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Stability analysis of shallow tunnels subjected to eccentric loads by a boundary element method

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ABSTRACT

In this paper, stress behavior of shallow tunnels under simultaneous non-uniform surface traction and symmetric gravity loading was studied using a direct boundary element method (BEM). The existing full-plane elastostatic fundamental solutions to displacement and stress fields were used and implemented in a developed algorithm. The cross-section of the tunnel was considered in circular, square, and horseshoe shapes and the lateral coefficient of the domain was assumed as unit quantity. Double-node procedure of the BEM was applied at the corners to improve the model including sudden traction changes. The results showed that the method used was a powerful tool for modeling underground openings under various external as well as internal loads. Eccentric loads significantly influenced the stress pattern of the surrounding tunnel. The achievements can be practically used in completing and modifying regulations for stability investigation of shallow tunnels.

1. Introduction

Along with the population growth, tunnel excavation and subsurface openings have become a major requirement for urban transportation, especially in big cities. Tunnels should be designed in such a way that could be sufficiently powered against static and dynamic loads. On the other hand, urban tunnels that are mainly close to the ground surface in urban areas can affect the behavior of the existing structures such as buildings, roads and railways. Therefore, it is necessary for engineers to utilize appropriate tools as well as efficient methods to determine more precise ground responses.

Technically speaking, there are several studies on stability analysis of shallow tunnels. Stability of circular tunnels has been extensively studied at Cambridge since the 1970s, for example, the works reported by Atkinson and Cairncross (1973), Cairncross (1973), Mair (1979), Seneviratne (1979), and Davis et al. (1980). Before the 1990s, most of the published works have focused on the stability of circular tunnel in undrained clayey soil. Later, theoretical solutions for circular tunnel problems in drained conditions have been determined by Muhlhaus (1985) and Leca and Dormieux (1990). Recently, using the theoretical approach proposed by Fraldi and Guarracino (2009), they presented a full analytical solution for collapse mechanisms of tunnels with arbitrary excavation profiles based on plastic Hoek–Brown criterion (Fraldi and Guarracino, 2010).

In recent decades, a large number of numerical methods have been proposed to calculate the responses of underground structures and estimate failure of surrounding rock mass. Among these methods are finite element method (FEM) and finite difference method (FDM). Rowe and Kack (1983) predicted soft ground settlement located above the tunnel using FEM. In order to model the tunnel geometry and determine the failure mechanisms, FEM was used by Koutsabeloulis and Griffiths (1989). Lee and Rowe (1991) calculated the deformations that occur in clayey soil surrounding the tunnel by three-dimensional (3D) FEM. Jao and Wang (1998) performed an extensive study on various soils in terms of the stability of shallow foundations located on underground tunnels. They simulated soil models and used FEM to investigate the effect of different tunnel positions. To investigate the behavior of unconsolidated soils with inclined layers, Park and Adachi (2002) applied an experimental procedure as well as finite element (FE) analysis.

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Azevedo et al. (2002) evaluated the response of residual soils in the presence of shallow tunnels using elastoplastic FE back analysis and found a good agreement between the displacements obtained from field studies and numerical results.

The application of FE limit analysis to the undrained stability of shallow tunnels was first considered by Sloan and Assadi (1993). They investigated the case of a plane-strain circular tunnel using linear programming techniques in cohesive soil whose shear strength varied linearly with depth. Later, Lyamin and Sloan (2000) considered the stability of a plane-strain circular tunnel in a cohesive-frictional soil using a developed nonlinear programming technique. Yamamoto et al. (2011a, b, 2012, 2013) investigated the stability of plane-strain single/dual circular as well as square tunnels in cohesive-frictional soils subjected to surcharge loading using FE limit analysis technique. They found that the failure mechanisms of shallow square tunnels were completely different from those of shallow circular tunnels due to the absence of curved geometries. Fraldi and Guarracino (2011) carried out a comparative study between numerical and analytical approaches for modeling plastic collapse in circular tunnels. They indicated that the numerical modeling of the evolution of progressive failure leading to collapse in the tunnels remains a complicated issue, which requires great care in preparing the model and analyzing the results.

Despite the simple formulation and also development in elastoplastic problems, FEM and FDM are continually accompanied by a large volume of calculations for analyzing problems including unlimited boundaries. Therefore, data and computation time are subsequently increased. Also, by applying approximate boundary conditions to truncated boundaries, the models become complicated and the accuracy is reduced. On the other hand, the boundary element method (BEM) can be practically used for the problems in which the domain includes infinite as well as semi-infinite boundaries because of discretizing boundaries instead of domain. It is worth mentioning that full-plane BEM has been completely developed for linear elastostatic problems (Brebbia and Dominguez, 1989).

Although the BEM has been formed over four decades, qualitative improvements of computers in recent decades have accelerated the development of BEM as well as other engineering issues. For the first time, boundary discretizing was used in 1903 for the potential flow equations (Fredholm, 1903). After a few decades, many researchers developed boundary integral equations (BIEs) to solid mechanics (Massonnet, 1965; Benjumea and Sikarskie, 1972; Banerjee and Driscoll, 1976). Since 1980, BEM has been also used for solving rock/soil mechanics problems and some researchers have used it for studying opening models in the continuous infinite space (Banerjee and Butterfield, 1981; Crouch and Starfield, 1983). During the same period, between 1980 and 1983, Hoek and Brown presented a criterion called the Hoek–Brown failure criterion for the intact/fractured rock resistance, whose complete version was presented in 1992 (Hoek and Marions, 2007). Commercial softwares, such as FLAC^{2D} and EXAMINE^{2D}, which have been used by some researchers for modeling two-dimensional (2D) underground structures, could not create the foundation shallow loads (Shah, 1992; Martin et al., 1999; Kooi and Verruijt, 2001). Panji (2007), Asgari Marnani and Panji (2007, 2008), and Panji et al. (2011, 2012, 2013) have recently prepared an algorithm based on full/half-plane elastostatic fundamental solutions of direct BEM and used it to analyze geotechnical structures with different cases of effective loads.

The literature review showed that the presence of shallow foundations on underground tunnels could cause the interaction of the induced stresses by shallow loads with tunnels, which reduces stability as well as bearing capacity (Banerjee and Driscoll, 1976; Panji et al., 2012). In previous studies, tunnels with in-situ loads have been only considered. Therefore, the purpose of the present study was to observe the behavior of shallow tunnels including different cross-sections subjected to various simultaneous gravity and shallow loadings. In this regard, the effect of one of the key parameters, i.e. the eccentricity of shallow loads, was studied by a developed algorithm based on full-plane elastostatic BEM.

2. Full-plane BEM

After applying the weighted residual integral to Navier's elastostatic equilibrium equation, regardless of body forces, the following equation can be obtained (Brebbia and Dominguez, 1989):

$$\int_{\Omega} \sigma_{kj,j} u_k^* \mathrm{d}\Omega = 0 \tag{1}$$

where u_k^* is the weight function or Kelvin's fundamental solution for full-plane, Ω indicates the domain, and $\sigma_{kj,j}$ is the stress component. After twice integration by parts of Eq. (1), Green's equation is obtained as follows:

$$\int_{\Omega} \sigma_{kj,j}^* u_k \mathrm{d}\Omega = -\int_{\Gamma} p_k u_k^* \mathrm{d}\Gamma + \int_{\Gamma} u_k p_k^* \mathrm{d}\Gamma \tag{2}$$

where p_k and u_k indicate the stress and boundary displacements, respectively; p_k^* is the full-plane stress fundamental solution; and Γ specifies the boundary. Using the Dirac delta method, the following BIE is presented after omitting the domain terms:

$$c_{lk}^i u_l^i + \int_{\Gamma} p_{lk}^* u_k \mathrm{d}\Gamma = \int_{\Gamma} u_{lk}^* p_k \mathrm{d}\Gamma \tag{3}$$

where $c_{lk}^i = 1 - \theta_i/(2\pi)$, θ_i is the boundary fraction angle of node i; u_{lk}^* and p_{lk}^* are the full-plane displacement and traction fundamental solution, respectively (Brebbia and Dominguez, 1989). After solving Eq. (3), boundary unknowns including displacements as well as tractions can be obtained for each boundary node i. It is noteworthy that in order to determine stresses at any defined point within the domain, full-plane stress fundamental solutions can be obtained using the displacement fields as follows:

$$\sigma_{ij} = \int_{\Gamma} D_{kij}^* p_k \mathrm{d}\Gamma - \int_{\Gamma} S_{kij}^* u_k \mathrm{d}\Gamma \tag{4}$$

where D_{kij}^* and S_{kij}^* are the internal stress fundamental solutions which can be found in Brebbia and Dominguez (1989).

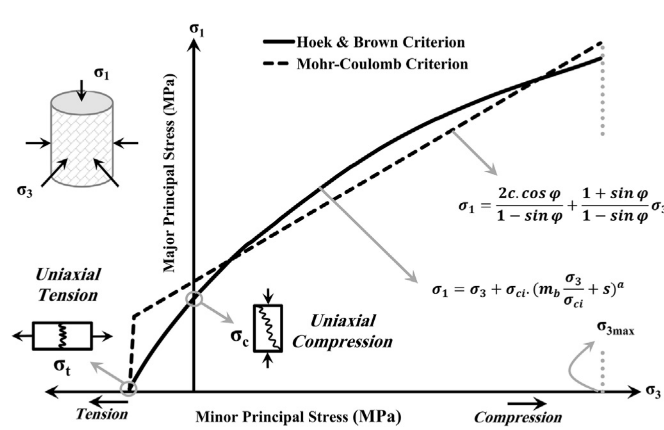

Fig. 1. Relationship between maximum/minimum principal stresses of Hoek–Brown criterion and equivalent values obtained using Mohr–Coulomb criterion.