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ABSTRACT

The rock mass properties are typically influenced by the excavation technique and the changes in the state of stress due to the rock excavation. The amount and extent of damage introduced in the rock mass depends on the excavation technique and practice quality. The influence of blasting in the rock mass near the tunnel periphery is far more significant due to the energy of waves and redistribution of stresses and the severity of the damage diminishes as the radial distance from the tunnel opening increases. Therefore, it is important to consider the effect of the damaged zone when analyzing the stresses and deformations around a tunnel. This study aimed at providing a new numerical solution for determination of the ground response (reaction) curve with the consideration of the non-uniform damage zone around the tunnel periphery. A deep circular tunnel subjected to hydrostatic stress condition and excavated in rock materials obeying the Hoek-Brown failure criterion is considered. A solution for the determination of stresses, strains, and deformations around the circular deep tunnel is presented in order to correctly assess the attenuation of damaged rock as the dis-

tance from the tunnel perimeter increases considering the loads applied to the supporting structure.

KEY WORDS: Disturbance factor**;** Convergence-confinement method; Damaged zone; Numerical solution

ABBREVIATION AND SYMBOLS

BIDZ Blast-Induced Damaged Zone;

CCC Convergence-Confinement Curve;

CCM Convergence-Confinement Method;

EDZ Excavation Damage Zone;

GSI Global Strength Index;

RQD Rock Quality Designation;

TBM Tunnel Boring Machines;

UCS Unconfined Compressive Strength;

a_p Peak strength parameters of Hoek and Brown;

D Disturbance factor;

D_{in} Initial value of the disturbance factor;

m_i Parameter that depends on the type of intact rock;

$m_{b,p}$ Peak strength parameters of Hoek and Brown;

E_{rm} Elastic modulus of the rock mass;

E_{res} Elastic modulus of the rock mass in residual conditions;

R Radius of the tunnel;

R_p Radius of the circular failure zone;

p_{cr} Critical pressure;

p_i Internal pressure;

p_0 Original lithostatic stress in the rock;

r	Radius of the damaged zone;
$r_{ext,i}$	Distance of the external edge of the ith ring from the tunnel center;
$r_{int,i}$	Distance of the internal edge of the ith ring from the tunnel center;
s_p	Peak strength parameters of Hoek and Brown;
t_{dam}	Thickness of the damaged zone;
u	Radial displacement;
$u_{ext,i}$	Radial displacement on the external edge of the ith ring;
$u_{int,i}$	Radial displacement on the internal edge of the ith ring;
ν_{rm}	Poisson coefficient of the rock;
ε_r	Radial strain;
ε_θ	Circumferential strain;
$\varepsilon_\perp$	Perpendicular strain to the plane comprising the radial and circumferential strains;
φ_{res}	Residual friction angle of the rock;
ψ	Dilatancy angle of the rock;
σ_{ci}	Unconfined compressive strength of the intact rock;
σ_r	Radial stress;
$\sigma_{R_{pl}}$	Radial stress on the plastic radius;
$\sigma_{r,ext,i}$	Radial stress on the external edge of the ith ring;
$\sigma_{\theta,ext,i}$	Circumferential stress on the external edge of the ith ring;
σ_θ	Circumferential stress;
$\sigma_\perp$	Perpendicular stress to the plane comprising the radial and circumferential stresses.

INTRODUCTION

Any underground excavation or opening is surrounded by zones that have been damaged or disturbed to some extent due to the redistribution of rock stresses that occur upon the creation of an underground excavation or as an effect of the excavation itself (Emsley et al., 1997). Therefore, it is necessary to understand the behavior of the rock mass, and to determine the stresses and displacements around circular openings (Hedayat, 2016). The degree and extent of the Excavation Damage Zone (EDZ) varies significantly based on the selected method of excavation (Read, 1996). In tunnels excavated mechanically by Tunnel Boring Machines (TBM), the effect of the damage in the surrounding rock mass is negligible. In the drill-and-blast (D&B) method, however, the influence of excavation disturbance in the rock mass near the tunnel radius is far more significant (Emsley et al., 1997; Martino and Chandler, 2004; Bastante et al., 2012; Zhang et al., 2017).

The effect of the damage is evidently greater on the perimeter of the tunnel and tends to decrease until disappearing at a certain distance from it. For instance, Emsley et al. (1997) and Kwon et al. (2009) stated that only in the near-filed (<2 m) the excavation method plays a role causing a damaged zone, with Rock Quality Designation (RQD) values decreasing around 10-15% in comparison with the RQD in undisturbed rock mass (Kwon et al., 2009; Verna et al., 2014).

It is, therefore, important to consider the effects of a blast-induced damaged zone (BIDZ) when analyzing the stresses and deformations around an excavation. The development of a BIDZ has a considerable effect on the strength and stiffness of the rock mass. This damage to the material is assumed to form a cylindrical zone of influence at a constant extent (Hedayat et al., 2018). Emsley et al. (1997) discussed the boundary between the damaged (irreversible changes in rock properties, see Saiang and Nordlund (2009)) and the disturbed (recoverable changes in rock properties, see Palmström and Singh (2001)), zone is gradational, i.e. there is no distinct boundary but a change which may be defined as the boundary beyond which (at greater distances from the walls) any changes within the rock mass caused by the effects of the excavation are recoverable, thus the disturbed zone, i.e. zone beyond the BIDZ is an elastic region, characterized by undamaged material properties.

The assessment of the BIDZ is important and its importance in the design phase varies in different mining, tunneling or petroleum fields (Olsson and Ouchterlony, 2003; Mandal et al., 2005). Daemen (2011) emphasized on the importance of BIDZ assessment in design of nuclear waste repositories, especially at locations where permanent seals are to be installed and extent and characterization of damaged zone pertaining to design and development of high-level nuclear waste disposal repositories have been extensively studied (Martino and Chandler, 2004; Hudson et al., 2009; Walton et al., 2015). BIDZ in in underground mining and tunneling has, however, received relatively less attention (Scoble et al., 1997). Mandal and Singh (2009) suggested that the damaged zone beyond overbreak zone, i.e. the zone beyond the minimum excavation line of the designed periphery from where rock blocks/slabs detach completely from the rock mass (Verna et al., 2018), should be considered in the design of the tunnel support systems.

Interaction between the ground and the support system can be determined by the convergence-confinement method (CCM), which describes relationship between the internal applied pressure and the radial displacements of a tunnel wall considering an elasto-plastic analysis of a circular tunnel subjected to hydrostatic far-field stress and uniform internal pressure (Rechsteiner and Lombardi, 1974; Panet, 1995; Peila and Oreste, 1995; Oreste, 2009; 2014; Spagnoli et al., 2016; 2017).

Over the years, the convergence-confinement method has also been applied to rock masses with a non-linear failure criterion, as described by Hoek and Brown (1980). When the internal pressure, p_i, in tunnels falls below a critical pressure, p_{cr}, a plastic zone develops around the tunnel (see Fig. 1). When BIDZ is included in the analysis, the dead weight of this broken zone exerts higher pressures to the support system at the crown (roof) of the tunnel which leads to safety factor decrease (e.g. Torbica and Lapčević, 2015; Hedayat et al., 2018). This needs to be considered in the elasto-plastic analysis of the tunnel. However, in order to represent these rock masses in the calculation, some simplifying assumptions are necessary, in particular as far as the strains in the plastic field are concerned (Brown et al., 1983; Carranza-Torres and Fairhurst, 2000).

The current state of practice for the effect of BIDZ is to consider a constant zone of damage for the rock mass with a constant thickness around the tunnel perimeter e.g. Hoek and Karzulovic (2000); González-Cao et al. (2018). Obviously, such simplification can lead to errors in the assessment of the convergences that the tunnel can manifest and of the loads that can be applied to the supporting structure. For this reason, in this work the effect of the damage of the rock mass on the static behavior of the tunnel has been investigated, considering a damage that progressively decreases until it disappears at a certain distance from the perimeter of the tunnel.

A numerical solution to the CCM method has been developed in this study, which makes it possible to analyze the behavior of circular openings in rock masses, without the need of introducing any added simplifying assumptions. The utilized approach is the same as that used when a numerical solution is introduced into the CCM (Oreste, 2014). After having presented the formulation necessary to be able to describe the convergence-confinement curve (CCC) of a tunnel excavated in rock masses, some significant results for typical variation intervals of the Disturbance Factor are reported. The objective of this study is to highlight the importance of certain assumptions about the characteristics of the damaged zone and in particular the need to correctly assess the attenuation of damage to the rock mass as the distance from the perimeter of the tunnel increases.

THE GEOMECHANICAL PARAMETERS OF THE DAMAGED ZONE

To describe the damage zone, the parameter introduced by Hoek and Brown (2018), called Disturbance Factor (D) is used. This parameter, which varies from 0 to 1 (i.e. 0 in the absence of damage and 1 in case of maximum damage), has the merit of being able to produce an immediate estimate of the geomechanical parameters of the rock mass: not only the parameters of the strength criterion of Hoek and Brown of rock masses (the parameters m_b, s

and a) (Hoek and Brown, 2018), but also the elastic modulus E_{rm} (Hoek and Diederichs, 2006):

$$m_b = m_i \cdot e^{\frac{GSI-100}{28-14 \cdot D}} \tag{1}$$

$$s = e^{\frac{GSI-100}{9-3 \cdot D}} \tag{2}$$

$$a = \frac{1}{2} + \frac{1}{6} \cdot \left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}} \right) \tag{3}$$

$$E_{rm}(MPa) = 10^5 \cdot \frac{1 - {}^{D}/_{2}}{1 + e^{\frac{75+25 \cdot D - GSI}{11}}} \tag{4}$$

where:

m_i is a parameter that depends on the type of intact rock (it can be obtained in the laboratory based on the results of triaxial tests);

GSI is the Global Strength Index (Marinos and Hoek, 2000), which measures the geomechanical quality of the rock mass;

D is the Disturbance Factor.

Through these parameters, therefore, it is possible to characterize in detail the rock mass in the damaged zone that is present at the edge of the tunnel. More specifically, it is possible to define the geomechanical parameters of the rock mass at each point of the damaged zone, from the perimeter of the tunnel (where the damage is greatest, $D = D_{in}$) to the extreme of the damaged belt where the damage disappears ($D = 0$). For simplicity, a linear trend of the Disturbance Factor was considered within the damaged zone.

The tunnel problem considered in this paper is shown in Figure 1. In order to analyze the effects of the presence of the damaged zone on the behavior of the tunnel, the convergence-confinement method of the deep and circular tunnel was used, made of a homogeneous and isotropic material with a hydrostatic stress condition.

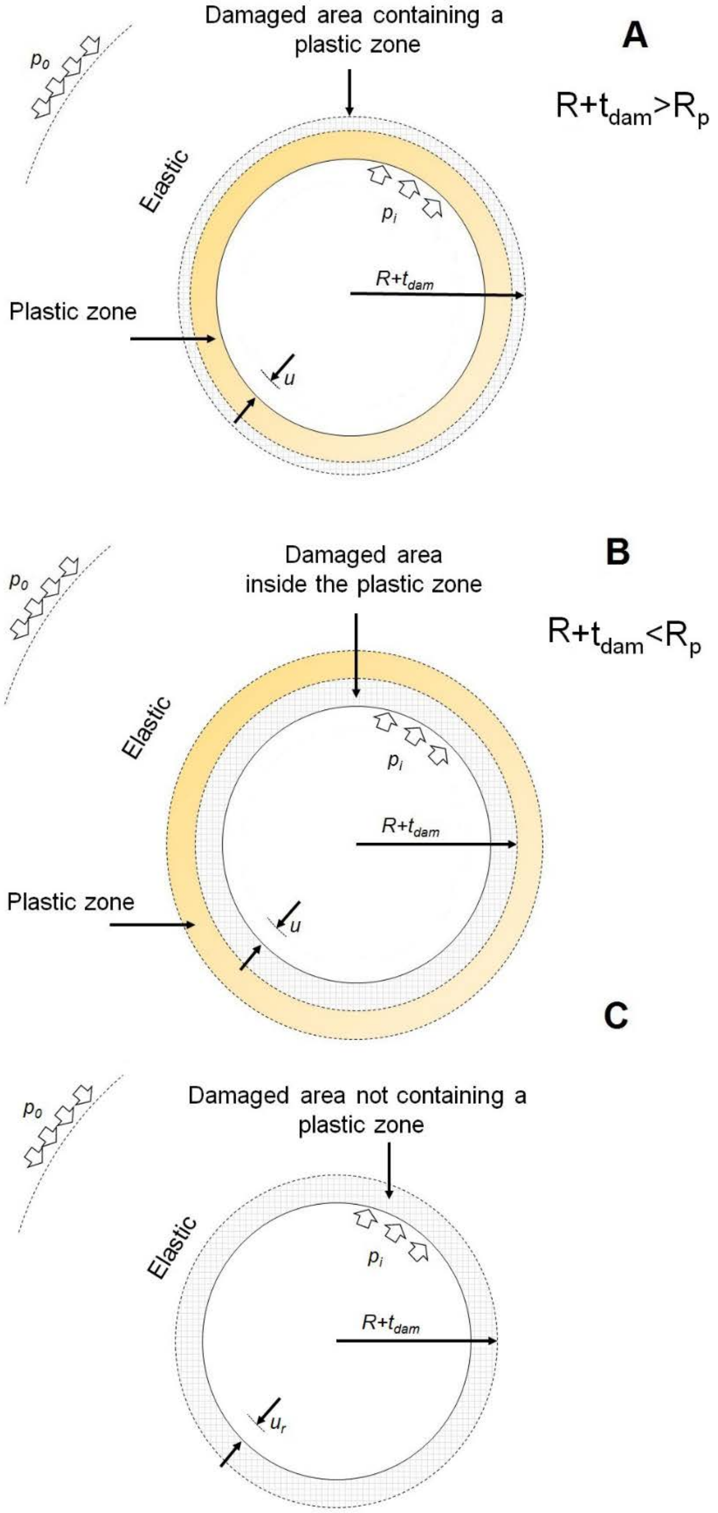

Fig. 1 Sketch of a deep circular tunnel subjected to a hydrostatic stress field with a damaged zone containing a plastic zone (A); with a plastic zone extending beyond the damaged area (B); and with a damaged zone without considering a plastic zone (C).

Because of the blasting impact, a cylindrical BIDZ is developed around the tunnel with different material properties than the rest of the medium. For this tunnel problem, a uniform support pressure of p_i is assumed to act radially on the interior of the tunnel. As the internal pressure decreases, the tunnel radial convergence u_r increases. The ground reaction curve is by definition the relation between the decreasing internal support pressure and increasing ground convergence. When the radial internal pressure falls below a critical pressure, p_{cr}, a circular failure zone of radius R_p develops around the tunnel. More specifically, the solution proposed by Oreste (2014) was used to determine the convergence-confinement curve of the tunnel excavated in rock masses. This solution:

1. considers the generalized failure criterion of Hoek-Brown (updated version of 2002) and a law of brittle elasto-plastic behavior;
2. develops a detailed analysis of plastic deformations in the plastic field;
3. assumes the dilatation ψ as a percentage value of the residual friction angle φ_{res} of the rock mass, evaluated according to the slope of the strenght criterion on the $\sigma_1 - \sigma_3$ curve
4. uses a finite difference numerical solution, which provides for the discretization of the plastic zone in 1000 concentric rings.

Basically, the rock mass obeys a law of brittle elasto-plastic Hoek-Brown behavior (Hoek et al. 2002). The original solution has been modified and integrated to be able to consider the presence of the damaged zone with the value of the variable parameter D inside it. The changes led to a calculation sequence of this type:

- evaluation of the fictitious convergence-confinement curves on the external perimeter of the damaged band (at a distance $r = R + t_{dam}$, where R is the radius of the tunnel and t_{dam} is the thickness of the damaged zone), with the determination of several sets of values of the radial and circumferential stresses and of the radial displacements ($\sigma_r; \sigma_\theta; u$) for different values of the radial stress σ_r;

- division of the damaged zone into 1000 concentric rings in which the stresses and deformations (as well as the radial displacements) are evaluated starting from the most external ring up to the inner ring next to the tunnel perimeter;
- determination of the radial stress and of the radial displacement on the inner edge of the last ring next to the tunnel perimeter; this pair of values represents a point of the convergence-confinement curves on the $p - u$ diagram, in the presence of the damaged rock area.

The numerical solution adopted was suitable to be able to easily deal with the problem of damage variability.

THE CONVERGENCE-CONFINEMENT METHOD APPLICATION IN ROCK MASSESS

The calculation starts from determining the radial stress on the plastic radius ($\sigma_{R_{pl}}$) through the following expression, numerically solved:

$$p_0 - p_{cr} = \frac{\sigma_{ci}}{2} \cdot \left(m_{b,p} \cdot \frac{\sigma_{Rpl}}{\sigma_{ci}} + s_p\right)^{a_p} \qquad (5)$$

The subscript "p" indicates the reference to the peak conditions, different from the residual ones, represented by the "res" subscript throughout the paper, σ_{ci} is the unconfined compressive strength (UCS) of the intact rock.

For pressures inside the tunnel higher than p_{cr}, the rock mass is entirely elastic and, therefore, the equations describing the stresses and the strains in the elastic field in the axisymmetric geometry are considered valid. For these pressure values the convergence-confinement curve appears with a linear trend in the internal pressure-radial displacement of the perimeter diagram ($p - u$).

For pressures below p_{cr}, a plastic zone around the tunnel appears. In this area, in addition to the strength criterion of Hoek and Brown (in the residual conditions) the following two differential equations are valid:

1. The differential equation deriving from the equilibrium of the forces of an infinitesimal element of rock in the polar coordinates:

$$\frac{d\sigma_r}{dr} = \frac{\sigma_\theta - \sigma_r}{r} \quad (6)$$

Where σ_θ and σ_r are respectively the circumferential and radial stresses;

r is the distance from the tunnel centre.

2. The differential equation deriving from the evaluation of strains in the plastic field:

$$\frac{du}{dr} = \frac{(1-\nu_{rm}{}^2)}{E_{res}} \cdot \left[(\sigma_r - p_0) \cdot \left(1 - N_\psi \cdot \frac{\nu}{1-\nu}\right) + (\sigma_\theta - p_0) \cdot \left(N_\psi - \frac{\nu}{1-\nu}\right)\right] - N_\psi \cdot \frac{u}{r} \quad (7)$$

Where:

ν_{rm} is the Poisson coefficient of the rock mass;

p_0 is the original lithostatic stress in the rock mass;

u is the radial displacement;

E_{res} is the elastic modulus of the rock mass in residual conditions;

$N_\psi = \frac{1+sin\psi}{1-sin\psi}$;

ψ is the dilatancy angle in the rock mass, expressed as a fraction of the residual friction angle.

The geomechanical parameters relating to the residual conditions are evaluated by referring to a reduced GSI value:

$GSI_{res} = GSI$ for GSI<35

$GSI_{res} = 35 + \frac{1}{2} \cdot (GSI - 35)$ for GSI≥35

The friction angle of the rock mass is evaluated in apparent terms, starting from the tangent to the strength criterion, for a given value of the minimum main stress (confinement stress).

By varying the pressure inside the tunnel, from the value p_0 to the null value, it is possible to obtain the convergence-confinement curve, i.e. the relation that links the internal pressure to the radial displacement of the tunnel perimeter (Oreste, 2014).

THE VARIABILITY OF THE DISTURBANCE FACTOR IN THE DAMAGED ZONE

Considering the stress and strain state of the damaged zone, it is important to consider the variation of the Disturbance Factor, D, in the numerical solution. D will be lineary changed from an initial value, D_{in}, at the edge of the tunnel ($r = R$) until to a null value D=0 at the boundary of the damaged zone. The adopted numerical procedure for finite differences involves starting from the external radius of the damaged zone ($r = R + t_{dam}$), considering one at a time the 1000 concentric rings of equal thickness in which the damaged zone is divided, until reaching the last ring next to the perimeter of the tunnel.

From the extreme radius of the damaged zone ($r = R + t_{dam}$) the three values ($\sigma_r; \sigma_\theta; u$) are considered which are obtained from the analysis of the natural rock through the evaluation of the convergence-confinement curves of the fictitious radius tunnel ($r = R + t_{dam}$). For each ring considered, it is evaluated whether the stress state is such as to produce the failure of the rock mass, by checking the achievement of the maximum (major) principal stress value of the Hoek and Brown strength criterion:

$$\sigma_\theta \lesseqgtr \sigma_r + \sigma_{ci} \cdot \left(m_{b,p} \cdot \frac{\sigma_r}{\sigma_{ci}} + s_p\right)^{a_p} \tag{8}$$

where,

$m_{b,p}$, s_p and a_p are the peak strength parameters of Hoek and Brown, evaluated in relation to the value of D attributed to the distance r of the considered ring, being D a linear function of r.

If $\sigma_\theta \leq \sigma_r + \sigma_{ci} \cdot \left(m_{b,p} \cdot \frac{\sigma_r}{\sigma_{ci}} + s_p\right)^{a_p}$ an elastic behavior of the damaged rock is observed, therefore the following equations are valid:

$$\varepsilon_r = \frac{(\sigma_r - p_0)}{E_{rm}} - \nu_{rm} \cdot \frac{(\sigma_\theta - p_0)}{E_{rm}} - \nu_{rm} \cdot \frac{(\sigma_\perp - p_0)}{E_{rm}} \tag{9a}$$

$$\varepsilon_\vartheta = \frac{(\sigma_\theta - p_0)}{E_{rm}} - \nu_{rm} \cdot \frac{(\sigma_r - p_0)}{E_{rm}} - \nu_{rm} \cdot \frac{(\sigma_\perp - p_0)}{E_{rm}} \tag{9b}$$

$$\varepsilon_\perp = \frac{(\sigma_\perp - p_0)}{E_{rm}} - \nu_{rm} \cdot \frac{(\sigma_r - p_0)}{E_{rm}} - \nu_{rm} \cdot \frac{(\sigma_\theta - p_0)}{E_{rm}} \tag{9c}$$

Where:

ε_r, ε_θ and $\varepsilon_\perp$ are respectively the radial, circumferential and perpendicular (to the plane comprising the first two) strains;

σ_r, σ_θ and $\sigma_\perp$ are respectively the radial, circumferential and perpendicular (to the plane comprising the first two) stresses;

p_0 is the natural lithostatic stress;

E_{rm} and ν_{rm} are respectively the elastic modulus and the Poisson coefficient of the rock mass; being E_{rm} function of the Disturbance Factor D, it varies at each ring in relation to the distance r of the ring from the tunnel centre.

From the previous equations, being $\varepsilon_\perp$=0 and $\varepsilon_r = du/dr$ we obtain:

$$\frac{du}{dr} = \frac{1}{E_{rm}} \cdot [(1-\nu_{rm}^2) \cdot (\sigma_r - p_0) - (\nu_{rm} + \nu_{rm}^2) \cdot (\sigma_\theta - p_0)] \tag{10}$$

Which in numerical terms can be written in the following way for the generic ith ring:

$$u_{int,i} = \frac{1}{E_{rm,i}} \cdot \left[(1-\nu_{rm}^2) \cdot \left(\sigma_{r,ext,i} - p_0\right) - (\nu_{rm} + \nu_{rm}^2) \cdot \left(\sigma_{\theta,ext,i} - p_0\right)\right] \cdot \left(r_{ext,i} - r_{int,i}\right) + u_{ext,i} \tag{11}$$

where,

$u_{ext,i}$ and $u_{int,i}$ are respectively the radial displacements on the external and internal edge of the ith ring;

$\sigma_{r,ext,i}$ and $\sigma_{\theta,ext,i}$ are respectively the radial and circumferential stresses on the the external edge of the ith ring;

$r_{ext,i}$ and $r_{int,i}$ are respectively the distances of the external and internal edges of the ith ring from the tunnel center. These distances are known, since the damaged zone of thickness t_{dam} was subdived in 1000 concentric rings of equal thickness ($r_{ext,i} - r_{ext,i} = \ t_{dam}/1000$).

The values of strain and stress on the outer edge of the ring i are the same obtained on the inner edge from the calculation of the previous ring (i-1).

Furthermore, the following equation of equilibrium of the forces in the radial direction of the infinitesimal element of rock is always valid (see equation 6). Eq. 6, resolved in numerical incremental terms, allows to obtain the following equation able to supply the radial stress on the inner edge of the generic ring i:

$$\sigma_{r,int,i} = \sigma_{r,ext,i} - \frac{\sigma_{\theta,ext,i}-\sigma_{r,ext,i}}{(r_{ext,i}+r_{int,i})/2}\cdot\left(r_{ext,i}-r_{int,i}\right) \tag{12}$$

Because $\varepsilon_\theta = u/r$ and by substituting where needed, the incremental numerical equation is obtained which gives the value of the circumferential stress on the inner edge of the generic ring i:

$$\sigma_{\theta,int,i} = \left[\frac{E_{rm,i}\cdot\frac{u_{ext,i}+u_{int,i}}{r_{ext,i}+r_{int,i}}+\left(\nu_{rm}+\nu_{rm}^2\right)\cdot\left(\frac{\sigma_{r,ext,i}+\sigma_{r,int,i}}{2}-p_0\right)}{1-\nu_{rm}^2}+p_0\right]\cdot 2-\sigma_{\theta,ext,i} \tag{13}$$

If $\sigma_\theta > \sigma_r + \sigma_{ci}\cdot\left(m_{b,p}\cdot\frac{\sigma_r}{\sigma_{ci}}+s_p\right)^{a_p}$ there is a plastic behavior of the damaged rock and, therefore, the following equation is valid along with equation 6:

$$\sigma_\theta = \sigma_r + \sigma_{ci}\cdot\left(m_{b,res}\cdot\frac{\sigma_r}{\sigma_{ci}}+s_{res}\right)^{a_{res}} \tag{14}$$

By performing the necessary substitutions, the following incremental numerical formulas are obtained, capable of evaluating the stress state on the edge inside the generic ring i, once the stress state on the outer edge is known:

$$\sigma_{r,int,i} = \sigma_{r,ext,i} - \frac{\sigma_{ci}\cdot\left(m_{b,res}\cdot\frac{\sigma_{r,ext}}{\sigma_{ci}}+s_{res}\right)^{a_{res}}}{r_{ext,i}}\cdot\left(r_{ext,i}-r_{int,i}\right) \tag{15}$$

$$\sigma_{\theta,int,i} = \sigma_{r,int,i} + \sigma_{ci}\cdot\left(m_{b,res}\cdot\frac{\sigma_{r,int,i}}{\sigma_{ci}}+s_{res}\right)^{a_{res}} \tag{16}$$

In the plastic field the displacements are governed by the following differential equation (Oreste, 2014):

$$\frac{du}{dr} = \frac{1-\nu_{rm}^2}{E_{res}}\cdot\left[(\sigma_r-p_0)\cdot\left(1-N_\psi\cdot\frac{\nu_{rm}}{1-\nu_{rm}}\right)+(\sigma_\theta-p_0)\cdot\left(N_\psi-\frac{\nu_{rm}}{1-\nu_{rm}}\right)\right]-N_\psi\cdot\frac{u}{r} \tag{17}$$

Where $E_{rm,res}$ is the elastic modulus of the rock mass in the residual conditions, calculated from the GSI in the residual conditions (GSI_{res}). From which the following incremental numerical equation is obtained, which allows to obtain the radial displacement of the inner edge of the generic ring i, once all the other parameters on the outer edge and on the inner edge are known:

$$u_{int,i} = \left\{ u_{ext,i} - \frac{(1-\nu_{rm}^2)\cdot(r_{ext,i}-r_{int,i})}{E_{rm}} \cdot \left[\left(\frac{\sigma_{r,ext,i}+\sigma_{r,int,i}}{2} - p_0 \right) \cdot \left(1 - N_\psi \cdot \frac{\nu_{rm}}{1-\nu_{rm}}\right) + \left(\frac{\sigma_{\theta,ext,i}+\sigma_{\theta,int,i}}{2} - p_0\right)\cdot\left(N_\psi - \frac{\nu_{rm}}{1-\nu_{rm}}\right)\right] + N_\psi \cdot \frac{u_{ext,i}\cdot(r_{ext,i}-r_{int,i})}{(r_{ext,i}+r_{int,i})}\right\} \cdot \frac{1}{1-N_\psi \cdot \frac{(r_{ext,i}-r_{int,i})}{(r_{ext,i}+r_{int,i})}} \qquad (18)$$

Once the last ring is reached, close to the tunnel wall, the values of the radial stress ($\sigma_{r,int,1000}$) and of the radial displacement ($u_{,int,1000}$) on the internal edge represent the pair of $p-u$ values of the convergence-confinement curve of the tunnel. In this way, at every point of the fictitious convergence-confinement curve for a radius $r = R + t_{dam}$, corresponds a point on the real convergence-confinement curves, evaluated considering the presence of the damaged zone.

RESULTS AND DISCUSSION

The above calculation procedure has been applied to a specific case, in order to evaluate the effects of a damaged rock area with variable and decreasing intensity as it moves away from the tunnel wall. The variation of the Disturbance Factor D was considered linear from an initial value D_{in} on the perimeter of the tunnel up to a null value at the end of the damaged zone.

The case of a circular tunnel with a radius R = 3.6 m, excavated in a rock mass having GSI = 45 (GSI in residual conditions: GSI_{res} = 40) has been studied. The lithostatic stress state p_0 was assumed to be 6 MPa, corresponding to a tunnel installed at a depth of about 250 m from the ground surface. For the intact rock the following characteristic values have been considered: the uniaxial compressive strength σ_{ci} = 30 MPa, the m_i parameter of Hoek and Brown equal to 8. The elastic modulus of the rock mass has been obtained from the equation of Hoek and Diederichs (2006) both for the peak value (E_{rm}) and residual conditions (E_{res}). The Poisson ratio of the rock mass (ν_{rm}) was assumed to be constant (i.e. 0.3). The dilatancy angle ψ has been assumed on each point of the plastic zone as 50% of the residual friction angle at that same point. It is therefore variable within the plastic zone in relation to the stress state (radial stress) variation of the existing at each point with changing distance from the tunnel centre, r. Initially the calculation was developed assuming the thickness of the damaged zone of 2 m (t_{dam} = 2 m) and a value of the Distubance Factor on the tunnel wall equal to 0.5 (D_{in} = 0.5) which corresponds to mechanical or hand excavation in poor quality rock masses (no blasting) resulting in minimal disturbance to the surrounding rock mass (Hoek, 2007). D varies linearly and decreases progressively as the distance r increases, until it reaches 0 at the end of the damaged zone ($r = R + t_{dam}$). The result of the convergence-confinement curves for this hypothesis is reported in Fig. 2, together with two other cases studied: 1) the absence of the damaged zone; 2) the case of the presence of a damaged zone of the same thickness (t_{dam} = 2 m), but with a constant value of D (D = 0.5). This last case represents the traditional hypothesis that is adopted for simplicity, considering a constant value of the Disturbance Factor within the damaged zone.

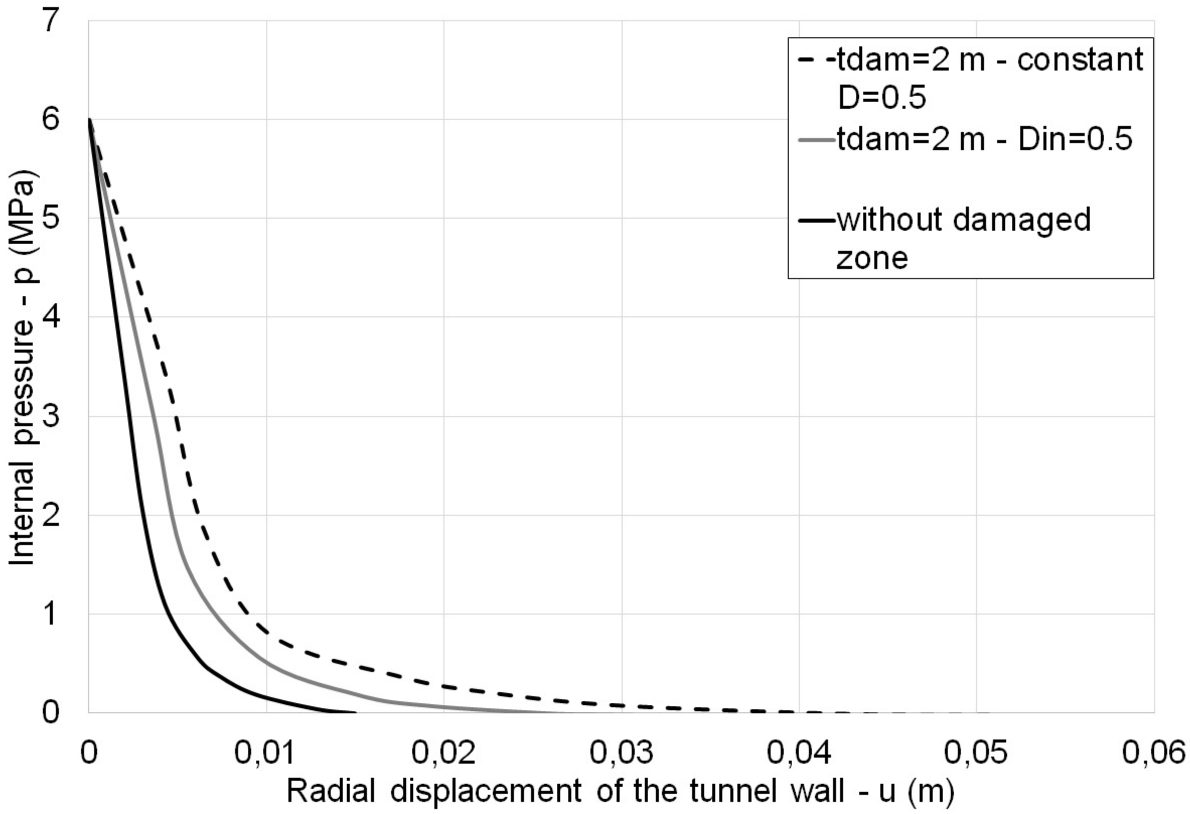

Fig. 2 Convergence-confinement curves for the case of the tunnel studied with attenuation of the value of the Disturbance Factor in the damaged band (grey line), together

with other two cases: absence of the damaged zone (black line) and presence of a damaged zone with value of D constant (dashed black line).

From the analysis of the figure it is possible to see how the effect of the damage on the static behavior of the tunnel is considerably reduced if we consider a linear variation of the Disturbance Factor D (grey line) with respect to the case where D remains constant (simplified approach, dashed black line). On the other hand, the presence of the damaged rock zone cannot be neglected, since there is also a certain difference between the curve obtained from the calculation with the proposed method (grey line) and the case relating to the absence of damaged rock (black line).

Then the results of two different values of the thickness of the damaged rock zone (t_{dam} = 1 and 2 m) and two different initial values of the Disturbance Factor (D_{in} = 0.5 and 1) were compared, considering the linear variation of D inside of the damaged zone. The results are shown in the figure 3.

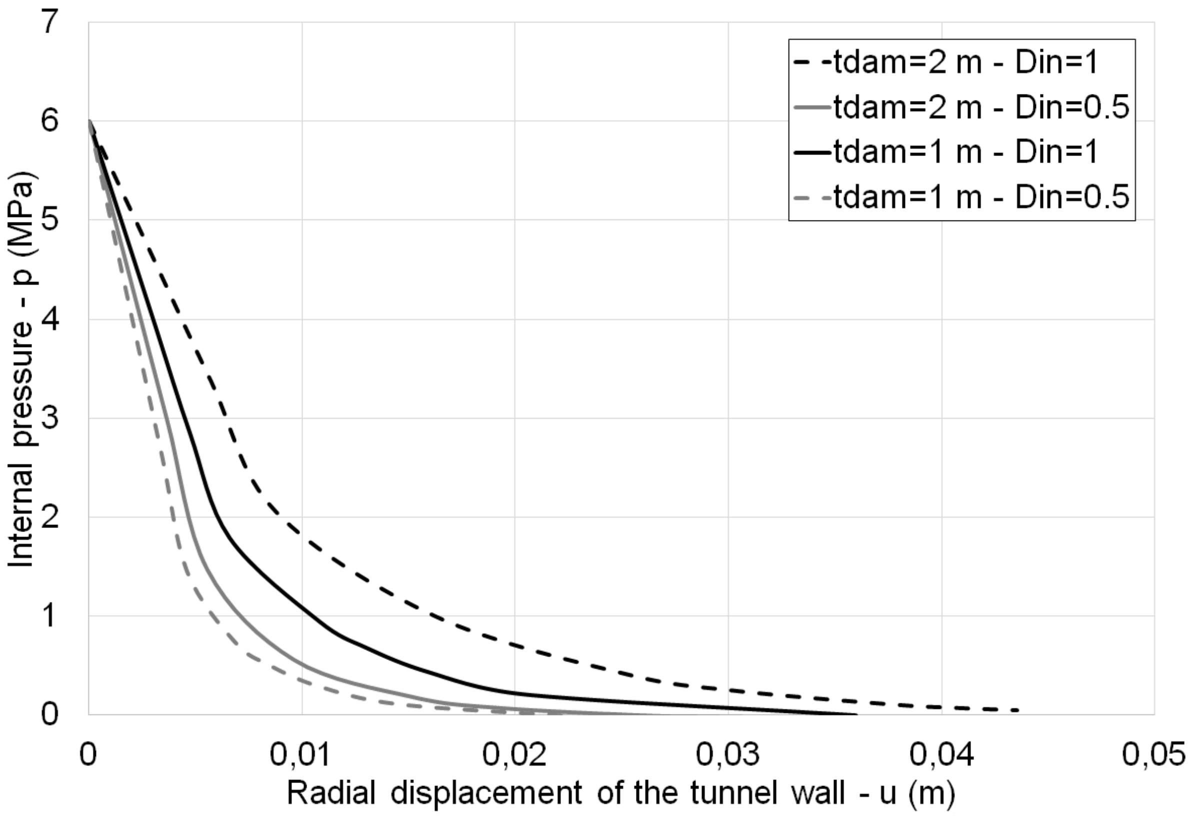

Fig. 3 Convergence-confinement curves for the 4 analyzed cases, varying the thickness of the damaged zone (t_{dam} = 1 and 2 m) and the initial value of D on the tunnel wall (D_{in} = 0.5 and 1).

From the figure 3 it can be noted that the estimation of the maximum value of the damage (D_{in}) on the tunnel wall and the thickness of the damaged zone (t_{dam}) are very important. Both of these parameters strongly influence the trend of convergence- confinement curves, with the relative repercussions on the convergences of the tunnel and, therefore, also on the loads applied to the supporting structures. The intensity of the damage on the perimeter of the tunnel seems, however, to have a more important role than the thickness of the damaged zone.

For this reason, great care should be placed on the correct estimation of the factor D at the tunnel wall and also on the thickness of the damaged zone. When it is not possible to obtain these parameters, it is necessary to define a variability interval for them, possibly associating the estimate of this interval with the probability that the real value falls within it. Subsequently, it is possible to proceed with the evaluation of the convergence-confinement curve assuming the extreme values of the interval of variability, in order to understand the effect of the limit values of D and t_{dam} on the convergences of the tunnel and on the loads applied to the supporting structures.

Subsequently, in order to study the effect of the damaged rock in terms of pressures and displacements on the perimeter of the excavation, the presence of the same damaged zone around tunnels of different geometry and depth and in the presence of a rock mass was considered with different geomechanical quality.

Another 7 cases were analyzed, varying the depth of the tunnel (about 500m with p_0=12MPa, in addition to the case of 250m with p_0=6 MPa), its radius R (7.2m in addition to the 3.6m case) and the geomechanical quality of the rock mass (GSI=75, in addition to the case of GSI=45). The case of p_0 = 6MPa, R = 3.6m and GSI = 45 has already been previously discussed.

In all cases the presence of a damaged zone of 2m (t_{dam} = 2m) and an initial parameter D (D_{in}) on the perimeter of the tunnel equal to 1 was considered. For each of the cases the convergence-confinement curve (CCC) was obtained, which was then compared with the corresponding CCC in natural conditions, i.e. without the presence of the damaged zone. Figs 4 and 5 show the results obtained by the calculation.

From the analysis of the figures it can be noted that in the low/medium geomechanical rock masses (GSI=45) the effect of the presence of a damaged are is very important: the characteristic curve, considering the presence of the damaged zone, moves upwards in a non-negligible way, both for small/medium-sized and large tunnels. For the tunnels of small/medium size the effect is even greater, considering a damaged area of constant thickness in all the cases analyzed. These effects can be found both in shallower tunnels (6MPa) (Figure 4) and in the deeper tunnels (12MPa) (Figure 5).

Comparison of the CCCs between the damaged and natural conditions reveal that as the damage occurs around the tunnel, for the same amount of internal pressure applied, the convergence is significantly larger. Similarly, for the same level of final convergence using the support system, the required internal pressure and correspondingly the support pressure will be significantly higher for the damage case than the natural case. For this reason, it is very useful to know the effect of the presence of the damaged zone on the CCC, in order to be able to correctly design the supports of the tunnel.

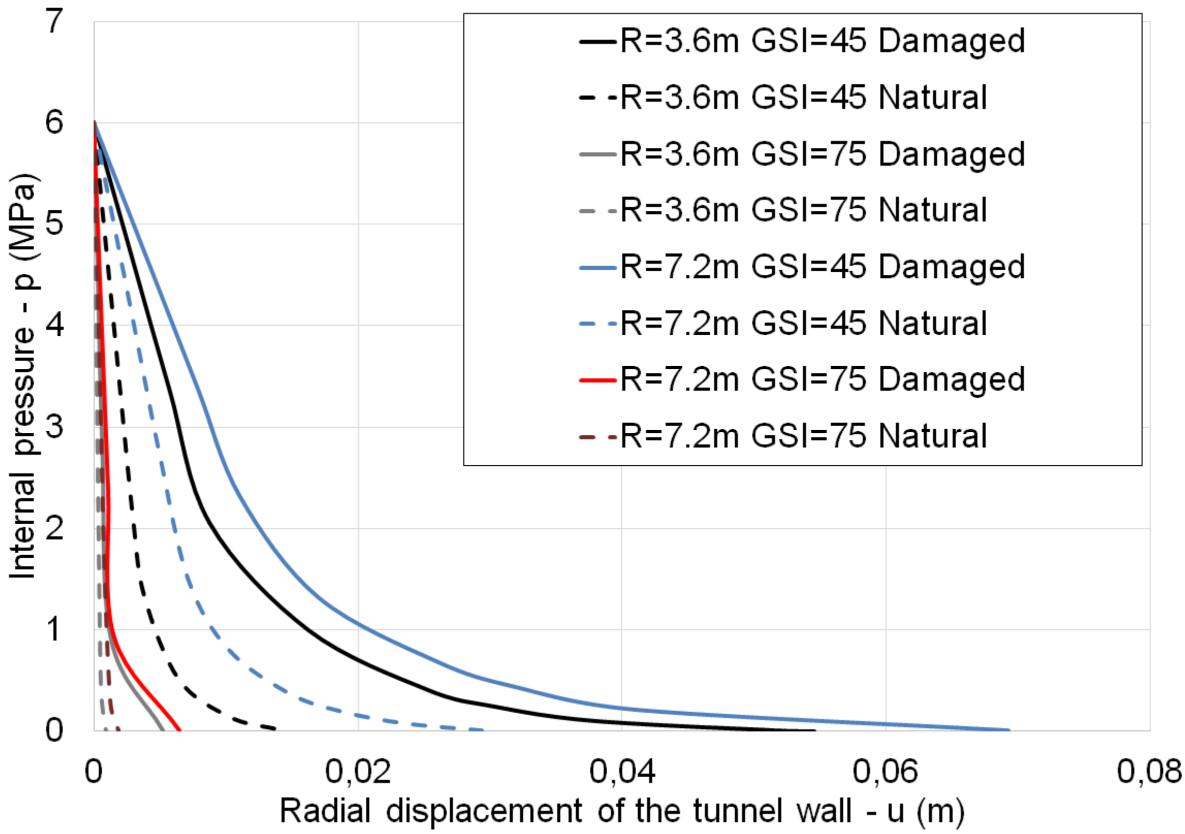

Fig. 4 Convergence-confinement curves for a tunnel radius of 3.6m and 7.2m (R = 3.6m, 7.2m), a tunnel depth of 250m (p_0 = 6MPa) and GSI index of 45 and 75 (considering the presence of a damaged zone with t_{dam} = 2 m and D_{in} = 1 and without the presence of a damaged zone).

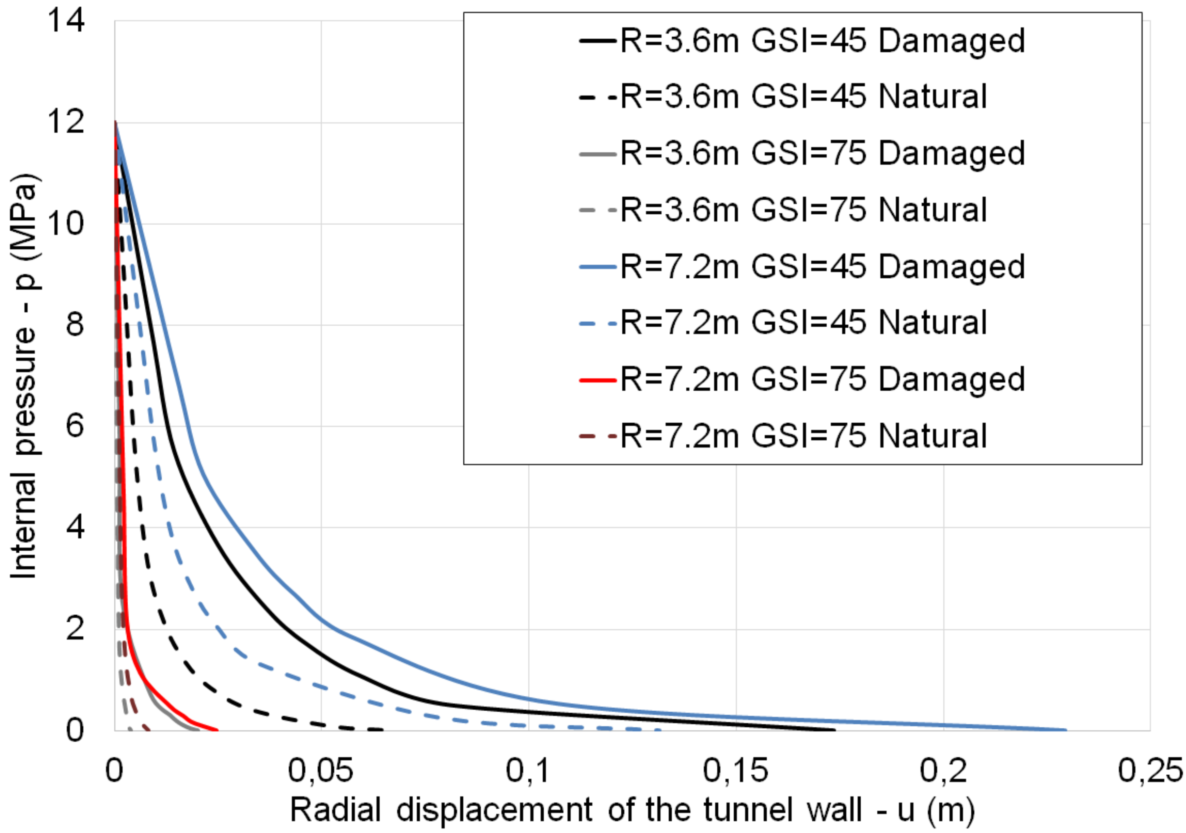

Fig. 5 Convergence-confinement curves for a tunnel radius of 3.6m and 7.2m (R = 3.6m, 7.2m), a tunnel depth of 500m (p_0 = 12MPa) and GSI index of 45 and 75 (considering the presence of a damaged zone with t_{dam} = 2 m and D_{in} = 1 and without the presence of a damaged zone).

CONCLUSIONS

In this study, a numerical solution was developed with the consideration of the degree and extent of the blast induced damage zone around a tunnel. To analyze the effects of the presence of the damaged zone on the behavior of the tunnel, the convergence-confinement method of the deep and circular tunnel made of a homogeneous and isotropic material subjected to a hydrostatic stress condition was used. The damage zone with variable D factor was considered in this study. The presented solution in this paper is novel and allows tunnel engineers to assess the effect of blasting quality on the ground and support interaction in tunnels. Several cases were presented to aid with the application of the presented method. From the example shown previously it was possible to see how the extent to consider the damage area constant for the entire thickness (a simplified approach widely used in practice) can lead to non-negligible errors on the development of the CCC and, therefore, to a considerable overestimation of the loads on the supporting structures. The calculation also allowed to note that the estimate of the thickness of the damaged zone and of the initial damage on the wall of the tunnel have a considerable effect on the CCC. This analysis phase needs special care in the design phase of the tunnel.

A limited parametric analysis on 8 cases, in which the radius and depth of the tunnel and the geomechanical quality of the rock were changed, allowed to detect how the effect of the presence of a certain damaged area around a tunnel can be especially important in small/medium-sized tunnels in rock masses with low/medium geomechanical properties. The depth, i.e. the original stress state of the rock mass, does not seem to play a fundamental role in influencing the trend of the CCC in the presence of a damaged zone: in fact, the same effects were noted both for the shallower and deeper tunnels.

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