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# Tunnel face stability under seepage flow conditions

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ABSTRACT

The present paper investigates the problem of the stability of the tunnel face under seepage flow conditions based upon the so-called "method of slices". This computational model improves the limit equilibrium method of Anagnostou and Kovári (1996) by treating the equilibrium in the wedge consistently with the overlying prism, i.e. without an *a priori* assumption concerning the distribution of the vertical stresses. Furthermore, it shows that tensile failure of the wedge may be more critical than shear failure, if the gradient of the hydraulic head in the ground ahead of the face is high. For an approximate distribution of the hydraulic head in the ground around the tunnel face, we derive a closed-form solution for the necessary face support pressure. In addition, we provide normalized diagrams, which allow for a quick assessment of the stability of the tunnel face.



## 1. Introduction

Seepage flow is unfavorable for the stability of the tunnel face because it is associated with the occurrence of hydraulic head gradients in the ground ahead of the excavation face. The hydraulic head gradient acts as a body force, the so-called "seepage force", which is directed towards the face and is, therefore, unfavorable with respect to its stability.

The effect of seepage flow on the face stability was already investigated in previous works with different methods. Anagnostou and Kovári (1994, 1996) applied a limit equilibrium method including the effect of seepage forces. The latter were computed numerically by means of three dimensional, steady state flow analyses. They presented nomograms for the assessment of the required effective support pressure in cohesive frictional soils under several hydraulic boundary conditions. Lee and Nam, (2001, 2006) and Lee et al. (2003) applied the upper bound solution of Leca and Dormieux (1990), also using numerical seepage flow analyses for the determination of the seepage forces. Lee et al. investigated only the case of purely frictional soils, while Park et al. (2007) investigated with the same method the stability in a cohesive frictional soil characterized by a linear variation of cohesion with the depth. Ströhle and Vermeer (2010) as well as Vermeer et al. (2002) assessed the stability of the tunnel face for specific parameter sets by means of numerical stress analyses according to the finite element method.

Similar to Anagnostou and Kovári (1992, 1996), the present paper investigates face stability under drained conditions by considering the wedge and prism mechanism of Fig. 1a, but analyses the equilibrium of the wedge based upon the method of slices (Anagnostou 2012). In analogy to the silo theory, the method of slices assumes proportionality between the horizontal stress $\sigma_y'$ and the vertical stress $\sigma_z'$:

$$\sigma_y' = \lambda \, \sigma_z', \tag{1}$$

where the coefficient of lateral stress $\lambda$ is assumed to be constant. The method of slices eliminates thus the need for an *a priori* assumption about the distribution of the vertical stress $\sigma_z'$ in the wedge. The computational predictions of the method of slices agree very well with published results of experimental tests in dry soil when $\lambda$ is taken equal to 1.0 (Anagnostou 2012). For this reason, the assumption of $\lambda = 1$ will be made throughout the present paper instead of the value $\lambda = 0.8$ suggested by Anagnostou and Kovári (1992, 1994).

In order to calculate the distribution of the vertical stresses $\sigma_z'$ inside the wedge, the equilibrium of an infinitesimally thin slice is considered (Fig. 1b). This makes it possible to analyze cases with non-uniform face support, heterogeneous ground (consisting of horizontal layers) or non-uniform distribution of the seepage forces along the height of the face.

In this paper we consider a homogenous soil obeying the Mohr–Coulomb failure criterion, a uniform support pressure and an approximate distribution of the hydraulic head, which is obtained by fitting the results of three dimensional seepage flow analyses (Fig. 2). All computational examples assume, furthermore, that

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**Notation**

| | |
|---|---|
| $a$ | coefficient (Eq. (3)) |
| $b$ | coefficient (Eq. (2)) |
| $B$ | width of the tunnel face |
| $c'$ | effective cohesion of the ground |
| $C_s$ | coefficient (Eq. (A12)) |
| $C_c$ | coefficient (Eq. (A14)) |
| $C_{\Delta h}$ | coefficient (Eq. (17)) |
| $C_v$ | coefficient (Eq. (A15)) |
| $C_\gamma$ | coefficient (Eq. (A13)) |
| $F$ | coefficient (Eq. (A16)) |
| $F_1$ | coefficient (Eqs. (26)) |
| $F_2$ | coefficient (Eqs. (27)) |
| $F_3$ | coefficient (Eqs. (28)) |
| $F_x$ | resultant seepage force in the x- direction |
| $F_y$ | resultant seepage force in the y-direction |
| $F_z$ | resultant seepage force in the z- direction |
| $G'$ | submerged weight |
| $h$ | hydraulic head |
| $h_0$ | undisturbed hydraulic head, elevation of water table |
| $h_F$ | hydraulic head on the tunnel face |
| $\Delta h$ | hydraulic head difference between water table and tunnel face |
| $H$ | height of the tunnel face |
| $i_{av}$ | average vertical hydraulic gradient in the prism at the elevation $z$ (Eq. (5)) |
| $i_{av}{}^*$ | limit average vertical hydraulic gradient in the prism (Eq. (6)) |
| $M$ | coefficient (Eq. (A6)) |
| $M_c$ | coefficient (Eq. (A10)) |
| $M_\gamma$ | coefficient (Eq. (A9)) |
| $N'$ | effective normal force |
| $P$ | coefficient (Eq. (A7)) |
| $P_1$ | coefficient (Eq. (20)) |
| $P_2$ | coefficient (Eq. (21)) |
| $P_3$ | coefficient (Eq. (22)) |
| $P_4$ | coefficient (Eq. (24)) |
| $P_c$ | coefficient (Eq. (A11)) |
| $P_s$ | coefficient (Eq. (A8)) |
| $R$ | ratio of the volume to circumferential area of the prism |
| $s'$ | effective support pressure |
| $s'_1$ | required effective support pressure (criterion: maximum tensile stress on the sliding surface is equal to $c'/\tan\phi'$) |
| $s'_2$ | required effective support pressure (criterion: no tensile stress on the sliding surface) |
| $s'_3$ | required effective support pressure (criterion: silo load is equal to the bearing capacity of the wedge) |
| $S'$ | effective support force |
| $t$ | overburden |
| $t^*$ | part of the overburden with non zero silo pressure (Eq. (10)) |
| $T$ | shear force at the inclined slip plane |
| $T_s$ | shear force at the lateral slip plane |
| $V'$ | effective vertical force |
| $V'_{silo}$ | effective vertical load exerted by the prism upon the wedge |
| $x$ | horizontal co-ordinate parallel to the tunnel axis |
| $y$ | horizontal co-ordinate perpendicularly to the tunnel axis |
| $z$ | vertical co-ordinate |
| $z^*$ | integration limit for the determination of the silo pressure (Eq. (7)) |

*Greek symbols*

| | |
|---|---|
| $\bar{\alpha}$ | coefficient (Eq. (9)) |
| $\gamma'$ | submerged unit weight of the soil |
| $\gamma_w$ | unit weight of the water |
| $\lambda$ | coefficient of lateral stress |
| $\Lambda$ | coefficient (Eq. (A5)) |
| $\sigma'_n$ | effective normal stress |
| $\sigma'_y$ | effective horizontal stress perpendicularly to the tunnel axis |
| $\sigma'_z$ | effective vertical stress |
| $\phi'$ | effective friction angle of the ground |
| $\omega$ | angle between face and inclined sliding plane of the wedge |
| $\xi$ | normalized $z$ co-ordinate |

the water table is at (or higher than) the soil surface ($h_0 \geqslant t + H$) and that the tunnel has a circular cross-section. The latter is approximated for computational simplicity by a square ($H = B$ in Fig. 1).

Section 2 describes the assumptions and typical results of the numerical seepage-flow analyses and presents equations approximating the distribution of the hydraulic head. These equations are used in order to compute approximate seepage forces, which are introduced in the limit equilibrium analysis (Section 3). Section 4 discusses the application of the method to weak rocks, which may exhibit tensile strength due to cementation. Section 5 shows by means of comparative calculations that the error induced by the approximate hydraulic head of Section 3 is acceptably small. The approximate hydraulic head is then used in a comprehensive parametric study, which is the basis for elaborating dimensionless design normalized diagrams (Section 6). Finally, Sections 7 and 8 compare the predictions of the proposed model with those of Anagnostou and Kovári (1996) and other methods, respectively.

## 2. Seepage flow analysis

We determine numerically the three dimensional, steady state hydraulic head field around the tunnel face assuming Darcy's law with a uniform ground permeability. The permeability coefficient does not influence the stationary hydraulic head field in a homogeneous ground. A no-flow boundary condition and a constant piezometric head $h_F$ are prescribed to the tunnel wall (impervious lining) and to the tunnel face, respectively. At the far-field boundary, the piezometric head is taken equal to the initial elevation of the water table $h_0$. Assuming a sufficient groundwater recharge from the surface, the drainage effect of the tunnel does not cause a draw-down of the water table. This is taken into account by prescribing the boundary condition $h = h_0$ to the water table. Fig. 2a shows the central part of the finite element mesh adopted for the calculations. The latter were performed by the finite element program COMSOL®. A square tunnel cross-section is considered for simplicity (analog to the limit equilibrium model). Due to the vertical symmetry plane, the computational domain consists of one half of the system. Fig. 2b shows typical numerical results (contour lines of hydraulic head).

Fig. 3 shows the numerically computed distribution of the normalized hydraulic head along the vertical axis $z$ above the tunnel (Fig. 3a) as well as along three horizontal lines ahead of the face in the axial direction. The distribution of the hydraulic head depends only slightly on the normalized overburden $t$: the smaller the overburden, the higher the hydraulic gradients will be. The