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Determination of critical distance for deep tunnels with a longitudinal water-filled cavity positioned above tunnel roof

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A R T I C L E I N F O

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1. Introduction

Stability in tunnel engineering involves stability at the face and roof. With respect to the former, it is divided into active failure (i.e., the collapse), and passive failure (a blow-out). The investigation of face stability has been a classical issue and extensively studied. For cohesive soils, a load factor N was introduced to estimate stability of tunnel face in purely cohesive soils.[1–4] According to the definition, $N = (\sigma_s + \gamma H - \sigma_t)/c_u$ where σ_s=surcharge loads applied on the ground surface; γ=soil self-weight; H=depth of the tunnel axis to the ground surface; σ_t=uniaxial tensile strength; c_u=soil undrained cohesion. To obtain the magnitude of N, centrifuge tests, limit equilibrium method and limit analysis approach of plasticity theory were utilized in the above references respectively.

In the case of face stability in frictional soils, investigation has been carried out with experimental, numerical and analytical methods. Chambon and Corté[5] carried out centrifuge experiments to characterize the failure mechanism and obtain the value of critical pressure for preventing active failure. The results indicate that the collapse mechanism under the limit state is similar to a shape like a chimney which may not extend to the ground surface. Based on this, the failure mechanism composed of logarithmic spirals was proposed and widely adopted by researchers and engineers in this field. Employing the collapse mechanism proposed by Horn,[6] Anagnostou and Kovari[7] derived the closed-form solutions of critical face pressure with the use of limit equilibrium method. It is noted that the results obtained however were based on prior assumptions on failure shape and normal stress distribution of detaching blocks in the front of tunnel face. The precision of results is hence highly dependent on the level of simplification. In order to overcome the drawback of the former approach, the kinematic method proposed in this paper provides an innovative approach. On the basis of the assumption of rigid geomaterials, Leca and Dormieux[8] obtained rigorous solutions of face stability with a two-block collapse mode based on the framework of limit analysis theory.

The kinematic approach among many kinds of research methodology has been used extensively to derive upper bound solutions. Mollon et al.[9] adapted the work of Leca and Dormieux[8] and proposed a translational 3D failure mechanism which constitutes five conical sliding blocks. However, these moving blocks may not be the most suitable for circular tunnel face which failures elliptically. To overcome this shortcoming, Mollon et al.[10] refined the failure mechanism with the use of a spatial discretization technique. Through the point-to-point technique, the three-dimensional failure surface was generated and the results were greatly improved. However the proposed velocity discontinuity may not reflect the rotational characteristics of failure blocks. To overcome this shortcomings, Mollon et al.[11] constructed a rotational failure mode with the spatial discretization technique which formed an 'inclined' velocity field in the whole tunnel face.

Considering circumstances where tunnels are designed in soils with cavities are well developed, potential threats of collapse exist in tunnel excavation. In the presence of a longitudinal cavity positioned above the tunnel crown, the face stability is rarely investigated. This paper aims to estimate the effect of water-filled cavity on face stability.

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2. Theoretical framework

2.1. Limit analysis theory

The focus of this study is on the derivation of supporting pressure for tunnel face. It is well established that limit analysis principle based on plasticity theory provides an effective approach and has been extensively adopted to evaluate engineering problems in geotechnical engineering, such as minimal supporting pressure, stability analysis, ultimate bearing capacity, due to its concise and simple means of solving practical issues. In the case of upper bound theorem, unlike limit equilibrium method, it considers the constitutive properties of rock materials where the kinematic approach is closer to the actual solution.

Since the 1970s, the limit analysis theory, particularly upper bound theorem, is regarded as an alternative and effective way of tackling geotechnical issues.[12] In the framework of plasticity theory, the upper bound theorem denotes that the load computed by equating the external work rate to the internal energy dissipation rate in any kinematically admissible velocity field is no less than the actual collapse load when the boundary satisfying deformation conditions.

$$\int_{\Omega} \sigma_{ij}\dot{\varepsilon}_{ij}d\Omega \geq \int_{S} T_i v_i dS + \int_{\Omega} X_i v_i d\Omega \tag{1}$$

where σ_{ij} and $\dot{\varepsilon}_{ij}$ correspond to the stress tensor and strain rate in the kinematically admissible velocity field, respectively. T_i refers to surcharge loading on the boundary S, X_i is the body force, Ω the volume of the moving blocks, and v_i represents the velocity along with the detaching surface.

2.2. Pore water effect

The pore water effect on face stability should also be incorporated into upper bound theorem. This can be taken as an external or internal load. Meanwhile, considering the fact that it is more straightforward to regard pore water pressure as external loading, this approach is often adopted in the analysis of many engineering problems. Based on Viratjandr and Michalowski,[13] the pore water pressure is regarded as an external load which applies on soil skeletons and the kinematic admissible velocity field boundaries. Based on this, the work rate generated by pore water effect gives

$$\dot{W}_u = -\int_{\Omega} u\dot{\varepsilon}_{ii}d\Omega - \int_{s} n_i\, uvds \tag{2}$$

in which u refers to pore water pressure, soil weight per volume γ, and the vertical distance h from a random underground point to the water level, and can be expressed as $u = r_u\gamma h$; $\dot{\varepsilon}_{ii}$ represents the volumetric strain rate, while n_i indicates the normal vector perpendicular to the failure surface.

In the application of upper bound theorem, the assumption of rigid materials is generally made to simplify the calculation procedure. In this case, the first part in the right of Eq. (2) is made equal to zero since the volumetric strain rate $\dot{\varepsilon}_{ii} = 0$, and therefore the influence of pore water pressure is accounted by the second component in the right hand side of the equation. And at the same time, the geomaterials should conform to the associated flow rule and must be perfect plastic when employing the kinematic approach.

2.3. Modified Hoek-Brown (HB) failure criterion

In rock engineering, numerous experiments have proved that the rock masses present obvious anisotropy under certain circumstances, such as faults, weak interlayers and joints; therefore, the traditional constitutive relationship is no longer accurate to describe the stress-strain relationship at failure. In the presence of anisotropic characteristics, it would be rather complicated to conduct theoretical reasoning with the numerous joints. To overcome this difficulty, the jointed rock masses are assumed to be homogeneous and isotropic in practice. This makes sense when the number of joints is large and the following conditions are satisfied: There exist 1) no faults or bedding planes, 2) directions of discontinuity surfaces are sufficient randomly distributed, 3) the joint separation is small when compared with the magnitude of rock structures, and 4) the discontinuity surfaces must be sufficiently dense, which means that the spacing between adjacent discontinuities is small enough compared to the overall dimension of rock structures.

On the basis of the isotropic and homogeneous assumption, many geotechnical tests have proved that the failure envelope of almost all rock materials presents nonlinear characteristics. Subsequently, many kinds of failure criteria have been proposed and developed, including nonlinear Mohr-Coulomb (MC) criterion, the Power-Law criterion, Leon-Torre strength criterion, and the widely used HB yield criterion. Initially, the HB criterion was proposed for intact rock materials, however it could also potentially satisfy very poor rock masses as a continuum like material. The modified HB criterion in isotropic and homogeneous rock masses can be expressed as

$$\sigma_1 - \sigma_3 = \sigma_c\left(\frac{m\sigma_3}{\sigma_c} + s\right)^n \tag{3}$$

where σ_1, σ_3 are the major and minor principal stress at failure, separately, σ_c corresponds to the uniaxial compressive stress of the rock mass, m, s, n refer to the rock properties determined by the geological strength index (GSI) of which the magnitude is greatly influenced by the rock mass structure and the surface condition of the joints. With reference to Hoek et al.[14] the parameters m, s, n gives

$$\frac{m}{m_i} = \exp\left(\frac{GSI - 100}{28 - 14D}\right) \tag{4}$$

$$s = \exp\left(\frac{GSI - 100}{9 - 3D}\right) \tag{5}$$

$$n = \frac{1}{2} + \frac{1}{6}\left[\exp\left(-\frac{GSI}{15}\right) - \exp\left(-\frac{20}{3}\right)\right] \tag{6}$$

where D represents a disturbance coefficient with its values ranging from 0 for the case of undisturbed rock materials to 1 for very disturbed. The value of m_i could be obtained from the measured inclination of the fracture planes in compression tests. Therefore, the magnitude of m_i should theoretically vary with respect to specific rock strata. For simplification, the approximate values are broadly utilized to conduct theoretical analysis with ease in the absence of available experimental data. Hoek [15] categorized the following five kinds of rock materials, as specifically, $m_i \approx 7$ for carbonate rock masses with well-developed crystal cleavage; $m_i \approx 10$ for the case of the lithified argillaceous rock; $m_i \approx 15$ for arenaceous rocks with strong crystals and poorly-developed crystal cleavage; $m_i \approx 17$ for the case of fine-grained polyminerallic igneous crystalline rock materials; and $m_i \approx 25$ for the coarse-grained polyminerallic igneous and metamorphic rocks.

2.4. Generalised tangential technique

In this study, the nonlinear HB failure criterion is adopted to estimate the stability of tunnel face under the condition of water-filled cavity. To obtain the upper bound solution of supporting pressure required to maintain face stability, some approaches should be taken to express the nonlinear properties of the yield criterion. An effective approach is to simplify the nonlinear criterion into a linear one. Based on this approach, the generalised tangential technique was first proposed by Yang.[16] Thereafter, this methodology was extended to evaluate slope stability and ultimate bearing capacity of foundations with nonlinear failure criterion and limit analysis theory.[17–19]

With reference to Yang,[16] a tangential line for characterizing the