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Rigid block based lower bound limit analysis method for stability analysis of fractured rock mass considering rock bridge effects

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ABSTRACT

A rigid block based lower bound limit analysis method for analyzing stability of fractured rock mass in 2D and 3D conditions is proposed. The rock bridge effects are considered in the general formation. No assumptions are imposed on the inter-element forces, and the solution obtained is statically admissible. The proposed method is theoretically rigorous and simple. The validation and efficiency of the proposed method have been demonstrated through three typical types of slopes, indicating that apart from the fractures, rock bridge plays a key role in stabilizing rock blocks, which should be greatly concerned in stability analysis of rock mass.

1. Introduction

For the stability analysis of fractured rock mass, in recent decades, the key block method and its extension (e.g. the key group method) have been developed and widely used [22,9,23,26,27,21,16]. However, the major assumption in the key block method that blocks move with virtually no cracking, which would lead to that only the removable blocks are considered for stability analysis [22,9]. In fact, the rock bridges (i.e., the intact regions between coplanar or noncoplanar fractures where a combined shear-cracking plane takes place) may lead to the failure of nonremovable blocks, which are often more unfavorable than the removable blocks [8,11,4]. Nevertheless, most of the key block and its extension methods did not provide a procedure to analyze the stability of rock blocks considering the rock bridges (e.g., [22,9,26,27,16,7]). Additionally, the key block method and its extension need to calculate the stability of all the possible combinations of element blocks one by one. Assuming that there are n blocks, the number of all the possible combinations of element blocks is $2^n - 1$. That is, the possible combination increases exponentially with n. Therefore, it is inefficient to traverse all the combinations of element blocks.

Due to the capability of obtaining the upper or lower bound of accurate solution, limit analysis proved efficient to solve the stability problem and gets extensive application (e.g., [5,15,18,28,1,19]). In contrast, it has been recognized by various researchers that the lower bound solution is more valuable in practice compared to the upper bound solution, as it results in a safe design (e.g., [20,13,24,17]). The lower bound limit analysis method based on discontinuous media approach such as rigid finite-element method (RFEM) [10] has been proposed and it has attracted significant attentions in recent years [30,14]. These methods seem to provide new insight for solving the rock mass stability problem. Nevertheless, the mechanical properties of rock mass with the intersection of fractures are complex in space, the stability analysis of rock mass by assuming a circular slip surface [30] or using two-dimensional approach is not reasonable [30,14].

In this study, to address the issues mentioned above, a novel lower bound limit analysis method based on rigid blocks for analyzing the stability of fractured rock mass in both 2D and 3D conditions is proposed. The validation and efficiency of the proposed method have been demonstrated through three typical types of slopes.

2. Description of rock mass system and identification of rock bridges

Rock bridges play an important role in stabilizing the rock blocks, in addition to the fractures themselves [11,4]. Thus,

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identification of the possible cracking rock bridges is significant. Additionally, a proper representation of the rock mass system is also essential. In this study, the boundary surfaces defining the interest domain are divided into two types: excavation surfaces (natural ground or excavated surfaces) and fixed surfaces as shown in Fig. 1a. Fixed surfaces are the faces that separate the problem domain from the infinite rock mass. If a block has one or more fixed faces, then it is defined as an infinite block; otherwise, it is defined as a finite block [29]. For the purpose of analyzing the stability of the interest domain, similar to the concept by Yu et al. [29], the interest domain is firstly subdivided into a finite number of convex subdomains. Then, the convex subdomains are decomposed into convex blocks with infinite fractures (Fig. 1b). Further, the infinite fractures are restored to finite discs (Fig. 1c). The key-block or key-group block method assembles the interest domain and classifies the complex blocks before analyzing (Fig. 1d). It ignores the rock bridge effect due to that the complex block usually is classified by enclosing the fractures and boundary surfaces. In contrast, in this work, the rigid block is identified in the procedure of restoring the infinite fractures to finite discs (Fig. 1c). The rigid block is enclosed by the fractures, the boundary surfaces and the possible cracking rock bridges which are represented by the dotted lines. As a result, the instability region obtained by the proposed method can be a single key rigid block or groups of rigid blocks. Note that, in this study, the failure of rock bridges is assumed to occur along the fractures considering that cracking rock bridges are usually small pieces of intact rock between fractures.

3. Formulations of lower bound method based on rock mass system

The lower bound theorem states that the collapse load obtained from any statically admissible stress field will underestimate the true collapse load. A statically admissible stress field is one which (a) satisfies the equations of equilibrium, (b) satisfies the boundary conditions and (c) does not violate the yield criterion.

3.1. Equilibrium conditions

Assume that the generalized forces at a rock bridge b of a rock block involve the shear force along S_{1b}, S_{2b} and normal force along n_b in the local coordinate system (S_{1b}, S_{2b}, n_b) as shown in Fig. 2. They can be denoted in a vector form as

$$Q_b = [V_{1b} \quad V_{2b} \quad N_b]^T, \quad b = 1, 2, \ldots, n_b \tag{1}$$

where n_b denotes number of all rock bridges. Then, the global force vector for rock bridges can be written collectively as follows:

$$Q = [Q_1^T \quad Q_2^T \quad \ldots \quad Q_{n_b}^T]^T \tag{2}$$

In the same way, the global force vector for fractures can be written as:

$$P = [P_1^T \quad P_2^T \quad \ldots \quad P_{n_k}^T]^T \tag{3}$$

where $P_k = [V_{1k} \quad V_{2k} \quad N_k]^T$ denotes the force vector in the local coordinate system (S_{1k}, S_{2k}, n_k). $k = 1, 2, \ldots, n_k$, n_k denotes number of all fractures. For fracture k as shown in Fig. 2, V_{1k}, V_{2k} and N_k denote shear force along S_{1k}, S_{2k} and normal force along n_k, respectively.

Then, following global equilibrium equation can be drawn:

$$C^T R + F = 0 \tag{4}$$

where $C = [C_1 \quad C_2]^T$, $R = [P \quad Q]^T$, $F = [F_1^T \quad F_2^T \quad \ldots \quad F_n^T]^T$, $F_i = [f_{Xi} \quad f_{Yi} \quad f_{Zi}]^T$, $i = 1, 2, \ldots, n$, n denotes number of all rock blocks. C_1 is matrix assembled by transformation matrix which

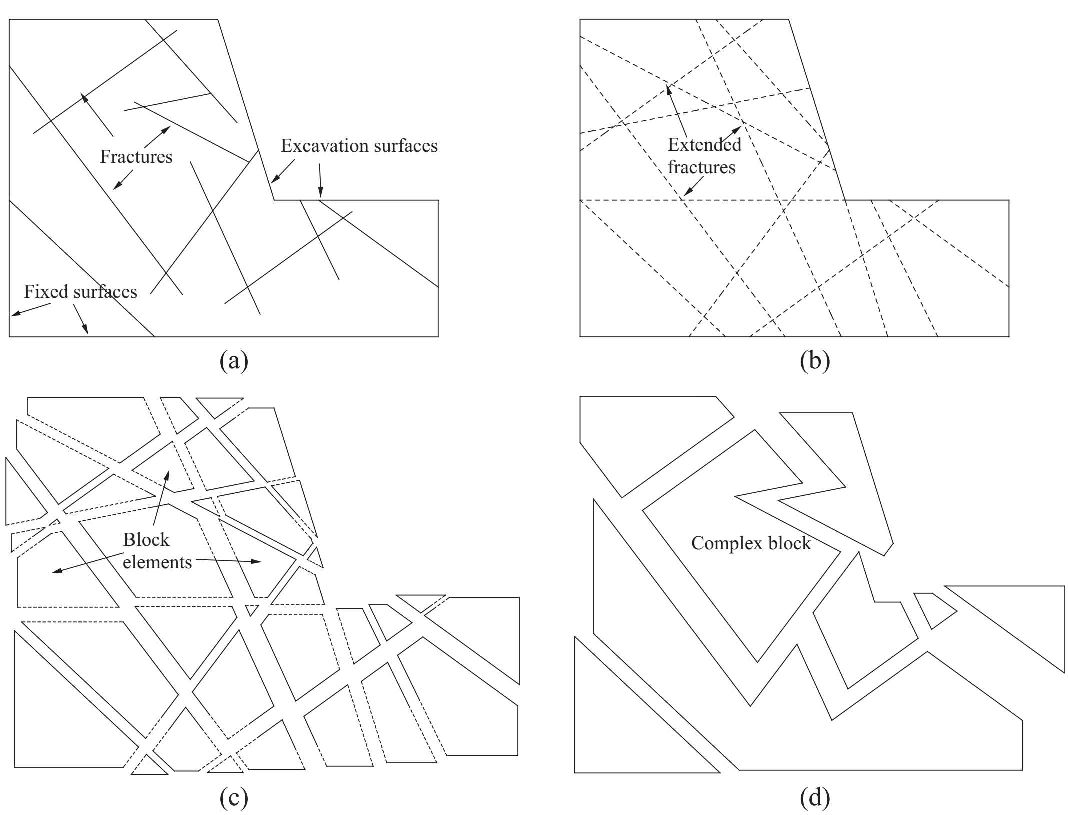

Fig. 1. Subdividing the modeling domain into rigid blocks.