating the value of u_{R0} knowing the concrete lining thickness, the shaft depth and the shaft radius, for the different considered qualities of rock, without having to use a specific numerical modelling.

The slope values of Tab. 1 were plotted vs. the lining thickness value in order to obtain the value c (slope of the lines in Fig. 6A). The intercept values were plotted vs. the lining thickness to obtain the value e (slope of the lines in Fig. 6B). Both relations were assumed to be linear. The respective slope and intercept values, called c and d for Fig. 6A and e and f for Fig. 6B, respectively, were in turn plotted vs. the radius values, R (Fig. 6C). The relations in this case were assumed to be polynomial (second degree) and the results gave back the equations 3 to 6 (for the rock category A).

In the case of an intermediate quality rock among those considered, it is possible to interpolate the values of the u_{R0}/R ratio obtained considering the properties close to that under examination. Once the value u_{R0} is known it is possible then to proceed using the CCM in the usual way, by correctly positioning the reaction line on the CCC of the circular cavity. The evaluation of the load acting on shaft lining can quickly proceed by determining the intersection point of the CCC of the circular cavity with the reaction line of the lining.

Application of the model

The following describes an applicative example for the equations described above to employ the CCM and to assess the load on the concrete lining. A shaft with a radius R=1.75m installed at a depth of 1,000m in a non-dilatant weak rock with a specific weight of 25kN/m^3 and with the geomechanical qualities previously described, is assumed (rock categories A, B and C). The lining thickness is hypothesized to be 0.25m. From equation 2, u_{R0}/R x 1000 values of 9.77, 2.23 and 1.02 for poor, medium and good rock quality respectively are obtained. The resulting u_{R0} values are 5.60, 1.27 and 0.58mm respectively (see Tab. 2).

From the numerical modelling results it was possible to observe how the radial displacement of the shaft walls at the temporary shaft bottom (u_{R0}) for weak rocks having poor geomechanical properties is actually bigger, i.e. 5.6mm, than the one obtained by using the formulation of Vlachoupulos and Diederichs (2009) and the CCM with the iterative procedure (see previous paragraph and Oreste, 2015), i.e. 1.9mm. The reaction line therefore seems to be right-shifted in the CCC graph of the circular cavity and the final applied load on the lining (given by the intersection of the CCC with the reaction line) is lower with respect to the results obtained by using the formulation of Vlachoupulos and Diederichs (2009). For weak rocks with poor geotechnical properties, the final acting load on the assumed lining shaft is estimated as 10.6MPa instead of 13.3MPa calculated with the CCM, which is 21% lower (see Fig. 7A). The final acting load value allows verifying the suitability of the lining for the shaft support and to proceed to any possible thickness variation (decrease or increase) as a function of the resulting comparison between the load acting on the lining and the maximum load the lining with that determined thickness is able to safely withstand. The results show instead for medium and good rock qualities a different trend. For the medium rock (type B) the difference decreases until reaching almost the same load value, i.e. 10.0MPa for the CMM and 10.2MPa for the numerical modelling (see Fig. 7B). Regarding the rocks with good mechanical properties (type C) the numerical modelling slightly overestimates the acting load on the assumed shaft lining giving a value of 9.14MPa instead of 8.57MPa calculated with the analytical approach (see Fig. 7C).

This difference is due to the fact that the Vlachoupulos and Diederichs equation (2009) was obtained in the case of non-supported shaft, whereas the parametric study obtained with the numerical model shown in this paper was able to consider the presence of linings and the successive phase of excavation and support installation.

Conclusions

The CCM has been already proposed as a tool to predict the ground radial displacements and the formation pressure on a shaft. The radial wall displacement, u_{R0}, cannot be easily obtained by using analytical methods. This research showed an extensive parametric analysis of the bi-dimensional axisymmetric numerical modelling in order to investigate the strain of the shaft wall close to the excavation bottom in order to properly design the lining in weak rocks categorized as with poor, medium and good qualities. The modelling analyzed the stress and strain state developing in the rock and in the lining during the construction phase. It was possible to obtain the trend of the radial displacements at the lateral shaft contour, by varying the distance from the temporary shaft bottom. It was also possible to observe the interaction mechanism between the lining and the shaft, during the construction phase. The numerical model developed in this research was able to detect the support structure influence from the radial displacement at the lateral shaft contour in order to correctly position the reaction line on the CCC of the circular cavity as a function of the lithostatic stress p_0, the lining thickness s and shaft radius R. From the results of the parametric analysis obtained by the numerical modelling, it was possible to obtain an equation giving the shaft wall displacement, $u_{R0,}$ at the temporary excavation bottom. This value represents the displacement at the instant of the lining installation, and it is important in order to correctly position the reaction line in the CCC graph for the circular cavity. With this equation, it is possible to preliminary design the support structure in a circular shaft in non-dilatant weak rocks. The new equation should not be used for the detailed design of tunnels in more complex rock masses and in situ stress fields.

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Figure caption

Fig. 1 Shaft installed with the underpinning method.

Fig. 2 Characteristic curve (modified after Spagnoli et al. 2016).

Fig. 3 Detail of the mesh, close to the temporary shaft bottom, for the half shaft longitudinal section of the developed numerical modeling in the axisymmetric configuration for the model with R=5m and s=0.2. The number of elements in the horizontal direction is 140, whereas in the vertical direction is 256. The width of the model is 36.5m, while the height is 76.8m. The elements height in the excavation zone is 0.3m, the elements width in the excavation zone and for the linings is 0.1m, whereas outside is 0.35m. The left side is the symmetric axis of the shaft, the lower edge is blocked for vertical displacements, at the upper edge and the right side the lithostatic pressure are applied by considering a constant and homogeneous state for deep-problem conditions (i.e. the lithostatic stress does not change in the proximity of the analyzed case).

Fig. 4 Generic trend of the ratio u_R/u_{Rmax} (u_R radial wall displacement; u_{Rmax} maximum value of u_R) by changing the distance from the temporary excavation bottom (positive upwards towards the excavated rocks). Key: the red dot represents the u_R/u_{Rmax} ratio, obtained at the temporary shaft bottom, where the support excavation activates.

Fig. 5 Plot of the linear relation U_{R0}/R x 1000 for changing p_0, R and s for the CCM (A) and the numerical modelling (B) for the rock category A (i.e. poor geotechnical properties). Key: symbols in 5A represent the results obtained using the Vlachoupulos and Diederichs' equation and the iterative procedure shown in Oreste (2015); lines in 5B represent results from the parametric analysis using the axisymmetric numerical model. Continuous lines indicate shaft radius of 1m, broken lines indicate shaft radius of 3m; dotted lines indicate shaft radius of 5m. The increased bold labelling indicates the increased lining thickness value as indicated by the number close to the lines.

Fig. 6 (A) Plot lining thickness (0.1, 0.2 and 0.3m) vs slope values from Tab. 1 for different R for the weak rock with poor geotechnical properties in order to obtain the value c, i.e. -1.7335, -1.563, -1.259 and d, i.e. 0.7715, 1.3371, 1.4975; (B) Plot lining thickness vs intercept value from Tab. 1 for different R for the weak rock with poor geotechnical properties in order to obtain the value e, i.e. -13.917, -11.879, -8.725, and f, i.e. 6.3644, 11.312. 11.377; (C) Plot radius values vs the slope and intercept values, called c and d respectively coming from the example (A) and slope and intercept values, called e and f respectively coming from the example (B). The parameters c, d, e and f, depend only on the radius R and they vary for the three different rock categories.

Fig. 7 Comparison of the shaft-support interaction using the Vlachoupulos and Diederichs' equation and an iterative procedure for obtaining u_{R0} (Oreste, 2015) (CCM) and using eq.1 obtained from the developed parametric study with the numerical modelling, for rocks with poor (A), medium (B) and good (C) mechanical properties.

Geometrical parameter	Slope a	Intercept b (negative value)
R5-s0.1	1.3821	10.565
R5-s0.2	1.2248	9.512
R5-s0.3	1.1303	8.82
R3-s0.1	1.1868	10.128
R3-s0.2	1.0126	8.9289
R3-s0.3	0.8742	7.7522
R1-s0.1	0.6265	5.2267
R1-s0.2	0.368	3.073
R1-s0.3	0.2798	2.4433

Tab. 1 Variables a (slope) and b (intercept) of the correlation from Fig. 5 (for the rock category A).

	Rock type A	Rock type B	Rock type C
u_{R0}/R x 1000 ratio	9.77	2.23	1.02
u_{R0} value (mm)	5.60	1.27	0.58

Tab. 2 Results obtained by using the novel developed approach for a shaft with R=1.75m, at a depth of 1,000m with rock specific weight of 25kN/m^3 and s=0.25m for rock types A, B and C