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Estimation of rock pressure during an excavation/cut in sedimentary rocks with inclined bedding planes

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ABSTRACT

The estimation of rock pressure induced by an excavation/cut in sedimentary rocks is addressed in this study. A simplified stochastic model is proposed to model this rock pressure to account for sliding along parallel bedding planes as well as random friction angles on these bedding planes. Simulations show that the classical Rankine and Coulomb theories typically give active pressures much larger than those predicted by the proposed model. A simplified reliability-based design approach is developed to calibrate the required partial factors for the determination of design rock pressure. The proposed approach is demonstrated over a case study for northern Taiwan. Design charts are developed to facilitate the determination of design rock pressures induced by excavation/cut in sedimentary rocks.

1. Introduction

Excavation/cut in soils typically requires a retaining or brace system such as retaining wall and diaphragm wall to resist the induced lateral active force. Estimation of such lateral active force can be done with a classical earth pressure theory such as Rankine and Coulomb theories [1]. Lateral forces induced by excavation/cut in rocks are more complicated. For hard rocks that can stand on their own, the rock pressure may be zero, but for weak rocks, the rock pressure may be greater than zero if the rock wedge starts to slide. This is especially true for sedimentary rocks with dip bedding planes. In contrast to joints in rocks, these bedding planes are typically fairly smooth and the shear strength may be susceptible to wetting. Excavation in such sedimentary rocks deserves further attention.

As mentioned above, bedding planes in sedimentary rocks are somewhat different from rock joints in the following ways:

(a) Rock joints are not smooth. The shear strengths of rock joints are affected by the joint roughness coefficient (JRC) as well as the joint asperity strength [2,3]. However, bedding planes in sedimentary rocks are relatively smooth (sometimes very smooth).
(b) Rock joints may not be persistent. Failure along these joints may be accompanied by crossing of intact rock bridges. The slope stability analysis with bridging joints is discussed in [4]. However, bedding planes in sedimentary rocks are typically persistent.
(c) The friction angle (ϕ) for bedding planes in sedimentary rocks can be susceptible to wetting, especially for shale and mudstone. A Taiwan database shows that the friction angle after wetting (ϕ_s) can decrease to a ratio of ϕ_s/ϕ = 55–95%, depending on the type of sedimentary rocks (lower bound for shale and upper bound for sandstone).

Due to the above discussions, planar sliding is considered in this study for excavation/cut in sedimentary rocks with bedding planes for the following two reasons:

(a) As mentioned earlier, bedding planes are persistent. The sliding plane may lie entirely within a single bedding plane;
(b) There are typically four sides (e.g., east, south, west, and north) in excavation. Regardless of the strike direction of the bedding plane, it is likely that one of the four sides will encounter a potential planar sliding with dip bedding planes. This planar sliding mode is considered to be the most critical failure mode. We do not consider wedge failure, as we believe the planar sliding mode is more critical.

Moreover, joint roughness that is relevant to rock joints is not considered in this study, as bedding planes are relatively smooth.

For the planar sliding mode, current practice of estimating the rock pressure is to implement the classical calculation methods

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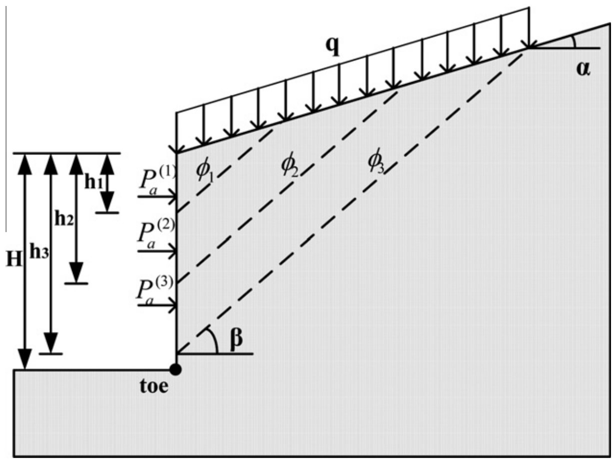

Fig. 1. Schematic for excavation/cut in sedimentary rocks with bedding planes

for soils [e.g., Rankine theory $P_a = 0.5\gamma H^2 K_a - 2cHK_a^{0.5}$, where P_a is the lateral active force, γ is the unit weight, c is the cohesion, H is the slope height, $K_a = \tan(45^\circ - \phi/2)^2$], with (c, ϕ) equal to the (c, ϕ) values along the bedding plane. The underlying assumption for these calculation methods is that the sliding plane will pass through the toe (see Fig. 1) and inclines at an angle of 45° + ϕ/2. However, these assumptions may deviate from the reality – the bedding plane may not pass through the toe and may incline at an angle of $\beta \neq 45 + \phi/2$ (see Fig. 1). As a result, the use of the classical earth pressure methods becomes questionable.

Despite its relevance to design practice, discussion for the rock pressure is rare in literature. Whiteside [5] proposed a simple formula for this purpose based on limit equilibrium. The derivation assumes that there is a single weak plane with known location and known friction angle. In reality, site investigation is rarely able to gain 100% certainty for the number of bedding planes, their exact locations, and their friction angles. As a result, the rock pressure is fairly uncertain. For instance, consider a case where the bedding plane dips with an angle $\beta = 20^\circ$ and where direct shear test indicates that $c = 0$ and $\phi = 30^\circ$ on the bedding plane. In the case that the above information is 100% accurate, the design lateral force is simply zero because $\beta < \phi$ so that the rock wedge will not move. However, the actual lateral force may be greater than zero if (a) there is a significant measurement error in the direct shear test or (b) there is significant spatial variability in the friction angles. To account for such uncertainties, stochastic models for jointed rocks have been proposed by Kulatilake [6], Einstein [7], Kulatilake et al. [8], Meyer and Einstein [9], Duzgun et al. [10], and more recently by Jimenez-Rodriguez and Sitar [11]. However, our target is to estimate the design lateral force for excavations in sedimentary rocks with bedding planes, rather than rock joints.

This study develops a more realistic calculation method for determining the lateral force P_a for excavation/cut in sedimentary rocks with parallel bedding planes. Besides proposing a new calculation model for the rock pressure, the key issue of determining the design lateral force in the presence of uncertainties will be addressed, and guidelines will be given to allow practitioners to easily obtain the required partial factors. The end result is a simplified reliability-based partial-factor design method for determining the design value of P_a that is more realistic for excavation/cut in sedimentary rocks.

2. Calculation model for P_a

Consider a vertical excavation/cut in sedimentary rocks with a set of n_p parallel bedding planes with dip angle of β (Fig. 1). For conservatism, we constrain ourselves in the case where $c = 0$, as indicated by Whiteside [5] that cohesion on weak planes may be easily reduced by weathering and hence unreliable. In the case where all bedding planes have identical friction angle ϕ, the safety factors for all wedges will be identical and equal to $\tan(\phi)/\tan(\beta)$. In the more realistic case where the n_p bedding planes have variable friction angles ($\phi_1 \neq \phi_2, \ldots \neq \phi_{n_p}$), the safety factor for the i-th wedge is $\tan(\phi_i)/\tan(\beta)$. If $\tan(\phi_i)/\tan(\beta) < 1$, the i-th wedge loses it equilibrium state and starts to slide, and vice versa. For a dry condition (no water), the force produced by the sliding of a wedge can be computed easily by limit equilibrium:

$$F = 1(\beta > \phi) \times \left(\frac{1}{2}\gamma h^2 K_W + qhK_W\right) \tag{1}$$

where F is the sliding force; ϕ is the friction angle on the bedding plane; 1(.) is the indicator function; q is the surcharge pressure; h is the height of the wedge (see Fig. 1);

$$\begin{aligned} K_W &= \frac{\cos(\beta)\cos(\alpha)}{\sin(\beta - \alpha)} \tan(\beta - \phi) \\ K_q &= \frac{\cos(\beta)}{\sin(\beta - \alpha)} \tan(\beta - \phi) \end{aligned} \tag{2}$$

α is the slope angle (see Fig. 1). Note that the above equation is not suitable for cases where $\alpha \geq \beta$, as when $\alpha \geq \beta$ the wedge will have an infinite mass. The above derivations for the planar sliding mode and the resulting sliding force bears the same spirit as those in [12] for plane failure. Basically, the same limit equilibrium is solved to determine the safety factor (SF), and if SF < 1, the sliding force can be determined based on the unbalanced forces.

For the case with water, Whiteside [5] considered the scenario where static water (no seepage) is present so that the entire excavation/cut is submerged (submerged case),

$$F = 1(\beta > \phi^s) \times \left(\frac{1}{2}\gamma' h^2 K_W^s + qhK_q^s\right) + \frac{1}{2}\gamma_w H^2 \tag{3}$$

where $\gamma' = \gamma_{sat} - \gamma_w$; γ_{sat} is the saturated unit weight, and γ_w = 9.81 kN/m^3; ϕ^s is the submerged friction angle on the bedding plane; K_{Ws} and K_{qs} are the same as Eq. (2) except that ϕ is replaced by ϕ^s. The submerged case can be quite realistic for excavation without dewatering and for a permanent basement wall. The friction angle on a bedding plane may degrade when water is present. This degradation is especially true in shale, mudstone, or shale–sandstone interlayers. As seen later in a case study in Taiwan, the ratio ϕ^s/ϕ may range from 55% to 95%. This degradation was also observed in the shear strengths of rock masses. Based on experiments on 44 rock mass samples, Mehrotra [13] reported that saturating a rock mass may cause a reduction up to 70% and 30% for its cohesion and friction angle, respectively.

The actual sliding force F, denoted by F_a, may not be the same as the model F in Eqs. (1) and (3). In fact, $F_a = M \times F$, where M is called the model factor – a random variable that characterizes the modeling error in the proposed model. If the model is unbiased, M has mean value $\mu_M = 1$. The variability of the model factor is characterized by its coefficient of variation (COV) V_M . The lateral force P_a is simply the maximum value among ($F_{a,1}, \ldots, F_{a,n_p}$):

$$P_a = \max\left(F_{a,1}, F_{a,2}, \ldots, F_{a,n_p}\right) \tag{4}$$

The wedge with the maximum F_a value is the most critical wedge. Let us denote the height of the most critical wedge by h^*.

The main difference between the proposed calculation model and the Rankine model $P_a = 0.5\gamma H^2 \tan(45 - \phi/2)^2$ or the Coulomb model is as follows: (a) the failure can only occur on a existing bedding plane in the proposed model, rather than shear through with angle of 45° + ϕ/2 in the Rankine case; (b) the height of the most critical wedge h^* is always less than the total height H in the