Tunnelling and Underground Space Technology 50 (2015) 365–375

Contents lists available at ScienceDirect

Tunnelling and Underground Space Technology

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

The impact of the physical model selection and rock mass stratification on the results of numerical calculations of the state of rock mass deformation around the roadways

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ARTICLE INFO

Article history:
Received 30 July 2014
Received in revised form 23 June 2015
Accepted 4 August 2015

Keywords:
Rock mass deformations
Numerical calculations
Underground roadways
Rock mass stratification
In situ measurements

ABSTRACT

At present, the basic methods used for designing and evaluating the stability of mine workings are numerical models. The finite element method is the most popular method for engineering purposes. However successful calculations depend not only on the proper selection of geomechanical properties of rocks but mainly on the proper selection of a physical model describing the behavior of the rock mass and a selection of the correct failure criterion. The best way of verifying results of the calculations is to carry out investigation in the field, then.

This article shows how the choice of a numerical model affects the size of the calculated damage zone around the working. To that end, numerical calculations considering elastic and elastic–plastic models were performed for six roadways. The rock mass was further differentiated in terms of its stratification and approach to mechanical properties of the rock mass. The results of these calculations were compared with measurements of mine convergence and the damage zone range in the roof. Such measurements were carried out at hard coal mine roadways.

1. Introduction

At present the basic methods used for designing and evaluating the stability of mine workings, are numerical models. Their rapid development over the past 20 years has resulted in many programs written specifically for supporting mine engineering operations. The finite element method is the most popular method used for solving engineering problems due to the ease of modeling discontinuities, anisotropy, and changes in boundary conditions as well as consideration for dynamic phenomena, although the boundary element method (BEM) and finite differences method (FDM) are also applied. Currently, the world's leading programs in terms of modeling rock mass are: FLAC, UDEC, Cosmos, Adina, Phase^2, MidasGTS. The fundamental representations of the rock mass are elastic–plastic models (Jing, 2003; Lisjak and Grasselli, 2014). The PFC program is becoming more commonly used for issues related to loose soils, as well as rock, although in this case the principles for building the model is a little different (Park et al., 2005; Kidybiński, 2011; Lisjak and Grasselli, 2014). It should be noted that the ability to build a three-dimensional structure (3D models), available in most of the above mentioned programs is not without significance, providing a more thorough analysis of the intersections of excavations and tunnels, pillar areas as well as faults and untypical rock mass stratification (Feng et al., 2012).

The basis for assessing the stability of an excavation, as well as designing appropriate way to reinforce it (i.e. through bolting) is in determining of the size of fracture zone around the roadway and the possible displacements of its contour (Majcherczyk and Małkowski, 2001; Toraño et al., 2002; Yavuz & Fowell, 2004; Małkowski et al., 2008; Круковский, 2012; Prusek, 2010; Kidybiński, 2011; Xiao, 2011; Niedbalski et al., 2013; Zhu et al., 2014).

Taking the efficiency of solving geomechanical problems using numerical methods into account, three crucial points seem to emerge (Jing, 2003):

- proper selection of geomechanical properties of the rocks;
- proper selection of the physical model describing the behavior of the rock mass;
- selection of the correct failure criterion.

Unfortunately, only a few calculations are verified in the field.

This article shows how the choice of a numerical model affects the size of the calculated fracture zone around the working. To that end, numerical calculations were performed in 16 variants for six roadways, and the results of these calculations were compared

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http://dx.doi.org/10.1016/j.tust.2015.08.004
0886-7798/

with the measurements of roadway convergence and the damage zone ranges in the roof. The measurements discussed in this paper were carried out at hard coal mine roadways.

Phase^2 software was chosen to the study. This software has been developed especially for mining engineering and, together with FLAC, seems to be the most universal for a rock mass modeling. Moreover Phase^2 software allows to define the different types of support quickly. UDEC, for example, is more suitable for blocky massifs and Midas GTS for geotechnical design applications.

This new approach to the subject of roadways stability is that rock mass was differentiated in terms of its stratification, considering quasi-homogenous and stratified rock mass. It has been proved that the bedded roof entails the problems with the proper quantitative rock mass movements assessments around the roadways. For such massifs, usually built with sedimentary rocks, numerical solutions should also be verified by research in the field.

2. The choice of computational models and failure criterion

Every material behaves elastically under stress. In the case of rock, brittle failure is the next phase of loading. The examination of rock mass stress and strain should therefore begin with an elastic model. Hence, the elastic deformation and a strength factor should be calculated first. Then for certain time intervals or stress intervals one can follow with non-linear calculations after the creep and/or plastic parameters selection has been indentified. Under certain conditions an elastic model can also produce satisfactory engineering solutions for the rock mass, in line with reality (Małkowski et al., 2008). The only condition is the choice of the proper geomechanical rock mass properties (Fossum, 1985).

Due to evident differences between the mechanical parameters of rock samples and the rock mass, many authors recommend lowering the compressive strength and Young's modulus (Bieniawski, 1978; Hoek and Diederichs, 2006; Dinc et al., 2011). Both parameters are usually determined by rock mass quality indexes. It should be noted, that while there are other approaches, such as empirical considerations, these have not yet found wider application (Dinc et al., 2011). Therefore, two different Young modules were applied for model considerations – one obtained from laboratory tests and one calculated for the rock mass using the Hoek and Diederichs modulus estimation method (2006), assuming $GSI = RMR$.

The Hoek–Brown failure criterion was selected, replacing the *GSI* index with *RMR* determined for each rock strata appearing around the selected workings. This criterion, repeatedly verified through geological and geotechnical observations (Hoek and Brown 1980, 1988; Hoek et al., 2002; Hoek & Marinos, 2007; Prusek, 2008), is currently the leading criterion applied to rocks. The quantitative assessment of various strength criteria performed by Catrin Edelbro from Technical University of Luleå indicates that the Hoek–Brown criterion is one of the systems providing rock stress and displacement values are the closest to reality (Edelbro et al., 2006).

The following procedure is therefore applied in the selection of physical models for rock mass evaluation: first a simple elastic model is developed, which is then enriched with elements of the support structure as well as variable deformation properties of rocks and anisotropy of the mass. A similar procedure is then performed for the elastic–plastic models. A total of eight elastic and eight elastic–plastic models were developed.

The first computational model was an elastic model (model 1) with elasticity modules compatible with values obtained in the laboratory (sample values – intact rock E_i). The second also featured steel yielding support modeled to the working (model 2) with no possibility of a slide. The choice of such a calculation variant was concluded due to the results of the mine research, where the amount of clamp slides in yielding steel frames were so small that the support structure usually remained quite stiff for a period of more than a year.

Due to the apparent anisotropy planes in the Carboniferous rock mass (sedimentary rocks), three models for transversally isotropic rock mass were made. In many cases, the solutions of these models provide satisfactory results for the area of Upper Silesia (Tajduś, 2009). There are five independent constants $E_1 = E_2$, E_3, $\nu_{12} = \nu_{21}$, ν_{31}, G_{12}, and differences in the values of the directional modules depend on the degree of fracturing in the rock mass. The model assumed the same values of Poisson's ratio in all directions and elasticity modules in accordance with sample values for the three variants $E_1 = E_2 = 0.1\ E_3$ (model 3), $E_1 = E_2 = 0.2{\cdot}E_3$ (model 4), $E_1 = E_2 = 0.5{\cdot}E_3$ (model 5).

Subsequent models adopted elastic modules of rock layers with values corresponding to the rock mass, calculated according to the Hoek and Diederichs method (for jointed rock mass E_m). In the first variant – without a support (model 6), the second – with a steel yielding support appropriate to a given working, without possibility of slide (model 7), the third – a yielding support with a 2% possibility of slide relative to the circumference of the steel frame set (model 8).

For rock mass with elastic–plastic properties, calculations were first performed for the model without a support structure, with elastic modulus values corresponding to the rock samples analyzed and post-failure values equal to modules prior to destruction (model 9). The support structure was then modeled to the working (model 10) and the post-failure strength of the rock strata was reduced by 10 times (elastic–plastic model with softening – No.

Table 1
The numerical models description.

Model no	Physical model	Young modulus E	Anisotropy	Post-failure strength reduction	Support	Joints
1	Elastic	Intact rock samples	No	n/a	No	No
2	Elastic	Intact rock samples	No	n/a	Yes, steel yielding with no slide	No
3	Elastic	Intact rock samples	$E_1 = E_2 = 0.1E_3$	n/a	No	No
4	Elastic	Intact rock samples	$E_1 = E_2 = 0.2E_3$	n/a	No	No
5	Elastic	Intact rock samples	$E_1 = E_2 = 0.5E_3$	n/a	No	No
6	Elastic	Jointed rock mass	No	n/a	No	No
7	Elastic	Jointed rock mass	No	n/a	Yes, steel yielding with no slide	No
8	Elastic	Jointed rock mass	No	n/a	Yes, steel yielding with slide	No
9	Elastic–plastic	Intact rock samples	No	No	No	No
10	Elastic–plastic	Intact rock samples	No	No	Yes, steel yielding with no slide	No
11	Elastic–plastic	Intact rock samples	No	Yes, 10 times	No	No
12	Elastic–plastic	Intact rock samples	No	Yes, 10 times	No	Yes
13	Elastic–plastic	Intact rock samples	No	No	No	Yes
14	Elastic–plastic	Jointed rock mass	No	No	No	No
15	Elastic–plastic	Jointed rock mass	No	No	Yes, steel yielding with no slide	No
16	Elastic–plastic	Jointed rock mass	No	Yes, 10 times	No	No