Tunnelling and Underground Space Technology 73 (2018) 162–169

Contents lists available at ScienceDirect

Tunnelling and Underground Space Technology

journal homepage: www.elsevier.com/locate/tust

Convergence-confinement curve analysis of excavation stress and strain resulting from blast-induced damage

J. González-Cao[1], L.R. Alejano, E. Alonso*, F.G. Bastante

Natural Resources and Environmental Engineering Department University of Vigo, Campus Lagoas-Marcosende s/n, Vigo, E-36280 Pontevedra, Spain

ARTICLE INFO

Keywords:
Convergence confinement method
Blast-induced damage
Tunnels

ABSTRACT

Using the convergence-confinement curves methodology, we analysed excavation behaviour for a range of rock masses of different geotechnical qualities, taking into account blast-induced damage. The novelty of the research is (i) we include blast-induced damage to the rock mass in ground reaction curve construction, and (ii) we analyse results for 54 rock mass and rock geotechnical quality scenarios. The research, an application of a previously developed methodology (González-Cao et al., 2013), provides practical guidelines for the preliminary design phase for an excavation resulting from blasting. Our main conclusions are (i) that rock mass quality has a greater bearing on the plastic radius and excavation maximum displacement than blast-induced damage, and (ii) that the plastic radius and maximum displacement around an excavation increase with the level of blast-induced damage, most especially for poor quality rock masses. This would justify the need to limit blast-induced damage in poor quality rock masses.

1. Introduction

The convergence-confinement method can be used to evaluate a support/reinforcement system to be installed in an excavation. The method is based on analysing interactions between three curves: (i) the ground reaction curve (GRC), (ii) the longitudinal deformation profile (LDP), and (iii) the support characteristic curve (SCC).

The GRC relates excavation wall deformation to support/reinforcement stress p_i. For homogeneous rock masses the GRC can be constructed using either numerical (Alonso et al., 2003; Guan et al., 2007; Park et al., 2008; Lee and Pietruszczak, 2008; Wang et al., 2010; Zhang et al., 2012) or analytical solutions (Salençon, 1969; Panet, 1993; Duncan, 1993; Carranza-Torres and Fairhurst, 1999; Carranza-Torres, 1998; Carranza-Torres, 2004). Numerical solutions are used in the case of elastoplastic behaviour with strain-softening, whereas analytical solutions are used for elastic perfectly plastic or elasto-brittle behaviour. However, neither of the above solutions is appropriate for GRCs for non-homogeneous rock masses. González-Cao et al. (2013) developed a methodology to numerically construct the GRC when a zone of material of a certain thickness around the excavation wall has elastoplastic properties that are different from those of the intact rock mass.

The LDP relates excavation wall deformation to the distance to the face. To obtain the LDP we used the method developed by Vlachopoulos and Diederichs (2009) according to which excavation wall displacement in function of distance to the face can be obtained from the GRC results (axisymmetric 2D and 3D models). In applying this methodology, the influence of the excavation wall displacement estimate at the moment of placing the support is crucial.

Finally, the SCC represents the stress-strain behaviour of the support/reinforcement system. To build the SCC, we used the method described by Oreste (2003) and compared the results obtained with those of Barton's geomechanical classification Barton and Grimstad (1994).

The convergence-confinement methodology is a simple and non-expensive approach to the preliminary design of support for an excavation. It needs to be applied, however, within a much broader design and construction philosophy based on in-situ measurements and a design-as-you-go approach.

The main limitations are associated with the construction of the ground reaction curve. The usual hypotheses are consideration of the hydrostatic field stress state and/or a circular excavation, which would imply a limitation on application of the methodology. In regard to these two hypotheses, a number of studies provide solutions for non-hydrostatic stress states and/or non-circular excavations, namely, González-Nicieza et al. (2008), Carranza-Torres et al. (2013), Su et al. (2014) and Mousivand et al. (2017). However, note that in those studies, rock mass behaviour that does not consider post-failure behaviour is assumed, or the methodology is applied using commercial software, which may pose

* Corresponding author.
E-mail addresses: jgcao@uvigo.es (J. González-Cao), alejano@uvigo.es (L.R. Alejano), ealonso@uvigo.es (E. Alonso), bastante@uvigo.es (F.G. Bastante).
[1] Present address: Environmental Physics Laboratory, Universidade de Vigo, 32004, Orense, Spain

https://doi.org/10.1016/j.tust.2017.12.005
Received 21 March 2017; Received in revised form 12 July 2017; Accepted 6 December 2017

an additional limitation.

In our opinion, the most important limitations of the methodology are those related to the three-dimensionality issue, obtaining reliable parameters to properly characterize the behaviour of the rock mass and the issue of the homogeneity of the rock mass.

In relation to construction of the ground reaction curve and the three-dimensionality issue, we can mention the works of Vlachopoulus and Diederichs (2014), Mohammadi et al. (2016) and Kaneko et al. (2016).

In relation to characterizing rock mass behaviour, noteworthy are the findings of Alejano (2010), who concludes that one of the most important limitations to application of the methodology is the correct characterization of post-failure behaviour in the plastic zone and correct construction of a longitudinal deformation profile.

Regarding the limitation posed by rock mass homogeneity, we describe the application of the methodology for the case where the stress state is hydrostatic and the excavation is circular, but the rock mass is non-homogeneous (González et al., 2013). This is (homogenous rock mass) one of the most common starting hypotheses for the construction of the ground reaction curve that also conforms least to reality. It is also worth emphasizing the value of using a solution that allows the ground reaction curve to be constructed by programming the solution in a free code (e.g., Tahoe, 2003).

2. Problem statement

2.1. Rock mass behaviour model

The behaviour of a rock mass can be generally considered as: (i) elastic perfectly plastic (EPP) (ii) elastoplastic with strain-softening (SS), or (iii) elasto-brittle (EB). In our research we considered SS behaviour, given that this is the more general case from which EPP and EB behaviours can be deduced as extreme or asymptotic cases.

SS behaviour can be described by a flow rule, f, and a plastic potential, g, both of which are dependent on the stress in the mass and on a variable that measures the plasticity of the material, η (plastic or softening parameter). The variable η is defined, following Alonso et al. (2003), as follows:

$$\eta = \varepsilon_\theta^p - \varepsilon_r^p \tag{1}$$

The flow rule and the plastic potential are both considered to be of the Mohr-Coulomb type, defined, respectively, as:

$$f(\sigma_\theta,\sigma_\theta,\eta) = \sigma_\theta - K_p(\eta)\sigma_r - q_u \tag{2}$$

$$g(\sigma_\theta,\sigma_\theta,\eta) = \sigma_\theta - K_\psi(\eta)\sigma_r \tag{3}$$

In Eqs. (2) and (3), K_p, K_ψ and q_u are defined as follows:

$$K_p(\eta) = \frac{1+\sin(\varphi(\eta))}{1-\sin(\varphi(\eta))} \tag{4}$$

$$K_\psi = \frac{1+\sin\psi}{1-\sin\psi} \tag{5}$$

$$q_u(\eta) = 2C(\eta)\sqrt{K_p(\eta)} \tag{6}$$

The cohesion and friction functions in the above equations are defined as proposed by Alonso et al. (2003):

$$\chi(\eta) = \chi_p - \frac{\chi_p - \chi_r}{\eta^*}\eta;\text{for } \eta \leqslant \eta^* \tag{7}$$

$$\chi = \chi_r;\text{for } \eta > \eta^* \tag{8}$$

In Eqs. (7) and (8) the generic variable χ can be interpreted both for friction $\phi(\eta)$ and for cohesion $C(\eta)$ and subscripts p and r refer to peak and residual values. Dilatancy ψ is considered to be constant. In Eq. (7), η^* is the value of the softening parameter that marks the transition between softening and residual regimes.

2.2. Calculation methods

2.2.1. Ground reaction curve construction

González-Cao et al. (2013) drew on the finite elements method to construct the GRC for non-homogeneous rock masses, assuming axisymmetry in the excavation.

Non-homogeneity in a mass is reflected in a zone of material around the excavation wall, of a certain thickness R_c, where, due to blast-induced damage, the elastoplastic parameters vary in relation to the intact rock mass. This variation is quantified by the damage parameter D, introduced by Hoek et al. (2002). The thickness of this zone is correlated with the damage parameter D, in accordance with Garcia Bastante et al. (2012). Garcia Bastante et al. (2012) described a new model for predicting the extent of blast-induced damage in rock masses, based on Langefors' theory of rock blasting and the hypothesis that the maximum burden parameter defined by Langefors is a good indicator of damage. These authors incorporated the effect of decreased internal energy of the gases as they expand to reach the walls of the borehole before fitting the model to experimental data.

In relation to the application of the convergence-confinement methodology considering blast-induced damage, we can mention Alejano et al. (2010b), who considered the whole rock mass to be damaged (i.e., for an unlimited distance around the excavation). The methodology described in González-Cao et al. (2013), as used in this work, considers that blast-induced damage involves only a limited area around the excavation. The non-homogeneity of the rock mass is explained by the fact that, in addition, it is considered that there is a specific zone of material around the excavation that is damaged by blasting. Beyond this zone the rock mass is assumed not to be affected by the blast. The non-homogeneity is therefore due to the fact that we consider two zones around the excavation with differing behaviour: an excavation-damaged area and a non-damaged area.

The appendix to González-Cao et al. (2013) contains both a verification of the developed code and a validation of the numerical aspects of the algorithm.

González-Cao et al. (2013) constructed the weak form of the equilibrium equation incrementally, taking into consideration the axisymmetry of the problem, and resolved it in a previously discretized domain of interest, as follows:

$$\int \Delta^{k+1}\sigma\varepsilon(v)d\Omega = \int \Delta^{k+1}\sigma v dS \tag{9}$$

The advance of the excavation face can be understood as a matter of resolving a series of problems in which the support/reinforcement stress in the excavation wall (p_i) will decrease from an initial value p_i^O (σ^0 = field stress) to zero (full unloading, null stress).

Fig. 1 graphically depicts the problem to be solved, which requires integrating the constitutive equations for the material in each unloading step (superscript K).

Analysing the range of solutions given in González Cao et al. (2013) for the construction of the ground reaction curve, it is concluded that, for the case where elastoplastic behaviour with softening "degenerates" to elasto brittle plastic behaviour, the incremental stress field can no longer be computed by integrating the system of ordinary differential equations.

The value η^* (Eq. (7)) determines the resolution method to be used, which depends on whether values are lower or higher that a certain critical value η^{crit} — a function, among others, of the type of flow rule. For values lower than η^{crit} (elasto brittle plastic rock mass behaviour), the GRC is constructed by an implicit numerical method; and for values higher than η^{crit} (strain-softening rock mass behaviour), the GRC is constructed by an explicit numerical method (González-Cao et al., 2013). For Mohr-Coulomb type flow rules and plastic potentials, η^{crit} can be obtained from:

$$\eta^{crit} = \frac{(1-\nu)(1+K_\psi)}{2G}K_1 \tag{10}$$