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Analysis of the numerical stability of soil slope using virtual-bond general particle dynamics

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ABSTRACT

The stability of soil slope is a critical parameter in geological engineering. In order to better understand the failure of soil slope, the virtual-bond general particle dynamics (VB-GPD) method was developed to simulate the slope stability. Virtual-bond failure was defined by the Drucker-Prager yield criterion, and the initiation and growth of plastic deformation and failure of the soil slope was determined via the plastic-flow rule. Two numerical cases were presented to demonstrate the validity and feasibility of the proposed method. After the safety factor and failure process of these cases were derived, the numerical results obtained via VB-GPD were found to concur with the results obtained by FEM, demonstrating that the VB-GPD method can overcome the excessively skewed mesh for FEM. Hence, it was concluded that the VB-GPD method is efficient at simulating soil slope stability and studying the failure of soil slopes.

1. Introduction

The stability of both natural and anthropogenic slopes is a crucial aspect in both geological and geotechnical engineering (Park and Michalowski, 2017). The limit equilibrium method (LEM) is widely used to conduct slope stability analyses. The most common limit equilibrium technique is the slice method, such as the ordinary method of slices (Fellenius, 1936) and the simplified Bishop method (Liu et al., 2015). A number of studies on slope stability have been performed using the LEM (Zhou and Cheng, 2013; Cheng and Zhou, 2015; Sun et al., 2015; Chakraborty and Dey, 2016; Yu et al., 2018). However, this method tends to give approximate values for the safety factor and does not consider the internal stress-strain relationship, because of which the failure process of the slope cannot be determined (Chen, 2004). Therefore, numerical simulation methods are used to analyze the soil slope stability as they compensate for the drawbacks of the LEMs. For example, Farias and Naylor (1998) used the finite element method (FEM) and Khan et al. (2016) used the finite difference method (FDM) to study slope stability. However, only an approximate safety factor can be determined via FEM and a highly refined mesh is needed to increase its accuracy (Farias and Naylor, 1998). Also, while a highly refined mesh may provide a more precise safety factor, there will still be a defect caused due to the severe mesh distortion that cannot be resolved and will lead to inaccurate calculations (Baghini et al., 2016). Therefore, these limitations of the FEM have motivated scholars and engineers to implement mesh-free techniques to analyze slope stability.

Over the past few decades, mesh-free techniques have quickly evolved in the field of engineering, and many geological engineering problems have been solved via meshless methods, such as the element-free Galerkin (EFG) method (Belytschko et al., 1994), the reproducing kernel particle method (RKPM) (Liu et al., 1995), and smooth particle hydrodynamics (SPH) (Lucy, 1977; Monaghan and Gingold, 1983). However, imposing essential boundary conditions for RKPM continues to be an issue, and subsequently the EFG takes more time for calculation when compared to the SPH method (Chinesta et al., 2011).

Smooth particle hydrodynamics (SPH) was proposed (Lucy, 1977; Monaghan and Gingold, 1983) and, initially, used to model fluid flow. Later on, Swegle et al. (1995), Bui et al. (2011) and Peng et al. (2015) applied and enhanced the SPH method to analyze slope stability and post-failure behavior. In order to simulate tensile crack initiation and propagation, a pseudo-spring-based fracture model was introduced to the SPH framework by Chakraborty and Shaw (2013), in which the efficient interaction between immediate neighbors is formulated by creating a suitable spring-like connectivity among discrete particles. However, the pseudo-spring-based fracture model (Chakraborty and Shaw, 2013) only considered the longitudinal stress and neglected the tangential and shear stresses between particles. Therefore, the pseudo-spring-based fracture model can only be used to simulate the tensile failure of materials caused by elastic damage. However, the failure of soil slopes, which is related to elastoplastic characteristics, is induced

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by longitudinal, tangential, and shear stresses.

The code for general particle dynamics (GPD) (Zhou et al., 2015) was developed using the framework of SPH. The GPD method adopts the concept of life-dead particles and overcomes the shortcomings of the SPH and FEM methods during the simulation of local damage or problems with large deformations. Zhou et al. (2015), Zhou and Bi (2016), Bi et al. (2016a), and Zhou and Zhang (2017) highlighted the mechanisms of crack propagation, fracture and fragmentation by using the GPD method, followed by an analysis of the progressive failure processes of reinforced rock slopes. In the previous GPD code, a particle is killed when its stresses satisfy a critical value, and the killed particle has no effect on its neighbors during the brittle failure process analyzed using GPD. However, the plastic failure process of materials is different from their brittle failure process, and the particle, being in a plastic state, will continue to have an impact on other particles during the plastic deformation process. Therefore, the issue of plastic deformation, which occurs in soil slope instability, cannot be studied well by using the traditional GPD method.

In order to improve the study of plastic deformation using the GPD method, a virtual-bond general particle dynamics (VB-GPD) was proposed to study the failure process and analyze the stability of soil slope. Thus, the virtual-bond model was introduced into the GPD theory and used to describe the interaction between any two particles. Virtual bonds between any two particles are intrinsic. When the internal bonding force satisfies the yield criterion, the virtual bonds between any two particles turn into plastic bonds and localization strain is initiated in these areas. Thus, the VB-GPD can effectively overcome the stress instability of SPH and predict the plastic deformation characteristics of the soil slope with greater accuracy. Also, two numerical examples are provided to validate the accuracy and feasibility of the VB-GPD method in predicting the formation of strain zone and progressive failure process in the soil or/and soil slope. The final numerical results of the safety factor and critical slip surface obtained from the proposed VB-GPD method are very similar to the FEM solutions. Moreover, the change in the surface and toe arrangement of the slope, obtained from VB-GPD, is much more evident than that obtained from FEM, which implies that the proposed VB-GPD method could be a promising numerical method for possible future application in the analysis of large deformations in geotechnical engineering.

This paper is structured as follows: the main steps of VB-GPD are presented in Section 2, the introduction and derivation of virtual bonds are given in Section 3, the numerical results pertaining to the soil sample under axial compression are illustrated in Section 4, an example of soil slope analysis is given in Section 5, and finally the conclusions drawn from the numerical results are in Section 6.

2. Introduction to the VB-GPD algorithms

2.1. Governing equations

The governing equations for the soil slope behavior based on continuum mechanics are given as follows (Lucy, 1977; Monaghan and Gingold, 1983; Libersky and Petscheck, 1991; Shaw and Reid, 2009):

$$\frac{d\rho^{\alpha}}{dt} = -\rho\frac{\partial v^{\beta}}{\partial x^{\beta}} \tag{1}$$

$$\frac{dv^{\alpha}}{dt} = -\frac{1}{\rho}\frac{\partial \sigma^{\alpha\beta}}{\partial x^{\beta}} \tag{2}$$

$$v^{\alpha} = \frac{dx^{\alpha}}{dt} \tag{3}$$

where ρ is the soil initial density, v^{α}denotes the velocity vector, $\sigma^{\alpha\beta}$denotes the total stress tensor, $x^{\alpha}$is spatial coordinate of the particles, ${}^{d}/_{dt}$ is the time derivative within the Lagrangian frame, and the superscripts α and $\beta = 1$, 2 are integer indices indicating the two spatial directions.

2.2. Elastic-plastic constitutive Laws

The total stress tensor in Eq. (2) is composed of hydrostatic and deviatoric stresses that are shown as follows:

$$\sigma^{\alpha\beta} = \tau^{\alpha\beta} - p\delta^{\alpha\beta} \tag{4}$$

where $\delta^{\alpha\beta}$ is called Kronecker's delta, $\alpha = \beta$ when $\delta^{\alpha\beta} = 1$, and $\alpha \neq \beta$ when $\delta^{\alpha\beta} = 0$.

In this paper, the hydrostatic pressure p is expressed as follows:

$$p = -\frac{\sigma^{\gamma\gamma}}{3} = -\frac{1}{3}(\sigma^{xx} + \sigma^{yy} + \sigma^{zz}) \tag{5}$$

where σ^{xx}, $\sigma^{yy}$and, σ^{zz} denote the components of stress tensor in the x, y and, z directions, respectively.

Generally, the stress rate is assumed to be proportional to the strain rate, which is defined by:

$$\dot{\tau}^{\alpha\beta} = 2G\dot{\varepsilon}^{\alpha\beta} \tag{6}$$

where G is shear modulus, $\dot{\tau}^{\alpha\beta}$ is stress rate, and $\dot{\varepsilon}^{\alpha\beta}$ is strain rate. Using the definition of the total stress tensor in Eq. (4) and the hydrostatic pressure in Eq. (5), the stress rate can be derived as follows:

$$\dot{\sigma}^{\alpha\beta} = 2G\left(\dot{\varepsilon}^{\alpha\beta} - \frac{1}{3}\dot{\varepsilon}^{\gamma\gamma}\,\delta^{\alpha\beta}\right) \tag{7}$$

When considering the issue of deformation, a stress rate must be considered for the sake of objectivity. This is known as the Jaumann stress rate $\hat{\sigma}^{\alpha\beta}$ and is defined by:

$$\hat{\sigma}^{\alpha\beta} = 2G\left(\dot{\varepsilon}^{\alpha\beta} - \frac{1}{3}\dot{\varepsilon}^{\gamma\gamma}\,\delta^{\alpha\beta}\right) + \sigma^{\alpha\gamma}\dot{\omega}^{\beta\gamma} + \sigma^{\gamma\beta}\dot{\omega}^{\alpha\gamma} \tag{8}$$

where the strain rate $\dot{\varepsilon}^{\alpha\beta}$is defined as follows:

$$\dot{\varepsilon}^{\alpha\beta} = \frac{1}{2}\left(\frac{\partial v^{\alpha}}{\partial x^{\beta}} + \frac{\partial v^{\beta}}{\partial x^{\alpha}}\right) \tag{9}$$

and $\dot{\omega}$ is the spin rate tensor given by:

$$\dot{\omega}^{\alpha\beta} = \frac{1}{2}\left(\frac{\partial v^{\alpha}}{\partial x^{\beta}} - \frac{\partial v^{\beta}}{\partial x^{\alpha}}\right) \tag{10}$$

The total strain rate tensor $\dot{\varepsilon}^{\alpha\beta}$ is obtained by the following formula (Bui et al., 2011):

$$\dot{\varepsilon}^{\alpha\beta} = \dot{\varepsilon}_{e}^{\alpha\beta} + \dot{\varepsilon}_{p}^{\alpha\beta} \tag{11}$$

where the elastic strain rate tensor $\dot{\varepsilon}_{e}^{\alpha\beta}$ is expressed as follows:

$$\dot{\varepsilon}_{e}^{\alpha\beta} = \frac{\dot{\tau}^{\alpha\beta}}{2G} + \frac{1-2\nu}{3E}\dot{\sigma}^{\gamma\gamma}\delta^{\alpha\beta} \tag{12}$$

where G is the shear modulus, $\dot{\tau}^{\alpha\beta}$ is the deviatoric stress rate tensor, E is the Young's modulus, and ν is the Poisson's ratio.

The plastic strain rate tensor $\dot{\varepsilon}_{p}^{\alpha\beta}$is defined by plastic flow rules as follows:

$$\dot{\varepsilon}_{p}^{\alpha\beta} = \dot{\lambda}\,\frac{\partial Q}{\partial \sigma^{\alpha\beta}} \tag{13}$$

where Q is the potential function, which controls the direction of the plastic strain gradient; $\dot{\lambda}$ is the rate of change of the plastic multiplier λ. The value of the plastic multiplier λ is calculated by using the consistency condition expressed as follows:

$$df = \frac{\partial f}{\partial \sigma^{\alpha\beta}}d\sigma^{\alpha\beta} = 0 \tag{14}$$

When the internal friction angle ϕ is relatively small, it leads to results related to non-associated flow that were employed in this study. The plastic potential function can be expressed as: