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Research Paper

A new model for response of laterally loaded piles in soil-rock mixtures

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ABSTRACT

The S-RM is treated as an equivalent homogenous *c-φ* geomaterial. A hyperbolic *p–y* curve, which is governed by initial stiffness and ultimate resistance, is used to describe the nonlinear interaction between the S-RM and the pile. The initial stiffness and ultimate resistance are derived based on two-phase composite material theory and empirical equations for friction angle and cohesion of S-RMs, respectively. The predicted responses of laterally loaded piles in S-RMs with different volumetric block proportions are validated with 3D numerical simulations. The influencing factors on the *p–y* curves are also investigated in this paper.

1. Introduction

Piles are often used in a foundation design to support infrastructure, both on land and in the ocean, such as bridges, high-rise buildings, platforms, and windmills. The pile's response under lateral loading from the infrastructure is usually analyzed with the *p–y* curve method, assuming that the soil is a series of nonlinear springs connected to the pile segments. Different types of geomaterials, such as clay, sand, and rock mass, have different formulas to express the *p–y* curves [1–4]. In practice, foundations composed of soil and coarse aggregates, such as rock blocks, gravel, and cobble particles, are very common. This type of mixture is generally referred to as soil-rock mixture (S-RM) [5] or bimrock [6]; in this study, the term S-RM is used. Because the S-RMs contain oversized rock blocks, conventional triaxial or direct shear test cannot be used to investigate the influence of oversized rock blocks on the mechanical behavior of S-RMs due to the size limitation of the test equipment. For the design of laterally loaded piles in the S-RM, however, it is essential to develop a *p–y* curve specifically for S-RM so that the mechanical properties of the S-RM are correctly accounted for.

The motivation to develop *p–y* curves for S-RMs is based on the fact that the mechanical properties of S-RMs are different from those of soils or rock masses. Finding the threshold between soils and rock blocks is the first task to be addressed in practice. Medley and Lindquis proposed a characteristic engineering dimension (L_c), which can be defined as, for example, the height of the slope in slope engineering and the diameter of the sample in triaxial tests, and 5% of L_c was suggested as the threshold to differentiate soil matrix and rock blocks in S-RMs [6,7]. This has become the standard for the preparation of samples for in situ and laboratory tests of S-RMs. In these tests, the S-RMs are often characterized as a *c-φ* geomaterial, and thus the Mohr-Coulomb criterion is used to describe the shear strength of the S-RMs. Through in situ shear tests, Xu et al. [5] found that the shear strength of the S-RM was related to the weight proportion of the rock blocks. Coli et al. [8] found an overall larger friction angle and lower cohesion for S-RMs than for a clayey matrix. Xu et al. [9] found that the deformation and fracture mechanisms of the S-RM were controlled by the volumetric block proportion (*VBP*) of the rock blocks. The *VBP* is a critical parameter of the shear strength of S-RMs, a fact also verified by numerical simulations and laboratory tests on S-RM samples. Vallejo and Mawby [10] attributed the influence of rock blocks on the shear strength of S-RMs to the different porosities corresponding to different *VBP*s. By using discrete element method (DEM) simulations, Xu et al. [11] concluded that the shear strength of S-RMs was higher than that of soils because of the existence of rock blocks. Li [12] proposed two equations for the estimation of the constant volume friction angle, based on laboratory tests on mixtures of fine and coarse fractions. Ruggeri et al. [13] investigated the possibility of estimating the shear strength of S-RMs based on laboratory experiments. The identification of the matrix fraction in well-graded mixtures and the effect of variations in the grading of the granular fraction were discussed. Jin et al. [14] conducted laboratory and numerical experiments on S-RM samples with and without cements. They found the influence of the *VBP* on the S-RM strength was dependent on the skeleton effect of the rock blocks. Based on statistical and regression analyses of laboratory and in situ test data, Kalender et al. [15] proposed equations to predict the cohesion and friction angle of S-RMs. These equations describe the influence of the

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VBP, the soil–rock interface strength, and the repose of the rock blocks on the overall strength of the S-RM.

For the design of laterally loaded piles in S-RMs, both the friction angle and cohesion should be considered in the *p–y* curve method. However, existing *p–y* curves are only suitable for cohesive soil, cohesionless soil, or rock masses. For cohesive soil, Matlock [16], Reese and Welch [17] obtained the *p–y* curves by in situ testing of soft marine clay sites and stiff clay foundations, respectively. For cohesionless soil, Reese et al. [2] developed *p–y* curves for laterally loaded piles in sand. Fan and Long [18] evaluated *p–y* curves for sand using the finite element method. For rock masses, Reese [3] proposed a *p–y* curve for weak rock based on two sets of in situ experimental data. Cho et al. [19] modified Reese's weak rock model [3] based on in situ test data. Liang et al. [4] proposed *p–y* curves for rock masses based on theoretical and numerical analyses. The *p–y* curves were verified by monitoring the pile response. Although previous researchers did not provide straightforward *p–y* curves for S-RM, they found that the curves were not only related to the pile design parameters, such as pile diameter and stiffness, but also to the mechanical properties of the geomaterials, such as the initial stiffness and ultimate resistance of soils or rock masses. These critical parameters and the methods by which *p–y* curves were developed provide a possible path for the development of *p–y* curves for laterally loaded piles in S-RMs. Considering the inhomogeneity of S-RMs, the variability of rock block position, and the rock block size, the *p–y* curves for S-RMs could be used for reliability analyses of both active and passive piles in S-RMs.

In this study, we first develop *p–y* curves based on the hyperbolic equation. A framework for developing the *p–y* curves of S-RMs is proposed. Second, the *p–y* curve for S-RM is implemented in the laterally loaded pile software *Pypile* (www.yongtechnology.com) that uses the Java programming language. The responses of the piles simulated with *Pypile* and the three-dimensional numerical simulations are compared to verify the proposed *p–y* curve method. Third, the influences of the *VBP*, the soil–rock interface strength, and the repose of the rock blocks on the proposed *p–y* curve are investigated. Finally, we discuss the proper ranges of *VBP* and rock block diameters for the proposed method.

2. Methodology

2.1. Hyperbolic formula of the p–y curve

Several mathematical formulas, such as the hyperbolic equation and the exponential function, have been used to match *p–y* curves obtained from laterally loaded pile tests. Yang [20] investigated different mathematical formulas based on the goodness of fit with *p–y* curves derived from field test data. It was concluded that the hyperbolic equation could be used to provide the best fit for the general shape of the *p–y* curves deduced from the tests. The hyperbolic equation was also successfully used to express *p–y* curves in sand [21], clay [1,22], and rock masses [4]. This illustrates that the hyperbolic equation is suitable for both soils and rocks. Therefore, the hyperbolic equation is adopted in this study to express the *p–y* curves for S-RMs. The hyperbolic equation is mathematically expressed as

$$p = \frac{y}{\frac{1}{K_i} + \frac{y}{p_u}} \tag{1}$$

where K_i is the initial stiffness of the *p–y* curve (initial tangent slope of the *p–y* curve) and p_u is the ultimate resistance of the S-RM per unit pile length. The shape of the hyperbolic *p–y* curve is controlled by the values of K_i and p_u according to Eq. (1).

2.2. Initial stiffness of S-RMs

Because there are no existing mathematical formulas to express the initial stiffness of S-RM, two initial stiffness formulas are evaluated in this paper. The first one is suitable for laterally loaded piles in clay, which is proposed by Bowles [23] and Rajashree and Sitharam [24] and expressed as:

$$K_1 = \frac{1.3E}{(1-\nu^2)}\left(\frac{ED^4}{E_pI_p}\right)^{\frac{1}{12}} \tag{2}$$

where E is Young's modulus of the soil; υ is Poisson's ratio of the soil; D is the pile diameter; and E_pI_p is the flexural rigidity of the pile. Georgiadis and Georgiadis [25] proved that Eq. (2) estimated the initial stiffness with reasonable accuracy in clayey soils, with discrepancies not exceeding 15%.

The second initial stiffness formula is suitable for rock masses, which is proposed by Liang et al. [4] and expressed as:

$$K_2 = E\left(\frac{D}{D_{ref}}\right)e^{-2\nu}\left(\frac{E_pI_p}{ED^4}\right)^{0.284} \tag{3}$$

where D_{ref} is a reference pile diameter and equal to 0.305 m.

These two formulas reflect the difference between initial stiffness of a laterally loaded pile in a relatively soft geomaterial (clay) and in a relatively hard geomaterial (rock masses). These two geomaterials are common components the of the S-RM (e.g. the matrix and block). Thus, Eqs. (2) and (3) are selected as the initial stiffness of the *p–y* curve for S-RMs. The applicability of these equations will be evaluated later. Correspondingly, the soil parameters, E and υ in Eqs. (2) and (3), should represent the elastic modulus and Poisson's ratio of S-RMs, respectively.

2.3. Ultimate resistance of the S-RM

The Brinch-Hansen method [26] is introduced to derive the ultimate resistance of S-RMs over the depth of the pile. Since S-RMs are c-φ geomaterials, the methods for cohesive soil ($\varphi = 0$) or cohesionless soil ($c = 0$) is not suitable for the calculation of the ultimate resistance. However, the Brinch-Hansen method can be an alternative to the theories for cohesive or cohesionless soils by setting $\varphi = 0$ or $c = 0$. Thus, the ultimate resistance is expressed as

$$p_u = (K_q\gamma z + K_c c)D \tag{4}$$

$$\gamma = (1-VBP)\gamma_m + VBP\gamma_b \tag{5}$$

where p_u is the ultimate resistance per unit pile length, which is related to the difference between passive and active earth pressures; γ, γ_m, and γ_b are the unit weights of the S-RM, the soil matrix, and the rock blocks, respectively; c is the cohesion of the S-RM; D is the pile diameter; z is the depth below the ground surface; K_q and K_c are passive resistance coefficients, and are functions of φ and normalized z/D, respectively. The equations related to K_q and K_c are provided in Appendix A. Typical values of K_q and K_c with different friction angles are shown in Fig. 1.

3. Mechanical properties of S-RMs

3.1. Elastic modulus and Poisson's ratio

Based on the theorems of minimum potential energy and minimum complementary energy of the theory of elasticity, Hashin [27] proposed an overall model for the elastic modulus and Poisson's ratio of composite materials composed of an elastic matrix and rigid particles. The Hashin equations [27] to calculate the elastic modulus and the Poisson's ratio of S-RMs are re-written as follows:

$$E = E_m[1 + \frac{3(1-\nu_m)(5\nu_m^2-\nu_m+3)}{(1+\nu_m)(4-5\nu_m)}VBP] \tag{6}$$

$$\nu = \nu_m + \frac{3(1-\nu_m)(1-5\nu_m)(1-2\nu_m)}{2(4-5\nu_m)}VBP \tag{7}$$