

Assessment of tunnel's face support pressure

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Abstract. The evaluation of support pressure ensuring tunnel's face stability during excavation using tunnel boring machines is one of the fundamental problems in tunnelling. This paper presents various approaches to this problem. The analytical models, based either on the principle of limit equilibrium or the limiting analysis (low boundary or upper boundary limiting methods), requires considerable simplification (circular tunnel, homogeneous rock mass etc.). But, on the other hand they are less time-consuming in comparison with the numerical models based on the finite element method. This paper describes the comparison of results of minimum face pressure obtained on the basis of different calculation methods with respect to the specific geometrical and material characteristics of the problem. Geotechnical software Midas GTS NX (based on finite element method) was used for presented numerical analysis. The paper also provides a comparison of modelling results with the results of the face pressure corresponding to the real excavation of the tunnel.

1. Introduction

One of the fundamental tasks in a tunnel excavation using tunnel boring machines is determination of the optimum face support pressure, which ensures the safety of the excavation itself, helps to minimize a surface settlement and ultimately prevents the collapse of the entire tunnel. The value of the face support pressure generally depends on many factors, primarily on the size of the tunnel, the depth below the surface, cohesion and internal friction angle of the rock environment, unit weight of the soil environment, permeability of the rock environment, machine type, excavation speed, depth of groundwater level, etc.

Research concerning the determination of the optimum tunnel face pressure is globally conducted by experimental methods, including centrifugal tests (e.g Kamata [9], Kimura, Mair [10]), as well as by various mathematical (analytical and numerical) methods [3].

2. Basic outline and principles of calculation methods to determination of the face pressure

The methods to calculate pressure on the tunnel face differ in their mathematical approach, calculation assumptions, the degree of conservatism of results, required computing time and calculation efficiency, which is a very important factor, particularly when practical problems are solved.

Analytical calculation methods to determine the minimum tunnel face pressure are either based on the limit equilibrium methods (Broms and Bennmark [4], Krause [11], Jancsecz and Steiner [8], Anagnostou and Kovári [1], Broere [3], Carranza-Torres [5]), or the limit analysis method (Atkinson and Potts [2], Davis, et al. [6], Leca and Dormieux [12]). Although derived analytical methods to determine the minimum tunnel face pressure are usually derived under quite restrictive conditions,



mainly relating to the homogeneity of the rock environment, circular cross-section of the tunnel, hydrostatic state of primary stress, disregarding the nature of the lining behind the face in the excavated area, etc., they are less time consuming in comparison with numerical methods.

Analytical calculation methods to determine the minimum face support pressure based on limit analysis can be divided into two basic groups. The first group of methods is based on the so-called “lower bound theory”. These methods allow for the determination of a statically acceptable solution for minimum face pressure, that satisfies both balance and rock failure conditions. The basic underlying assumptions for these types of methods imply, that they allow obtaining conservative value of the minimum tunnel face support pressure.

The second group of computational methods is based on the so-called “upper bound theory”, which assumes kinematically acceptable solution subject to a certain accepted rock failure mechanism. Methods of this type assume that, given a certain rock failure mechanism, the compatibility of displacements between individual blocks of material in a rock failure process is not breached. Unlike solutions based on lower bound theory, these computational approaches to obtain a kinematically acceptable solution allow obtaining a non-conservative value of face pressure.

Computational schemes, which foresee the emergence of a particular rock failure mechanism before the face, are based on the Horn model [7], which replaces the circular shape of the tunnel face with a square shape, and approximates the corresponding rock failure area before the tunnel face using rigid blocks – wedge and the follow-up prism (chimney).

3. Characteristics of the numerical model for calculation of the face support pressure

For the comparative analysis of the various approaches to determining the required minimum face pressure, the authors of this paper used the numerical model of an excavated tunnel of a circular cross-section. Both software Midas GTS NX and Plaxis 3D were used for numerical analysis. These codes were developed for the stress-strain and stability analysis of geotechnical problems and are based on a numerical finite element method. Mentioned software allow the simulation of linear and nonlinear behavior of soil, provide to model both hydrostatic and hydrodynamic effects of water in soils and the interaction between structures (reinforcement, anchors and wall) and the soil.

The basic model dimensions 70x50x75 m were considered (see Figure 2). The basic tunnel diameter $D = 5$ m was considered at different depths $h = 10, 20$ or 30 meters below the surface. The ratio between the diameter of the tunnel and the depth (h/D) achieves value 6, 4 and 2 respectively. In the next part of research next variants of diameters $D = 8$, resp. 10 m were considered. The length of the excavated tunnel was considered 25 meters. Due to a more objective comparison with analytical methods, the calculations assume the hydrostatic state of primary stress, i.e. the coefficient of lateral pressure is equal to one. Influence of ground water was not considered in the calculation. The input material properties of the rock environment and concrete structure are given in Table 1. Angle of internal friction was considered from 25° to 35° with a step of 5° . The model used the final tetrahedron-shaped elements. Elastic- perfectly plastic Mohr-Coulomb constitutive model of soil was applied in the model. The concrete lining of thickness of $d = 35$ cm was characterized by elastic constitutive model. This lining will be installed immediately in one step after excavation (no gap between the excavation and lining installation was assumed). The standard boundary deformational conditions (rigid box) were assumed.

For simulation of stabilizing pressure at the face the pressure on full face was considered. The values of pressure were changed from 10 to 800 kPa. The first calculation stage involved deactivation of the volume elements inside the tunnel and activation of the shell elements corresponding to the tunnel lining. In the first stage the full pressure load on the face was activated. Each loading phase was solved separately.

Table 1. Properties of the rock environment and concrete structure.

	Model	E (MPa)	ν	γ (kN/m ³)	c (kPa)	ϕ (°)
Soil	M/C	45	0.35	20	2	from 25 to 35
Concrete	Elastic	20 000	0.2	24	-	-

4. Assessment of minimum tunnel face support pressure obtained by analytical and numerical methods

To the assessment of minimal tunnel face pressure both analytical and numerical methods were applied. Figure 1 shows the comparison of the results of the selected analytical approaches to determining the minimum stabilization face pressure. Equations were published earlier [11, 14, 15]. Due to the practically non-cohesive nature of the investigated rock environment (cohesion is equal to 2 kPa), methods working with undrained shear strength c_u are not considered in the presented study. Analyzed methods don't reflect the impact of tunnel depth.

Based on the results of the calculations, it is possible to formulate the following conclusions:

- The range of the minimum face support pressure for all the compared analytical methods is 20 to 80 kPa for internal friction angle $\phi = 20^\circ$, values of 15 to 50 kPa for $\phi = 30^\circ$ and 10 to 38 kPa for $\phi = 40^\circ$.
- The most conservative values of the minimum face support pressure result from the Krause method, considering the quadrant rock failure mechanism.
- In terms of the face support pressure, the least conservative analytical method is the Krause method considering hemispherical rock failure mechanism.
- The model indicates more significant dependence of the face support pressure on the internal friction angle in the case of lower values of the internal friction angle. [15]

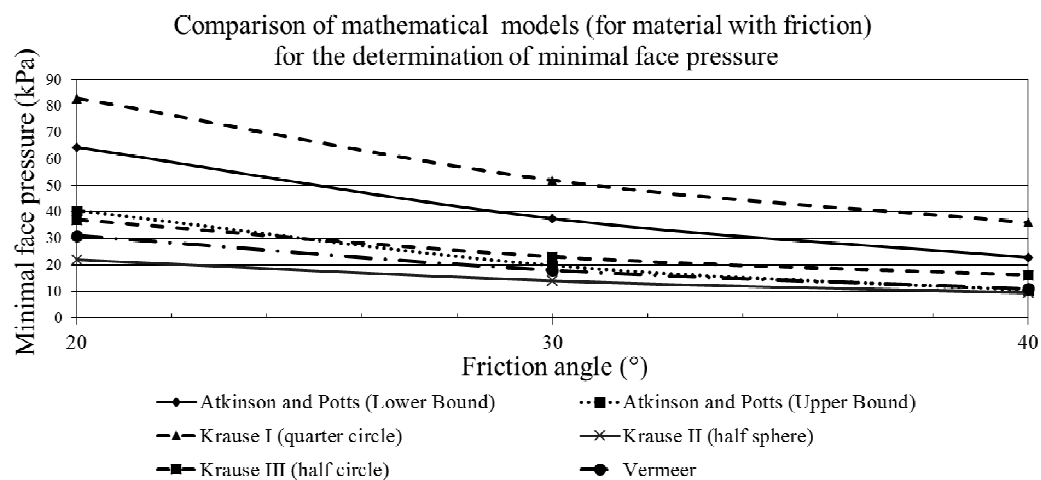


Figure 1. Comparison of the values of minimum face support pressures determined by analytical computational methods.

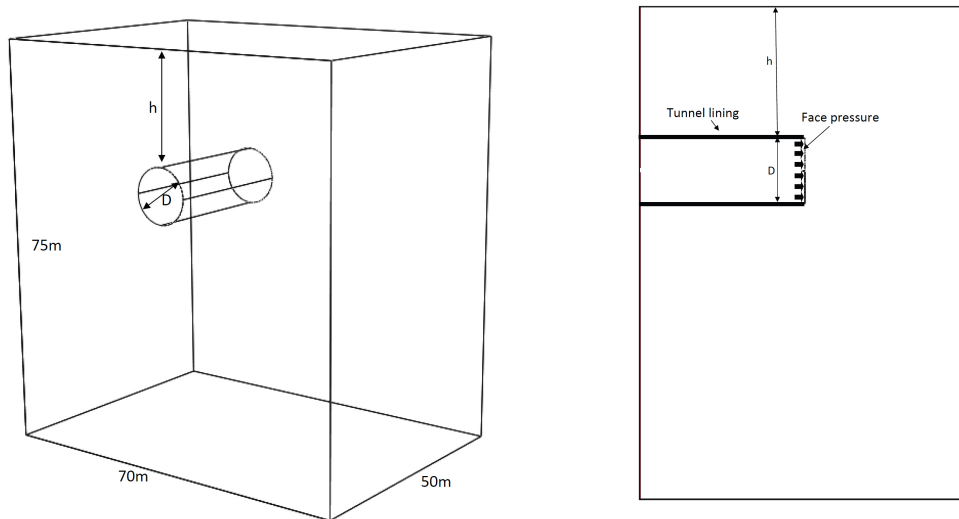


Figure 2. Conceptual model.

Three different tunnel diameters D were assumed ($D = 5, 8$ or 10 m respectively) in the performed numerical analysis. Moreover, the variant friction angle between 25 and 35° was assumed. Depth h of the tunnel below the surface was considered 20 m.

Based on the numerical model, all alternative depths, internal friction angles and face loads were assessed for horizontal face displacements corresponding to face extrusion into the space behind the face.

Figure 3 illustrates the dependence of the maximum horizontal face displacements depending on the face pressure for three different values of excavation diameter D at the same tunnel depth of 20 m. Friction angle of 30° was considered. For the diameter $D = 10$ m under the same face pressure of 50 kPa the face displacements are increased 3.5 times in comparison with the diameter $D = 5$ m. For the higher value of face pressure the corresponding face displacements are practically identical independently on the diameter. The face pressure, which causes such a qualitative change in the developments of horizontal displacements and more significant face extrusion, is considered the required minimum face pressure. Based on this approach, the minimum face support pressure determined by the model for $D = 10, 8$ and 5 m reaches 150 kPa, 100 kPa and 75 kPa respectively. Face pressure of approximately 600 kPa leads to virtually no signs of face extrusion, with horizontal displacements being zero. At pressures exceeding 600 kPa, the calculation model shows developments of horizontal face displacements in the opposite direction (the face is pushed back by the pressure).

Figure 4 illustrates the dependence of the maximum horizontal face displacements depending on the face pressure for three different values of internal friction angle and identical tunnel depth below the surface of 30 m. This dependence allows us to conclude that, depending on the internal friction angle, up to a certain face pressure, the horizontal face displacements are not very significant and there is no significant extrusion into the free space behind the face. The minimum face support pressure reaches 60 kPa for the highest considered internal friction angle of 35° , approximately 75 kPa for $\varphi = 30^\circ$ and 100 kPa for $\varphi = 25^\circ$.

Figure 5 documents the range of the deviatoric strain for safety calculation related to the face support of 10 kPa, the tunnel diameter of 5 m, the tunnel depth of 20 m and friction angle of 30° . Range of maximum deviatoric strain zone in front of the face is about 3.8 m and the shape of this zone corresponds to the Horn's scheme (including chimney) [7]. Angle of the wedge from the vertical position is about 37.5° , range of this zone above the roof (in the overburden of the tunnel) is 6 m (approximately the same as the tunnel diameter).

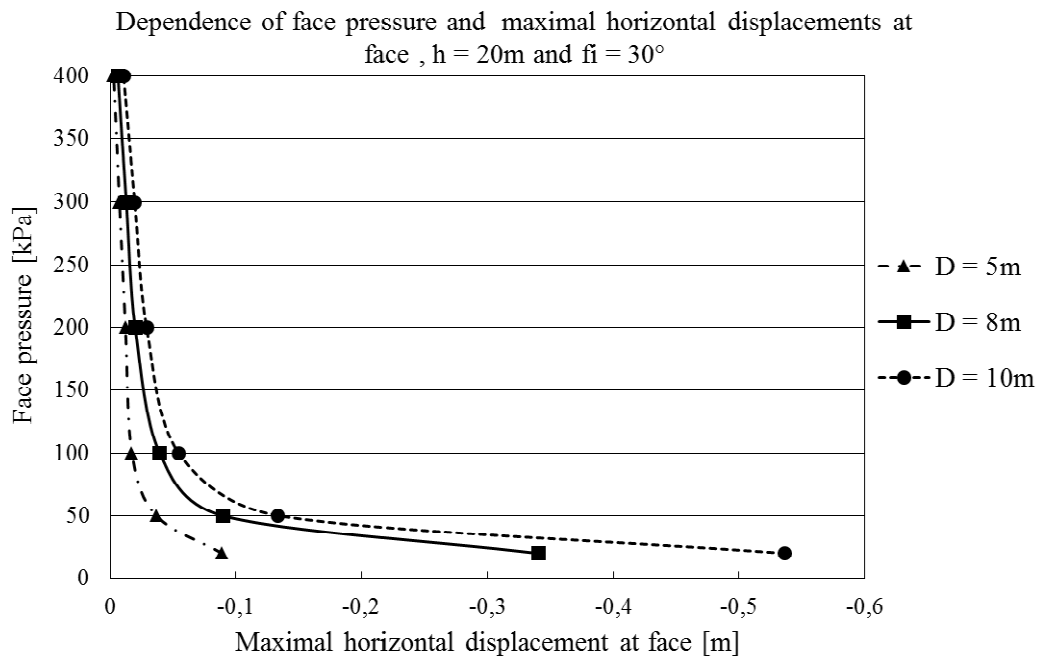


Figure 3. Dependence of face pressure and maximal horizontal displacements at face for various tunnel diameter.

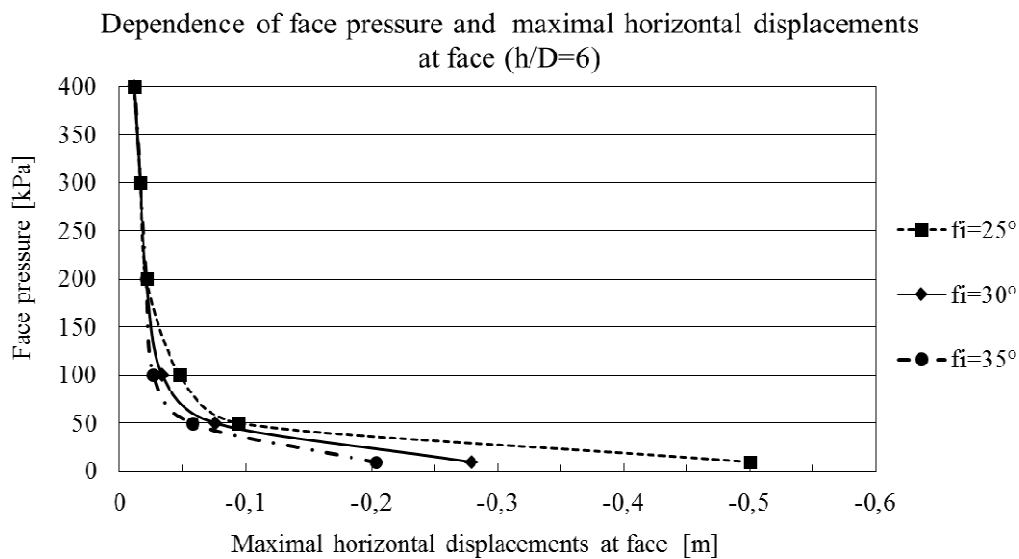


Figure 4. Dependence of face pressure and maximal horizontal displacements at face for various friction angle.

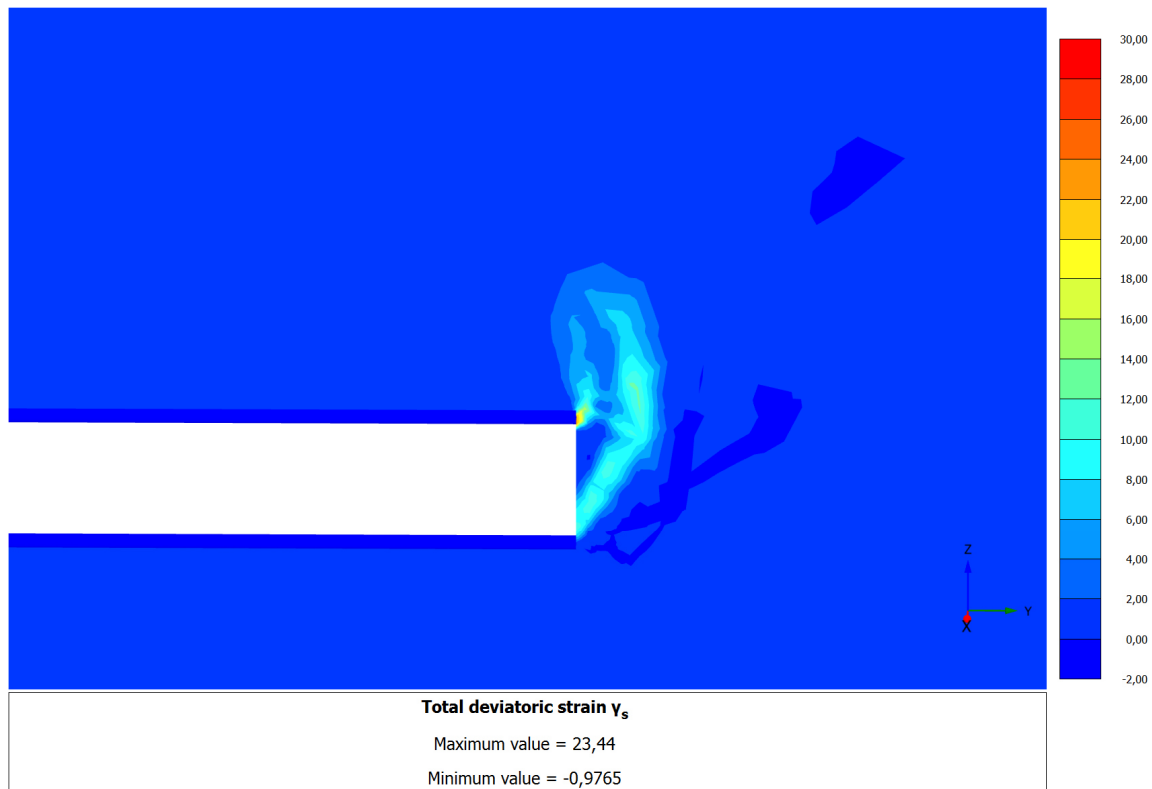


Figure 5. Pattern of total deviatoric strain in front of the face (PLAXIS).

5. Conclusion

The results of analytical computational models and numerical 3D model created in MIDAS GTS (finite element method) indicate minimum face pressure values during the excavation of the specified tunnel with circular cross-section excavated by full tunnel face. Using the finite element method for a depth of 30 m, the given model task results in 20% higher minimum face support pressure compared with the most conservative analytical Krause method (which, however, disregards tunnel depth). For the tunnel depth of 20 m, the values of the maximum face support pressures are comparable for the most conservative analytical method as well as the finite element method. For the tunnel depth of 10m, the value of the minimum face support pressure is smaller in comparison with the most conservative analytical method, but the value is greater than the value determined by the least conservative analytical method. The zone of maximum deviatoric strain, resulted from the 3D PLAXIS calculation, confirms the shape of Horn's failure area in front of the face.

When formulating these conclusions, account must also be taken of the fact that the finite element method is a deformation method, and calculation thus depends not only on strength parameters of the soil environment, but also on deformation parameters (Young's modulus and Poisson's ratio), which are not even included in the analytical methods to determine the minimum support face pressure. However, although the quantitative values of face deformations (equivalent to horizontal displacements on the face) can be expected to naturally be different for different values of the Young's modulus of the soil environment, they would not be very different qualitatively. To verify these particular conclusions, it is necessary to conduct further, more extensive numerical analysis and comparison with a real face pressure during an actual tunnel excavation.

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