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Non-limit passive soil pressure on rigid retaining walls

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ABSTRACT

This paper aims to reveal the depth distribution law of non-limit passive soil pressure on rigid retaining wall that rotates about the top of the wall (rotation around the top (RT) model). Based on Coulomb theory, the disturbance degree theory, as well as the spring-element model, by setting the rotation angle of the wall as the disturbance parameter, we establish both a depth distribution function for sand and a nonlinear depth distribution calculation method for the non-limit passive soil pressure on a rigid retaining wall under the RT model, which is then compared with experiment. The results suggest that under the RT model: the non-limit soil pressure has a nonlinear distribution; the backfill disturbance degree and the lateral soil pressure increase with an increase in the wall rotation angle; and, the points where the resultant lateral soil pressure acts on the retaining wall are less than 2/3 of the height of the wall. The soil pressure predicted by the theoretical calculation put forward in this paper are quite similar to those obtained by the model experiment, which verifies the theoretical value, and the engineering guidance provided by the calculations are of significance.

1. Introduction

At present, the calculations of soil pressure on retaining walls or supporting structures in civil engineering are mostly based on Rankine theory and Coulomb theory, which are both ultimate-state soil pressure theories. However, the displacement of the wall and soil when reaching the ultimate state is often not allowed in civil engineering [1]. These two theories are the two fundamental approaches for analyzing the passive soil pressure, but both assume the backfill being able to reach the ultimate equilibrium state. The soil pressures obtained by Rankine or Coulomb theory show a linear increase with an increase of depth, which is more similar to the distribution of the soil pressure under the translation mode (T mode) of the retaining wall. As a specific example, coal-mining activities can result in uneven surface subsidence, leading to the formation of a subsidence basin [2–4], where the soil body in the compression zone squeezes abutment foundations. As for abutments or other retaining structures in mining areas, there might be rotation around the top (RT) of the abutment as the top is constrained by a rigid deck. Many valuable experiments and numerical simulations associated with soil pressure against retaining walls have been conducted [5–11]. It has been found that the amount of wall movement and the particulars of the movement modes have great influence on the passive soil pressures against rigid retaining walls. Bang et al. believe that the distribution of the passive soil pressure on rigid retaining walls varies with the amount of the displacement and the displacement mode, based on which they proposed the concept of the non-limit state. Fang et al. carried out a model experiment related to this concept with different displacement modes, as well as different densities of soil [5–8]. Zhou carried out a model experiment for clay with different displacement modes [9]. Xu et al. conducted a comparative study of a model experiment and a numerical analysis [10]. Yang carried out a non-limit soil pressure experiment on an inclined retaining wall with an inclined surface of sand [11].

Many researchers have studied the non-limit soil pressure theory based on the RT model. Wang et al. calculated the ultimate active soil pressure under the RT model by adopting the method of level-layer analysis [12]. Gong improved the level-layer analysis method to calculate the non-limit active soil pressure under the RT model [13]. Zhang calculated the ultimate passive soil pressure under the RT model based on the static equilibrium of the soil mass [14]. Peng regarded the backfill behind the rigid retaining wall as a composed body with one ideal rigid-plastic body and a series of spring elements. Yet, the application range of this model is limited

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since his model did not consider the case of the non-limit soil pressure [15,16]. Zhu applied the disturbance state concept (DSC) to the calculation of the non-limit soil pressure under the T mode but such a method does not apply to the RT model [17,18].

This paper studies the depth distribution law of non-limit passive soil pressure under the RT model. In this paper, the backfill is regarded as a combination of a series of spring elements and a rigid-plastic body, and the rigid-plastic body is determined by the disturbance internal friction angle, the disturbance external friction angle, and the disturbance slip angle. Based on Coulomb theory, the force of the soil wedge in the disturbance state is analyzed, the formula for the non-limit lateral soil pressure on a retaining wall under the RT model is derived, and a model test is carried out, which verifies the proposed method.

2. Non-limit passive soil pressure under the RT model

2.1. Disturbing function

Desai first proposed the DSC, and applied it to the description of the softening characteristics of the over-consolidated soil [19]. Zhu applied the DSC by analyzing the non-limit soil pressure against the retaining wall under the T mode [18]. In most cases, the at-rest state of the soil is assumed to be the relative intact (RI) state, and the corresponding value S is zero, and S is the horizontally disturbed displacement of the retain wall. When the amount of the wall movement approaches a certain value (S_p), the backfill will reach a passive ultimate equilibrium state, which can be considered as the fully adjusted (FA) state. When $0 < S < S_p$, the backfill can be regarded as a mixture of the relative intact (RI) part and the fully adjusted (FA) part. The non-limit soil pressure obtained by the DSC shows a linear distribution with the change of depth. However, under the RT model, the distribution of the soil pressure is nonlinear with depth, which is inconsistent with Zhu's theoretical formula. Based on the theoretical model proposed by Zhu, combined with the spring element model, a novel approach related to the disturbance degree is proposed in this paper. β is the rotation angle defined by the amount, and the wall rotates around the top of the wall. The at-rest state is assumed to be the RI state. When the backfill behind the wall is in the RI state, the corresponding value β_0 is zero. When the amount of the rotation angle reaches a threshold value (β_p), the backfill will reach the passive ultimate equilibrium state. Thus, the ultimate state can be considered as an FA state. When $0 < \beta < \beta_p$, the backfill is regarded as a mixture of the initial continuum, or RI part, and the FA part (Fig. 1).

According to the DSC, the value of the disturbance degree d ranges from 0 to 1. When we assume the at-rest state as the RI state, and the ultimate state as the FA state, the disturbing function can be defined as follows:

$$d = \left(\frac{\tan\beta_d}{\tan\beta_p}\right)^{\xi} \tag{1}$$

where β_d is the disturbance rotation angle which obeys $0 \le \beta_d \le \beta_p$; β_p the rotation angle in the passive ultimate state; and ξ a disturbance parameter. When the rotation angle is zero, the backfill causes no disturbance; therefore, the disturbance degree is zero; when the rotation angle is β_p, the backfill is in the FA state and the disturbance degree is one.

When the backfill is disturbed, the internal friction angle can be defined as

$$\varphi_d = \arctan[\tan\varphi_0 + d(\tan\varphi_{\max} - \tan\varphi_0)] \tag{2}$$

The external friction angle can be defined as

$$\delta_d = \arctan[\tan\delta_0 + d(\tan\delta_{\max} - \tan\delta_0)] \tag{3}$$

where φ_d is the internal friction angle when the sand is disturbed; φ_0 the initial internal friction angle when the sand is in the RI state; $\varphi_{\max}$ the internal friction angle when the sand is in the FA state; δ_d the external friction angle between the sand and the wall when the sand is disturbed; δ_0 the initial external friction angle when the sand is in the RI state; and $\delta_{\max}$ the external friction when the sand is in FA state.

When the sand is not disturbed, and we ignore the influence of the external friction angle δ_0, the internal friction angle and external friction angle can be defined as [20]:

$$\varphi_0 = -\arcsin\left|\frac{1-K_0}{1+K_0}\right| \tag{4}$$

$$\delta_0 = -\varphi_{\max}/2 \tag{5}$$

At the same time, for normal consolidation soil: $K_0 = 1 - \sin\varphi_{\max}$, and K_0 is coefficient of static soil pressure.

2.2. Disturbed soil pressure

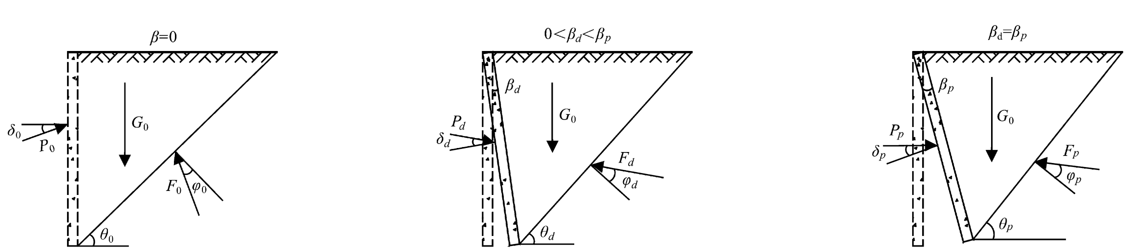
In this paper, we assume that: the height of the retaining wall is h, the back of the wall is vertical; the surface of the backfill is horizontal, the backfill is composed of sand, the disturbance internal friction angle of the backfill is φ_d, and the disturbance external friction angle is δ_d (Fig. 2). The basic assumptions are as follows: (1) the wall is rigid, the back of the wall is vertical, and the surface of the backfill is horizontal; (2) the backfill is ideal sand, and its cohesion is zero; (3) when displacement and deformation happen to the wall, a potential sliding soil body will appear in the backfill, which is regarded as a rigid body; and (4) the sliding surface after the backfill is a plane that crosses the wall heel, and the frictional force is uniformly distributed on it.

Now, the formula of the disturbed soil pressure is as follows:

According to the literature, the resultant soil pressure of the wall is as follows [21].

$$P_d = \frac{\gamma h^2}{2} \cdot \frac{\cos(\theta_d - \beta_d)}{\cos\beta_d \cdot \sin\theta_d} \cdot \frac{\sin(\varphi_d + \theta_d)}{\cos(\theta_d + \varphi_d + \delta_d - \beta_d)} \tag{6}$$

The lateral resultant soil pressure on the retaining wall can be expressed as



(a) At-rest state=(RI state) (b) Non-limit passivestate=(disturbed state) (c) The limit passive state=(FA state)

Fig. 1. Disturbed state of backfill.