ARTICLE IN PRESS

Computers and Geotechnics xxx (xxxx) xxx–xxx

Contents lists available at ScienceDirect

Computers and Geotechnics

journal homepage: www.elsevier.com/locate/compgeo

Technical Communication

Continuous field based upper bound analysis for three-dimensional tunnel face stability in undrained clay

Maosong Huang[a], Sen Li[a], Jian Yu[a,*], Jorgin Qi Wen Tan[b]

[a] Department of Geotechnical Engineering, Tongji University, Shanghai 20092, China
[b] Geotechnical and Tunneling, Land Transport Authority, 219428, Singapore

ARTICLE INFO

Keywords:
Undrained clay
Tunnel face stability
Upper-bound limit analysis
Finite element
Continuous field

ABSTRACT

A new numerical upper-bound method applicable to three-dimensional stability problems in undrained clay is proposed and applied in the analysis of tunnel face stability. Comparisons with the elasto-plastic finite element method (FEM) reveal that this method could provide sufficiently accurate upper-bound prediction for the tunnel face stability factors. For tunnels with large cover depth, the obtained results greatly improve the existing upper-bound solutions. Their corresponding velocity fields are investigated, which show a similar evolving pattern with those of the FEM. A discussion is made to compare the various advantages of multi-block mechanisms and continuous mechanisms in tunnel face stability.

1. Introduction

Underground tunnel excavation has become commonplace in infrastructure projects worldwide. An important design parameter in tunnel face stability is the support pressure that is applied at the tunnel face. Due to the practical significance of three-dimensional (3D) tunnel face stability, substantial research investigations have been conducted with various methods including the empirical method [2], lower bound limit analyses [3,8,13], upper-bound limit analyses [3,5,6,8–11,13] and elasto-plastic finite element (FE) analyses [14,16]. Broms and Bennermark [2] first proposed a hand calculation design expression for tunnel face stability in undrained clay by empirically relating the expected deformations with various factors such as support pressure, surcharge, soil unit weight, cover depth and tunnel diameter to determine whether the tunnel is sufficiently supported. The first theoretically rigorous analysis of 3D tunnel face stability was carried out by Davis et al. [3] using both upper and lower bound limit analyses. Leca and Dormieux [8] extended the work of [3] into fictional soil. Ukritchon et al. [14] and Zhou [16] further utilized the finite element method (FEM) to re-evaluate the accuracy of the previously published results.

As a widely used analysis method, the applications of the upper-bound method in 3D tunnel face stability are further reviewed. The upper-bound method can be divided into three categories according to their underlying mechanisms. The first approach is to postulate mechanisms composed of either translational [3,8,10] or rotational rigid blocks [11] where the energy dissipation exists solely at the interfaces between adjacent blocks. The second approach is to develop continuous failure mechanisms with no discontinuity, where the energy is solely dissipated by plastic deformation throughout the continuums [5,6,9]. The third approach is to construct failure mechanisms with both deformable zones and discontinuity surfaces [13]. The continuous velocity field of Klar et al. [6] was derived from both the elasticity solution corresponding to the original plastic problem and the concept of sinks and sources. The continuous mechanism of Mollon et al. [9] was obtained from the elasto-plastic finite difference method (FDM). The finite element limit analysis (FELA) was employed by Sloan [13] to investigate 3D tunnel face stability. However, the difference between the upper and lower bound for 3D tunnel problem is quite large when compared with the 2D FELA solutions [1].

Much effort has been put into constructing upper-bound solutions for 3D undrained tunnel stability. However, a significant gap still exists between the upper-bound results and the elasto-plastic FEM results for tunnels with large cover depth. Such a gap might be attributed to the difficulty in constructing complicated 3D upper-bound velocity fields. This paper attempts to propose a generally applicable 3D upper-bound method for undrained clay based on the total loading extended mobilisable strength design (T-EMSD) proposed by Huang et al. [4] and Yu et al. [15], which can acquire the upper-bound solution through elastic iterative analysis and thus could be conveniently replicated through an elastic displacement-based FEM package. The method will be applied to 3D undrained tunnel stability, and the obtained upper-bound solutions will be thoroughly compared with existing solutions.

* Corresponding author.
E-mail address: 002yujian@tongji.edu.cn (J. Yu).

http://dx.doi.org/10.1016/j.compgeo.2017.09.014
Received 27 June 2017; Received in revised form 30 August 2017; Accepted 19 September 2017
0266-352X/

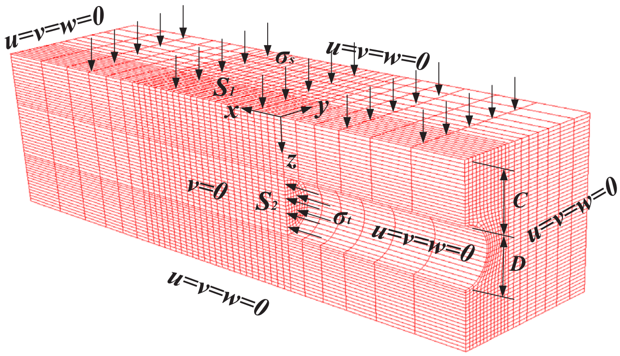

Fig. 1. Problem notation and example finite element layout of 3D undrained tunnel face stability.

2. Method of analysis

The T-EMSD, developed from the EMSD, is a total loading method to obtain an upper-bound load-displacement solution of a plane-strain problem through iterative computation, rather than optimal computation. It has been verified in the study of the soil resistance to the lateral movement of a 2D circular pile. The T-EMSD is able to directly calculate an upper-bound lateral resistance under a given total pile displacement (without the need for an incremental loading process). Large enough displacements would enable the resistance to approach an upper-bound lateral limit load. This section will extend the T-EMSD theory to a generic 3D condition and illustrate its application in obtaining the upper-bound tunnel face stability factor.

2.1. Theory

Shield and Drucker [12] demonstrated that the upper-bound theorem for Tresca material, conventionally adopted for undrained clay, can be expressed as

$$\int_s T_i v_i dS + \int_V f_i v_i dV \leqslant \int_V 2S_u|\dot{\varepsilon}_{\max}|dV + \int_A S_u \Delta v_t dA = \int_S T_i^* v_i dS + \int_V f_i v_i dV \quad (i = 1,2,3) \tag{1}$$

where T_i and f_i are the vectors of surface traction and body force acting on the boundary S and V respectively; v_i is the kinematically admissible velocity field; s_u is the undrained shear strength of the soil; $|\dot{\varepsilon}_{\max}|$ is the absolute largest principal component of the plastic stain rate $\dot{\varepsilon}_{ij}$ compatible with v_i; Δv_t is the magnitude of the relative velocity change across the discontinuity surfaces A; and T_i^* is the upper-bound collapse load.

For a continuous velocity field, Eq. (1) becomes

$$\int_V 2s_u|\dot{\varepsilon}_{\max}|dV = \int_S T_i^* v_i dS + \int_V f_i v_i dV \quad (i = 1,2,3) \tag{2}$$

Klar and Osman [7] demonstrated that the upper-bound theorem of Eq. (1), expressed in the form of rate, could be rewritten as the energy conservation of the internal and external increment of work

$$\overbrace{\int_V 2c_m(\varepsilon_s)d\varepsilon_{\max}dV}^{\text{Internal increment of work}} = \overbrace{\int_S T_i^* du_i dS + \int_V f_i du_i dV}^{\text{External increment of work}} \quad (i = 1,2,3) \tag{3}$$

where $d\varepsilon_{\max} = \max(|d\varepsilon_1|,|d\varepsilon_3|,|d\varepsilon_3|)$ is the increment of the largest absolute principal strain; $c_m(\varepsilon_s)$ is the mobilised strength, which is a function of engineering shear strain ε_s $(= \sum_1^n |d\varepsilon_1 - d\varepsilon_3|$, n is the number of the loading steps); for an elastic perfectly-plastic material $c_m(\varepsilon_s)$ equals to $Min(G_s\varepsilon_s, s_u)$, where G_s is the shear modulus of the soil. With ε_s accumulating during loading, $c_m(\varepsilon_s)$ tends to s_u such that T_i^* tends to an

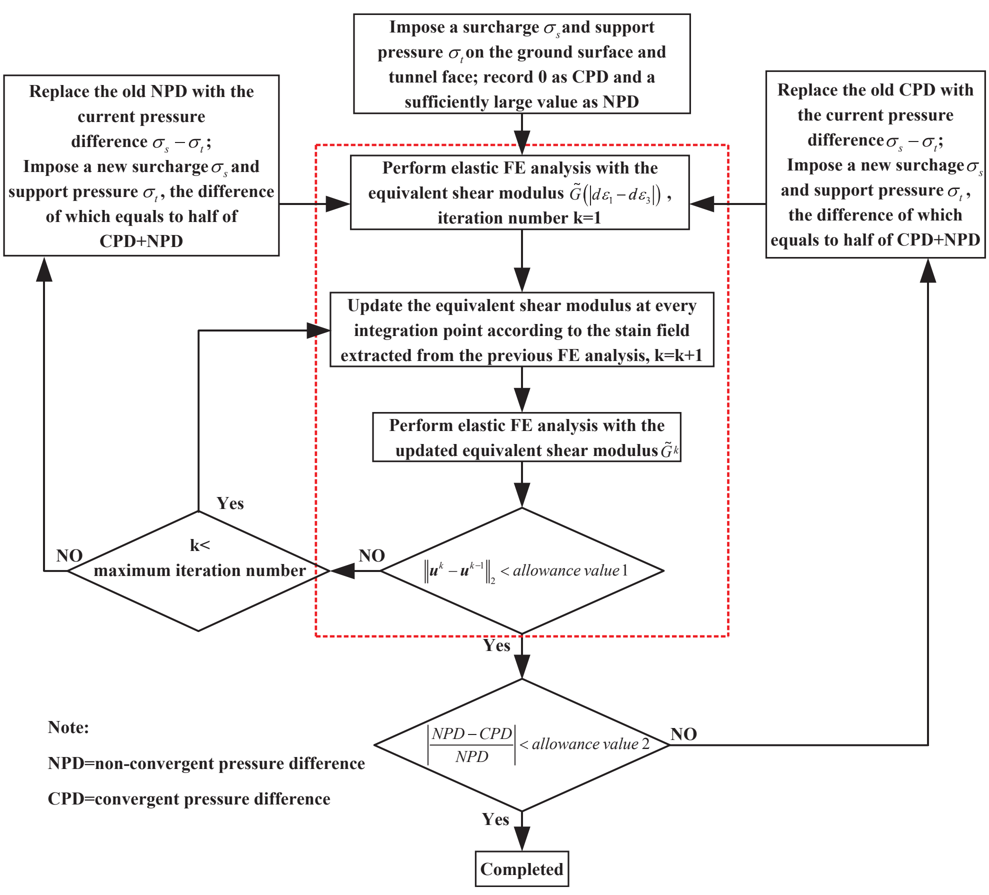

Fig. 2. Flow diagram for tunnel stability.