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

14 The Convergence-Confinement Method (CCM) applies to circular tunnels in an in situ stress 15 field in which all three principal stresses are equal and where the rock mass exhibits elasto-16 perfectly plastic shear failure. As the radial wall displacement cannot be easily obtained by 17 using analytical methods, an extensive parametric analysis of the bi-dimensional numerical 18 modelling in order to investigate the strain of the shaft wall close to the excavation bottom 19 was performed. 81 cases were derived from the combination of the geometrical parameters 20 and three weak rock categories. By processing the data relating to u_{R0} (radial displacement of 21 the shaft wall at the excavation bottom) values obtained by numerical calculation in the dif-22 ferent cases studied, it was possible to calculate the u_{R0}/R ratio as a function of the lithostatic 23 stress p₀, 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 24 the shaft radius, for the different qualities of rock considered. 25

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

28 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).

34 Typical shaft lining materials are steel, concrete, fiberglass and corrugated metal (Henn 35 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 36 (Peck and Lee 2007; Spagnoli et al. 2016) because of their poor mechanical properties. 37 Deep shafts excavated in rocks with poor geomechanical properties need to have a suitable 38 39 lining design in order to guarantee the stability of the walls (e.g. Jia et al. 2013). The calcula-40 tion 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 41 42 analytically assuming a state of hydrostatic stress, considering a failure criterion and deter-43 mining the pressure of internal support that will prevent the rupture zone, which develops 44 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 labora tory 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).

56 The chance of using the bi-dimensional axisymmetric numerical modelling further simplifies 57 the implementation of the model and reduces the calculation time. However, the simulation of 58 the excavation process and the installation of the support for a certain shaft section (about 10 59 times the diameter), is a time-consuming process. The stress and strain states both in the 60 rock and in the support structure, when the shaft reaches the central part of the model, are 61 analyzed. As the variation of the excavation step barely influences the final result (Oreste et 62 al. 2016), for the sake of simplicity, exaggerated excavation step (higher than those observed 63 in the reality) can be considered, in order to reduce the calculation time. Due to the complexi-64 ty 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 cal-65 culation is performed. After having analyzed the results, it is possible to see whether the 66 67 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 under-68 69 goes less stresses than the lining was designed for, the successive hypothesis must consid-70 er a less robust support structure, and vice versa. The progressive fine-tuning of the support 71 structure can require several numerical models and the process can be time-consuming. For 72 this reason, analytical methods are often used. They allow a simplified and quick analysis of 73 the interaction between the shaft lining and the surrounding rocks, and they can give rather 74 precise results regarding the interaction problem. Numerical modelling is, therefore, only 75 used to verify the results in detail. The most used analytical method is the CCM. This re-76 search considers the CCM as a tool to predict the radial ground displacements and the for-77 mation pressure on a shaft. As the radial wall displacement, u_{R0} , cannot be easily obtained 78 by using analytical methods, an extensive parametric analysis of the bi-dimensional numeri-79 cal modelling developed in this research is able to detect the support structure influence from 80 the radial displacement at the lateral shaft contour. This allows to correctly position the reac-81 tion line on the convergence-confinement curve (CCC) of the circular cavity as a function of the lithostatic stress p₀, the lining thickness s and shaft radius R, for (non-dilatant) weak 82 rocks categorized as with poor, medium and good qualities. 83

84 The convergence-confinement method for designing support structures

85 The CCM, usually applied to tunnels (e.g. Duncan-Fama 1993; Nguyen-Minh and Guo 1996; 86 Oreste 2003, 2014; Carranza-Torres and Fairhurst 2000), has been proposed by Wong and 87 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 88 89 shaft surface is represented by the convergence-confinement curve, which is very important 90 for analyzing the interaction between the rock and the lining (Spagnoli et al. 2016). The CCM 91 has also been validated for real underground structures (e.g. Kitagawa et al. 1991; Mariee et 92 al. 2009; Svoboda and Masin 2010). This method is based on the assumption that the rock at 93 the shaft bottom provides an initial support pressure equal to the in situ stress p_0 . As the 94 shaft excavation advances and the bottom moves away from the section under considera-95 tion, the support pressure gradually decreases until it reaches zero at some distance behind 96 the shaft (Hoek et al. 2008). The extent of the plastic zone can also be estimated by this 97 method at each stage of the process (Wong and Kaiser 1988; Vlachopoulos and Diederichs 98 2009) by controlling the internal support pressure p_{i} , applied by the linings (Hoek et al. 2008). 99 The CCM allows an analysis considering the interaction between the pressure applied to the 100 circular shaft wall and the corresponding radial displacements. The method considers the 101 following hypotheses:

- 102 1. Circular and deep shaft;
- 103 2. In situ homogeneous and isotropic rock around the shaft with ideal (or brittle) post-104 failure behavior;
- 105 3. Constant and isotropic lithostatic stress p_0 (K₀ = 1) around the shaft (e.g. Oreste 106 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).

116 In the case a curvilinear failure criterion (typical of rocks) is employed, such as the one of 117 Hoek and Brown (1980), the solution is no longer a closed-form one. It is rather a finite dif-118 ference numerical solution necessary (Oreste 2014). In this case, instead of cohesion and 119 friction angle, Hoek and Brown's m and s parameters as well as the compressive strength of 120 the intact rock, σ_c , have to be considered. The final result is nonetheless the CCC, generally 121 representing a first linear trend (for internal pressure between the so-called critical value, p_{cr}, 122 and the virgin in situ lithostatic stress, p₀), which changes to a curvilinear path (for internal 123 pressure between 0 and p_{cr}) with a downwards concavity (Carranza-Torres and Fairhurst 124 2000). In order to consider an interaction mechanism between the rock and the lining, it is 125 necessary to intersect the CCC of the shaft and the reaction line of the support. The reaction 126 line represents the relation between the applied pressure to the support and the correspond-127 ing radial displacement of the tunnel wall. Assuming an elastic lining behavior, the relation is 128 a linear one. The reaction line has a slope depending on the lining stiffness.

129 Another parameter needed to correctly position the reaction line on the CCC is the radial wall 130 displacement, u_{R0}, at the point where the lining is installed. Generally speaking, support 131 structures are installed in proximity of the excavation face (i.e. the temporary shaft bottom). 132 Therefore, u_{R0} coincides with the radial shaft wall displacement in correspondence with the 133 temporary excavation bottom. This parameter cannot be easily obtained by using analytical 134 methods. Vlachoupulos and Diederichs (2009) could estimate, by means of the numerical 135 modelling, the variation of the radial wall displacement of the circular cavity by varying the 136 distance from the excavation face, in the absence of support lining and for different rock 137 types. The equations provided by Vlachoupulos and Diederichs (2009) allow to preliminary 138 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 139

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.

144 Through this procedure, it is possible to obtain an estimation of the radial shaft wall dis-145 placement at the excavation face, and therefore to correctly position the reaction line on the 146 CCC. In this way, the intersection point, which gives the final applied pressure on the rock by 147 the lining (p_{eq}) , is obtained. By knowing p_{eq} , it is possible to verify the suitability of the hy-148 pothesized support structure. By means of the CCC, the design iterative procedure can be 149 quicker. Based on recent numerical analysis (Oreste et al.2016), the use of the CCC method 150 combined the equations of Vlachoupulos and Diederichs (2009), in order to define the start-151 ing point of the reaction line, leads to a non-exact evaluation of the final pressure acting on 152 the lining (Oreste et al. 2016). The equations of Vlachoupulos and Diederichs (2009), origi-153 nally obtained in absence of supporting structure, cannot be used for positioning in a reliable 154 way the reaction line on the CCC of the shaft. For this reason, this research shows an exten-155 sive parametric analysis of the bi-dimensional numerical modelling in order to investigate the 156 displacement of the shaft wall close to the excavation bottom. The findings of this parametric 157 analysis may be useful for a proper design by using the CCC procedure.

158 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 (al-ready 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 accen tuated;
- 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 (p₀): 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 repre-senting 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.

205 The excavation step was chosen as 1.2m, since it was possible to observe its marginal influ-206 ence on the final results (see Oreste et al. 2016). The excavation phase and support installa-207 tion were considered as simultaneous, i.e. during the excavation from the shaft bottom the 208 installation of the support was simulated. The excavation has been modeled through a sim-209 ple elimination of the elements belonging to the excavated rock, whereas the shaft installa-210 tion was simulated through the reactivation of the elements in the zone occupied by the sup-211 port structure. These reactivated elements were considered as having zero stress state. The 212 stresses within the lining grow with the successive lowering of the temporary excavation bot-213 tom. Although it has been reported that some rocks present a nonlinear stress-strain behav-214 ior (e.g. Nawrocki et. al. 1998), this paper assumes a single constant elastic modulus for the 215 depth in the model. The elastic modulus has been considered, for simplicity, isotropic and 216 constant around the shaft at the investigated depth (Spagnoli et al. 2016). By considering the 217 mechanical properties described above and using the well-known Mohr-Coulomb failure cri-218 terion equation the unconfined compressive strength (UCS) values for the weak, medium 219 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 . 220

221
$$UCS = \frac{2c \cos \varphi_i}{(1-\sin \varphi_i)}$$

(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, p₀) 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).

230 This trend, with the typical S shape, is very important as it represents the strain evolution of 231 the rock, with the presence of the support structures, both above and below the temporary 232 shaft bottom. From this trend, it is also possible to observe the interaction mechanism be-233 tween the lining and the shaft, during the construction phase. A similar trend regarding the 234 radial displacement of the circular cavity contour was obtained by Vlachoupulos and 235 Diederichs (2009) for the case without support structures. The authors were able to describe 236 with analytical equations the radial displacement in the excavated section and ahead of the 237 excavation face, as a function of the ratio maximum plastic radius (in the rock) and shaft ra-238 dius. The numerical model developed in this research is able to detect the support structure 239 effect from the radial displacement at the lateral shaft contour. It is very important for the 240 CCM to know a particular radial displacement value (u_{R0}) at the temporary excavation bot-241 tom, where the lining is installed (see Fig. 4). From the u_{R0} value it is possible to correctly 242 position the reaction line on the CCC of the circular cavity.

243 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:

248
$$\frac{a_{R0}}{R} \cdot 1000 \cong (c \cdot s + d) \cdot p_0 - (e \cdot s + f)$$
 (2)

- 249 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:
- 253 Weak rock with poor quality:

254
$$c = 0.0167 \cdot R^2 + 0.0185 \cdot R - 1.7687$$
 (3)

- 255 $d = -0.0507 \cdot R^2 + 0.4854 \cdot R + 0.3368$ (4)
- $256 e = 0.1395 \cdot R^2 + 0.461 \cdot R 14.518 (5)$
- 257 $f = -0.6103 \cdot R^2 + 4.9151 \cdot R + 2.0596$ (6)
- 258 Weak rock with medium quality:
- 259 $c = 0.0071 \cdot R^2 0.031 \cdot R 0.3226$ (7)
- $260 d = -0.0067 \cdot R^2 + 0.086 \cdot R + 0.0892 (8)$
- $261 e = 0.0941 \cdot R 3.3904 (9)$
- 262 $f = -0.1013 \cdot R^2 + 0.9468 \cdot R + 0.4928$ (10)
- 263 Weak rock with good quality:
- $264 c = 0.0067 \cdot R^2 0.0367 \cdot R 0.0869 (11)$
- $265 d = -0.0018 \cdot R^2 + 0.0309 \cdot R + 0.0392 (12)$
- $266 \quad e = 0.0759 \cdot R^2 0.4762 \cdot R 0.7117 \tag{13}$

267
$$f = -0.0591 \cdot R^2 + 0.5068 \cdot R - 0.0182$$
 (14)

268

269 By inserting in a graph the linear trends observed by the ratio u_{R0}/R (given by numerical 270 modelling by varying p_0) and the values u_{R0}/R , obtained with an iterative procedure using the 271 equations of Vlachoupulos and Diederichs (Oreste, 2015), it is possible to observe differ-272 ences for estimating u_{R0} (see Fig. 5). The equation of Vlachoupulos and Diederichs can de-273 scribe the radial displacement of the shaft wall by changing the distance from the excavation 274 bottom (shaft bottom). This equation is however obtained in the case of absence of linings, 275 but it is usually employed when support structures are present, as in the literature there are 276 no other equations able to consider the presence of the linings within the shafts. Besides, the 277 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 278 279 the problem. The differences for estimating u_{R0} between the equations of Vlachoupulos and 280 Diederichs (2009) and the model shown in this research can give errors regarding the esti-281 mation of the radial loads on the lining (peg). The equation 2 allows correctly evaluating the 282 value of u_{R0} knowing the concrete lining thickness, the shaft depth and the shaft radius, for 283 the different considered qualities of rock, without having to use a specific numerical model-284 ling.

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.

298 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/m³ 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).

306 From the numerical modelling results it was possible to observe how the radial displacement 307 of the shaft walls at the temporary shaft bottom (u_{R0}) for weak rocks having poor geomechan-308 ical properties is actually bigger, i.e. 5.6mm, than the one obtained by using the formulation 309 of Vlachoupulos and Diederichs (2009) and the CCM with the iterative procedure (see previ-310 ous paragraph and Oreste, 2015), i.e. 1.9mm. The reaction line therefore seems to be right-311 shifted in the CCC graph of the circular cavity and the final applied load on the lining (given 312 by the intersection of the CCC with the reaction line) is lower with respect to the results ob-313 tained by using the formulation of Vlachoupulos and Diederichs (2009). For weak rocks with 314 poor geotechnical properties, the final acting load on the assumed lining shaft is estimated as 315 10.6MPa instead of 13.3MPa calculated with the CCM, which is 21% lower (see Fig. 7A). 316 The final acting load value allows verifying the suitability of the lining for the shaft support 317 and to proceed to any possible thickness variation (decrease or increase) as a function of the 318 resulting comparison between the load acting on the lining and the maximum load the lining 319 with that determined thickness is able to safely withstand. The results show instead for medi-320 um and good rock qualities a different trend. For the medium rock (type B) the difference 321 decreases until reaching almost the same load value, i.e. 10.0MPa for the CMM and 322 10.2MPa for the numerical modelling (see Fig. 7B). Regarding the rocks with good mechani-323 cal properties (type C) the numerical modelling slightly overestimates the acting load on the 324 assumed shaft lining giving a value of 9.14MPa instead of 8.57MPa calculated with the ana-325 lytical 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.

330 Conclusions

331 The CCM has been already proposed as a tool to predict the ground radial displacements 332 and the formation pressure on a shaft. The radial wall displacement, u_{R0}, cannot be easily 333 obtained by using analytical methods. This research showed an extensive parametric analy-334 sis of the bi-dimensional axisymmetric numerical modelling in order to investigate the strain 335 of the shaft wall close to the excavation bottom in order to properly design the lining in weak 336 rocks categorized as with poor, medium and good qualities. The modelling analyzed the 337 stress and strain state developing in the rock and in the lining during the construction phase. 338 It was possible to obtain the trend of the radial displacements at the lateral shaft contour, by 339 varying the distance from the temporary shaft bottom. It was also possible to observe the 340 interaction mechanism between the lining and the shaft, during the construction phase. The 341 numerical model developed in this research was able to detect the support structure influ-342 ence from the radial displacement at the lateral shaft contour in order to correctly position the 343 reaction line on the CCC of the circular cavity as a function of the lithostatic stress p₀, the 344 lining thickness s and shaft radius R. From the results of the parametric analysis obtained by 345 the numerical modelling, it was possible to obtain an equation giving the shaft wall displace-346 ment, u_{R0} at the temporary excavation bottom. This value represents the displacement at the 347 instant of the lining installation, and it is important in order to correctly position the reaction 348 line in the CCC graph for the circular cavity. With this equation, it is possible to preliminary 349 design the support structure in a circular shaft in non-dilatant weak rocks. The new equation 350 should not be used for the detailed design of tunnels in more complex rock masses and in 351 situ stress fields.

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438 **Figure caption**

439 Fig. 1 Shaft installed with the underpinning method.

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

441 Fig. 3 Detail of the mesh, close to the temporary shaft bottom, for the half shaft longitudinal 442 section of the developed numerical modeling in the axisymmetric configuration for the model 443 with R=5m and s=0.2. The number of elements in the horizontal direction is 140, whereas in 444 the vertical direction is 256. The width of the model is 36.5m, while the height is 76.8m. The 445 elements height in the excavation zone is 0.3m, the elements width in the excavation zone 446 and for the linings is 0.1m, whereas outside is 0.35m. The left side is the symmetric axis of 447 the shaft, the lower edge is blocked for vertical displacements, at the upper edge and the 448 right side the lithostatic pressure are applied by considering a constant and homogeneous 449 state for deep-problem conditions (i.e. the lithostatic stress does not change in the proximity 450 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.

455 Fig. 5 Plot of the linear relation $U_{R0}/R \times 1000$ for changing p_0 , R and s for the CCM (A) and 456 the numerical modelling (B) for the rock category A (i.e. poor geotechnical properties). Key: 457 symbols in 5A represent the results obtained using the Vlachoupulos and Diederichs' equa-458 tion and the iterative procedure shown in Oreste (2015); lines in 5B represent results from 459 the parametric analysis using the axisymmetric numerical model. Continuous lines indicate 460 shaft radius of 1m, broken lines indicate shaft radius of 3m; dotted lines indicate shaft radius 461 of 5m. The increased bold labelling indicates the increased lining thickness value as indicat-462 ed 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 inter-

466 cept value from Tab. 1 for different R for the weak rock with poor geotechnical properties in
467 order to obtain the value e, i.e. -13.917, -11.879, -8.725, and f, i.e. 6.3644, 11.312. 11.377;
468 (C) Plot radius values vs the slope and intercept values, called c and d respectively coming
469 from the example (A) and slope and intercept values, called e and f respectively coming from
470 the example (B). The parameters c, d, e and f, depend only on the radius R and they vary for
471 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

478 Tab. 1 Variables a (slope) and b (intercept) of the correlation from Fig. 5 (for the rock catego479 ry A).

480

	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

481 Tab. 2 Results obtained by using the novel developed approach for a shaft with R=1.75m, at

482 a depth of 1,000m with rock specific weight of 25kN/m³ and s=0.25m for rock types A, B and

483 C