Tunnelling and Underground Space Technology 49 (2015) 230–240

Contents lists available at ScienceDirect

Tunnelling and Underground Space Technology

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

The stability of shallow circular tunnels in soil considering variations in cohesion with depth

Nima Khezri [a], Hisham Mohamad [a,*], Mohsen HajiHassani [a], Behzad Fatahi [b]

[a] UTM Construction Research Centre, Faculty of Civil Engineering, Universiti Teknologi Malaysia, 81310 Skudai, Malaysia
[b] School of Civil and Environmental Engineering, University of Technology, Sydney, Australia

ARTICLE INFO

Article history:
Received 6 March 2014
Received in revised form 28 February 2015
Accepted 13 April 2015

Keywords:
Tunnel face pressure
Upper bound solution
Tunnel face stability

ABSTRACT

This paper presents an upper bound investigation of the three dimensional stability of a tunnel face in a deposit of soil whose strength varies with depth. The upper bound theorem of limit analysis incorporating the linear variation of the soil cohesion with depth was used to calculate the pressure at the tunnel face of a closed face excavation. For an open face excavation, the factor of safety against the tunnel face instability was calculated using the strength reduction technique and the upper bound theorem. The results, in terms of the minimum required face pressure, were then compared with other solutions available from the literature for verification, and the numerical results in the form of dimensionless design charts are also presented. In addition, a comparative study between the simplified approaches adopting a singular soil cohesion parameter representing the whole layer instead of considering its actual variation with depth is presented. It was concluded that adopting the mean soil cohesion that does not vary with depth would lead to a conservative design, that is, a higher minimum face pressure being required during construction and a lower factor of safety against face instability. However, adopting the local cohesion obtained from the tunnel face may result in underestimating the required face pressure and may lead to an unsafe design.

1. Introduction

The tunnel face stability and ground surface settlement due to underground tunnelling are two important criteria to be considered in the design of shallow tunnels. In mechanised tunnelling, the imposing tunnel face pressure must be sufficient to maintain its stability (prevent the face from collapsing) during boring but not excessively high, as this may cause a surface blow-out.

Numerous researchers have investigated the problem of tunnel face stability by laboratory and experimental methods (Broms and Bennermark, 1967; Kimura and Mair, 1981; Chambon and Corte, 1994; Takano et al., 2006). Based on the laboratory tests and field observation for tunnelling in purely cohesive soils, Broms and Bennermark (1967) introduced the stability ratio N as follows:

$$N = \frac{\sigma_s + \gamma H - \sigma_T}{c_u} \tag{1}$$

where σ_s is the surcharge acting on the ground surface, γ is the soil unit weight, H is the tunnel depth, σ_T is the tunnel face pressure, and c_u is the undrained shear strength of the soil. Eq. (1) shows that the higher the ratio N is, the less stable it is, for example, Broms and Bennermark (1967) suggested that where $N < 6$, stability is maintained. Kimura and Mair (1981) conducted centrifuge tests and derived stability charts for clays. Their test results suggested a wider range for the stability factor N of between 5 and 10, depending on the ratio of the depth to the tunnel diameter (H/D) and the unlined length of the tunnel. Chambon and Corte (1994) conducted model centrifuge tests to study the stability of a tunnel face in sand. Their research aimed to determine the minimum face pressure to be applied to the face of the tunnel to optimise the cost of excavation as well as to prevent ground surface heave. They investigated the effects of the geometry and material parameters on the failure mechanism and the shape of the failure surface. Chambon and Corte (1994) concluded that the shape of the failure surface is like a chimney in the longitudinal direction where the magnitude of the limiting pressure is related to the unsupported length of the tunnel. Their results showed that the depth of a tunnel had an insignificant effect on limiting pressure; it was the diameter of the tunnel that was the influencing parameter. Takano et al. (2006) conducted some model tests using an X-ray computed tomography scanner to obtain a three dimensional (3D) visualisation of the failure zone. Their results showed that the failure surface was like a logarithmic-spiral curve in a longitudinal direction with

* Corresponding author. Tel.: +60 75533109; fax: +60 75566157.
E-mail address: mhisham@utm.my (H. Mohamad).

http://dx.doi.org/10.1016/j.tust.2015.04.014
0886-7798/

Nomenclature

C	cover depth of the tunnel
D	diameter of the tunnel
D_i	diameter of the *i*th cone
E	horizontal length of the sliding block
H	depth of the tunnel
K_P	passive earth pressure coefficient
L_i	lateral area of the *i*th cone
L_{ie}	lateral area of the vertical element
N	stability ratio
N_s, N_γ	weighting coefficients
P_e	power of the external loads
P_V	dissipated power of the internal loads
Q_s	surcharge parameter of Leca and Dormieux (1990)
Q_T	tunnel face pressure parameter of Leca and Dormieux (1990)
Q_γ	soil unit weight parameter of Leca and Dormieux (1990)
SF	safety factor
$\boldsymbol{V}$, $\boldsymbol{V_d}$	velocity of sliding block
V_i	volume of the *i*th cone
V_{ie}	volume of the *i*th element
a	major semi-diameter of the ellipse
b	minor semi-diameter of the ellipse
c_o	cohesion at the ground level
c_{ave}	average (mean) cohesion
c_d	mobilised cohesion
c_f	cohesion at the face (centreline) of tunnel
c_i	cohesion of the *i*th element
c_{ij}	cohesion at the point *j* of the *i*th element
c_u	undrained shear strength of the soil
h_i	horizontal length of the *i*th cone
k	integer counter of Gaussian points
n	integer counter of number of elements
w_{ij}	weight of Gaussian points
z	*z* coordinates
Σ_d	discontinuity surface
α	angle of the velocity vector with horizon
γ	soil unit weight
δh	element's width
θ_d	angle between the velocity and the discontinuity surface
π	pi number
ρ	rate of the cohesion increment with depth
σ_{co}	unconfined compression strength of the soil
$\sigma_{T(face\ \ cohesion)}$	tunnel face pressure obtained by the use of the cohesion at the centreline line of the tunnel
σ_s	surcharge
σ_T	tunnel face pressure
$\sigma_{T(average)}$	tunnel face pressure obtained by use of average cohesion
$\sigma_{T(linear)}$	tunnel face pressure obtained by use linear cohesion
φ	internal friction angle
φ_d	mobilised friction angle

an elliptical cross section. They also introduced a new failure mechanism based on the limit equilibrium theory.

Many researchers have also used theoretical and analytical procedures to examine the stability of the tunnel face in purely cohesive soils (Davis et al., 1980; Augarde et al., 2003; Osman et al., 2006; Klar et al., 2007) or frictional soils (Chambon and Corte, 1989; Leca and Dormieux, 1990; Anagnostou and Kovari, 1996; Broere, 1998; Mollon et al., 2009; Mollon et al. 2010;). With purely cohesive soils, the stability ratio N defined by Eq. (1) can be utilised to evaluate the stability of the tunnel. Davis et al. (1980) used limit theorems of plasticity and derived bound solutions for three cases of tunnel headings in homogeneous soil with constant undrained shear strength and concluded that the stability ratio N was significantly influenced by the depth of the tunnel and cover to diameter (C/D) ratio. The finite element formulation of limit analysis was used by Augarde et al. (2003) to study a plane strain heading in an undrained condition. Here they represented the upper and lower bounds of the load factor for different tunnel geometries and soil conditions, and also investigated the effects of a linear increase in the shear strength with depth on the load factor. Osman et al. (2006) used the upper bound theorem to investigate the plane strain stability of a tunnel face in undrained clays. They used the upper bound theorem to determine the width of a subsurface settlement trough in which a plastic deformation mechanism with distributed shear was incorporated. To obtain the velocity field, Osman et al. (2006) used a Gaussian settlement trough near the ground surface, whereas Klar et al. (2007), instead of using a Gaussian settlement trough, acquired the admissible velocity field directly from the elasticity equations. Klar et al. (2007) claimed that their results showed a slight improvement over those obtained by Davis et al. (1980) in some cases.

Leca and Dormieux (1990) adopted the upper bound theorem to investigate the 3D face stability of a shallow tunnel in a cohesive–frictional soil. They proposed three mechanisms of failure, consisting of rigid truncated or perfect cones. The face pressure resulting from the method developed by Leca and Dormieux (1990) was in good agreement with the centrifuge tests conducted by Chambon and Corte (1989). Later, Mollon et al. (2009) modified the possible failure mechanisms by using a finite number of rigid conical blocks, while Anagnostou and Kovari (1996) utilised the wedge stability theory to analyse the problem of face stability in drained conditions. While the study by Anagnostou and Kovari (1996) was limited to machine operation in Earth Pressure Balance (EPB) mode, their results showed the relationship between the limit pressure and the hydraulic head in the muck; it should be noted that the applied hydraulic head reduces seepage from the tunnel face. They concluded that the pore water pressure and the limit pressure must be controlled during excavation to obtain the optimum support pressure.

Broere (1998) used a wedge stability model to calculate the tunnel face pressure in a layered soil and to estimate the slip angle of the wedge. Moreover, Mollon et al. (2010) used the upper bound theorem and a spatial discretization technique to develop a new model based on their previous multi-block mechanism of Mollon et al. (2009). This new mechanism includes the whole circular area of the tunnel face instead of the encompassed ellipse in the tunnel face. Park (2011) utilised the two-conical mechanism of Leca and Dormieux (1990) and derived an analytical solution for the stability of the tunnel face below the groundwater table in cohesive–frictional soils with a linear increase of cohesion with depth. However, the author oversimplified the expression of the dissipation power, assuming a constant lateral area parameter for each block's interface instead of double integrating the change of unit surface and cohesion with depth.

There is a limited amount of literature on tunnel face stability which considers variations of the shear strength parameters of the soil with depth. Such studies have been limited to the two dimensional (plane strain) condition (Augarde et al., 2003) or suffer from mathematical inadequacies in dissipated power calculations (Park, 2011). Variations of soil cohesion with depth so far