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1 **The effect of gravity of the plastic zones on the behavior of supports in very deep tunnels**
2 **excavated in rock masses**

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13 **ABSTRACT**

14 The vertical load acting on a support structure is affected by the loss of self-bearing capacity of
15 the rock inside the plastic zone. This load can then be accounted for by analytical calculation
16 methods capable of evaluating the stresses in the tunnel support system to proceed with the
17 tunnel design. Generally, the effect of the rock's own weight in the plastic zone is considered in a
18 simplistic way by evaluating an additional vertical load given by the weight of the rock due to
19 the thickness of the plastic zone. This approach leads to a significant increase in the vertical load
20 with the risk of overdesigning the support structure. In this work, the effect of the rock's own
21 weight in the plastic zone was considered by modifying the numerical solution of the
22 convergence-confinement method for tunnels built in rock. In this way, through the intersection

23 of the characteristic curve of the tunnel and the intersection line of the support structure, it is
24 possible to determine both the vertical loads (with the effect of the weight of the rock) and the
25 horizontal load (without the effect of weight of the rock). The application of the method to a
26 project in the Alps allowed to detect the magnitude of the percentage increase of the vertical load
27 and a significant increase in the thickness of the plastic zone with the consequences that this may
28 have on the designing of the radial bolting length in that zone. Increasing the plastic radius leads
29 to an increase in the length of the bolts. This is interesting because in the area of the crown where
30 the weight of the plasticized rock is considered the bolts are usually installed with a greater
31 length. In the final part of the paper a new procedure is illustrated to define the vertical and
32 horizontal loads acting on the support structures, starting from the convergence-confinement
33 curves, obtained for the crown and for the lateral areas (sides).

34 **KEY WORDS:** convergence-confinement method; hyperstatic reaction method; base tunnel;
35 plastic radius; rock

36

37 **ABBREVIATIONS AND NOTATION LIST**

- 38 a exponent of the failure criterion of Hoek and Brown
- 39 A_s area of the lining section
- 40 D parameter that varies between 0 and 1, which considers the disturbance of the rock mass
- 41 due to the excavation operations
- 42 E_r elastic modulus of the rock mass in the plastic field
- 43 F_n limit strength of the elastic-plastic behavior of the normal springs
- 44 f_r component of body forces per unit volume in the radial direction
- 45 f_θ component of body forces per unit volume in the tangential directions
- 46 I_s inertia moment along the lining section
- 47 k stiffness of the lining
- 48 K_n stiffness of the normal interaction springs
- 49 K_s stiffness of the shear interaction springs
- 50 k_0 earth pressure at rest
- 51 M bending moments along the support
- 52 m_b strength parameter, which depend on the GSI (Geological Strength Index)
- 53 m_i strength parameter that refers to intact rock and which depends on the typology of the
- 54 rock
- 55 N normal forces along the support

- 56 p pressure inside the tunnel
- 57 p_{cr} critical pressure
- 58 p_0 natural lithostatic stress at the tunnel depth
- 59 r_0 tunnel radius
- 60 R_{pl} plastic radius
- 61 s strength parameter, which depend on the GSI (Geological Strength Index)
- 62 u_{crown} support displacements at the crown of the tunnel
- 63 u_R radial displacement
- 64 u_{sides} support displacements at the sides of the tunnel
- 65 $\varphi_{res,app}$ apparent residual friction angle of the rock mass
- 66 σ_{Rpl} radial stress at the plastic radius
- 67 σ_r radial stress
- 68 $\sigma_{R,crown}$ vertical load applied at the crown of the support
- 69 $\sigma_{R,sides}$ horizontal load applied at the side of the support
- 70 $\sigma_{r,i}$ incremental radial stress
- 71 $\sigma_{\theta,i}$ incremental tangential stress
- 72 σ_θ tangential stress
- 73 σ_1 principal maximum stress

- 74 $\sigma_{I,lim}$ maximum principle stress upon failure of the rock mass
- 75 σ_3 principal minimum stress
- 76 $\tau_{R\theta}$ shear stress
- 77 θ angle representing the evaluation point in the circular support in the tunnel
- 78 γ weight of the rock
- 79 ψ dilatancy expressed in radians
- 80 ν Poisson ratio of the rock mass
- 81 CC Characteristic curve
- 82 CCM Convergence-confinement method
- 83 GSI Geological Strength Index
- 84 HRM Hyperstatic reaction method

85 **INTRODUCTION**

86 Determination of the stresses and displacements around circular openings has been one of the
87 most fundamental problems in geotechnical, petroleum, and mining engineering. Design of
88 tunnel liners and the validation of numerical models are among the practical applications of
89 displacement analysis around circular openings. The most common support design techniques
90 are based on analytical approaches, such as the hyperstatic reaction method, HRM (Do et al.,
91 2014a; Oreste et al., 2018a; 2018b) and the Einstein and Schwartz (1979) method. However,
92 these analytical methods require knowledge of the loads acting on the support structures. The
93 loads depend on:

- 94 • The dimension and depth of the tunnel;
- 95 • The geomechanical characteristics of the ground;
- 96 • The stiffness characteristics of the support structure itself and;
- 97 • The distance from the excavation face where the structure is to be installed.

98 Interaction between the rock mass and the support system is generally evaluated with the
99 convergence-confinement method (CCM). CCM describes the relation between the decreasing
100 tunnel internal pressure and the increasing tunnel radial convergence and can be constructed
101 from elasto-plastic analysis of a circular tunnel subjected to hydrostatic far-field stress and
102 uniform internal pressure (see Panet, 1995; Peila and Oreste, 1995; Oreste, 2009; 2014; Spagnoli
103 et al., 2016; 2017). CCM allows to obtain an estimate of the loads acting on the supporting
104 structures, proceeding with the intersection of the ground characteristic curve of the tunnel and
105 the characteristic curve of the supporting structure.

106 In the base tunnels in rocks, the extension of the plastic zone around the tunnel may be
107 significant, above all when the GSI value is low, the strength of the intact rock is reduced, the
108 tunnel radius is high and the lithostatic pressure is high. The development of an adequate plastic
109 zone is, however, necessary also to be able to contain the loads on the supporting works. Rock
110 with plastic behavior loses a significant part of its resistance and does not generally have the
111 ability to self-sustain itself. For this reason, it is prudent in the design phase, to consider the rock
112 with plastic behavior in the calculation of the applied loads to the support system with its own
113 weight.

114 A large body of work currently exists on the stress and deformation analysis of tunnels with the
115 consideration of different failure criteria and rock mass behaviors including the elastic-perfectly
116 plastic, elastic-brittle-plastic and elastic-strain-softening models (e.g., Brown et al., 1983;
117 Carranza-Torres and Fairhurst, 1999; Carranza-Torres, 2004; Alonso et al., 2003; Park and Kim,
118 2006; Lee and Pietruszczak, 2008; Park et al., 2008; Fahimifar and Hedayat, 2008; 2009;
119 Fahimifar et al., 2010; Hedayat, 2016). CCM is also applied to rock masses, presenting a non-
120 linear rupture criterion, as described by Hoek and Brown (1980).

121 In the present work a procedure to obtain the characteristic curve of the tunnel considering the
122 effect of the rock weight in the plastic zone is presented. An example of a calculation, relating to
123 a tunnel built in the Alps will allow to evaluate the weight of the rock in the plasticized zone on
124 the loads acting on the support. The final analysis with HRM will allow to detect the importance
125 of the additional load on the tension state that develops in the support work and, therefore, on the
126 stability conditions of the support itself.

127 **THE CONVERGENCE-CONFINEMENT METHOD CONSIDERING GRAVITY EFFECT**

128 When the internal pressure in tunnels falls below a critical pressure, p_{cr} , a plastic zone develops
129 around the tunnel. The tunnel conditions typically assumed for elasto-plastic analysis include a
130 deep circular tunnel in a continuous, homogeneous, isotropic, and initially elastic rock mass
131 subjected to the hydrostatic far-field stress.

132 In base tunnels with high overburden pressure, the initial stress state of the rock mass generally
133 approaches the hydrostatic conditions (k_0 is approximately equal to 1). As the internal pressure
134 decreases, the tunnel radial convergence increases. Brown et al. (1983) summarized a large
135 number of solutions obtained for an axisymmetric tunnel problem and presented a closed-form
136 solution for rocks with elastic-brittle plastic behavior as well as a step-wise sequence of
137 calculations for rock with an elastic-strain-softening behavior. Wang (1996) improved the
138 accuracy of the solution in predicting the plastic radius. Carranza-Torres (2004) proposed a
139 rigorous, elasto-plastic solution by rewriting the generalized Hoek-Brown failure criterion in
140 terms of transformed stress quantities.

141 In the present work a detailed analysis method of CC for rock tunnels will be employed, based
142 on a numerical solution to finite differences (Oreste, 2014). The rock around the tunnel is
143 subdivided into several thin concentric rings, in which the values of apparent cohesion and of the
144 apparent friction angle are continuously determined, linearizing the Hoek-Brown strength
145 criterion according to the value of the radial tension reached. The dilatancy is determined for
146 each concentric ring, as a predefined percentage of the apparent residual friction angle of the
147 specific ring considered. The calculation is repeated for each concentric ring, varying the internal
148 pressure acting on the perimeter of the tunnel. Provided that the inner pressure falls below a
149 critical pressure, p_{cr} , a plastic region of radius R_{pl} develops around the tunnel. Because the dead
150 weight of the broken zone around the tunnel can significantly increase the required support

151 pressure at the roof, the effect of gravity must be considered in the interaction between the
152 ground and the support system. In other words, the dead weight of the broken zone exerts higher
153 pressures to the support system at the crown of the tunnel and needs to be considered in the
154 elasto-plastic analysis of the tunnel. It is suggested that the readers refer to Oreste (2014) for
155 more detailed information. Limited work has been conducted on the effect of the gravitational
156 forces acting on the ground (Hoek and Brown, 1980; Detournay, 1984; Panet 1995; Zareifard
157 and Fahimifar, 2012). To account for the gravity effect, Hoek and Brown (1980) and Panet
158 (1995) suggested an increase in the required support pressure by the amount of $\gamma(R_{pl} - r_0)$,
159 where γ is the unit weight of the rock and r_0 is the tunnel radius. This adjustment assumes that
160 the full weight of the broken zone at the tunnel crown will be transferred to the support system,
161 resulting in ground response curves that are too conservative. Therefore, there is a critical need
162 to study the true interaction between the ground and the supporting system. The weight of the
163 broken zone around the tunnel can be so high that it may affect the tunnel stability. The most
164 critical point is the tunnel crown and the stability assessment needs to be carried out by the
165 construction of the ground response curve at the crown.

166 Assuming a state of hydrostatic stress field and plane strain condition around a circular tunnel,
167 the equilibrium equations in the radial and tangential directions taking the gravity forces into
168 account are defined by assessing the stress state. Consider a small element ABCD shown in Fig.
169 1, since the tunnel problem is under plane strain condition, all stress components are functions of
170 radial and tangential directions. Hence, it is possible to use polar coordinate instead of cylindrical
171 one. Let f_r and f_θ be the components of the body forces per unit volume in radial and tangential
172 directions, respectively (see Fig. 1).

173 Summation of forces parallel to the radial direction through the center of element yields:

$$\begin{aligned}
174 \quad & \left(\sigma_r + \frac{\partial \sigma_r}{\partial r} dr \right) (r + dr) d\theta - \sigma_r r d\theta - \left[\sigma_\theta + \frac{\partial \sigma_\theta}{\partial \theta} d\theta - \sigma_\theta \right] dr \sin \frac{d\theta}{2} + \left[\tau_{r\theta} + \frac{\partial \tau_{r\theta}}{\partial \theta} d\theta - \right. \\
175 \quad & \left. \tau_{r\theta} \right] dr \cos \frac{d\theta}{2} + f_r r dr d\theta = 0 \tag{1}
\end{aligned}$$

176 Since $d\theta$ is infinitesimal, $\sin \frac{d\theta}{2}$ and $\cos \frac{d\theta}{2}$ can be replaced by $\frac{d\theta}{2}$ and unity, respectively.

177 Neglecting the small quantities of higher order and dividing the above equation by $r dr d\theta$, the
178 equation of equilibrium in radial direction can be found as follows:

$$179 \quad \frac{\partial \sigma_r}{\partial r} + \frac{1}{r} \frac{\partial \tau_{r\theta}}{\partial \theta} + \frac{\sigma_r - \sigma_\theta}{r} + f_r = 0 \tag{2}$$

180 Similarly, summation of tangential components of forces may result in the equilibrium equation
181 in direction.

$$182 \quad \frac{1}{r} \frac{\partial \sigma_\theta}{\partial \theta} + \frac{\partial \tau_{r\theta}}{\partial r} + \frac{2\tau_{r\theta}}{r} + f_\theta = 0 \tag{3}$$

183 As the mentioned tunnel problem has axial symmetry, the radial and tangential stresses in the
184 rock mass will be principal stresses (i.e. $\sigma_r = \sigma_3$ and $\sigma_\theta = \sigma_1$) and consequently the radial
185 equilibrium equations can be reduced to:

$$186 \quad \frac{\partial \sigma_r}{\partial r} + \frac{\sigma_r - \sigma_\theta}{r} + f_r = 0 \tag{4}$$

187 In above equation, f_r is the component of body forces per unit volume in the radial direction.

188 Since the construction of ground response curve at the roof of the tunnel, which is the most
189 critical point, is of great importance, γ , unit weight of the rock mass, must be substituted for
190 f_r in the equilibrium equation. In fact, at the roof of the tunnel, the gravity forces per unit volume
191 are radially toward the tunnel center and are equal to γ . Thus, the equilibrium equation within the
192 broken zone can be rewritten as:

$$193 \quad \frac{\partial \sigma_r}{\partial r} + \frac{\sigma_r - \sigma_\theta}{r} + \gamma = 0 \tag{5}$$

194 Fig. 2a demonstrates schematically the body forces acting at the roof of the tunnel according to
195 equation (4). Figure 2b shows the graphical representation of the theoretical ground response
196 curve at the roof with and without consideration of the gravity effect from the plastic zone. The
197 ABC curve represents the ground response curve without consideration of the gravity forces
198 while curve ABMD demonstrates the ground response curve with consideration of gravity
199 forces. The curve ABMD represents comparably larger values of convergence at a given internal
200 pressure. The existence of a minimum M in the ground response curve shows the importance of
201 the dead weight of the broken zone compared with the in situ stresses.

202 Zareifard and Fahimifar (2012) developed an interesting elasto-plastic, analytical-numerical
203 solution, which considers the curvilinear strength criterion of the previous solution of Hoek and
204 Brown (1988). This solution applies to a strain-softening behavior of the rock and links some
205 fundamental rock parameters in the plastic field to the actual deviatoric plastic strain and to the
206 critical value of the deviatoric plastic strain. In order to represent the rock behavior correctly,
207 sophisticated laboratory tests are necessary in order to define in detail the parameters required by
208 the adopted behavior model. Taking into account that the solution of Zareifard and Fahimifar
209 (2012) does not consider the updated generalized failure criterion (i.e. Hoek et al., 2002),
210 equation 6 has also been used in the numerical procedure formulated by Oreste (2014), in order
211 to obtain the description of the CC of the tunnel considering the effect of the weight of the plastic
212 zone around the tunnel. Besides, regarding the dilatation of the rock mass (difficult to evaluate in
213 the laboratory), this is linked in the proposed procedure to the residual friction angle of the rock
214 mass, as a fixed percentage of the latter.

215 **THE PROPOSED NUMERICAL MODEL**

216 The last update of the failure criterion of Hoek and Brown (Hoek et al., 2002) for rock masses
217 has the following expression:

$$218 \quad \sigma_{1,lim} = \sigma_3 + \sigma_{ci} \cdot \left(m_b \cdot \frac{\sigma_3}{\sigma_{ci}} + s \right)^a \quad (6)$$

219 where: $\sigma_{1,lim}$ is the maximum principle stress upon failure of the rock mass;

220 σ_3 is the minimum principle stress (confinement);

221 σ_{ci} is the uniaxial compression strength (UCS) of the intact rock;

222 m_b and s are the strength parameters, which depend on the GSI (Geological Strength
223 Index) (Marinos and Hoek, 2000; Marinos et al., 2005) and on the parameter D :

$$224 \quad m_b = m_i \cdot e^{\left(\frac{GSI-100}{28-14 \cdot D}\right)}; \quad s = e^{\left(\frac{GSI-100}{9-3 \cdot D}\right)}$$

225 D is a parameter that varies between 0 and 1, which considers the disturbance of the rock
226 mass due to the excavation operations ($D=0$ for non-disturbed mass; $D=1$ for intensely
227 disturbed mass);

228 m_i is a strength parameter that refers to intact rock and which depends on the typology of
229 the rock;

230 a is the exponent that is present in eq. 6: $a = 0.5 + \frac{1}{6} \cdot \left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}} \right)$.

231 For the good and medium-quality rock masses exhibiting a fragile elasto-plastic behavior it is
232 necessary to describe two failure criteria, one for the peak conditions and another for the residual
233 conditions. The former governs the stress state in the rock mass at the elastic limit, the latter the
234 stress state in the plastic zone. The resistance parameters are duplicated for the two-different
235 peak and residual criteria: m_{bp} , s_p , a_p , m_{bres} , s_{res} , a_{res} . Peak parameters can be determined by using

236 the GSI referring to the initial conditions of the rock mass, whilst the residual ones to a suitably
 237 reduced GSI (GSI_{res}) (Oreste, 2014).

238 In order to deal with the study of the conditions of the rock mass in the plastic zone, the criterion
 239 of residual failure of Hoek and Brown can be locally linearized by deriving it from the minimum
 240 principle stress σ_3 , which is the radial stress σ_r :

$$241 \quad \frac{d\sigma_{1,lim}}{d\sigma_3} = 1 + a_{res} \cdot m_{bres} \cdot \left(m_{bres} \cdot \frac{\sigma_3}{\sigma_{ci}} + s_{res} \right)^{a_{res}-1} \quad (7)$$

242 This derivative allows to directly obtain the apparent residual friction angle $\varphi_{res,app}$ of the rock
 243 mass, according to the existing minimum stress σ_3 :

$$244 \quad \sin\varphi_{res,app} = \frac{a_{res} \cdot m_{bres} \cdot \left(m_{bres} \cdot \frac{\sigma_3}{\sigma_{ci}} + s_{res} \right)^{a_{res}-1}}{a_{res} \cdot m_{bres} \cdot \left(m_{bres} \cdot \frac{\sigma_3}{\sigma_{ci}} + s_{res} \right)^{a_{res}-1} + 2} \quad (8)$$

245 The numerical procedure for defining the characteristic curve of the tunnel occurs is as follows
 246 (Oreste, 2014):

247 1. Calculation of the radial stress at the plastic radius (σ_{Rpl}): if this value is less than 0, no
 248 plastic zone is created around the tunnel; if it is greater than zero, a plastic zone is created for
 249 pressures inside the tunnel (p) lower than σ_{Rpl} ; the evaluation of σ_{Rpl} occurs by solving the
 250 following equation numerically:

$$251 \quad p_0 - \sigma_{Rpl} = \frac{\sigma_{ci}}{2} \cdot \left(m_{bpl} \cdot \frac{\sigma_{Rpl}}{\sigma_{ci}} + s_p \right)^{a_p} \quad (9)$$

252 2. In the presence of a plastic zone, we proceed with a finite difference method dividing the
 253 rock at the outline of the tunnel in concentric rings. Starting from the tunnel outline, where
 254 the radial stress is equal to the applied internal pressure (p), the stress is increased radially by
 255 a small amount of $\Delta\sigma_r$ value at each ring, until the σ_{Rpl} value is reached. At each value of $\sigma_{r,i}$,

256 the corresponding value of $\sigma_{\theta,i}$ is used, employing the residual failure criterion of Hoek and
 257 Brown;

258 3. By replacing in Equation 6 the term $(\sigma_{\theta}-\sigma_r)$ derived from the criterion of residual failure of
 259 Hoek and Brown:

$$260 \quad \sigma_{\theta} - \sigma_r = \sigma_1 - \sigma_3 = \sigma_{ci} \cdot \left(m_{bres} \cdot \frac{\sigma_3}{\sigma_{ci}} + S_{res} \right)^{a_{res}} \quad (10)$$

261 and transforming the equation obtained in incremental terms, the following expression is
 262 obtained:

$$263 \quad \frac{\sigma_{r,i+1} - \sigma_{r,i}}{r_{i+1} - r_i} = \frac{\sigma_{ci} \cdot \left(m_{bres} \cdot \frac{(\sigma_{r,i+1} + \sigma_{r,i})/2}{\sigma_{ci}} + S_{res} \right)^{a_{res}}}{(r_{i+1} + r_i)/2} - \gamma \quad (11)$$

264 which allows to obtain all the values of r_{i+1} useful for determining the distances of the lateral
 265 surfaces of the concentric rings, comprised between the outline of the tunnel and the plastic
 266 radius R_{pl} ; this procedure also allows to obtain the value of the plastic radius R_{pl} ;

267 4. Starting then from the plastic radius backwards towards the tunnel contour, the deformation
 268 problem is solved. Knowing the stress state in all the concentric rings, the value of the radial
 269 displacement u is obtained on each surface of the concentric rings on the basis of the
 270 following differential equation valid for the plastic field under axisymmetric conditions:

$$271 \quad \frac{du}{dr} = \frac{(1-\nu^2)}{E_r} \cdot \left[(\sigma_r - p_0) \cdot \left(1 - N_{\psi} \cdot \frac{\nu}{1-\nu} \right) + (\sigma_{\theta} - p_0) \cdot \left(N_{\psi} - \frac{\nu}{1-\nu} \right) \right] - N_{\psi} \cdot \frac{u}{r} \quad (12)$$

$$272 \quad \text{where: } N_{\psi} = \frac{1 + \sin \psi}{1 - \sin \psi}$$

273 ψ is the dilatancy expressed in radians (dilatancy is an angle that can vary between
 274 0 and the residual friction angle of the material);

275 ν is the Poisson ratio of the rock mass;

276 E_r is the elastic modulus of the rock mass in the plastic field (it can be evaluated
277 referring to the value of GSI_{res}).

278 5. This differential equation is also transformed into incremental terms in order to be able to use
279 it in the adopted finite difference method;

280 6. The dilation angle ψ is changed at each concentric ring, determining it as a percentage value
281 of $\varphi_{res,app}$ evaluated through equation 9, where for σ_3 the mean radial stress at the specific ith-
282 ring is considered:

$$283 \quad \sigma_3 = \frac{(\sigma_{r,i+1} + \sigma_{r,i})}{2} \quad (13)$$

284 Once the outline of the tunnel is reached, it is possible, therefore, to obtain the radial
285 displacement u_R to be associated with the acting internal pressure p . The pairs of values $p - u_R$
286 allow to trace the characteristic curve of the tunnel, considering the weight of the rock material
287 present in the plastic zone. A simplified sketch of the plastic zone for the case considered in this
288 paper is shown in Fig. 3.

289 **RESULTS AND DISCUSSION**

290 The proposed solution was adopted in relation to a tunnel built in the Alps in metamorphic rocks
291 such as calcareous schists with a GSI of 35 ($GSI_{res} = 35$). The general geology of the area
292 comprises zones of contact between units of the continental crust and units preserving the
293 characters of oceanic crust. The unit where the tunnel is designed consists of a crystalline
294 basement and limited-strength permo-mesozoic metasediments mainly micascists and calcareous
295 schists. The initial section consists of calcareous schists then a gneiss with massive-to-foliated
296 structure is encountered.

297 This tunnel was assumed to be circular with an equivalent radius of 5m. The average overburden
298 was assumed as 170m ($p_0 = 5\text{MPa}$). The parameters of the rock mass are assumed as following:

- 299 • σ_{ci} of the intact rock is 60MPa;
- 300 • $m_i = 12$;
- 301 • disturbance factor $D = 1$;
- 302 • angle of dilatancy ψ equal to the angle of residual friction (associated flow rule assumption);
- 303 • Poisson ratio, ν , equal to 0.3.

304 On the basis of these data, the following parameters have been estimated:

- 305 • elastic module $E = 1635\text{MPa}$;
- 306 • parameter of resistance m_b equal to 0.115;
- 307 • parameter of resistance s equal to 2×10^{-5} ;
- 308 • parameter of resistance $a = 0.519$;
- 309 • the unit weight γ of the rock was evaluated at 30 kN/m^3 .

310 Fig. 4 shows the CCs of the tunnel, in the radial displacement range between 0.05 and 0.25m.

311 The curves referring to three cases are shown:

- 312 1. Case 1: Without considering the weight of the rock in the plastic zone (black continues
313 line);
- 314 2. Case 2: Considering the weight of the rock in the plastic zone according to the
315 calculation procedure illustrated in this article (grey dashed line);
- 316 3. Case 3: Considering the additional load due to the weight of the plasticized rock,
317 according to the conservative approach of Hoek and Brown (1980) and Panet (1995)
318 (black dotted line).

319 In the same figure, the reaction line of the lining is shown in black, consisting of a 25cm thick
320 fiber-reinforced sprayed concrete lining. Assuming an average elastic modulus during the setting
321 time equal to 10000MPa, a stiffness, k , of the lining is obtained equal to 105MPa/m, which
322 provides the slope to the reaction line. The load on the lining is obtained from the intersection of
323 the reaction line with the characteristic curve:

- 324 1. Case 1: 1.57MPa
- 325 2. Case 2: 1.62MPa
- 326 3. Case 3: 1.87MPa

327 It can be noted that the solution proposed in this article leads to an increase in load on the linings
328 of 3.2% due to the weight of the rock in the plasticized zone, while the simplified solution
329 considers an additional load equal to the weight of the rock multiplied by the thickness of the
330 plastic zone (case 3) leading to a considerable increase of 19.1% of the original load (case 1).

331 Figure 5 shows the trend of the plastic zones for case 1 (absence of weigh of the rock) and case 2
332 (proposed solution). In case 3, the trend coincides with case 1. It can be seen that the proposed
333 solution leads to a noticeable increase of the plastic radius until reaching a value of 29.4m for
334 internal pressure p equal to 0, compared to 19.4m of the case 1.

335 The proposed solution of the convergence-confinement method, therefore, provides vertical
336 loads slightly higher than the original ones, however with a significant increase in the thickness
337 of the plastic zone with respect to the case a) (without considering the weight of the rock inside
338 the plastic zone).

339 Three different load conditions on the lining have been studied from the three cases examined in
340 Fig. 4:

- 341 • condition a) refers to the presence of the same load in the crown and on the sides of the
342 tunnel, without considering the effect of the rock's own weight in the plastic zone (case 1);
- 343 • condition b) with the vertical load obtained from the proposed calculation procedure
344 (modified by Oreste, 2014) (case 2) considering the weight of the rock in the plastic zone and
345 the horizontal load without considering the weight of the rock in this zone (case 1);
- 346 • condition c) considering the simplified method of Hoek and Brown (1980) to determine the
347 effect of the weight of the rock in the plastic zone at the crown (case 3, simplified method)
348 and the horizontal load without considering the weight of the rock (case 1).

349 Figure 6 shows the trend of the bending moments along the development of half of the support
350 (starting from the center of the inverted arch up to the center of the crown) by applying the loads
351 as described above:

- 352 • Condition a): 1.57MPa in the vertical direction (case 1) and 1.57MPa in the horizontal one
353 (case 1);
- 354 • Condition b): 1.62MPa in the vertical direction (case 2) and 1.57MPa in the horizontal one
355 (case 1);
- 356 • Condition c): 1.87MPa in the vertical direction (case 3) and 1.57MPa in the horizontal one
357 (case 1);

358 These results were obtained using the hyperstatic reaction method (see Oreste, 2007; Do et al.
359 2014a; 2014b; Oreste et al., 2018a and 2018b for more details). This method provides for the
360 subdivision of the support into one-dimensional elements placed in succession, so as to represent
361 its entire development. The connection points between the elements are called nodes and on
362 them the springs simulating the interaction of the support with the rock face, both in the normal
363 direction and in the shear direction, are anchored. The loads acting on the support are represented

364 with nodal forces and the solution of the problem consists in obtaining the displacements of the
365 nodes of a support structure subjected to vertical and horizontal loads. Once the nodal
366 displacements have been obtained along the structure it is then possible to obtain the trend of the
367 bending moments and of the normal forces, useful for verifying the static conditions of the
368 support. The following data were used in the calculation:

- 369 • Stiffness of the normal interaction springs K_n : 490.5MN/m;
- 370 • Stiffness of the shear interaction springs K_s : 245.2MN/m;
- 371 • Limit strength of the elastic-plastic behavior of the normal springs F_n : 0.26 MN;
- 372 • Cohesion of the support-rock interface: 0.2MPa;
- 373 • Friction angle at the support-rock interface: 25°.

374 Because of the non-symmetry of the applied load, bending moments appear along the lining,
375 which reaches maximum values in the center of the invert, in the center of the crown and in the
376 middle of the side wall of the tunnel. These maximum bending moments, together with the
377 values of the normal force (Fig. 7) in correspondence with the same points at maximum moment,
378 allow to derive the internal stresses developing in the sprayed concrete linings. By comparing the
379 maximum internal stresses acting in the sprayed concrete with the strength of the sprayed
380 concrete, it will be possible to verify whether the hypotheses on the thickness of the lining and
381 on the quality of the sprayed concrete are compatible with the stability of the tunnel.

382 The analysis of the results shown in Fig. 6 allows to verify how the load condition c), which is
383 based on the evaluation of the vertical load in a simplified way, would lead to an overestimation
384 of the maximum bending moments of 600% with respect to the load condition b), for which the
385 evaluation of the vertical load was performed using the proposed calculation method. In the load
386 condition a), the bending moments are zero, thanks to the symmetry of load in the two directions

387 (vertical and horizontal). With regard to the normal forces (figure 7), a constant value is detected
388 along the support in the load condition a); in the load condition of b) the maximum normal force
389 is 3.2% higher than the value of the load condition a); finally, in the condition of load c) the
390 maximum normal force is about 19% higher than that found in the load condition a) and about
391 15% with respect to that obtained under load condition b). It can be noted that the simplified
392 methods, proposed for the determination of vertical load due to the weight of the plastic rock
393 band, lead to a sensible overestimation of the maximum bending moments in the support and a
394 minor overvaluation even of the maximum normal forces.

395 **ANALYSIS OF THE LINING-TUNNEL INTERACTION USING THE OBTAINED** 396 **CONVERGENCE-CONFINEMENT CURVES OF A BASE TUNNEL**

397 Considering the two convergence-confinement curves at the crown and for the sides of the tunnel
398 that can be obtained from the indicated procedure (Fig. 8A), there is a link load-displacement
399 characterizing the perimeter of the tunnel in the crown point and at the sides, based on the
400 behavior of the rock mass present at the boundary of the tunnel. It is also possible to determine
401 the behavior of a supporting structure in a simplified manner on the basis of its axial and flexural
402 stiffness, referring to the studies by Einstein and Schwartz (1979). By combining the two
403 behaviors (of the tunnel and of the supporting lining) it is possible to reach the exact
404 determination of the actual loads acting in the crown and at the sides of the tunnel, the
405 displacements shown by the lining and finally the bending moments and the normal forces that
406 they develop along the support structure. In this way it is possible to make an initial hypothesis
407 about the type and size of the support structure and then verify whether it is able to resist the
408 bending moments and the normal forces induced inside it. More specifically, referring to the
409 work of Einstein and Schwartz (1979), it is possible for the absence of sliding between the

410 support and the rock wall (no-slip case) to obtain the following equations of the support
 411 displacements at the crown (u_{crown}) and at the sides (u_{sides}) of the tunnel:

$$412 \quad u_{crown} = \frac{(d \cdot \sigma_{R,crown} - e \cdot \sigma_{R,sides})}{2 \cdot (d^2 - e^2) \cdot E} \cdot R \cdot (1 + \nu) \cdot \left\{ a_0^* \cdot \left[1 + \frac{1}{d} \cdot \left(\frac{\sigma_{R,sides} \cdot (d^2 - e^2)}{(d \cdot \sigma_{R,crown} - e \cdot \sigma_{R,sides})} - e \right) \right] - \right.$$

$$413 \quad \left. \left[1 - \frac{1}{d} \cdot \left(\frac{\sigma_{R,sides} \cdot (d^2 - e^2)}{(d \cdot \sigma_{R,crown} - e \cdot \sigma_{R,sides})} - e \right) \right] \cdot h \right\}$$

414 (14)

$$415 \quad u_{sides} = \frac{(d \cdot \sigma_{R,crown} - e \cdot \sigma_{R,sides})}{2 \cdot (d^2 - e^2) \cdot E} \cdot R \cdot (1 + \nu) \cdot \left\{ a_0^* \cdot \left[1 + \frac{1}{d} \cdot \left(\frac{\sigma_{R,sides} \cdot (d^2 - e^2)}{(d \cdot \sigma_{R,crown} - e \cdot \sigma_{R,sides})} - e \right) \right] + \right.$$

$$416 \quad \left. \left[1 - \frac{1}{d} \cdot \left(\frac{\sigma_{R,sides} \cdot (d^2 - e^2)}{(d \cdot \sigma_{R,crown} - e \cdot \sigma_{R,sides})} - e \right) \right] \cdot h \right\}$$

417 (15)

418 where: $\sigma_{R,crown}$ is the vertical load applied on the support at the crown;

419 $\sigma_{R,sides}$ is the horizontal load applied on the support at the sides;

420 R is the tunnel radius;

421 E and ν are respectively elastic modulus and Poisson modulus of the rock mass;

$$422 \quad e = \frac{1}{2} \cdot (1 - a_0^*) - (1 - 6 \cdot a_2^* + 4 \cdot b_2^*);$$

$$423 \quad d = \frac{1}{2} \cdot (1 - a_0^*) + (1 - 6 \cdot a_2^* + 4 \cdot b_2^*);$$

$$424 \quad h = 4 \cdot (1 - \nu) \cdot b_2^* - 2 \cdot a_2^*;$$

$$425 \quad f = \frac{1}{d} \cdot \left[\frac{\sigma_{R,sides} \cdot (d^2 - e^2)}{(d \cdot \sigma_{R,crown} - e \cdot \sigma_{R,sides})} - e \right];$$

$$426 \quad a_2^* = \beta \cdot b_2^*;$$

427
$$\beta = \frac{(6+F^*) \cdot C^* \cdot (1-\nu) + 2 \cdot F^* \cdot \nu}{3 \cdot F^* + 3 \cdot C^* + 2 \cdot C^* \cdot F^* \cdot (1-\nu)}$$

428
$$b_2^* = \frac{C^* \cdot (1-\nu)}{2 \cdot [C^* \cdot (1-\nu) + 4 \cdot \nu - 6 \cdot \beta - 3 \cdot \beta \cdot C^* \cdot (1-\nu)]}$$

429
$$a_0^* = \frac{C^* \cdot F^* \cdot (1-\nu)}{C^* + F^* + C^* \cdot F^* \cdot (1-\nu)}$$

430
$$C^* = \frac{E \cdot R \cdot (1-\nu_s^2)}{E_s \cdot A_s \cdot (1-\nu^2)}$$
 (where A_s is the area of the lining section);

431
$$F^* = \frac{E \cdot R^3 \cdot (1-\nu_s^2)}{E_s \cdot I_s \cdot (1-\nu^2)}$$
 (where I_s is the inertia moment of the lining section).

432 Considering the convergence-confinement curve referred to the conditions at the crown (CCC
 433 crown, obtained with the presence of the weight of the rock in the plastic zone), for each point
 434 belonging to the CCC, the values of $\sigma_{R,crown}$ and u_{crown} are determined and are inserted in
 435 equation 14. From this equation $\sigma_{R,sides}$ is obtained and inserted in equation 15. Then, from this
 436 u_{sides} is obtained.

437 We proceed moving on the convergence-confinement curve referred to the crown conditions,
 438 until the pair of values $\sigma_{R,sides} - u_{sides}$ obtained from equations 14 and 15 are compatible with
 439 the convergence-confinement curve referred to the conditions at the sides (CCC sides, obtained
 440 without considering the weight in the plastic zone).

441 When compatibility is found, i.e. the correspondence between the behavior of the rock to the
 442 tunnel contour and the behavior of the support, the procedure stops and the values of the loads
 443 assessed on the crown $\sigma_{R,crown}$ and on the sides of the tunnel $\sigma_{R,sides}$ are used to determine
 444 bending moments (M) and normal forces along the support (N), by changing the angle θ which
 445 is the evaluation point in the circular support in the tunnel (see Fig. 8B):

446
$$M = \frac{(d \cdot \sigma_{R,crown} - e \cdot \sigma_{R,sides}) \cdot R^2}{4 \cdot (d^2 - e^2)} \cdot (1 - f) \cdot (1 - 2 \cdot a_2^* + 2 \cdot b_2^*) \cdot \cos(2\theta) \quad (16)$$

447
$$N = \frac{(d \cdot \sigma_{R,crown} - e \cdot \sigma_{R,sides}) \cdot R}{2 \cdot (d^2 - e^2)} \cdot [(1 + f) \cdot (1 - a_0^*) + (1 - f) \cdot (1 + 2 \cdot a_2^*) \cdot \cos(2\theta)] \quad (17)$$

448 **CONCLUSIONS**

449 During design of the supporting structures it is often necessary to evaluate the load applied by
 450 the rock mass to the outline of the tunnel, above all when using analytical methods of wide
 451 application such as the hyperstatic reaction methods. Generally, load estimation is performed
 452 using the convergence-confinement method, where the characteristic curve of the tunnel
 453 intersects with the support reaction line.

454 In the vertical direction, due to the loss of self-bearing of the rock present in the plastic zone, it is
 455 necessary to consider the weight of the rock in the study of the evolution of stress and
 456 deformations. In this work the methodology in which this aspect can be taken into consideration
 457 has been presented. The numerical solution of the convergence-confinement method introduced
 458 by Oreste (2014) has been modified to take into account the weight of the rock within the plastic
 459 zone. In this way it is possible to obtain the modified characteristic curve in order to reach the
 460 correct evaluation of the vertical load acting on the supporting work. As for the horizontal load,
 461 however, it is possible to still refer to the original convergence-confinement curve, without
 462 taking into account the weight of the plasticized rock.

463 The proposed solution was then applied to a case of a tunnel built in the Alps. Based on the
 464 available data, the convergence-confinement curves of the tunnel were drawn and the loads
 465 acting on the linings were evaluated. These loads were then used in the hyperstatic reaction

466 method in order to obtain the trend of the bending moments and of the normal forces along the
467 development of the lining.

468 From the study developed, it was possible to detect how the proposed solution allows to evaluate
469 vertical loads greater than a few percentage units compared to the case of weightlessness of the
470 plastic zone. The simplified solution considering an additional vertical load equal to the weight
471 of the rock by the thickness of the plastic zone, on the other hand, involves significant increases
472 of about 20%, which are not justifiable in practical terms. Moreover, from the example of
473 calculation carried out, it has been possible to identify a non-negligible increase in the value of
474 the plastic radius when considering the weight of the plastic zone. This increase in the plastic
475 radius can be of great interest due to the repercussions it may have in defining the length of the
476 radial bolting in the crown area.

477 Finally, a specific and quick procedure for the evaluation of the mechanical behavior of the
478 support of a base tunnel has been illustrated, referring to the convergence-confinement method
479 and to the method of analysis of a circular support by Einstein and Schwartz (1979). This
480 procedure starts from determining the two convergence-confinement curves of the tunnel
481 (relative to the crown zone, considering the weight of the plastic zone, and to the lateral areas,
482 without considering the weight of the plastic rock). More specifically, for each point of the
483 convergence-confinement curves of the crown zone (CCC crown) the values of the horizontal
484 load and of the horizontal displacement of the support at the sides of the tunnel are evaluated,
485 compatible with the mechanical behavior of the support. The procedure continues until this pair
486 of values is compatible with the convergence-confinement curves of the lateral zone of the
487 tunnel, i.e. until the stress and deformation analysis of the rock mass is compatible with the stress
488 and deformation analysis of the supporting structure. At that point it is possible to obtain the

489 actual loads (vertical and horizontal) acting on the supporting lining and, therefore, the trend of
490 the bending moments and of the normal forces along the development of the supporting
491 structure.

492 **DISCLAIMER**

493 BASF is not involved in any form with the research presented in this paper and it is only the
494 current affiliation of one of the authors.

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578

579 **FIGURE CAPTION**

580 **Fig 1. Stresses on element ABCD in polar coordinate.**

581 **Fig. 2. (a) Schematic representation of the body forces at the tunnel roof. b) Graphical**
582 **demonstration of the ground response.**

583 **Fig. 3. Schematic representation of the tunnel with radius, R , of 5m with an overburden**
584 **pressure, p_0 , of 5MPa excavated in a rock with GSI_{peak} of 35 and UCS, σ_{ci} , of 60MPa**
585 **showing a plastic radius R_{pl} of 7.3m without considering the weight effect (not to scale).**

586 **Fig. 4. Convergence-confinement curves of the tunnel analyzed in the three considered**
587 **cases**

588 **Fig. 5. Trend of the plastic radius of the tunnel as the internal pressure changes for case 1**
589 **(weightlessness of the rock in the plastic area) and case 2 (proposed solution).**

590 **Fig. 6. Trend of the bending moments along half of the support obtained with the**
591 **hyperstatic reaction method, applying the vertical and horizontal loads derived from the**
592 **analysis with the method of the characteristic curves and the solution proposed in this**
593 **work.**

594 **Fig. 7. Trend of normal forces along half the support obtained by the hyperstatic reaction**
595 **method, applying the vertical and horizontal loads derived from the analysis with the**
596 **method of the characteristic curves and the solution proposed in this work.**

597 **Fig. 8. A). Procedure for the determination of the load values on the support ($\sigma_{R,crown}$ and**
598 **u_{crown}) from the convergence-confinement curve referred at the condition at crown (CCC**
599 **crown) and from that referred at the condition at sides (CCC sides). Key: p is the internal**

600 pressure in the tunnel; u is the radial displacement of the tunnel wall; p_0 is the natural
601 lithostatic load at the tunnel depth; p_{cr} is the internal pressure below which a plastic region
602 at the tunnel contour is formed; $\sigma_{R,crown}$ and $\sigma_{R,sides}$ vertical load acting on the support
603 respectively at the crown and at sides; u_{crown} and u_{sides} are the displacement support
604 respectively at the zone of crown and sides; B) Contact stresses at lining-rocks interface.