Excavation stability of deep tunnels in water-bearing fault zones

Gerold Lenz
iC consulenten ZT GesmbH, Bergheim / Salzburg, Austria

ABSTRACT: Groundwater is one major reason for tunnel collapses throughout history. Tectonic fault zones, characterized by heterogeneous rock mass composition and low strength, are particularly prone to such events. The article describes the groundwater conditions in heterogeneous ground at high overburden, revealing that high hydraulic gradients may develop close to the face in such situations. Numerical analyses with full hydraulic-mechanical coupling indicate a poorly confined region, subject to seepage forces, forming ahead of the face when approaching a fault zone. The governing hydraulic failure mechanisms for excavation stability can be distinguished into seepage-force driven mechanical failures (so called plug failure and cracking) and erosion processes. For the former, the article describes a novel solution to assess tunnel face stability subject to seepage forces, based on the method of slices applied to a hemispherical failure body.

Keywords: flowing ground, groundwater, seepage, fault zone, deep tunnel, face stability.

1 INTRODUCTION

Several historical and recent cases describe tunnel collapses or critical incidents related to high water or mud ingress into tunnels, often referred to as 'flowing' or 'swimming ground'. Tectonic fault zones are particularly prone to such events. On the one hand such zones usually exhibit low rock mass strength. On the other hand, strongly varying hydraulic parameters within the fault zone may influence the distribution of hydraulic heads around the tunnel and thus yield more adverse groundwater conditions than in homogeneous ground.

Although the geological - geotechnical knowledge and methods developed rapidly during the last decades, the geomechanical failure modes triggering flowing ground conditions in tunnelling are hardly described. Lacking understanding of the failure mechanisms, practicable methods to assess tunnel stability in water-bearing rock mass under high overburden are currently scarce. This paper shall contribute to understanding the hydraulic conditions and failure mechanisms for deep tunnels excavated in weak and water-bearing rock mass.

2 HYDRAULIC HEAD FIELD AND EFFECTIVE STRESSES

2.1 *Hydraulic head field*

For assessing groundwater-related failure modes, knowledge of the hydraulic head field (i.e. the distribution of pore pressures) in vicinity of the tunnel face is essential. In homogeneous rock mass, the hydraulic head field predominately depends on the hydraulic conductivity of the rock mass, the advance rate of the excavation, the size of the excavation and the initial groundwater level. The lower the hydraulic conductivity and the faster the excavation advance, the steeper are the hydraulic gradients occurring ahead of the face and the higher are the resulting seepage forces acting towards the tunnel.

A comparison between homogeneous and heterogeneous rock mass conditions (with several layers of varying hydraulic conductivities) show, that in the latter case the occurring gradients, and consequently seepage forces and inflow rates, are significantly higher than in homogeneous conditions (Leitner & Müller 2007, Zingg & Anagnostou 2012, Lenz 2020). The most adverse conditions regarding stability of the tunnel face develop when a tunnel is excavated in low-permeability rock mass and approaches rock mass with higher permeability, or when an excavation approaches a series of layers with varying permeability.

Numerical seepage flow analyses are conducted with the software FLAC3D (Itasca 2017) to assess the hydraulic head field ahead of tunnel excavation. The numerical model comprises a cylindrical tunnel with a radius of 5 m. Both, face and lining of the tunnel, are assumed as permeable and at atmospheric pressure. The analysis allows for the computation of steady as well as transient hydraulic states. The initial hydraulic head is 400 m. The tunnel advances step by step with a round length of 1 m at an advance rate of 4 m/d. In total, 100 rounds are computed. The ground model comprises of a low-permeability fault core with a thickness of 2 m, oriented perpendicular to the tunnel axis, and a 4 m wide high-permeability damage zone on either side (see Figure 1).

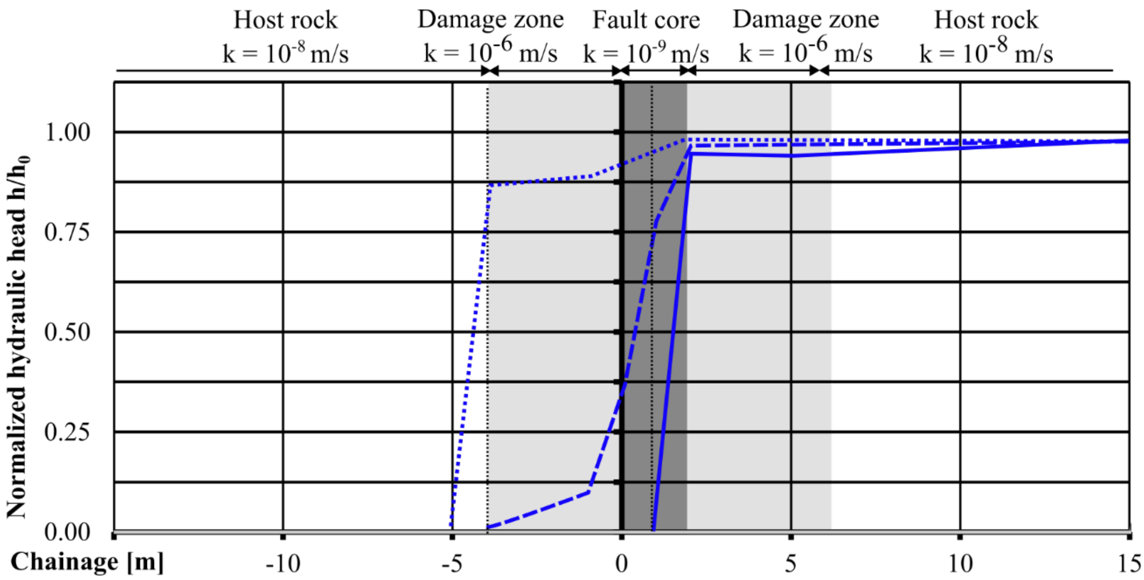


Figure 1. Selected state lines of pore pressure at the face when tunnelling through a fault zone with layers of different permeability.

Figure 1 displays the state line diagram for the pore pressure (normalized by the initial hydraulic pressure) for selected positions at the tunnel face. State lines are obtained by connecting the calculated pore pressures at the center point along the tunnel axis. When the excavation advances to one round before the first high-permeability damage zone, the hydraulic head in the damage zone is still approx. 85 % of the initial hydraulic head (Figure 1, dotted line). After entering the damage zone, the initially high heads are rapidly equalized by seepage and comparatively low hydraulic gradients occur close to the face (Figure 1, dashed line), because re-charge of the drained area is hindered by the fault core (in which steep hydraulic gradients occur). At the transition from fault core to damage zone, the maximum hydraulic gradient occurs, because drawdown in advance is hindered by the low-permeability fault core and this region is at the same time continuously re-charged from the second high-permeability damage zone (Figure 1, solid line). When entering the second damage zone, the hydraulic head is approximately 95 % of the initial head. This analysis reveals that hydraulic heads in the same range as the initial groundwater level and correspondingly high hydraulic gradients may occur when driving through fault zones, even if the extent of the fault zones is limited (as far as no advance drainage measures are applied).

2.2 Interaction of stresses and pore pressures

During tunnel excavation and the related stress redistribution, changes in pore pressure occur, provided that the differential hydraulic heads cannot be equalized fast enough by seepage flow, that is in rock mass with comparatively low permeability. In such cases, stresses and pore pressures interact. Vice versa, a change in the effective stress field causes a change in pore pressure. For rock mass with high stiffness, and consequently low changes in volumetric strains, this effect may be negligible, whereas in cases of weak rock mass with low permeability the coupling of deformation and pore pressure may play a decisive role. The interaction of stresses and pore pressure is studied in a numerical analysis considering full hydraulic-mechanical coupling according to the three-dimensional consolidation theory of Biot (1941). The calculation model is similar as previously described, however the ground model in this case comprises of one single high-permeability and low-strength damage zone with a thickness of 5 m. The relevant calculation parameters are specified in Table 1.

Table 1. Calculation parameters for numerical analysis with full hydraulic-mechanical coupling.

Parameter		Unit	Host rock	Damage zone
Specific weight	γ	kN/m³	25	25
Young's modulus	E	MPa	4000	2000
Poisson's ratio	v	-	0.25	0.27
Porosity	n	-	0.20	0.20
Friction angle	φ	°	25	22
Cohesion	c	MPa	1.20	0.10
Hydraulic conductivity	k	m/s	10^{-7}	10^{-5}
Primary stress / pore pressure	σ_0, p_0	MPa	1.80 / 0.72	
Lateral pressure coefficient	K_0	-	0.80	

The calculated effective stresses and pore pressures for calculation step '-3', when the remaining rock pillar between face and damage zone equals 3 m, are displayed in Figure 2. The stress plot reveals a stress concentration in the competent rock mass ahead of and behind the damage zone, whereas the damage zone is almost fully unloaded and unconfined (the minor principal stress goes to zero at the transition). At the same time, the pore pressure within the fault zone still is approx. 85 % of the initial value. For the following excavation step, no more equilibrium is reached, and face displacements of > 300 cm are computed. In fact, the rock mass ahead of the face fails in shear and tension subject to seepage forces acting onto the rock mass ahead of the face, which forms a 'plug' with low permeability (Figure 3a). In case of low permeability of the fault zone, negative pore pressures may develop close to the face, acting stabilizing on such mechanisms. However, such effects were not computed for the parameters used in the objective case.

3 HYDRAULIC FAILURE MODES IN DEEP TUNNELS

It is obvious that the failure mode described above does not comply with the commonly known hydraulic failure modes from soil engineering, for example described in the European code for geotechnical design, EN 1997- 1 or Eurocode (EC) 7. EC7 distinguishes four different hydraulic failure modes: failure by uplift; failure by heave; failure by internal erosion and failure by piping.

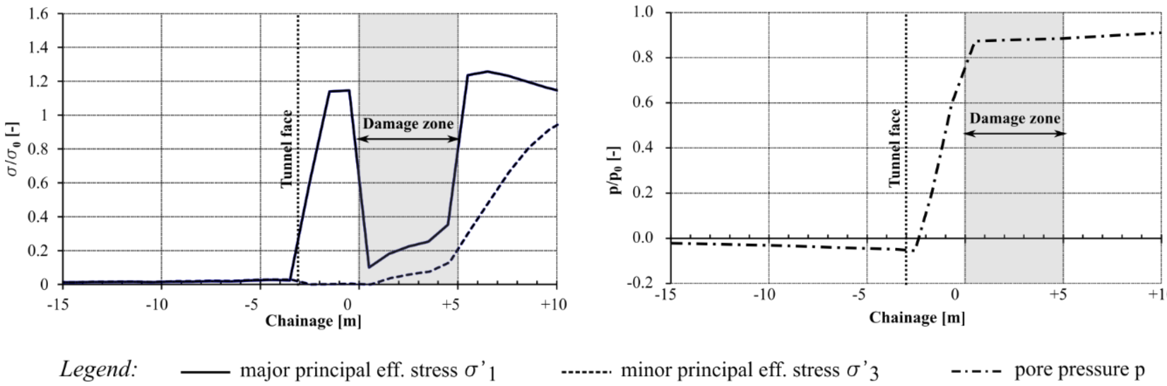


Figure 2. Normalized effective stresses and pore pressure at tunnel axis, 3 m before entering a damage zone with high permeability.

These failure modes typically apply for shallow structures in unlithified material, but for deep tunnels in rock, the definitions require adaptions. Failure by uplift is not of particular relevance, since seepage forces in vertical direction are usually low. The highest hydraulic gradients usually occur close to the face and the seepage forces are oriented towards the face. These seepage forces can trigger failure in the ground ahead of the tunnel face, e.g. when a low-permeability layer hinders seepage flow towards the tunnel face (as described in section 2.2). In other words, the rock mass forms a 'plug' in a rather poorly confined region, which is then pushed into the tunnel by seepage forces. In the following, this failure mode is therefore referred to as *plug failure*.

Failure by heave as per EC7 occurs when upwards-directed seepage forces act against the weight of the soil, reducing vertical effective stresses to zero. Soil particles are then lifted away by seepage flow. In deep tunnels, the rock mass typically exhibits at least a minimum tensile strength. Therefore, tension cracks are formed rather than spilling-out of particles. Although basically describing similar mechanical conditions, the term failure by heave would be misleading in this case. Rather, the term *cracking* is used to describe conditions, where the effective stress level exceeds the tensile strength of the rock mass in the poorly confined zone ahead of the face (plug). Cracking does not necessarily represent unstable conditions in a tunnel. In fact, the rock mass adjacent to the cracks may still be stable, e.g. due to shear and tensile strength or due to support measures at the face. However, the combination of newly formed cracks and seepage flow can trigger regressive erosion (see below), potentially leading to loss of rock mass interlocking or particle bond.

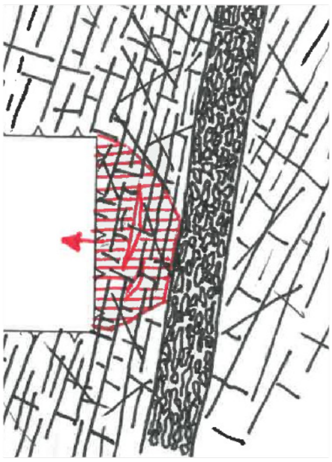
Internal erosion (suffosion) is produced by transport of particles within a soil stratum, at the interface of soil strata, or at the interface between the soil and the structure due to seepage flow. This failure mode can basically occur in any porous medium subject to seepage flow but is unlikely as soon as the ground exhibits a certain cohesion or tensile strength (Wudtke 2014). *Piping* is defined as a particular form of internal erosion, where the erosion process starts at a free surface and regresses, until a pipe-shaped channel is formed. In tunnelling, regressive erosion may be triggered by discrete geological features such as cracks, open joints, karst voids or boreholes. In such features, high flow velocities can occur, which may cause erosion in case of insufficient particle bonding.

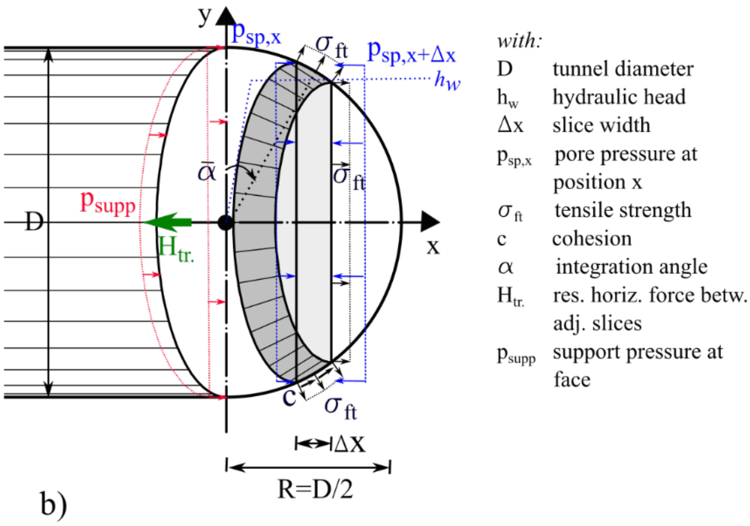
Plug failure and cracking are controlled by seepage forces and may thus be assessed with appropriate statical models, as will be elaborated in the following section. In contrast, erosion processes are highly dynamic and predominately depending on the actual seepage velocity as well as on the particle bonding. With respect to the extremely limited information on the micro-structure of lithified rock mass, such as pore, crack or discontinuity geometries as well as particle bonding, at the time being no adequate mechanical models exist to describe the erosion processes according to the author's knowledge. This topic represents a huge demand for further research for future contributors. Lenz (2020) provides a non-exhaustive collection of empirical criteria to tackle these failure modes in design.

4 ANALYSIS OF TUNNEL FACE STABLITY SUBJECT TO SEEPAGE FORCES

Three-dimensional numerical analysis undoubtedly represents the state of the art to assess excavation stability under seepage flow. However, such calculations are complex, require a significant modelling effort, a large number of input parameters and highly sophisticated software. Therefore, closed-form solutions may be used supplementary, e.g. for early design stages or for parametric studies with large parameter variabilities. Several analytical calculation models for tunnel face stability under seepage forces can be found in literature (Anagnostou & Kovari 1996, Lee et al. 2003), postulating wedge-shaped or conical failure bodies. However, these models do not reflect the failure modes described above (plug failure, cracking), being typical for deep tunnels in heterogeneous rock mass.

A novel solution based on the method of slices is therefore developed to assess face stability under seepage force action. The calculation model postulates a hemispherical failure body, free of stresses, ahead of the tunnel face (Figure 3b). Destabilizing forces are represented by horizontally oriented seepage forces acting onto the hemisphere.





a) b)

Figure 3. a) Schematic sketch of failure mode 'plug failure / cracking'; b) calculation model for assessment of face stability under seepage forces.

With respect to the results shown in section 2.1, a uniform hydraulic head (reaching up to the initial hydraulic head, depending on the geological conditions) may be assumed simplifying ahead of the face to estimate the seepage forces, with γ_w being the specific weight of water and $A_{shell,n}$ being the shell area of the slice:

$$F_{seep,h} = \gamma_w \cdot h_w \cdot A_{shell,n} \cdot \sin \bar{\alpha} \tag{1}$$

As retaining forces, rock mass cohesion and tensile strength as well as an optional support pressure on the tunnel face may be considered. The limit state of the failure body is computed by formulating the limit equilibrium in horizontal direction, starting with slice n most distant to the face (at $x = R$):

$$\sum F_h = -F_{seep,h} + C_h + T_{h,shell} + T_{h,slice} + H_{tr.,n} + H_{tr.,n-1} = 0 \tag{2}$$

Where C_h is the horizontal component of the resulting cohesive force at the shell, $T_{h,shell}$ and $T_{h,slice}$ are the resulting tensile forces from the shell and the slice respectively (if applicable) and $H_{tr.,n}$ and $H_{tr.,n-1}$ are the contact forces on the left and the right side of the slice. If equilibrium for slice n is not met ($H_{tr.,n} > 0$), the differential force to the limit equilibrium acts destabilizing on the adjacent slice and so on. Thus, stability of each combination of slices can be analyzed. If tension occurs between two adjacent slices (i.e. the contact force $H_{tr,n.}$ between two slices becomes negative), a tension crack is considered at this position and equilibrium is checked for the remaining section between the tension crack and the tunnel face. This allows distinction between the failure modes *plug failure* and *cracking*. If $H_{tr,n.}$ at position $x = 0$ exceeds the applied support pressure, the plug fails. The complete formulation of the equilibrium conditions for this model and empirical equations for estimating the seepage forces in both, homogeneous and heterogeneous rock mass, can be found in Lenz (2020).

It is acknowledged that this calculation model is based on several simplifications, e.g., it disregards equilibrium of vertical forces and moment equilibrium. Therefore, it shall be applied with caution for structural design checks in terms of EC7. Rather, the model is developed to provide a basis to determine the ground and system behavior, as described in the Guideline for the Design of Underground Structures with Conventional Excavation (Austrian Society for Geomechanics 2010).

5 CONCLUSION

Heterogeneous rock mass sequences with varying permeabilities and low strength are particularly prone to hydraulic failure of the tunnel face. The hydraulic heads in or close to highly permeable zones may reach up to the initial head even in utmost vicinity of the face, depending on the geotechnical conditions, yielding high seepage forces towards the face. With respect to the stress conditions in narrow fault zones close to the face, a hemispherical failure body is considered to best reflect the actual failure mode of a plug being pushed into the tunnel face by seepage forces. Stability of this plug may be assessed by the calculation procedure proposed within this article for an easy and fast assessment of ground and system behavior.

REFERENCES

Anagnostou, G. & Kovári, K. (1996). Face Stability Conditions with Earth-Pressure-Balanced Shields. *Tunnelling and Underground Space Technology*, 11 (2), 165–173.

Austrian Society for Geomechanics (2010). *Guideline for the Geotechnical Design of Underground Structures with Conventional Excavation (Translated from version 2.1 ed.).* Salzburg: Austrian Society for Geomechanics.

Biot, M. A. (1941). General theory of three-dimensional consolidation. *Journal of Applied Physics*, 12(2), 155–164.

EN 1997-1 (2014). *Eurocode 7: Geotechnical design, Part 1: General rules*. OeNORM, Austrian Standards Institute.

Itasca Consulting Group, Inc. 2017. FLAC3D: Fast Lagrangian Analysis of Continua in Three-Dimensions, Ver. 6.0. Minneapolis: Itasca.

Lee, I.-M., Nam, S.-W., & Ahn, J.-H. (2003). Effect of seepage forces on tunnel face stability. *Canadian Geotechnical Journal*, 40(2), 342–350.

Leitner, R. & Müller, H. (2007). Numerische Modellierung von Wasserzutritten für tiefliegende Tunnel in Störungszonen. *In: Massnahmen zur Beherrschung des Wassers in der Geotechnik*, volume 30, (pp. 293–309). Mitteilungsheft der Gruppe Geotechnik Graz.

Lenz, G. 2020. Ground and system behaviour in water-bearing fault zones. Doctoral thesis, Graz University of Technology, Austria.

Wudtke, R. B. (2014). Hydraulischer Grundbruch in bindigem Baugrund. Doctoral thesis, Bauhaus-Universität Weimar, Germany.

Zingg, S. & Anagnostou, G. (2012). Tunnel face stability in narrow water-bearing fault zones. *In Rock Engineering and Technology for Sustainable Underground Construction,* Stockholm, Sweden.