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A damage plasticity model for different types of intact rock

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A B S T R A C T

A constitutive model for describing the nonlinear mechanical behavior of different types of intact rock subjected to complex 3D stress states is presented. It is formulated on the basis of a combination of plasticity theory and the theory of damage mechanics along the lines of a damage-plastic model for concrete. Irreversible deformations, associated with strain hardening and strain softening, as well as degradation of stiffness can be modeled. The material parameters and the model parameters are identified by means of an optimization procedure combining an evolutionary and gradient based optimization algorithm with niching strategy. The proposed model is validated by numerical simulations of laboratory experiments conducted on specimens of marble, granite and sandstone. The simulations are performed at integration point level. Thereby, the capability of capturing key features of the constitutive behavior of different types of intact rock is demonstrated. Finally, the application of the model to structural analyses is demonstrated by simulating a boundary value problem, including the formation of shear bands.

1. Introduction

Intact rock comprises a class of frictional-cohesive materials exhibiting distinct nonlinear mechanical behavior, including irreversible deformations, degradation of stiffness, strain hardening and strain softening. The mechanical behavior of intact rock is investigated by performing laboratory experiments on small-scale specimens like uniaxial and triaxial compression tests [1,2].

Rock mass is composed of intact rock and discontinuities, e.g. bedding planes, joints, etc. [2–7]. The mechanical behavior of rock mass is determined by both, the behavior of intact rock and the discontinuities. The latter are typically not present in laboratory samples [1]. Hence, the mechanical behavior of rock mass is often assessed by investigating intact rock by classical laboratory experiments, conducted on small-scale specimens, and by quantifying the effects of discontinuities by empirical methods like rock mass classification systems and/or laboratory experiments focusing on the discontinuities themselves, e.g. by performing direct shear tests for investigating the behavior of joints [6,7].

The mechanical behavior of rock mass is of great interest for various kinds of underground constructions, e.g. for tunneling and for the construction of underground caverns [3,6]. Rock mass can be modeled either by the continuum approach or the discontinuum approach. In the continuum approach an equivalent continuum model of the rock mass based on a homogenization technique is applied. In cases of one or two dominant sets of small spaced discontinuities this may result in an anisotropic equivalent continuum, while, in case of heavily jointed (quasi-isotropic) rock mass homogenization may yield an isotropic equivalent continuum. In the discontinuum approach discontinuities are modeled directly [8,9].

Different types of numerical methods are available for analyzing 3D rock mechanical problems: continuum based methods for solving the governing partial differential equations of the equivalent continuum, e.g. the finite difference method (FDM), the finite element method (FEM) and the boundary element method (BEM), and discontinuum based methods for solving the equations of motion for rigid blocks of intact rock connected by discontinuities, e.g. the discrete element method (DEM) [5].

The choice of the appropriate approach depends on the type of engineering problem under consideration. At shallow depth, e.g. in case of shallow tunnels, failure of the rock mass is often controlled by the discontinuities present in the rock mass. In such cases discontinuum based models are well suited. On the contrary, underground excavations with high overburden exhibit large stress changes accompanied by plastic deformations, hence, continuum based models are commonly employed [6,9]. Another aspect is the problem size compared to the scale of the discontinuity system. If, on one hand, the size of individual blocks of intact rock is small compared to the problem analyzed, homogenization of the rock mass is appropriate. If, on the other hand, the block size is of the same scale as the problem size, a discontinuum approach is

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appropriate [4,5]. For many problems a combined approach is the most suitable choice. For heavily jointed rock mass with additional single distinct discontinuities, such as faults, a FE model can be adopted including discrete modeling of the individual distinct discontinuities. For coarsely jointed rock mass, for which deformation of the individual blocks should be considered, a DE model can be combined with a continuum approach for the individual blocks [5].

The available numerical tools are very powerful and allow for analyzing complex 3D rock mechanical problems. Nevertheless, in particular for underground constructions with high overburden, reliable estimates of displacements and stresses in the ground-support system are difficult to obtain. In this context the employed material model for describing the mechanical behavior of the rock mass plays an important role. Constitutive models have to be formulated for the equivalent continuum of the rock mass, e.g. the Hoek–Brown model [4,6,10], and/or for discontinuities, e.g. the Barton–Bandis model [11]. An overview on continuum and discontinuum constitutive models for rock is found in [5].

The present contribution addresses constitutive modeling of quasi-isotropic rock mass, aiming at numerical simulations of the advance of deep tunnels within the framework of the continuum approach. In practical rock mechanical engineering the most popular constitutive models for rock mass – adopting the isotropic continuum approach – are formulated on the basis of the flow theory of plasticity assuming linear elastic, perfectly plastic material behavior and non-associated plastic flow. Typically, a failure criterion for rock mass serves as yield function and/or plastic potential. Well-known strength criteria for rock mass, developed in the last decades [5,12], as well as recent developments, e.g. [13–15], can be found in the literature. The Hoek–Brown criterion is probably the most popular one. It combines a failure criterion for intact rock, which is a further development of the failure criterion for concrete proposed by Leon [16], with an empirical approach for considering discontinuities by down-scaling the strength and stiffness parameters of intact rock on the basis of a rock mass classification system [4,6,9,10]. Frequently, a smooth version of the Hoek-Brown criterion, proposed by Menétrey and Willam [17], is adopted. As Hoek–Brown models are only available in some commercial FE-programs, linear elastic, perfectly plastic models with a Mohr–Coulomb failure criterion, non-associated plastic flow and material parameters fitted to the Hoek–Brown criterion are also applied [18,19].

However, this class of models exhibits several shortcomings. Linear elastic behavior is predicted for loading in predominant hydrostatic compression, although the dependence of the deformation properties on the level of hydrostatic pressure is well known [19,20]. Furthermore, they are unable to model strain hardening in the pre-peak regime of the stress–strain curves or strain softening in the post-peak regime, although these characteristics may have a major impact on deformations and stability of underground excavations [1,4,7,21–25]. Hence, these relatively simple constitutive models have limited capabilities. Nevertheless, till now only few researchers proposed models incorporating at least some of these effects, e.g. [21,26–35].

The mentioned shortcomings are the motivation for proposing a constitutive model for different types of intact rock subjected to complex 3D stress paths. Serving as a prerequisite of a constitutive model for rock mass, it will be employed in FE-analyses for determining reliable estimates of displacements and stresses of the ground-support system in underground constructions, in particular with high overburden.

The proposed intact rock model is based on the combination of plasticity theory and the theory of damage mechanics along the lines of advanced material models in the field of concrete engineering, which emanated from the failure criterion, proposed by Leon for concrete,[16] and its modification for rock by Hoek and Brown. Based on the work of Willam, Warnke and Pramono [36,37], Etse [38] proposed the so-called extended Leon model (ELM), formulated in the framework of the flow theory of plasticity. Later it was enhanced by Pivonka [39]. Grassl and Jirásek [40] proposed a damage-plastic model (DPM), representing a further development of the ELM based on the combination of the flow theory of plasticity and theory of damage mechanics. Strain hardening is described on the basis of plasticity theory and strain softening by isotropic damage mechanics. An evaluation of both models by Valentini [41,42] revealed the superior performance of the DPM. Hence, the DPM serves as the basis of the constitutive model for intact rock of the present contribution.

The proposed constitutive model for different types of intact rock requires the determination of seven material parameters and four model parameters. The identification of such a relatively large number of parameters by hand fitting is almost impossible. Thus, a procedure for parameter identification from experimental data adopting inverse parameter identification is presented. Inverse parameter identification for constitutive models using optimization algorithms has been discussed extensively, e.g. in [43–50]. The proposed identification procedure for the presented constitutive model is characterized by a hybrid optimization algorithm combining an evolutionary optimization algorithm with niching strategy, proposed by Kučerová [45], with a gradient based optimization algorithm. In this way the shortcoming of a gradient based optimization method, delivering a parameter set representing a local minimum of the objective function, depending on the chosen initial values of the parameter set, is mitigated by the evolutionary algorithm, while benefit is taken from faster convergence of the gradient based method in the vicinity of local minima [43,44,47,48].

Employing the presented identification scheme, material and model parameters are identified for different types of intact rock from laboratory experiments, conducted by Stavrogin, Tarasov and Shirkes [1,51], based on numerical simulations at integration point level. The proposed constitutive model is validated by numerical simulations of laboratory experiments, which are not used for parameter identification.

Finally, FE-simulations of biaxial compression tests on sandstone, based on different FE-meshes, will be presented for demonstrating the capabilities of the proposed model for structural analyses involving the formation of shear bands.

2. Constitutive model for intact rock

The constitutive model for intact rock is formulated within the framework of plasticity theory and the theory of damage mechanics [40]. In the latter theory the use of the concept of the effective stress $\bar{\boldsymbol{\sigma}}$ is made. It is the “true” stress, which is carried by the intact or undamaged part of a damaged area element [52]. By contrast, the nominal stress $\boldsymbol{\sigma}$ is referred to the total area element and appears in the equilibrium equations. According to the theory of isotropic damage mechanics [52–54] the nominal stress is related to the effective stress by

$$\boldsymbol{\sigma} = \left(1 - \omega\right)\bar{\boldsymbol{\sigma}}. \tag{1}$$

ω $(0 < \omega \le 1)$ denotes the scalar damage variable with the limiting values ω=0 and ω=1 representing the undamaged and the completely damaged state of the material, respectively. For numerical reasons ω is restricted to values slightly smaller than one.

The model is formulated within the framework of the infinitesimal strain theory. Basic assumptions of the plasticity formulation are the additive decomposition of the total strains ε into