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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 p_0 , the lining thickness s , and the shaft radius R . Novel equations were obtained for
24 quickly estimating the value of u_{R0} knowing the lining concrete thickness, the shaft depth and
25 the shaft radius, for the different qualities of rock considered.

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

27 Method

28 **Introduction**

29 The choice of the lining system depends on the costs, ground conditions, contractor's prefer-
30 ence and the construction method. Shaft linings can be installed by using the underpinning
31 method (if the rock can stand unsupported, see Fig. 1), by using the caisson-sinking method
32 (in the case where the vertical excavated face is difficult to achieve), by using diaphragm
33 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
36 instance) are often the biggest challenge for site investigations, design, and construction
37 (Peck and Lee 2007; Spagnoli et al. 2016) because of their poor mechanical properties.
38 Deep shafts excavated in rocks with poor geomechanical properties need to have a suitable
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
41 the contact rock-lining is known (Öztürk and Ünal 2001). This pressure can be calculated
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:

- 45 • Analytical methods are the deterministic solutions of closed form, such as the Con-
46 vergence-Confinement Method (CCM) described by Wong and Kaiser (1988), Hoek
47 and Brown (1980), Panet (1995), which is also the most used analytical method.
- 48 • Empirical methods use equations based on different mines with respect to the labora-
49 tory and in-situ test results. All systems have quantitative estimation of the rock mass
50 quality linked with empirical design rules to estimate adequate rock support
51 measures, such as rock bolt, shotcrete and steel set (e.g. Barton, et al. 1974;
52 Bieniawski 1989; Hoek and Brown 1980).
- 53 • Numerical methods which consider nonlinear analysis, anisotropy and discontinuities
54 of the rock mass, complicated geometry of the problem and troublesome geological
55 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 struc-
65 ture is assumed (with certain dimensions and rigidity of the material) and the numerical cal-
66 culation is performed. After having analyzed the results, it is possible to see whether the
67 support structure previously hypothesized is correct or a further design step is needed. If the
68 results give an excessive applied pressure to the shaft wall, or the support structure under-
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
82 the lithostatic stress p_0 , the lining thickness s and shaft radius R , for (non-dilatant) weak
83 rocks categorized as with poor, medium and good qualities.

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
88 on a shaft (Fig. 2). The link between the radial stresses and the displacements of the lateral
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 ($K_0 = 1$) around the shaft (e.g. Oreste
106 2009).

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

- 111 • Elastic parameters (elastic modulus and Poisson's ratio);

- 112 • Strength parameters (cohesion and friction angle);
- 113 • Strain parameters in plastic field (dilatancy angle);
- 114 • Geometric parameters (shaft radius and depth, the latter giving the lithostatic stress
- 115 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, p_0), 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. Vlachopoulos 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 Vlachopoulos and Diederichs (2009) allow to preliminary
138 estimate the radial wall displacement at the excavation face, as a function of the final wall
139 displacement. Therefore, in order to use the CCC for designing the support structure, the

140 equations of Vlachopoulos and Diederichs (2009) require an iterative procedure, which
141 quickly converges. The final wall displacement depends on the radial displacement at the
142 excavation face and, therefore, only after having defined the latter, it is possible to determine
143 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 Vlachopoulos 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 Vlachopoulos 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**

159 The numerical modelling for this research was developed with the bi-dimensional explicit
160 finite difference program Flac 2D v.6.0 (Itasca 2008) used in the axisymmetric configuration.
161 The model analyzed the stress and strain state developing in the rock and in the linings dur-
162 ing the construction phases. The study considered a circular vertical shaft with a support
163 structure of concrete. The lithostatic stress state was hypothesized, for the sake of simplicity,
164 as homogeneous, i.e. the horizontal stress is constant independent of the direction. This
165 simplified assumption permits to accelerate the numerical calculation with a bi-dimensional

166 method in the axial-symmetric configuration. The following assumptions were also made (al-
167 ready considered by Spagnoli et al. 2016):

- 168 • The failure criterion adopted for the rock is the linear Mohr-Coulomb, generally
169 adopted for weak rocks, for which the curvature of the failure criterion is less accen-
170 tuated;
- 171 • The residual conditions were considered equal to those of the peak, assuming, there-
172 fore, an ideal elasto-plastic behavior of the rock in the phase of post-failure;
- 173 • The elastic modulus was considered for simplicity constant both in the elastic and
174 plastic phase;
- 175 • The dilatancy angle, ψ , which describes the strains behavior in the plastic range, has
176 been considered as equal to zero, assuming plastic strains at constant volume, as
177 described by Hoek and Brown (1997), Alejano and Alonso (2005) and Alejano et al.
178 (2010) in the case of deep rocks with poor mechanical properties;
- 179 • The lining was considered to be composed of concrete, with typical values of elastic
180 modulus (30,000MPa) and Poisson's ratio (0.15);
- 181 • The horizontal lithostatic stress was considered equal in the two different directions:
182 this condition is generally taken to the great depths to which the variability of the
183 lithostatic stress in the three directions of the space is drastically reduced.

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

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

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

199 The numerical model considers about 36,000 quadrilateral elements, employed for repre-
200 senting both the rock and the concrete constituting the support structure.

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

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 be-
220 tween the cohesion, c , and internal friction angle of the soil/rock, φ_i .

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

222 According to Santi (2006) rocks with UCS values less than 20MPa are empirically classified
223 as weak rocks. 81 cases were analyzed (i.e. 3⁴) by combining the geometrical parameters
224 (R, s, p₀) and the three rock categories previously introduced. This analysis is able to cover
225 as many cases as possible that may be encountered in the construction of medium-to-large
226 diameter deep shafts in weak rocks.

227 For each of the 81 cases investigated it was possible to obtain the trend of the radial dis-
228 placements at the lateral shaft contour, by varying the distance from the temporary shaft bot-
229 tom 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 Vlachopoulos 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**

244 By processing the data relating to u_{R0} values obtained by numerical calculations in the differ-
245 ent studied cases, it was possible to observe that the u_{R0}/R ratio is a linear function of both
246 the lithostatic stress p₀ and the lining thickness s (i.e. of its stiffness); whereas it depends on
247 the quadratic form of the shaft radius, R:

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

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

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

251 The parameters c , d , e and f , depend only on the radius R and they vary for the three differ-
252 ent rock categories considered as:

253 Weak rock with poor quality:

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

$$255 \quad d = -0.0507 \cdot R^2 + 0.4854 \cdot R + 0.3368 \quad (4)$$

$$256 \quad e = 0.1395 \cdot R^2 + 0.461 \cdot R - 14.518 \quad (5)$$

$$257 \quad f = -0.6103 \cdot R^2 + 4.9151 \cdot R + 2.0596 \quad (6)$$

258 Weak rock with medium quality:

$$259 \quad c = 0.0071 \cdot R^2 - 0.031 \cdot R - 0.3226 \quad (7)$$

$$260 \quad d = -0.0067 \cdot R^2 + 0.086 \cdot R + 0.0892 \quad (8)$$

$$261 \quad e = 0.0941 \cdot R - 3.3904 \quad (9)$$

$$262 \quad f = -0.1013 \cdot R^2 + 0.9468 \cdot R + 0.4928 \quad (10)$$

263 Weak rock with good quality:

$$264 \quad c = 0.0067 \cdot R^2 - 0.0367 \cdot R - 0.0869 \quad (11)$$

$$265 \quad d = -0.0018 \cdot R^2 + 0.0309 \cdot R + 0.0392 \quad (12)$$

$$266 \quad e = 0.0759 \cdot R^2 - 0.4762 \cdot R - 0.7117 \quad (13)$$

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

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 Vlachopoulos and Diederichs (Oreste, 2015), it is possible to observe differ-
272 ences for estimating u_{R0} (see Fig. 5). The equation of Vlachopoulos 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
278 presence of the concrete support, and, therefore, can be considered as a rigorous solution of
279 the problem. The differences for estimating u_{R0} between the equations of Vlachopoulos 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 (p_{eq}). 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.

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

292 In the case of an intermediate quality rock among those considered, it is possible to interpo-
293 late the values of the u_{R0}/R ratio obtained considering the properties close to that under ex-
294 amination. Once the value u_{R0} is known it is possible then to proceed using the CCM in the
295 usual way, by correctly positioning the reaction line on the CCC of the circular cavity. The
296 evaluation of the load acting on shaft lining can quickly proceed by determining the intersec-
297 tion point of the CCC of the circular cavity with the reaction line of the lining.

298 **Application of the model**

299 The following describes an applicative example for the equations described above to employ
300 the CCM and to assess the load on the concrete lining. A shaft with a radius $R=1.75\text{m}$ in-
301 stalled at a depth of 1,000m in a non-dilatant weak rock with a specific weight of 25kN/m^3
302 and with the geomechanical qualities previously described, is assumed (rock categories A, B
303 and C). The lining thickness is hypothesized to be 0.25m. From equation 2, $u_{R0}/R \times 1000$

304 values of 9.77, 2.23 and 1.02 for poor, medium and good rock quality respectively are ob-
305 tained. 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 Vlachopoulos 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 Vlachopoulos 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).

326 This difference is due to the fact that the Vlachopoulos and Diederichs equation (2009) was
327 obtained in the case of non-supported shaft, whereas the parametric study obtained with the
328 numerical model shown in this paper was able to consider the presence of linings and the
329 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 p_0 , 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.

352 **References**

353 Alejano, L.R., and Alonso, E. (2005). "Considerations of the dilatancy angle in rocks and rock
354 masses." *Int. J. Rock Mech. Min. Sci.*, 42(4):481-507.

355 Alejano, L.R., Rodriguez-Dono, A., Alonso, A., and Fdez.-Manín, G. (2009). "Ground reaction
356 curves for tunnels excavated in different quality rock masses showing several types of post-
357 failure behaviour." TUNN. UNDERGR. SP. TECH., 24(6):689-705.

358 Barton, N., Line, R. and Lunde, J. (1974). "Engineering classification of rock masses for the
359 design of tunnel support." Rock Mech., 6(4), 189-236.

360 Bieniawsky, Z. (1989). Engineering rock mass classifications, Wiley: New York.

361 British Tunnelling Society (2004). Tunnel lining design guide. Thomas Telford, London.

362 Carranza-Torres, C., and Fairhurst, C. (2000). "Application of the convergence–confinement
363 method of tunnel design to rock masses that satisfy the Hoek–Brown failure criterion." TUNN.
364 UNDERGR. SP. TECH., 15(2): 187–213.

365 Duncan Fama, M.E. (1993). "Numerical modeling of yield zones in weak rock." Comprehen-
366 sive Rock Engineering. J.A. Hudson, ed. Vol. 2. Pergamon, Oxford, 49–75.

367 Fabich, S., Bauer, J., Rajczakowska, M., and Świtoń, S. (2015). "Design of the shaft lining
368 and shaft stations for deep polymetallic ore deposits: Victoria Mine case study." Mining Sci-
369 ence, 22: 127-146.

370 Henn, R.W. (2003). AUA guidelines for backfilling and contact grouting of tunnels and shafts.
371 American Society of Civil Engineers, Reston.

372 Hoek, E., Carranza-Torres, C., Diederichs, M.S. and Corkum, B. (2008). "Integration of ge-
373 otechnical and structural design in tunnelling." Proceedings University of Minnesota 56th An-
374 nual Geotechnical Engineering Conference, 29 February 2008. Minneapolis, 1–53

375 Hoek, E., and Brown, E.T. (1980). Underground Excavations in Rock. Institution of Mining
376 and Metallurgy, London.

377 ITASCA 2008. Flac 6.0 Fast Lagrangian Analysis of Continua, Version 6.0-User's Manual.
378 367 ITASCA Consulting Group Inc, Minneapolis.

379 Jia, Y.D., Stace, R., and Williams, A. (2013). "Numerical modelling of shaft lining stability at
380 deep mine." Mining Technology: Transactions of the Institutions of Mining and Metallurgy:
381 Section A, 122(1):8-19.

382 Kitagawa, T., Kumeta, T., Ichizyo, T., Soga, S., Sato, M., and Yasukawa, M. (1991) "Applica-
383 tion of Convergence Confinement Analysis to the study of preceding displacement of a
384 squeezing rock tunnel." *Rock Mechanics and Rock Engineering*, 24(1): 31-51.

385 Mariee, A.A., Belal, A.M., and El-Desouky A (2009) "Application of the Convergence-
386 Confinement Approach to Analyze the Rock-Lining Interaction in Tunnels (Case Study:
387 Shimizu Tunnel)." 3th International Conference on AEROSPACE SCIENCES & AVIATION
388 TECHNOLOGY, May 26 –28, 2009, Cairo, Egypt.

389 Nawrocki, P.A., Dusseault, M.B., Bratli, R.K., and Xu, G. (1998). "Assessment of some semi-
390 analytical models for non-linear modelling of borehole stresses." *Int. J. Rock Mech. Min. Sci.*,
391 35(4):522.

392 Nguyen-Minh, D., and Guo, C. (1996). "Recent progress in convergence confinement meth-
393 od." ISRM International Symposium - EUROCK 96. G. Barla, ed, Balkema, Rotterdam,. 855-
394 860.

395 Oreste, P. (2003). "Analysis of Structural Interaction in Tunnels using the Convergence-
396 Confinement Approach." *TUNN. UNDERGR. SP. TECH.*, 18(4):347-363, DOI:
397 10.1016/S0886-7798(03)00004-X.

398 Oreste, P. (2009). "The convergence-confinement method: Roles and limits in modern geo-
399 mechanical tunnel design." *Am. J. Appl. Sci.*, 6(4), 757-771, DOI:
400 10.3844/ajassp.2009.757.771.

401 Oreste, P. (2009). "The Determination of the Tunnel Structure Loads Through the Analysis of
402 the Interaction between the Void and the Support Using the Convergence-Confinement
403 Method." *Am. J. Appl. Sci.*, 11(11), 1945-1954, DOI: 10.3844/ajassp.2014.1945.1954.

404 Oreste, P. 2014. "A Numerical Approach for Evaluating the Convergence-Confinement Curve
405 of a Rock Tunnel Considering Hoek-Brown Strength Criterion." *Am. J. Appl. Sci.*, 11(12),
406 2021-2030, DOI: 10.3844/ajassp.2014.2021.2030.

407 Oreste, P. 2015. "Analysis of the Interaction between the Lining of a TBM Tunnel and the
408 Ground Using the Convergence-Confinement Method." *Am. J. Appl. Sci.*, 12(4), 276-283.
409 DOI: 10.3844/ajassp.2015.276.283.

410 Oreste, P., Spagnoli, G., and Lo Bianco, L. (2016). "A combined analytical and numerical
411 approach for the evaluation of radial loads on the lining of vertical shafts." *Geotech. Geol.*
412 *Eng.*, 34(4), 1057-1065, DOI: 10.1007/s10706-016-0026-6.

413 Öztürk, H., and Ünal, E. (2001). "Estimation of lining thickness around circular shafts." *Pro-*
414 *ceedings of 17th International Mining Congress and Exhibition of Turkey- IMCET2001*, 437-
415 444.

416 Panet, M., (1995). *Le calcul des tunnels par la méthode convergence-confinement*. Presses
417 *de l'écolenationale des Ponts et chaussées*, Paris.

418 Peck, W.A., and Lee, M.F. (2007). "Application of the Q-system to Australian underground
419 metal mines." *Proceedings International Workshop on Rock Mass Classification in Under-*
420 *ground Mining*, C. Mark, R. Pakalnis, and R.J. Tuchman, eds. NIOSH, OH, pp. 129-140.

421 Santi, P. (2006). "Field methods for characterizing weak rock for engineering." *Env. Eng. Ge-*
422 *osci*, 12(1):1-11.

423 Spagnoli, G., Oreste, P., and Lo Bianco, L. (2016). "New equations for estimating radial
424 loads on deep shaft linings in weak rocks." *Int. J. Geomech*, 16(6): 06016006, DOI:
425 *10.1061/(ASCE)GM.1943-5622.0000657*.

426 Svoboda T. and Masin D. (2010) "Convergence-confinement method for simulating NATM
427 tunnels evaluated by comparison with full 3D simulations." *INTERNATIONAL CONFERENCE*
428 *UNDERGROUND CONSTRUCTION "TRANSPORT AND CITY TUNNEL"*, PRAGUE 2010.

429 Terzaghi, K, (1943), *Theoretical soil mechanics*. John Wiley and Sons, New York.

430 Vlachopoulos, N., and Diederichs, M.S. (2009). "Improved longitudinal displacement profiles
431 for convergence confinement analysis of deep tunnels." *ROCK MECH. ROCK ENG.*,
432 42(2):131-146, DOI: 10.1007/s00603-009-0176-4.

433 Waltham, T. (2009). *Foundations of Engineering Geology*. Spoon Press, Oxon.

434 Wong, R., and Kaiser, P. (1988). "Design and performance evaluation of vertical shafts: ra-
435 tional shaft design method and verification of design method." *Can Geotech J*, 25(2):320-
436 337.

437

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 $R=5\text{m}$ 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).

451 Fig. 4 Generic trend of the ratio u_R/u_{Rmax} (u_R radial wall displacement; u_{Rmax} maximum value
452 of u_R) by changing the distance from the temporary excavation bottom (positive upwards to-
453 wards the excavated rocks). Key: the red dot represents the u_R/u_{Rmax} ratio, obtained at the
454 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.

463 Fig. 6 (A) Plot lining thickness (0.1, 0.2 and 0.3m) vs slope values from Tab. 1 for different R
464 for the weak rock with poor geotechnical properties in order to obtain the value c , i.e. -
465 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.

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

476

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 category A).
479

480

	Rock type A	Rock type B	Rock type C
$u_{R0}/R \times 1000$ 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 $R=1.75\text{m}$, at
482 a depth of $1,000\text{m}$ with rock specific weight of 25kN/m^3 and $s=0.25\text{m}$ for rock types A, B and
483 C