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Stability analysis of soil behind a vertical free-face between supporting piles

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ABSTRACT

Estimating soil stability between supporting piles is of practical significance in geotechnical engineering. However, few methods exist that can solve this problem with proper usability. The primary purpose of this paper is to present a new analytical approach that can be used to calculate the factor of safety of soil with a vertical free-face between two adjacent supporting piles. This analytical approach combines the upper bound method with the strength reduction technique. After an appropriately simplified model of a three-dimensional slip surface is developed, the input energy and the energy dissipation of the instability mechanism can be calculated algebraically. Then, the factor of safety of the soil between the supporting piles can be obtained using the work–energy balance equation. A single-variable method is used to analyze the influence of geometry and soil properties on the factors of safety. The magnitude of the factor of safety is generally found to decrease nonlinearly with increasing height of the vertical free-face of the soil, decrease nonlinearly with increasing clear spacing between the two adjacent piles, increase with increasing internal friction angle and increase linearly with increasing soil cohesion. Numerical analyses are performed with the explicit finite difference code FLAC^{3D}, confirming these results. The relationships between the factor of safety and the input parameters obtained from the theoretical procedure are found to be generally similar to that obtained from the FLAC^{3D} numerical analyses. The factors of safety calculated by FLAC^{3D} show acceptable differences from the theoretical results in most calculation domains. In addition, an estimation approach of the seismic stability of soil between the supporting piles is proposed by combining the upper bound method with the pseudo-static method.

1. Introduction

Supporting piles are a popular retaining system used in foundation pit engineering and slope engineering (Martin and Chen, 2005; Lirer, 2012; Song et al., 2012; Zhou et al., 2014). Like every earth retaining structure, this system in practice must be checked for failures, which should consider seismic fortification in earthquake-prone regions. In the last several decades, research into supporting piles has been focused on the global stability of the supported slopes or excavation pits (Wei and Cheng, 2009; Yu et al., 1998; Yu and Deng, 2007) and the soil–pile interaction (Boulanger et al., 1999; Chen and Martin, 2002; Won et al., 2005). The stability of the soil between the supporting piles is usually estimated based on the designer's experience or empirical rules. Unfortunately, there are many engineering accidents that have been ascribed to the collapse of the soil between piles, particularly in earthquake-prone areas (Wang et al., 2009; Li and He, 2009). Even if lagging between the supporting piles is planned, severe damage may occur before lagging installation (Perko and Boulden, 2008). Thus, the stability of the soil behind a vertical free-face between supporting piles must be measured scientifically during design and construction. However, few studies concerning this problem have been reported.

The limit analysis method is a well-developed theory to determine the load-carrying capacity of plastic materials (Chen, 2013). Idealizing soil as a perfectly plastic material that obeys associated flow rules, two plastic bounding theorems (i.e., upper and lower bounds) can be determined. In the initial stage of its development, most operating procedures are restricted to two dimensions. Recently, publications regarding the limit analysis method in three dimensions have also been available (Chen et al., 2001). However, those existing methods are not suitable for soil between supporting piles due to a more complex boundary condition and sophisticated failure mechanism.

After developing a simplified model of a three-dimensional slip surface, this article presents an analytical approach to calculate the factor of safety of soil behind a vertical free-face between supporting piles and verifies its validity with numerical methods. Furthermore, the approach is extended to consider earthquake loads in combination with the pseudo-static method.

2. Definition of the problem

A section of an excavation pit with cantilevered supporting piles is used as a representative case of the problem in question (Fig. 1). The

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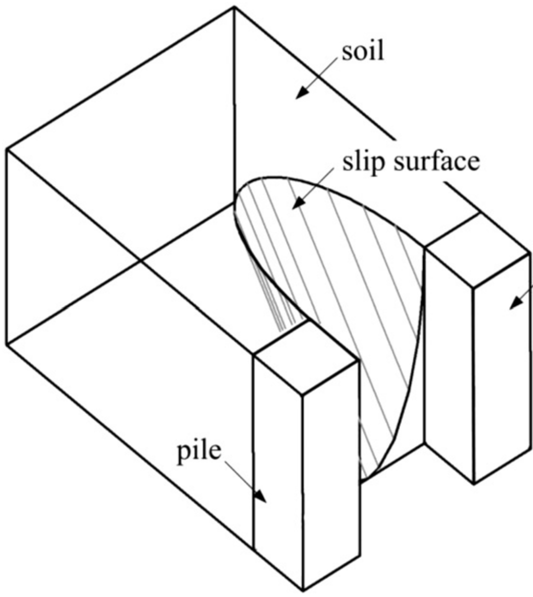

Fig. 1. Sketch plot of the representative case.

clear spacing between two adjacent piles is represented by w. The height of the vertical free-face of the soil between the piles is represented by h. The properties of the soil are described by the internal friction angle φ, the cohesion c, and the unit weight γ. The objective of this paper is to determine the stability of soil behind the vertical free-face, so the deformation and displacement of the piles are ignored. Hence, the supporting piles are considered rigid bodies without displacement.

3. Theoretical analysis procedure

3.1. Methodology

According to the upper bound theorem, if a set of external loads acts on a failure mechanism and the work performed by the external loads in an increment of displacement equals the work performed by the internal stress, then the external loads obtained are not lower than the true collapse loads (Chen, 2013). It is noted that the external loads are not necessarily in equilibrium with the internal stresses and that the mechanism of failure is not necessarily the primary failure mechanism. By examining different mechanisms, the best (i.e., lowest) upper bound value may be found. To ensure the validity of the limit theorems, classical calculation procedures of the upper bound method usually assume that the soil behaves as an associated plastic material with an angle of dilation, ψ, which equals the angle of friction, φ.

Factors of safety obtained from the strength reduction method are recommended as the stability index of soil between piles because this evaluation method is easy to use and can consider earthquake loads. The shear strength reduction technique reduces soil strength parameters from their initial values until the soil fails. When the Mohr–Coulomb criterion is considered, the factor of safety F_s can be calculated by the following equation:

$$F_s = c/c' = \tan\varphi/\tan\varphi' \quad (1)$$

where c and φ represent the initial soil strength parameters, and c' and φ' are the reduced shear strength parameters of the soil immediately prior to failure.

3.2. Simplified model of a three-dimensional slip surface

An important step of the upper bound analysis is to establish suitable instability mechanisms. After analyzing a series of model test results, the features of the slip surface of the soil between the two adjacent supporting piles have been summarized as follows (Zhang et al., 2014):

(1) The slip surface is symmetrical to the vertical plane (i.e., neutral plane), which is located in the middle of two piles;
(2) The intersection of the slip surface with a horizontal plane is a parabola;
(3) The intersection of the slip surface with a vertical plane (i.e., orthogonal to the free-face between piles) can be approximated to a straight line.

Based on these features, a simplified model of a three-dimensional slip surface can be created, as shown in Fig. 2. The simplified slip surface is generated by translating a generatrix (i.e., a straight line, according to the third feature of slip surfaces) along a directrix (i.e., a parabola, according to the second feature of slip surfaces). The directrix is on the top horizontal surface of the soil behind the piles, and the intersection of the directrix with the vertical free-face is located on the edge of the pile. The angle between the generatrix and the vertical free-face is represented by β; the height of the vertical free-face is h; and the width of the vertical free-face is w.

The soil between two adjacent supporting piles is divided into two parts by the three-dimensional slip surface. The soil mass above the slip surface slides down with velocity V; and the angle between velocity V and the slip surface is φ, as shown in Fig. 2.

Then, the input energy W, which is equal to the power of gravity W_g for a non-seismic condition, and the energy dissipation D can be calculated algebraically.

3.3. Input energy and energy dissipation

Because it is difficult to directly calculate the power of gravity W_g and the rate of internal energy dissipation along the slip surface, which is represented by D, a discrete procedure is suggested, as shown in Fig. 3.

Considering the sliding soil mass above the slip surface as the research subject, a set of auxiliary planes parallel to the plane *yoz* (coordinate system shown in Fig. 2) are drawn to divide the sliding soil mass into several slices. The number of slices is set to $2n$, and thus, the thickness of each slice is $w/(2n)$. For each soil slice, another plane is drawn parallel to the generatrix. Then, a *step surface* is generated to approximate the slip surface.

After the *step surface* is established, according to the second feature of slip surfaces mentioned above, the equation of the directrix can be written as:

$$y = ax^2 + b \quad (2)$$

where a and b are undetermined coefficients. Because the coordinates of points A and B are known, we can deduce that $a = -\frac{4h\tan\beta}{w^2}$ and $b = h\tan\beta$. In addition, the following geometrical relationships are true:

$$OE_i = -\frac{4h\tan\beta}{w^2}\left(i\cdot\frac{w}{2n}\right)^2 + h\tan\beta \quad (3)$$

$$E_0G_0 = \sqrt{h^2 + (h\tan\beta)^2} \quad (4)$$

$$OE_0 = h\tan\beta \quad (5)$$

$$E_iG_i = h\left(1-\frac{i^2}{n^2}\right)\sqrt{1+\tan^2\beta} \quad (6)$$

$$OG_i = h\left(1-\frac{i^2}{n^2}\right). \quad (7)$$