STATIC EFFECTS AND ASPECTS OF FEASIBILITY AND DESIGN OF DRAINAGES IN TUNNELLING

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Zurich, October 2016
Sara Zingg
Summary

This PhD thesis investigates the effectiveness of drainage measures with respect to two particularly important problems associated with tunnelling through water-bearing, weak ground: the stability of the tunnel face and the stability and deformation of grouting bodies. Water is an adverse factor with respect to the stability and deformation of underground structures due to, (i), the pore water pressure and, (ii), the seepage forces associated with seepage flow towards the tunnel. Drainage boreholes reduce the pore water pressure and the seepage forces in the vicinity of the cavity. Furthermore, loss of pore water pressure increases the effective stresses and thus the shearing resistance of the ground (‘consolidation’), which is favourable in terms the deformation occurring during and after tunnelling.

The goal of the PhD thesis is to elaborate a more detailed understanding of the interrelationships between drainage measures and the stability of the tunnel face and grouting bodies. The main objectives of the investigations relating to the tunnel face are: (i) analysis of face stability through limit equilibrium computations taking account of the numerically determined seepage flow conditions prevailing in the ground after the implementation of drainage measures; (ii) systematic investigation of tunnel face stability considering several different drainage layouts and working out design-nomograms; (iii) consideration of a series of aspects limiting pore pressure relief and thus the effectiveness of drainage measures and their impact on face stability. The main objectives of the investigations with regard to grouting bodies are: (i) a study of the stabilizing effect of the virtual case of ideal drainage on tunnel support and plastification in grouted fault zones in plane strain conditions; (ii) a comparison with the stabilizing effect of real drainage layouts, i.e. when considering pore pressure relief due to specific drainage borehole arrangements, (iii) application of the drainage measure both before and after the injection works.

The study of face stability is organised in four chapters. The first chapter investigates the effectiveness of various advance drainage schemes with respect to face stability in ground of uniform permeability. A suite of computations is carried out to quantify the effects of the geometric parameters of several different drainage schemes. The seepage forces, which are considered in the limit equilibrium computations, are determined numerically through steady-state, three-dimensional seepage flow analysis which takes account of the characteristics of a given drainage scheme. A dimensionless formulation of the required support pressure (or the required cohesion of the ground) is developed in order to produce design nomograms that can provide a quick assessment of face stability in cases involving partial pore pressure relief in advance of excavation.

Hydraulic heterogeneity due to alternating aquifers and aquitards may result in a hydraulic head field which is particularly adverse for face stability due to high gradients close to the face. In the second chapter, a suite of stability analyses are carried out in order to quantify the effects of the orientation, thickness, location, number and permeability ratio of the ground layers, paying particular attention to the effectiveness of a common advance drainage measure consisting of six axial boreholes drilled from the tunnel face. The computational results provide valuable information about whether and to what extent the required support pressure is higher or lower than in the case of uniformly permeable ground; which ground structures are critical for face stability and necessitate a higher support pressure; the extent to which advance drainage allows for a reduction in support pressure; and where the drainage boreholes have to be arranged in order to be most effective.

Several other factors may impose limits on pore pressure relief in the ground around advance drainage boreholes and thus limit their effectiveness with respect to face stability: (i) the hydraulic capacity of
the drainage boreholes hindering full pressure relief in highly permeable ground at high water table; (ii) the casings required for stabilizing the borehole, but which in turn restrict pore pressure relief to small openings; (iii) the lead-time in a poorly permeable ground, where pore pressure relief by advance drainage may take a prohibitively long time to work; (iv) environmental constraints with respect to the drawdown of the water table; (v) the magnitude of settlements, which may impose limits on the amount of admissible pore pressure relief and, (vi) the pumping capacity available on site, which may limit the quantity of water inflow. In the third chapter, the hydraulic capacity of the drainage boreholes is investigated by means of an equivalent conductivity model taking account of pipe- and open-channel flow hydraulics within the drainage borehole. The model makes it possible to determine the maximum ground permeability for which it is safe to consider the borehole wall as a seepage face. In addition, the minimum requirements for casings are elaborated based upon face stability considerations. The fourth chapter discusses the time required for lowering the hydraulic head field to practically steady state conditions and analyses the magnitudes of drawdown, settlement and water discharge caused by advance drainage boreholes drilled from the tunnel face. The computational results provide useful insights into potential risks related to advance drainage measures for face stability and indicate the limits of applicability of the design nomograms.

The stability of grouting bodies is studied for two crucial drainage measures: (i) drainage of the inner part of the grouting body to decrease the load and the risk of inner erosion due to the action of high hydraulic gradients, (ii) advance drainage of the area of future grouting bodies to increase the effective stresses and lead to consolidation of the ground prior to injection. A cylindrical tunnel is assumed to be excavated in ground considered as a porous, elasto-plastic medium obeying the principle of effective stress and Coulomb’s failure criterion and taking the seepage forces into account. For the virtual case of ideal drainage, i.e. complete pore pressure relief, an analytical solution is derived. Several specific arrangements of drainage boreholes are studied by means of hydraulic-mechanical coupled FE-modelling and the deviations from the analytical solutions are elaborated. The effect of the drainage measures is discussed by means of the characteristic line, i.e. stress as a function of the displacement at the excavation boundary of the tunnel, and the degree of plastification of the grouting body, which may serve as another dimensioning criterion for stability. The computational results provide valuable information about the static effects of number, length and spacing of drainage boreholes arranged inside- and outside the grouting body.

In summary, the contribution of this PhD thesis is the detailed investigation of the static effects of drainage measures during tunnelling in water-bearing ground with respect to the stability of the tunnel face and the grouting body as well as the supply of design aids capable of providing a quick assessment of face stability when considering a number of advance drainage schemes.
Zusammenfassung


Die Zielsetzung der Doktorarbeit ist es, ein umfassendes Verständnis der Zusammenhänge zwischen Drainagemassnahmen und der Ortsbruststabilität sowie der Stabilität von Injektionskörpern zu erarbeiten. Die Hauptziele der Untersuchung rund um die Ortsbrust sind: (i) die Beurteilung der Ortsbruststabilität mittels Grenzgleichgewichtsmodell, welches die sich infolge Drainage einstellende, numerisch ermittelte Sickerströmung berücksichtigt; (ii) die systematische Untersuchung der Ortsbruststabilität für verschiedene, praxisnahe Drainageanordnungen inklusive der Ausarbeitung von Dimensionierungs-Nomogrammen; (iii) die Berücksichtigung einer Reihe von Aspekten, die den Porenwasserdruckabbau somit die Wirksamkeit von Drainagemassnahmen limitieren. Die Hauptziele der Untersuchung rund um den Injektionskörper sind: (i) das Studium der stabilisierenden Wirkung der theoretischen, vollständigen Drainage auf Ausbauwiderstand und Plastifizierung eines Injektionskörpers im ebenen Verformungszustand; (ii) der Vergleich mit dem stabilisierenden Effekt von praxisnahen Drainageanordnungen, d.h. unter Berücksichtigung des Porenwasserdruckabbaus infolge ausgewählter Bohrschemata; jeweils, (iii), für Drainagebohrungen vor und nach dem Erstellen eines Injektionskörpers.


Die hydraulische Heterogenität infolge einer Wechsellagerung von Aquiferen und Aquitarden kann zu einer ungünstigen Druckverteilung mit grossen, zur Ortsbrust gerichteten hydraulischen Gradienten führen. Das zweite Kapitel quantifiziert am Beispiel einer gängigen Drainagemassnahme von sechs Drainagen ab Tunnelortsrust die Auswirkung von Orientierung, Dicke, Ort, Anzahl und Durchlässigkeitstoleranz der Baugrundschichten auf die Ortsbruststabilität. Die Berechnungsergebnisse liefern wertvolle Informationen darüber, ob und um wieviel der erforderliche Stützdruck von jenem in homogem Baugrund abweicht; welche Baugrundmodelle einen höheren Stützdruck erfordern und somit kritisch sind für die Ortsbruststabilität; welche Stützdruckreduktion durch...
Drainagemassnahmen erreichbar ist und wo die Drainagebohrungen angeordnet werden müssen, um möglichst ihre volle Wirkung zu entfalten.

Verschiedene, den *Porenwasserdruckabbau limitierende Faktoren* werden bezüglich ihrer Auswirkung auf die Ortsbruststabilität untersucht: (i) die hydraulische Kapazität der Drainagebohrungen kann im hochdurchlässigen Baugrund und unter hohem Wasserdruck den Porenwasserdruckabbau verunmöglichen; (ii) für die Bohrlochstabilität erforderliche Hüllrohre verringern die für den Druckabbau zur Verfügung stehende Fläche auf kleine Öffnungen; (iii) die Vorlaufzeit bis zum Erreichen des gewünschten Druckabbaus in geringdurchlässigem Baugrund kann zu lange sein; (iv) die Grundwasserspiegelabsenkung kann durch Umweltauflagen eingeschränkt sein; (v) die zulässigen Setzungen können den Betrag der Porenwasserdruckabsenkung begrenzen und; (vi), die auf der Baustelle bereitgestellte Pumpenkapazität kann die maximal abführbare Wassermenge vorgeben. Im dritten Kapitel wird ein Modell zur Erfassung der hydraulischen Kapazität der Drainagebohrungen erarbeitet, das das turbulente Druck- und Freispiegelabflussverhalten im Drainagerohr mittels eines porösen Mediums von äquivalenter Durchlässigkeit abbildet. Es erlaubt die Bestimmung der maximalen Baugrunddurchlässigkeit, für welche die Annahme von atmosphärischen Druckbedingungen entlang der Bohrlochwand noch zulässig ist. Weiter werden die aus Sicht der Ortsbruststabilität minimalen hydraulischen Anforderungen an Hüllrohre erarbeitet. Im vierten Kapitel wird schließlich die zum Erreichen einer nahezu stationären Porenwasserdruckverteilung erforderliche Vorlaufzeit quantifiziert. Im Weiteren werden die zusätzliche Grundwasserspiegelabsenkung, die Setzung der Geländeoberfläche und der Wasserzutritt infolge der Drainagen ab der Tunnelortsbrust analysiert. Die Berechnungsresultate liefern wichtige Hinweise auf die potentiellen Risiken von vorauslaufenden Drainagemassnahmen zur Erhöhung der Ortsbruststabilität und zeigen die Anwendungsgrenzen der Dimensionierungs-Nomogramme auf.


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1. Introduction

1.1 Problem statement

Tunnelling in water-bearing, low strength ground represents an engineering challenge. A key factor is water, which has a negative effect on the stability or deformation of underground openings due to: (i) the pore pressure, which reduces effective stresses and thus the shear resistance of the ground, (ii) the seepage forces associated with seepage flow towards the cavity. Thus, for example, a high pore pressure and/or a high pressure gradient may endanger the stability of the working face or favour the development of large convergences. Drainage of the ground decreases both the pore pressure and its gradient in the vicinity of the cavity. The consolidation that occurs due to the pore pressure relief is favourable with respect to the stability and deformation because it increases the mean effective stress in the ground and thus also its stiffness and resistance to shearing.

The improvement of ground behaviour by means of drainage measures is well-known from tunnelling practice. Although it is important to understand and quantify the static effects of pore pressure and drainage for rational decision-making in design and construction, relatively few investigations have been made (as shown below) into the interaction of seepage flow, pore pressure and ground response to tunnel excavation. Even fewer publications deal specifically with the effect of drainage. This was the motivation for the present PhD thesis, which investigates the stabilizing effect of drainage measures on two selected tunnelling problems: (i) the stability of the tunnel face, and (ii) the deformation of the excavation boundary of a grouting body in geological fault zones.

The thesis deals with static effects and aspects of the feasibility and design of drainage measures in tunnelling. The first key word points to the objective of elaborating a detailed understanding of the interrelationships between drainage measures and the stability of the tunnel face or the grouting body. In the first step, examination is based on the simplifying assumption of ideal drainage. In the second step, focus is placed on aspects of the feasibility and/or design, which may limit drainage effectiveness with regard to static effects. Factors limiting pore pressure relief achieved by drainage measures may include geometric restrictions concerning the arrangement of drainage boreholes (e.g. location, number, length of the boreholes), constraints due to the ground encountered on site (e.g. non-uniform permeability rendering some boreholes ineffective; high permeability, causing the boreholes to reach hydraulic capacity; unstable borehole walls requiring casings), as well as limitations due to environmental reasons (e.g. admissible groundwater drawdown or settlement) or operational reasons (e.g. lead-time or pumping capacity available on site).

1.2 State of research

1.2.1 Face stability

One of the most serious risks in tunnelling through weak ground is collapse of the tunnel face. The favourable effect of advance drainage on face stability is well known from tunnelling experience (e.g. Nilsen, 1999, 2011; Nilsen and Palmström, 2001, Wang et al., 2004; Hong et al., 2010; Fulcher et al., 2008; Sellner et al., 2008). Although there are many publications addressing face stability in water-bearing ground (e.g. Pellet et al. 1993; Anagnostou and Kovári 1996; de Buhan et al. 1999, Broere 2001; Vermeer et al. 2002; Droniuc et al. 2004; Lee and Nam 2001; Fellner and Gärber 2004; Höfle and Filibeck 2007; Ströhle and Vermeer 2009; Lu et al., 2014; Perazzelli et al. 2014), few research
works deal specifically with the effect of advance drainage measures on face stability, and those that do focus mainly on ground of uniform permeability (Lee et al., 2003, 2004; Anagnostou et al., 2010).

Tunnelling in weak water-bearing ground is demanding particularly under high water pressures and in heterogeneous formations exhibiting variable permeability. The hydraulic heterogeneity may result in locally high hydraulic gradients or impair effectiveness of advance drainage. The literature on face stability in water-bearing ground of non-uniform permeability deals mainly with tunnelling through fault zones (e.g. Brekke and Selmer-Olsen, 1965; Chang et al., 2005; Anagnostou, 2010); specific case histories (e.g. Könz, 1968; Barla, 2000; Bezrodny et al., 1992; Dahlø and Nilsen, 2002; Nilsen, 1999; 2001; 2011); or specific aspects such as the effect of pre-support (pipe roof) and grouting in fractured zones (Shi et al., 2015) or the mechanism of punching failure (Hong et al., 2010). As shown later in Chapters 4 and 5, the literature on situations where multiple factors are conspiring to limit the effectiveness of drainage measures is sparse and does not refer to face stability.

1.2.2 Grouting body

Water-bearing fault zones consisting of crushed rock or soil-like material with little or no cohesion represent a major challenge for the design and construction of deep tunnels. Sudden water and mud inflows as well as high ground and water pressures can have a disastrous impact on tunnelling operation and safety (see e.g. Theiler et al., 2013 for a historic example). Therefore, the timely application of special measures (or combinations thereof) such as grouting, systematic drainage or even artificial ground-freezing is required (Chang et al., 2005; Sturk and Stille, 2008).

Grouting bodies usually have an approximately cylindrical shape and a hollow cylinder remains after excavating the tunnel. The grouted body has increased strength and stiffness and, due to the filling of the pores by grout, lower permeability than untreated ground. The induced hydraulic heterogeneity of the ground may result in locally high hydraulic gradients and pore pressures. Both in turn may endanger stability and favour the development of large convergences: the pore pressure reduces effective stresses and thus the shear resistance of the ground; the gradients may overstress the grouted body or cause its inner erosion. Drainage is an effective measure for preventing these water-related dangers in ground behaviour.

Previous investigations considered two borderline cases of grouting body drainage: a perfectly sealing (i.e. fully impermeable) and an ideally drained (i.e. fully permeable) grouting body. The stability of grouting bodies for these borderline cases was extensively investigated e.g. by Ponimatkin (1972), Kovári (1992), Adachi et al. (2005), Egger et al. (1982), Egger (1988), Fukuchi (1989), Soriano et al. (1989), Anagnostou and Kovári (2003, 2005) or Bilfinger (2005). Some case histories are reported using grouting bodies in combination with drainage measures (Indraratna and Chu, 2005; Fulcher et al., 2008) and Nasberg and Ilyushin (1973) derived analytical solutions for the seepage analysis when considering several different drainage borehole arrangements, but there is a lack of research with respect to the effect of specific drainage borehole arrangements on the stability of grouting bodies.
1.3 Research objectives

The goal of the PhD thesis is to elaborate a more detailed understanding of the interrelationships between drainage measures and stability of the tunnel face and grouting bodies.

The main objectives relating to the tunnel face are:

- Analysis of face stability through limit equilibrium computations taking account of the numerically-determined seepage flow conditions prevailing in the ground after the implementation of drainage measures;
- Systematic investigation of tunnel face stability considering several different drainage layouts and working out design-nomograms;
- Consideration of a series of aspects limiting pore pressure relief and thus the effectiveness of drainage measures and their impact on face stability.

The main objectives of the investigations with regard to grouting bodies are:

- Study of the stabilizing effect of the virtual case of ideal drainage on tunnel lining support and plastification in grouted fault zones in plane strain conditions;
- Comparison with the stabilizing effect of real drainage layouts, i.e. when considering pore pressure relief due to specific drainage borehole arrangements, for application of drainage measures both before and after the injection works.

1.3.1 Methods

The PhD thesis is based on qualitative research by means of FE-modelling and analytical solutions.

The stability of the tunnel face is analysed after Anagnostou and Kovári (1996) by considering the limit equilibrium for a failure mechanism consisting of a wedge ahead of the tunnel face and a prism extending up to the surface. The computational model takes account of the mechanical action of the groundwater, (i) by analysing the limit equilibrium in terms of effective stress, and (ii) by introducing the seepage forces that act on the wedge and the prism into the equilibrium equations. The seepage force at any point of the sliding bodies is equal to the gradient of the pore pressure field. The latter is determined by means of three-dimensional, numerical seepage flow analyses assuming Darcy’s law when taking account of a specific advance drainage layout (performed with the finite element code COMSOL®).

The stability and deformation of the grouting body is analysed after Anagnostou and Kovári (2003) by considering a cylindrical tunnel, surrounded by a grouting body in the form of a thick-walled cylinder. Both treated and untreated ground is considered as a porous, elasto-plastic medium obeying the principle of effective stress, Coulomb’s failure criterion and taking account of the seepage forces according to Darcy’s law. Analytical solutions are derived for the virtual case of ideal drainage in a system fulfilling the condition of rotational symmetry. The effects of several different drainage borehole arrangements are studied by means of hydraulic-mechanical coupled FE-modelling (performed with the finite element code COMSOL®).

1.3.2 Limitations of scope

The thesis focuses on numerical modelling of the static effects of drainage measures in tunnelling. Research intended to develop or improve the application-oriented, technological implementation of drainage boreholes was not part of this study. No operational handling is therefore addressed in terms of drainage boreholes in tunnelling practice (such as drilling, insertion or efficiency control of drains).
The applicability of the results is discussed by means of analytical considerations and application examples of tunnels. No work was undertaken in field or laboratory testing.

The study considers the ground as a porous medium obeying Darcy’s law. In fractured rock, preferential seepage flow along the rock joints is to be expected. The stabilizing effect of drainage boreholes that consider such flow patterns is not part of the thesis.

1.4 Structure of the thesis

The PhD thesis is organized in six Chapters. After the introduction, Chapters 2-5 focus on face stability and Chapter 6 deals with the stability and deformation of the grouting body.

1.4.1 Face stability

Advance drainage improves face stability in water-bearing ground by reducing the pore water pressure and hydraulic head gradient in the vicinity of the tunnel face. However, several simplifying or common assumptions result in overestimation of the pore pressure relief (Table 1.1).

<table>
<thead>
<tr>
<th>Simplifying or common assumptions:</th>
<th>But:</th>
</tr>
</thead>
<tbody>
<tr>
<td>The borehole walls represent seepage faces under atmospheric pressure.</td>
<td>(i) High permeability and the water table may result in pressure development inside the boreholes, thus resulting in reduced pore pressure relief in the surrounding ground.</td>
</tr>
<tr>
<td>Sufficient time is available for pore pressure relief.</td>
<td>(ii) If the boreholes have casings, only their openings will represent seepage faces and consequently the pore pressure relief will be limited.</td>
</tr>
<tr>
<td>Sufficient number and/or length of boreholes to achieve the desired pore pressure relief.</td>
<td>(iii) The time available may be limited, with the consequence that pore pressures ahead of the face will be higher than at steady state conditions.</td>
</tr>
<tr>
<td></td>
<td>(iv) Geometric constraints: The equipment may impose constraints on the number and/or locations of boreholes, thus resulting in reduced pore pressure relief.</td>
</tr>
<tr>
<td></td>
<td>(v) Heterogeneous ground: Boreholes within aquitards are less effective. Heterogeneous ground may necessitate a large number of boreholes; otherwise the pore pressure relief will be less than in homogeneous ground.</td>
</tr>
<tr>
<td></td>
<td>(vi) Drawdown of water table: If the water table experiences an inadmissible drawdown, then the number and/or length of boreholes may have to be limited, which in turn results in reduced pore pressure relief.</td>
</tr>
<tr>
<td></td>
<td>(vii) Settlements: If the consolidation-induced settlements are inadmissible (independently of whether the water table experiences a drawdown or not), then the number and/or length of the boreholes may have to be limited, which in turn results in reduced pore pressure relief.</td>
</tr>
<tr>
<td></td>
<td>(viii) Water discharge: If the amount of water inflow is too large to be handled by the pumping system, the number and/or length of boreholes may have to be limited, which in turn results in reduced pore pressure relief.</td>
</tr>
</tbody>
</table>

These factors are discussed individually in the present PhD thesis. Chapter 2 starts with face stability analysis in ground of uniform permeability and investigates the effectiveness of various advance drainage schemes (item iv in Table 1.1). The effects of the geometric parameters for each drainage scheme are discussed assuming the borehole walls as seepage faces under atmospheric pressure. A dimensionless formulation of the required support pressure (or the required cohesion of the ground) is
developed in order to produce design nomograms capable of providing a quick assessment of face stability for a series of advance drainage layouts in tunnelling.

Hydraulically heterogeneous formations (item $v$ in Table 1.1) may originate from tectonic or formation history, as the latter may cause variations in the degree of fracturation or in the lithological composition and grain size distribution of the ground (for an overview see, for example, Anagnostou et al., 2014). Hydraulic heterogeneity may occur at different scales. Aquifers and aquitards may have a thickness ranging from decimetres to decametres and be oriented vertically, horizontally or with an arbitrary inclination to the tunnel axis. Frequently alternating sub-vertical zones of variable strength and permeability, for instance, result in great variability in the geotechnical behaviour of the ground during tunnelling, thus rendering the timely application of adequate auxiliary measures difficult. Single weak zones consisting of crushed rock or soil-like material of low cohesion, if encountered suddenly, may result in large-scale instability and subsequent inundation of a long portion of the tunnel. All alternating aquifers and aquitards may lead to hydraulic head distributions that are particularly challenging for face stability. Chapter 3 investigates the effects of the orientation, thickness, location, number and permeability ratio of ground layers with regard to the effectiveness of a common advance drainage measure consisting of six axial boreholes drilled from the tunnel face. Useful guide values are indicated as to potentially critical situations, the effectiveness of advance drainage and the adequate arrangement of drainage boreholes.

The combination of very high hydraulic gradients and highly permeable ground (e.g. in a subaqueous tunnel) may result in quantities of water inflow so high that advance drainage becomes ineffective with respect to pore pressure relief (item $i$ in Table 1.1). The flow regime within the borehole changes from open-channel to pipe flow and the water pressure developing within the boreholes may result in reduced pore pressure relief in the surrounding ground. In Chapter 4, an equivalent permeability$^1$ model is developed which accounts for the hydraulic characteristics of the drainage boreholes (roughness, diameter etc.). The model allows a numerical determination to be made of the hydraulic head field, considering the hydraulic capacity of the drainage boreholes and the interaction between seepage flow in the ground and turbulent pipe flow in the boreholes. The computational results provide an insight into factors influencing face stability under high inflow conditions and allow a determination to be made of the range of conditions (water level, ground permeability) for which advance drainage measures are fully effective with respect to pore pressure relief and the nomograms provided in Chapter 2 are thus applicable here. In addition, Chapter 4 investigates the limited capacity of drainage boreholes in cases where casings become necessary because the borehole walls are unstable (item $ii$ in Table 1.1). The screen of the casings impedes pore pressure relief by restricting the passage of water to small openings. The investigations indicate the minimum requirements for casings in order to account for pore pressure relief comparable to the relief when assuming the borehole wall is a seepage face.

Chapter 5 completes the study on the remaining factors of Table 1.1 and deals with operational and environmental limits on pore pressure relief and their impact on face stability. Focus is placed on a common advance drainage scheme consisting of axial boreholes from the tunnel face:

- In ground of low permeability, pore pressure relief by advance drainage boreholes may take a long time to occur (item $iii$ in Table 1.1). The lead-time required for the hydraulic head to fall far enough for the face stability considerations according to Chapter 2 is analysed.

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$^1$ Strictly speaking: “hydraulic conductivity”. Note that within this Thesis, the term “permeability” is used interchangeable with “hydraulic conductivity”.

In order to avoid disturbance to the hydrogeological conditions, it may be necessary to limit groundwater drawdown (item vi in Table 1.1). The additional drawdown due to drainage measures is studied and estimates of groundwater drawdown are provided for specific drainage arrangements.

Drainage increases the pore pressure relief and may thus lead to inadmissible settlements of the ground surface (item vii in Table 1.1). Potential consolidation settlement is estimated assuming linear-elastic ground behaviour. The investigations indicate the settlement due to the additional pressure reduction induced by drainage measures.

In the case of high water inflow (item viii in Table 1.1), the pumping system installed on site may become a limiting factor. The amount of water discharge from the tunnel face and from the boreholes is quantified for variable ground permeability considering the borehole walls as seepage faces.

1.4.2 Grouting body

The completion of tunnel sections in water-bearing fault zones is often possible only after strengthening and sealing the ground around the opening by grouting, which is carried out ahead of the tunnel excavation. Chapter 6 of the present PhD thesis picks up the investigations of Anagnostou and Kovári (2003), but with point to three crucial aspects of drainage measures:

- The effect of local drainage of the inner part of a grouting body to decrease the load and the risk of inner erosion due to the action of high hydraulic gradients;
- Advance drainage of the entire area of the future grouting body to increase the effective stresses and lead to consolidation of the ground prior to injection;
- Consideration of both the virtual case of ideal drainage (i.e. complete pore pressure relief) and the effects of several different borehole arrangements (i.e. considering the geometric constraints on the number, length and/or location of the boreholes).

The chapter starts with an analytical solution for the virtual case of ideal drainage. The partial pore pressure relief resulting from several different arrangements of drainage boreholes is then studied using hydraulic-mechanical coupled FE-modelling. The effect of the drainage measures is discussed by means of the characteristic line, i.e. stress as a function of the displacement at the excavation boundary of the tunnel. Another dimensioning criterion of for grouting body stability may be the extent of the plastic zone developing due to overstressing of the grouting body. Therefore, the degree of plastification is evaluated as a function of the lining support pressure.

The computational results provide valuable information about the number, length and spacing of drainage boreholes arranged inside- and outside of a grouting body, and quantify the derivation of the specific drainage measures compared to the ideal drainage case.

1.5 Publications

Parts of this thesis have been published or presented in conferences (Appendix A).

The thesis also includes post-processed data gained during numerical analyses carried out within the framework of several Master’s theses evolving under the lead and with close support of the author. The contributions are listed at the end of Appendix A.
2. An investigation into efficient drainage layouts for the stabilisation of tunnel faces in homogeneous ground

2.1 Introduction

Drainage measures comprise horizontal or inclined boreholes that are drilled either directly from the face (Fig. 2.1a) or from lateral niches or enlarged cross-sections (Fig. 2.1b). Deep below the water table, the equipment has to be protected against high water pressure by means of so-called "preventers". The installation of advance drainage boreholes from the tunnel face interferes with tunnel excavation and support installation. In addition, technical equipment and procedures limit the length of the drainage boreholes and thus the time available for pore pressure relief, which is a critical factor in low permeability grounds. Advance drainage from niches (Fig. 2.1b) in combination with directional drilling (allowing for longer boreholes) remedies these problems. Another option is to employ a pre-existing underground opening, such as a pilot tunnel inside or outside the cross-section of the main tunnel (Fig. 2.1c and d) or – in the case of twin tunnels – the tube constructed first (Fig. 2.1e). The drainage action of a pre-existing underground opening can be enhanced by drilling sufficiently long radial boreholes, i.e. extending beyond the axis of the main tunnel (Fig. 2.1d). Such drainage curtains are of course essential for pore pressure relief where the lining of the pre-existing opening is watertight (e.g. a TBM-driven safety gallery with a sealed segmental lining).

This chapter extends the face stability model of Anagnostou and Kovári (1996) to include pore pressure relief due to advance drainage. It analyses and presents design nomograms for the most common drainage layouts (Fig. 2.1). The chapter considers ground of uniform permeability sufficiently high for the necessary drainage time not to be a limiting factor (cf. Table 1.1). This condition is fulfilled where ground permeability is higher than about $10^{-8}$ m/s (Anagnostou and Kovári, 1996, Anagnostou et al. 2010). In addition, uncased drainage boreholes of sufficient hydraulic capacity are considered.

![Figure 2.1.](image)

**Figure 2.1.** Drainage via: (a) boreholes from the tunnel face; (b) boreholes from a niche or from a locally enlarged cross-section; (c) a co-axial pilot tunnel; (d) boreholes from an external pilot tunnel; (e) the first tube of a twin tunnel
After outlining the computational model (Section 2.2), basic aspects of the different drainage layouts in Figure 2.1 are discussed in Section 2.3 through comparative analyses of a cylindrical tunnel (Fig. 2.2). Section 2.4 presents dimensionless design nomograms allowing a quick estimate to be made of the necessary face support pressure in the presence of drainage measures. The practical applicability of the nomograms is illustrated in Section 2.5, making reference to two tunnelling projects.

### 2.2 Computational model

Face stability is analysed after Anagnostou and Kovári (1996), which considered a failure mechanism consisting of a wedge and a prism (Fig. 2.2), determined the seepage forces by means of numerical, steady state seepage flow analyses assuming Darcy’s law, and introduced these into the equilibrium equations. There are only two differences to the model of Anagnostou and Kovári (1996): (i) The latter takes the ratio of horizontal to vertical stresses which governs the frictional part of the shear resistance at the vertical slip planes of the failure mechanism to \( \frac{\lambda_p}{p} = 0.8 \) for the prism and \( \frac{\lambda_w}{w} = 0.4 \) for the wedge. Here, slightly higher values are considered (1.0 and 0.5, respectively), based on recent results in Anagnostou (2012). (ii) For the determination of the seepage forces, Anagnostou and Kovári (1996) considered the hydraulic head field that prevails when drainage occurs only through the tunnel face. Here, the effect of advance drainage measures (Fig. 2.1) on the hydraulic head field is taken into account.

![Figure 2.2. Failure mechanism (after Anagnostou and Kovári, 1996): (a) cross-section, (b) longitudinal section and (c) axonometric projection](image.png)

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2 The shear resistance of the vertical slip surfaces depends essentially on the horizontal stresses, which cannot be determined from the equilibrium equations. The assumed value of \( \lambda_w = 1 \) for the prism is according to Janssen's classic silo-theory (Janssen, 1895) and was also proposed by Terzaghi and Jelinek (1954) on the basis of trap-door tests. The assumption of a half-as-high coefficient \( \lambda_w \) for the wedge is based upon comparative analyses with the method of slices (see end of section 2 in Anagnostou and Kovári, 1994). A detailed investigation of this issue as well as comparisons with other methods and experimental results, which show the adequacy of \( \lambda_p = 1 \) and \( \lambda_w = 0.5 \), can be found in Anagnostou (2012) as well as in section 4 of Perazzelli and Anagnostou (2011).
2.2.1 Seepage flow analyses

The seepage flow domain extends either up to the ground surface $H$ (subaqueous tunnels) or up to the groundwater table $H_w$. The upper boundary of the numerical model is thus located at distance $T = \min(H, H_w)$ above the tunnel crown. The hydraulic head at the far-field boundaries is taken equal to the initial hydraulic head $h_0$, which is equal to the elevation of the water table above the tunnel axis. The water table is assumed to remain constant in spite of the drainage action of the tunnel. This assumption is true in the case of sufficient groundwater recharge from the surface and conservative in the case of a drawdown.

The tunnel face and the walls of the drainage boreholes are taken as seepage faces under atmospheric pressure, while the tunnel boundary is considered impervious up to the face (no-flow boundary condition, Fig. 2.3). Investigations by Wongsaroj (2005) indicated that seepage flow through the lining can be neglected if the lining permeability is lower than $0.1 K_g d_{lin}/T$, where $K_g$ and $d_{lin}$ denote the ground permeability and the lining thickness, respectively (cf. also Mair, 2008). Even if for a low permeability ground ($K_g = 10^{-7}$ m/s), a large cover ($T = 250$ m) and a thin lining ($d_{lin} = 0.2$ m), the threshold lining permeability amounts to about $10^{-11}$ m/s. Well applied shotcrete exhibits a lower permeability ($10^{-12}$ m/s, Franzen and Celestino, 2002) and can, therefore, be considered as practically impermeable. Low-quality shotcrete may exhibit a higher permeability (in the order of $10^{-10}$ m/s; Celestino et al., 2001) and allow for some additional drainage and pore pressure relief. This is particularly true for a perforated shotcrete lining or an open shield. In such cases, the no-flow boundary condition represents a simplification on the safe side.

![Figure 2.3. Example of spatial discretisation and hydraulic boundary conditions for advance drainage according to Figure 2.1a](image)

2.2.2 Support pressure

As it will be shown below, the support pressure that is needed in order to stabilize a specific wedge (characterized by the angle $\omega$; Fig. 2.2) for given drainage measures can be expressed as follows:

$$s = N_{cga} \gamma' \frac{D}{D} - N_{qa} c + N_{ha} \gamma_w h_0,$$

where $c$, $h_0$, $\gamma'$ and $\gamma_w$ denote the ground cohesion, the depth of the tunnel axis underneath the water table (Fig. 2.2), the submerged unit weight of the ground and the unit weight of the water, respectively. $N_{cga}$, $N_{qa}$ and $N_{ha}$ are dimensionless coefficients, which depend on the numerically computed hydraulic head field (and thus on the drainage layout), the friction angle $\phi$, the normalized cohesion $c/\gamma' D$, the ratio of dry unit weight $\gamma_d$ to $\gamma'$, the normalized overburden $H/D$, the normalized in
situ head $\gamma_n h_0/\gamma'D$ and the angle $\omega$ between the tunnel face and the inclined slip surface of the wedge (hereafter referred to as "wedge angle").

The critical wedge angle $\omega_{cr}$, i.e. the angle that results in the maximum support pressure $s$, is determined iteratively by repeating the computation for different values of $\omega$.

### 2.2.2.1 Mechanism

For the derivation of Eq. (2-1), a mechanism is considered that approximates the circular tunnel face by a square having the same centre point and an equal area, i.e. its side length $D' = \chi D$, where $\chi = 0.5\pi^{0.5} = 0.89$ (Fig. 2.2). Overscores denote normalized variables. Cohesion and all pressures are normalized by $\gamma'D$, where $\gamma'$ denotes the submerged unit weight of the ground. Lengths are normalized by the tunnel diameter $D$, with the exception of $h_0 (= 0.5D + H_w)$, which is normalized as follows: $\overline{h_0} = \gamma_w (0.5 + H_w)/\gamma'$.

### 2.2.2.2 Limit equilibrium

The required support force $S_{\omega}$ for a given angle $\omega$ is obtained by considering the limit equilibrium of the wedge. The latter is acted upon by the following forces: its submerged weight $G' (= 0.5\gamma'\chi D^3 \tan \omega)$; the vertical load $V'$ of the prism ($= \chi D^2 \sigma' \tan \omega$, where $\sigma'_v$ denotes the effective silo pressure); the resultant seepage forces $W_1$ and $W_3$; the unknown support force $S_{\omega}$; the unknown effective normal force $N'$ on the inclined slip plane; the shear resistance $T_v$ of the two lateral vertical slip planes of the wedge, which according to Anagnostou and Kovári (1994) reads as follows for $\lambda_w = 0.5$:

$$T_v = (\chi D)^2 \tan \omega \left( c + \tan \phi \left( \chi D\gamma' / 6 + \sigma'_v / 3 \right) \right) ; \quad (2-2)$$

and the shear resistance $T_i$ of the inclined slip plane of the wedge according to the Mohr-Coulomb failure criterion:

$$T_i = (\chi D)^2 c / \cos \omega + N' \tan \phi . \quad (2-3)$$

The equilibrium conditions normal and parallel to the sliding direction are:

$$N' = (G' + V' - W_3) \sin \omega + (S_{\omega} + W_i) \cos \omega , \quad (2-4)$$

$$T_i + T_v = (G' + V' - W_3) \cos \omega - (S_{\omega} + W_i) \sin \omega . \quad (2-5)$$

Eliminating $T_i$ and $N'$ from the last three equations leads to the following expression for the required support force:

$$S_{\omega} = -W_i + \frac{-W_3 + G' + V'}{\tan (\phi + \omega)} = \frac{T_v + (\chi D)^2 c / \cos \omega}{(\tan \phi + \tan \omega) \cos \omega} . \quad (2-6)$$

### 2.2.2.3 Seepage forces

By making use of Gauss' theorem, the seepage forces $W_k$ acting upon the wedge can be calculated by means of surface integrals:
\[ W_k = -\int \gamma_w \frac{\partial h}{\partial x_k} dV = -\gamma_w \int n_k h dS, \quad (2-7) \]

where \( h \) denotes the hydraulic head field; \( n_k \) is the unit normal vector of the wedge surface; and \( \gamma_w \) is the unit weight of the water. Taking into account that \( n_1 = 0 \) over the boundaries KON, LMP and MNOP (Fig. 2.2c), \( n_1 = -1 \) over KNML, \( n_1 = \cos \omega \) over KLPO, \( n_3 = 0 \) over KON, LMP and KNML, \( n_3 = 1 \) over NOPM and \( n_3 = -\sin \omega \) over KLPO, the resultant seepage forces in the axial and vertical directions (\( W_1 \) and \( W_3 \), respectively) read as follows (\( W_2 = 0 \) due to symmetry):

\[ W_1 = -\gamma_w h_0 D^2 \cos \omega I_1, \quad (2-8) \]

\[ W_3 = -\gamma_w h_0 D^3 (I_3 - I_1 \sin \omega), \quad (2-9) \]

where \( h_0 \) is the depth of the tunnel axis underneath the water table and \( I_k \) represents dimensionless coefficients:

\[ I_1 = \int_{-0.5 \chi / \cos \omega}^{0.5 \chi / \cos \omega} \int_{-0.5 \chi / \cos \omega}^{0.5 \chi / \cos \omega} \frac{h}{h_0} d\xi d\eta, \quad (2-10) \]

\[ I_3 = \int_{0}^{\chi / \tan \omega} \int_{-0.5 \chi / \cos \omega}^{0.5 \chi / \cos \omega} \frac{h}{h_0} d\xi d\eta, \quad (2-11) \]

where the normalized coordinates \( \zeta_k = x_k / D \).

### 2.2.2.4 Silo pressure

The effective vertical pressure \( \sigma'_v \) exerted by the prism upon the wedge (Fig. 2.2) can be expressed as follows (Anagnostou and Kovári, 1996):

\[ \sigma'_v = \sigma'_{wv} + \sigma'_{vd} + \sigma'_{vs}, \quad (2-12) \]

where \( \sigma'_{wv} \) is the contribution of the ground underneath the water table in the case of hydraulic equilibrium; \( \sigma'_{vd} \) represents the contribution of the ground above the water table; and \( \sigma'_{vs} \) is the contribution of the seepage flow.

The first r.h.s. term of Eq. (2-12) is obtained by applying silo-theory to the part of the prism underneath the water table. For \( \lambda_p = 1 \),

\[ \bar{\sigma}'_{w} = \frac{\bar{\sigma} - \bar{\sigma}}{\tan \varphi} (1 - e_s) , \quad (2-13) \]

where

\[ e_s = e^{-\frac{\tan \varphi}{\pi} \left[ \bar{\sigma} + 0.5(1 - \chi) \right]} ; \quad (2-14) \]

\( r_c \) is the ratio of the area to the circumference of the horizontal cross-section of the prism,
\[ r_e = 0.5 \chi D \tan \omega (1 + \tan \omega)^{-1}; \quad (2-15) \]

and \( T \) denotes the height of the prism part underneath the water table:

\[ T = \min (H, H_u); \quad (2-16) \]

The second r.h.s. term of Eq. (2-12) takes into account that the ground above the water table exerts the load

\[ \sigma'_{vr} = \frac{\overline{\gamma_d} - \overline{c}}{\tan \varphi} (1 - e) , \quad (2-17) \]

on the underlying prism, where \( \overline{\gamma_d} = \gamma_d / \gamma' \) is the normalized dry unit weight of the ground and

\[ e_d = e \frac{\tan c}{\nu - \nu_{wu}} \quad (2-18) \]

This load increases the silo pressure at the top of the wedge by

\[ \sigma'_{vd} = \sigma'_{vr} e_s . \quad (2-19) \]

This term does not apply if the tunnel is subaqueous \((H_u \geq H)\) or

\[ \overline{c} > \overline{c}_{lim, d} = \overline{\gamma_d} . \quad (2-20) \]

In the latter case, Eq. (2-17) yields a negative pressure, which means that the prismatic body above the water table is stable without support by the underlying prism. Consequently, the following may be written:

\[ \sigma'_{vd} = \delta \frac{\overline{\gamma_d} - \overline{c}}{\tan \varphi} (1 - e) e_s , \quad (2-21) \]

where

\[ \delta = \begin{cases} 1, & \text{if } \overline{c} < \overline{c}_{lim, d} \land H_u < H; \\ 0, & \text{if } \overline{c} \geq \overline{c}_{lim, d} \lor H_u \geq H. \end{cases} \quad (2-22) \]

The contribution of the seepage flow (third r.h.s. term of Eq. (2-12)) is determined based upon equation 19 of Anagnostou and Kovári (1996):

\[ \sigma'_{va} = \gamma_u \left( h_e e_s - h_{av} (0.5 \chi D) + \frac{\tan \varphi}{r_e} 0.5 D \int h_{av} (x_3) e^{-\frac{\tan c \left( \frac{\nu_{wu}}{\nu} - 0.5 \chi \right)}{\chi} \ dx_3} \right), \quad (2-23) \]

where \( h_{av}(x_3) \) is the average piezometric head over the horizontal cross-section of the prism at elevation \( x_3 \):

\[ h_{av} (x_3) = \frac{1}{\chi^2 D^2 \tan \omega} \int_0^{0.5 \chi D} \int_{-0.5 \chi D} h \ dx_2 \ dx_1. \quad (2-24) \]
Introducing Eq. (2-24) into (2-23) we obtain:

\[
\overline{\sigma}_{\text{sn}} = \frac{\gamma_s h_0}{\gamma' D} I_s \overline{h}_0 ,
\]

where the dimensionless coefficient is

\[
I_s = e_s - \frac{1}{\chi^2 \tan \omega} \int_0^{0.5 \chi} \left| \int_{-0.5 \chi}^{0.5 \chi} \frac{h}{h_0^{1/2} \tan \omega} \frac{d \xi_z d \xi_1}{s} + \right. \\
\tan \varphi \frac{1}{\tau_c} \int_0^{0.5 \chi} \left| \int_{-0.5 \chi}^{0.5 \chi} \frac{h}{h_0^{1/2} \tan \omega} \frac{d \xi_z d \xi_1}{s} \right| d \xi_z d \xi_1 .
\]

(2-26)

It can readily be verified that if the cohesion is sufficiently high, then Eq. (2-12) yields a negative silo pressure, which means that the prism would be stable without support by the underlying wedge. In this case, the silo pressure should be taken equal to zero. The limit cohesion is obtained from Eq. (2-12) (with \(\sigma' = 0\) and the r.h.s. terms after Eqs. (2-13), (2-21) and (2-25)) with respect to \(c\):

\[
\overline{\sigma}_{\text{lim,s}} = \frac{(1 - e_s + \delta_d \overline{\tau}_d (1 - e_d) e_s) \overline{\tau}_c + \tan \varphi \overline{h}_0 I_s}{1 - e_s + \delta_d (1 - e_d) e_s}. \]

(2-27)

In conclusion, based upon Eqs. (2-12), (2-13), (2-21) and (2-25), the silo pressure is

\[
\overline{\sigma}_s = \delta \left( \frac{\overline{\tau} (1 - e_s + \delta_d \overline{\tau}_d (1 - e_d) e_s)}{\tan \varphi} \cdot \frac{1 - e_s + \delta_d (1 - e_d) e_s}{\tan \varphi} + I_s \overline{h}_0 \right),
\]

(2-28)

where

\[
\delta = \begin{cases} 
1, & \text{if } \overline{\sigma} < \overline{\sigma}_{\text{lim,s}} \\
0, & \text{if } \overline{\sigma} \geq \overline{\sigma}_{\text{lim,s}}.
\end{cases}
\]

(2-29)

2.2.2.5 Support pressure

Introducing \(T_v, W_k, \sigma'\) (Eqs. (2-2), (2-8), (2-9) and (2-28)) into Eq. (2-6) results in the following expression for the normalized support pressure:

\[
\overline{K}_s = N_{\text{fso}} - N_{\text{cso}} \overline{\sigma} + N_{\text{hso}} \overline{h}_0 \cdot \overline{h}_0 , \]

(2-30)

where \(N_{\text{fso}}, N_{\text{cso}}\) and \(N_{\text{hso}}\) are dimensionless coefficients:

\[
N_{\text{fso}} = A \left( \frac{X}{2} + \delta \overline{\tau}_c \frac{1 - e_s + \delta_d (1 - e_d) e_s \overline{\tau}_d}{\tan \varphi} \right) ,
\]

(2-31)

\[
N_{\text{cso}} = \frac{\cos \varphi (1 + \sin \omega)}{\cos \omega \sin (\varphi + \omega)} + \delta \frac{1 - e_s + \delta_d (1 - e_d) e_s}{\tan \varphi} ,
\]

(2-32)
\[ N_{ho} = \frac{\sin \varphi}{\chi^2 \sin(\varphi + \omega)} I_1 + \frac{1}{\chi^2 \tan(\varphi + \omega)} I_3 + \delta_s A I_s, \]  
(2-33)

with

\[ A = \frac{\tan \omega}{\tan(\varphi + \omega)} \left(1 - \frac{\sin \varphi}{3 \cos(\varphi + \omega)}\right) \]  
(2-34)

and \( r_c, e_c, e_d, \delta_n, \delta_d, I_1, I_3 \) and \( I_s \) according to Eqs. (2-15), (2-14), (2-18), (2-29), (2-22), (2-10), (2-11) and (2-26), respectively. The hydraulic head field, which is needed for the computation of the last three coefficients, is determined by a numerical seepage flow analysis.

### 2.2.2.6 Dimensional analysis of the hydraulic head field

Consider, for the sake of simplicity, a tunnel with just one drainage borehole located at the tunnel axis (Fig. 2.4). According to Darcy’s law, the steady state hydraulic head at every point in the ground around the tunnel and ahead of the face depends in general on the location of the point considered (described by its coordinates \( x_k \)) as well as on all of the problem parameters, i.e. on the permeability \( K_g \) of the ground, the tunnel diameter \( D \), the distance \( T \) between tunnel crown and upper boundary of the seepage flow domain, the borehole length \( l_{dr} \), the borehole diameter \( d_{dr} \) and the height \( h_0 \) of the water table above the tunnel axis:

\[ h = f \left( x_k, D, T, h_0, K_g, l_{dr}, d_{dr} \right). \]  
(2-35)

Dimensional analysis shows that permeability does not play a role\(^3\), while the geometric parameters can be reduced by one, and consequently the hydraulic head field \( h \) can be expressed as follows:

\[ \frac{h}{D} = f \left( \frac{x_k}{D}, \frac{T}{D}, \frac{h_0}{D}, \frac{l_{dr}}{D}, \frac{d_{dr}}{D} \right). \]  
(2-36)

Therefore, the numerically determined hydraulic head field \( h(x_k) \) depends on geometrical parameters of the drainage measure (\( l_{dr}/D, d_{dr}/D \) of the drainage arrangement) and the location of the point considered (\( x_k/D \)) and the boundary conditions of the problem (\( h_0/D, T/D \)).

![Figure 2.4. Problem setup for the dimensional analysis of the hydraulic head field](image)

\(^3\) Combining the continuity equation when assuming no mass source with Darcy’s law at steady-state conditions shows that the hydraulic head field in a ground of uniform permeability is independent of the value of permeability.
2.3 Comparative analyses

2.3.1 Introduction

We consider the example of a 100 m deep subaqueous cylindrical tunnel (Fig. 2.5). The assumed parameters are given in Table 2.1. The effectiveness of each drainage scheme (Fig. 2.1) is evaluated in terms of the face support pressure \( s \) that is needed for stability. For the purposes of comparison we first consider the following two borderline cases (Section 2.3.2): (i) no drainage measures, \( i.e. \) pore pressure relief only due to the natural drainage action of the open tunnel face; (ii) ideal drainage, \( i.e. \) complete pore pressure relief in the ground ahead of the tunnel face. These two borderline cases bound the range of face support pressures that would be needed in combination with non-ideal, real world advance drainage. They thus serve as reference cases for an evaluation of the effectiveness of the various drainage layouts in Figure 2.1.

Subsequently (in Section 2.3.3), we show the effect of the number, length and location of drainage boreholes drilled either directly from the tunnel face (Fig. 2.1a) or from lateral niches or enlarged cross-sections (Fig. 2.1b). Sections 2.3.4 and 2.3.5 deal with the drainage action of a pilot tunnel (located inside or outside the cross-section of the main tunnel) or of the first tube of a twin tunnel, respectively. Finally, Section 2.3.6 investigates drainage curtains (Fig. 2.1d).

<table>
<thead>
<tr>
<th>Table 2.1. Parameters for the comparative analyses</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Problem layout</strong></td>
</tr>
<tr>
<td>Depth of cover ( H ) \hspace{1cm} 100 m</td>
</tr>
<tr>
<td>Elevation of water table ( H_w ) \hspace{1cm} 130 m</td>
</tr>
<tr>
<td>Tunnel diameter ( D ) \hspace{1cm} 10 m</td>
</tr>
<tr>
<td><strong>Ground</strong></td>
</tr>
<tr>
<td>Effective cohesion ( c ) \hspace{1cm} 0-400 kPa</td>
</tr>
<tr>
<td>Angle of eff. internal friction ( \varphi ) \hspace{1cm} 30°</td>
</tr>
<tr>
<td>Submerged unit weight ( \gamma' ) \hspace{1cm} 12 kN/m³</td>
</tr>
<tr>
<td>Unit weight water ( \gamma_w ) \hspace{1cm} 10 kN/m³</td>
</tr>
<tr>
<td><strong>Shear resistance of the vertical slip surfaces</strong></td>
</tr>
<tr>
<td>Coeff. of lateral stress in wedge ( \lambda_w ) \hspace{1cm} 0.5</td>
</tr>
<tr>
<td>Coeff. of lateral stress in prism ( \lambda_p ) \hspace{1cm} 1.0</td>
</tr>
<tr>
<td><strong>Drainage boreholes</strong></td>
</tr>
<tr>
<td>Diameter ( d_{dr} ) \hspace{1cm} 0.1 m</td>
</tr>
<tr>
<td>Length ( l_{dr} ) \hspace{1cm} 0.5-30 m</td>
</tr>
<tr>
<td>Number ( n ) \hspace{1cm} 0-12</td>
</tr>
<tr>
<td>Distance from tunnel axis ( r_{dr} ) \hspace{1cm} 1.5-11 m</td>
</tr>
<tr>
<td><strong>Drainage via a pilot tunnel or another tunnel</strong></td>
</tr>
<tr>
<td>Diameter of coaxial pilot tunnel ( d_p ) \hspace{1cm} 0.5-5 m</td>
</tr>
<tr>
<td>Diameter of adjacent tunnel ( d_p ) \hspace{1cm} 1-10 m</td>
</tr>
<tr>
<td>Vertical centre distance ( L_v ) \hspace{1cm} 14-92 m</td>
</tr>
<tr>
<td>Horizontal centre distance ( L_h ) \hspace{1cm} 14-90 m</td>
</tr>
<tr>
<td>Distance of drainage curtains ( a_{dr} ) \hspace{1cm} 4-20 m</td>
</tr>
</tbody>
</table>
2.3.2 Reference cases

Figure 2.6a shows the support pressure needed for face stability in a cohesionless ground as a function of the wedge angle $\omega$ in the case of ideal drainage (lower curve) and in the absence of drainage measures (upper curve). In the first case, the pore pressure in the wedge is atmospheric; seepage forces have to be taken into account only for the overlying prism. In the second case, the wedge is acted upon by seepage forces which are directed towards the tunnel face, thus leading not only to a considerably higher necessary support pressure ($s = 770$ vs. $100$ kPa), but also a more extended unstable region (critical wedge angle $\omega_{cr} = 63^\circ$ vs. $30^\circ$). It is, nevertheless, remarkable that even in the absence of drainage measures the necessary support pressure is considerably lower than the initial hydrostatic pressure ($s = 770$ vs. $1400$ kPa). This is due to the natural drainage action of the tunnel heading, which inevitably leads to some pore pressure relief in the ground ahead of the face.

The practical significance of these results becomes evident when we consider the fact that face support pressures of more than $200$ kPa cannot be managed in conventional tunnelling, even with heavy face bolt reinforcement (Anagnostou and Serafeimidis, 2007). In the present example, ground improvement by grouting or freezing would be indispensable in the absence of drainage measures. In the case of
complete pore pressure relief by advance drainage, the face would still need support, but this would be technically feasible considering the relatively moderate support pressure of about 100 kPa that is required.

The required support pressure (for the critical wedge) decreases with increasing cohesion of the ground (Fig. 2.6b). In the case of complete pore pressure relief, cohesion of just 45 kPa would be sufficient for the tunnel face to remain stable without support (point A). Without drainage measures the cohesion required for an unsupported face increases to 330 kPa (point B); even with heavy bolt reinforcements ($s = 180$ kPa), the ground would need a cohesion of at least 240 kPa (point C).

### 2.3.3 Drainage via boreholes from the tunnel face or niches

We consider horizontal boreholes of uniform diameter ($d_{dr} = 10$ cm) and investigate successively the effect of their number $n$, location, length $l_{dr}$ and distance $r_{dr}$ from the tunnel axis (Figs. 2.1a and 2.1b).

#### 2.3.3.1 Number of drainage boreholes

The effect of the number of boreholes is investigated assuming that they are 30 m long and located at $r_{dr} = 3.8$ m. Figure 2.7 shows the distribution of the hydraulic head along the $x_1$-axis ahead of the tunnel face and along the $x_3$-axis above the tunnel for 2 to 12 boreholes (dashed lines) as well as for the reference case without drainage boreholes (solid line). It is remarkable that just 2 to 4 boreholes result in considerable pore pressure relief, particularly ahead of the tunnel face ($x_1$-axis in Fig. 2.7). Advance drainage decreases the hydraulic head gradients close to the face (they occur only at a greater distance), which leads to narrower critical wedges and a substantial reduction in the required support pressure (Fig. 2.8). In a cohesionless ground, advance drainage with just two boreholes decreases the required support pressure to about 60% of the reference pressure, i.e. the pressure required in the absence of boreholes (compare points A and B in Fig. 2.8). The addition of four more drainage boreholes (point C in Fig. 2.8) leads to a further reduction in the support pressure to about 40% of the reference pressure; marginal utility diminishes with further boreholes. In a weak rock exhibiting a cohesion of 150 kPa, 6 boreholes would suffice for face stability (point D in Fig. 2.8). Trading the feasible pore pressure relief against the drilling effort, 4 - 6 drainage boreholes can be recommended as an efficient face stabilization measure in homogeneous ground.
Figure 2.7. Distribution of the hydraulic head $h$ above (l.h.s.) and ahead of (r.h.s) the tunnel face (parameters according to Fig. 2.5 and Table 2.1)

Figure 2.8. Required support pressure $s$ as a function of the number of drainage boreholes $n$ (borehole locations see Fig. 2.13; parameters according to Fig. 2.5 and Table 2.1)
2.3.3.2 Location of drainage boreholes

Figure 2.9 shows the support pressure $s$ as a function of the borehole number $n$ in ground without cohesion when considering all possible combinations of borehole location $d_{r_1}$-$d_{r_6}$ (see inset in Fig. 2.9). It becomes apparent that the exact location of the boreholes is of secondary importance for the necessary support pressure, provided that at least two boreholes are located in the upper third of the tunnel face and the remaining boreholes either arranged in the upper section or distributed evenly over the face (the recommended borehole arrangements are indicated with green crosses, the others are marked in red).

![Diagram of borehole arrangements](image)

Figure 2.9. Required support pressure $s$ as a function of the number of drainage boreholes $n$ (borehole locations see inset; parameters according to Fig. 2.5 and Table 2.1)

Figures 2.10a illustrates the example of 2 boreholes for two cases, where the deviation in required support pressure is considerable (see red and green crosses for $n = 2$ in Fig. 2.9). Figure 2.10b shows the hydraulic head fields in the case of 2 boreholes located either in the upper or in the lower part of the face. Boreholes arranged in the upper part of the cross-section reliefs the pore pressure particularly in the upper area ahead of the face and therefore results in favourable support pressures (Fig. 2.10c). The recommendation concerning borehole locations is also valid for other borehole diameters (e.g. $0.30 \text{ m}$ instead of $0.10 \text{ m}$; see Fig. 2.11) and other values of water table (e.g. $55 \text{ m}$ instead of $135 \text{ m}$ above the tunnel axis; see Fig. 2.12).

Figure 2.13 indicates the borehole locations which are taken into account for further consideration in the following sections.
Figure 2.10. Results of comparative computations concerning the location of the boreholes

Figure 2.11. Results of comparative computations concerning the location of the boreholes (same water table as in Fig. 2.10, but larger borehole diameter)
2.3.3.3 Length of drainage boreholes

We discuss the effect of borehole length $l_{dr}$ for the case of 6 boreholes (Fig. 2.14) considering the effective borehole location indicated in Figure 2.13. The required support pressure decreases rapidly with increasing borehole length $l_{dr}$ until the latter reaches about 15 m (point A in Fig. 2.14), i.e. until the boreholes extend sufficiently ahead of the largest critical wedge (remember that in the absence of drainage the critical angle $\omega_{cr}$ is equal to about 63°). Longer boreholes provide no benefit, because they cause pore pressure relief far ahead of the tunnel face (in a zone that is anyway non-critical for face stability; see below). In the present example (10 m diameter tunnel), 30 m long boreholes (a technically feasible length) every 15 m (one and a half tunnel diameters) would be a sensible choice. Longer boreholes would be advantageous in medium to low permeability ground because they provide a longer drainage period in advance of excavation. However, they may present execution difficulties due to instabilities or deformations in the borehole walls, friction when using casings or drilling accuracy.
Figure 2.14. Required support pressure $s$ as a function of the borehole length $l_d$ (borehole locations see inset; parameters according to Fig. 2.5 and Table 2.1)

An increase of the borehole length beyond a certain value (hereafter referred to as "characteristic borehole length $l_{d, char}$") does not increase face stability, because it causes pore pressure relief in the ground far away from the tunnel face, which is anyway non-critical for stability. The characteristic borehole length corresponds to the extent of the potentially unstable zone ahead of the face, i.e. to the depth $D \tan \omega_{cr}$ of the critical wedge (where $D' = 0.89D$ denotes the side length of equivalent rectangle to the tunnel diameter $D$ and $\omega_{cr}$ the critical wedge angle). As the critical wedge angle does not depend on $D$ (see Eq. (2-30) and more detailed later in Section 2.4.1: for a specific drainage layout and given ratios for horizontal to vertical stress ($\lambda_w$, $\lambda_p$) it is $\omega_{cr} = f(\phi, H, \gamma_d, c)$, $l_{d, char}$ is proportional to $D$.

These relations are shown in Figure 2.15 for our tunnel example. Figure 2.15a shows the required support pressure $s$ as a function of the wedge angle $\omega$ when considering no drainage boreholes. The critical wedge angle $\omega_{cr}$ decreases with increasing ground cohesion $c$ (Fig. 2.15b), thus also the characteristic borehole length $l_{d, char}$ (Fig. 2.15c). Figure 2.15d finally superimposes the characteristic borehole length (indicated with crosses) with Figure 2.14, which leads to the recommendation of always maintaining a minimum borehole length of 1.5$D$ (e.g. by drilling 3$D$ long boreholes every 1.5 tunnel diameter). This proposal covers even the worst-case of cohesionless ground.

Of course, the critical wedge angle (and thus the characteristic borehole length) increases with increasing hydraulic gradient. In case of $T/D > 5$ and considering cohesionless ground as decisive, the dependencies of the critical wedge angle simplify to $\omega_{cr} = f(\phi, \bar{h}_0)$. Figure 2.16 shows the normalized characteristic borehole length (according to Eq. (2-36)) for a wide range of normalized hydraulic head $\bar{h}_0$ and three angles of internal friction of the ground $\phi$. The normalized characteristic borehole length clearly increases with increasing normalized head and slightly with decreasing friction angle (Fig. 2.16).
(Side note: The minimum borehole length is supported by Atwa et al. (2000), who studied the hydraulic head field close to the tunnel face (without considering face stability) and stated that the hydraulic gradients do not change if the borehole has minimum length of $l_{dr}/D = 2$.)

Figure 2.15. (a) Support pressure $s$ as a function of the wedge angle $\omega$ when considering no drainage boreholes, (b) critical wedge angle $\omega_{cr}$ as a function of ground cohesion $c$, (c) characteristic borehole length $l_{dr, char}$ as a function of ground cohesion $c$, (d) superimposing Fig. 2.14 with the characteristic borehole length $l_{dr, char}$ (indicated with crosses; parameters according to Fig. 2.5 and Table 2.1)
Figure 2.16. Normalized characteristic borehole length $l_{d, char}$ as a function of the normalized hydraulic head $h_0$ for selected friction angle $\varphi$

Figure 2.17. Required support pressure $s$ as a function of the borehole location $r_{d}$ (borehole locations see inset; parameters according to Fig. 2.5 and Table 2.1)
2.3.3.4  Radial distance of drainage boreholes

Finally, we investigate whether there is any optimisation potential with respect to the distance \( r_{dr} \) of the boreholes from the tunnel centre. We consider six, 30 m long drainage boreholes located at distances of 1.5 - 11 m from the tunnel centre (the distances \( r_{dr} > D/2 = 5 \) m apply to boreholes drilled from niches), which are located either at the upper part of the cross-section or laterally in groups of three (see inset in Fig. 2.17, l.h.s. and r.h.s cross-section, respectively). Figure 2.17 shows the required support pressure \( s \) as a function of \( r_{dr} \) for these two arrangements (solid and dashed lines, respectively). As the dashed and solid lines are very close together but all curves exhibit a minimum at \( r_{dr} = 5 - 7 \) m, the conclusion is that the drainage boreholes should be arranged close to the periphery of the tunnel cross-section, while draining from a niche above the roof or from a lateral niche does not make any difference.

2.3.4  Drainage action of a pilot tunnel

We consider first a pilot tunnel that is coaxial with the main tunnel. Figure 2.18 shows the required face support pressure \( s \) as a function of the diameter \( d_p \) of the pilot tunnel for different values of the cohesion \( c \). In order to determine the required face support pressure, failure mechanisms involving the entire face or parts thereof were considered (mechanisms I, II and III in the inset in Fig. 2.18). Mechanism III proved to be decisive. The pore pressure relief from the pilot tunnel has a significant stabilizing effect. For example, face stability in a weak rock exhibiting a cohesion \( c \) of 150 kPa would require a very high support pressure of about 400 kPa in the absence of drainage measures (point A in Fig. 2.18). A 3 m diameter pilot tunnel would, however, provide sufficient pore pressure relief in advance of profile enlargement for the face to be stable without support (point B in Fig. 2.18).

![Figure 2.18](image_url)
The drainage effect of a pilot tunnel outside the cross-section of the main tunnel (Fig. 2.19) is, as might be expected, less pronounced than that of a coaxial pilot tunnel. It is still remarkable, however, considering the relatively long distance from the main tunnel (27 m in the example in Fig. 2.19). A 3 m diameter pilot tunnel causes a reduction in the required support pressure by 34 - 80% depending on the cohesion of the ground (compare points at $d_p = 0$ with points at $d_p = 3$ m in Fig. 2.19). In the case of a weak rock exhibiting cohesion of 150 - 200 kPa, for example, the necessary face support pressure can be reduced to a technically feasible level of 50 - 150 kPa by pre-constructing a pilot tunnel of 3 - 4 m diameter.

According to Figures 2.18 and 2.19, the drainage effect of a pilot tunnel is considerable even if its diameter is very small (as is the case with micro-tunnelling). The form of the $s (d_p)$ curves also shows that the benefit of excavating a larger diameter pilot tunnel is relatively small. In addition, the stability of the face in a larger diameter pilot tunnel may itself be problematic. (For small tunnel diameters, drainage measures alone often suffice for stability, even in ground of low cohesion; see also Zingg and Anagnostou, 2013.) From the point of view of face stability, a pilot tunnel of small diameter is therefore clearly preferable.

Figure 2.19. Required support pressure $s$ as a function of the diameter $d_p$ of an external pilot tunnel (parameters according to Fig. 2.5 and Table 2.1)
2.3.5 Drainage action of the first tube of a twin tunnel

The solid curves in Figure 2.20 show the support pressure $s$ required for face stability of the second tube as a function of its distance $L$ from the first tube. The crosses apply to the tube constructed first. The advance drainage of the ground due to the first tube has a marked effect on the face stability of the second tube even if the distance of the two tubes is relatively large ($L = 50-70$ m; compare solid lines with crosses in Fig. 2.20).

For a typical spacing of $L_h = 30$ m, the drainage action of the first tube leads to a reduction in the necessary face support pressure in the second tube of 44 – 77% depending on the cohesion of the ground. The construction of the second tube is, therefore, considerably easier than that of the first tube. Consider, for example, a twin tunnel in weak rock exhibiting a cohesion $c$ of 200 kPa. Face stabilization in the first tube would require a barely feasible support pressure of more than 200 kPa (point B in Fig. 2.20), or, alternatively, advance drainage by means of boreholes from the face, ground improvement by grouting or combinations thereof. In the second tube, however, the face would be stable without any support $s$ (point C in Fig. 2.20).

It should be noted that the decisive parameter for the drainage action of the first tube is the distance $L$, irrespective of the vertical or horizontal offset of the two tubes. This is illustrated by the dashed lines in Figure 2.20, which applies to the theoretical case of vertically arranged tunnel tubes with zero horizontal offset.

![Figure 2.20](image)

**Figure 2.20.** Required support pressure $s$ as a function of the centre distance $L$ of two twin tunnels (parameters according to Fig. 2.5 and Table 2.1)
2.3.6  Effect of drainage curtains from a pilot tunnel

Taking into account the results of the previous sections, we consider a pilot tunnel of 3 m diameter. The geometric parameters for drainage curtains are: their spacing $a_{dr}$, the number $n$ and the location of the boreholes of each curtain (Fig. 2.21). The length and the diameter of the boreholes are taken equal to 30 m and 10 cm, respectively. Both the case of a single tunnel (Fig. 2.21a) and that of a twin tunnel will be considered (Fig. 2.21b). The horizontal and vertical offsets of the pilot tunnel are taken equal to 20 m and 10 m, respectively, which mean that the drainage curtains reach the entire cross section of the main tunnel(s).

The numerical investigations were carried out for two different pilot tunnel linings: an impermeable lining (e.g. a sealed segmental lining) and a permeable lining (e.g. a shotcrete lining with pore pressure relief holes). In the first case, drainage occurs solely via the face of the main tunnel and via the radial boreholes. In the second case, pore pressure relief also occurs due to drainage through the pilot tunnel walls.

The computations were carried out for 1, 2, 4 or 6 boreholes per curtain; the benefit of a larger number of boreholes per curtain is marginal (cf. Section 2.3.3.1). For any specific number of boreholes per curtain, computations for different borehole arrangements in the plane of the tunnel cross-section were carried out to identify the one with the lowest necessary support pressure (see example in Zingg et al., 2013). The table in Figure 2.21 shows the optimum borehole arrangements.

The diagrams in Figure 2.22 apply to the case of a single tunnel (Fig. 2.21a) and show the distribution of the hydraulic head in the axial direction ahead of the tunnel face for 1, 2, 4 or 6 boreholes per curtain (diagrams from the top down) and a sealed (l.h.s. diagrams) or draining (r.h.s. diagrams) pilot tunnel lining. The curves in every diagram apply to curtain spacings $a_{dr} = 4 - 20$ m (Fig. 2.21). The diagrams also show for comparison the head distribution without drainage curtains (upper line in

![Diagram](image)

**Figure 2.21.** Location of the most effective boreholes ($dr_1 - dr_n$) of the drainage curtain
Figure 2.22. Axial distribution of the normalized hydraulic head $h$ ahead of the face of the main tunnel (borehole locations according to table in Fig. 2.21, parameters according to Table 2.1)
The hydraulic head is lower in the case of a draining pilot tunnel lining (r.h.s. diagrams), exhibits local minima at the locations of the drainage curtains and decreases with decreasing curtain spacing and with increasing number of boreholes per curtain. It is remarkable that just 2 boreholes per curtain spaced at 10 m intervals in the longitudinal direction suffice to reduce the hydraulic head to 45 – 60 % of its initial value in the vicinity of the face (see red curves in Figs. 2.22b and 2.22f). Considering the high cost but limited additional benefit of very closely spaced curtains, a spacing of 10 m (i.e. one tunnel diameter) represents a reasonable choice. Figure 2.23 shows the required support pressure $s$ for this spacing as a function of the cohesion $c$ for curtains consisting of 1, 2, 4 or 6 boreholes for an impermeable (Fig. 2.23a) and a permeable (Fig. 2.23b) pilot tunnel lining, a single tunnel (black lines) and a twin tunnel (red lines). The difference between single and twin tunnels is small (e.g. for $c = 0$: 3-12% for an impermeable, 3-7% for a permeable pilot tunnel lining).

In order to illustrate the significance of these results from the practical engineering point of view, consider a single tunnel crossing weak rock exhibiting a cohesion of 100 - 150 kPa. In the absence of drainage measures, the required support pressure would be 400 - 500 kPa (point A in Fig. 2.23a). This value, which is unfeasible in conventional tunnelling, can be reduced to a manageable level (of about 100 kPa) by drainage curtains consisting of 2-4 boreholes each (points B and C, Fig. 2.23a). In combination with a permeable pilot tunnel lining (Fig. 2.23b), the drainage curtains would even allow an unsupported face, and this in spite of the combination of weak ground with high in situ hydrostatic pressure.

**Figure 2.23.** Required support pressure $s$ as a function of cohesion $c$ for $n = 0$ to 6 drainage boreholes per curtain: (a) impermeable, (b) permeable pilot tunnel lining (borehole locations according to table in Fig. 2.21, parameters according to Fig. 2.5 and Table 2.1)
2.4 Design equation

This Section will show that the required face support pressure can be approximated by the following equation, which is sufficiently accurate for practical design purposes:

\[
\frac{s}{\gamma'/D} = F_0 - F_1 \cdot \frac{c}{\gamma'/D} + \left( F_2 - F_3 \cdot \frac{c}{\gamma'/D} \right) \gamma_0 \cdot h_0 \cdot \frac{1}{\gamma'/D}, \tag{2-37}
\]

where the dimensionless coefficients \( F_0 \) to \( F_3 \) depend only on the friction angle \( \varphi \) and the drainage layout. The coefficients were determined by means of a comprehensive parametric study and can be depicted from the design nomograms of Figures 2.24 to 2.30. Each figure applies to a different drainage layout; Table 2.2 provides an overview. Eq. (2-37) is sufficiently accurate for practical purposes provided that the upper boundary of the seepage flow domain is not too close to the tunnel crown (specifically, that \( \min(H, H_w) > 5D \)).

Table 2.2. Drainage layouts and belonging design nomograms

<table>
<thead>
<tr>
<th>Drainage layout</th>
<th>Nomogram</th>
</tr>
</thead>
<tbody>
<tr>
<td>None (only drainage action of the tunnel face)</td>
<td>Fig. 2.24 (( n = 0 ))</td>
</tr>
<tr>
<td>Axis-parallel, long boreholes through the tunnel face ( (l_d/D = 1.5) )</td>
<td>Fig. 2.24</td>
</tr>
<tr>
<td>Axis-parallel, short boreholes through the tunnel face ( (l_d/D = 3) )</td>
<td>Fig. 2.25</td>
</tr>
<tr>
<td>Axis-parallel boreholes from niches</td>
<td>Fig. 2.26</td>
</tr>
<tr>
<td>Co-axial pilot tunnel</td>
<td>Fig. 2.27</td>
</tr>
<tr>
<td>First tube of twin tunnel</td>
<td>Fig. 2.28</td>
</tr>
<tr>
<td>External pilot tunnel with permeable lining</td>
<td>Fig. 2.29 (( n = 0 ))</td>
</tr>
<tr>
<td>Radial boreholes from external pilot tunnel with permeable lining</td>
<td>Fig. 2.29</td>
</tr>
<tr>
<td>Radial boreholes from external pilot tunnel with impermeable lining</td>
<td>Fig. 2.30</td>
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</tbody>
</table>

2.4.1 Development of the design equation

The starting point for developing Eq. (2-37) and working out design nomograms is the rigorous Eq. (2-30). The latter gives the support pressure \( s_\omega \) for a specific value of the angle \( \omega \). The critical support pressure \( s \) is obtained by maximizing \( s_\omega \) with respect to \( \omega \):

\[
\bar{s} = N_\gamma - N_c \bar{c} + N_h \bar{h}, \tag{2-38}
\]

where \( N_\gamma = N_{\gamma u}(\omega_{cr}, ...) \), \( N_c = N_{c u}(\omega_{cr}, ...) \), \( N_h = N_{h u}(\omega_{cr}, ...) \) and \( \omega_{cr} \) is obtained by solving the non-linear equation

\[
\bar{c} \frac{dN_{\gamma u}}{d\omega} \bigg|_{\omega=\omega_{cr}} - \bar{h}_0 \frac{dN_{h u}}{d\omega} \bigg|_{\omega=\omega_{cr}} = \frac{dN_{\gamma u}}{d\omega} \bigg|_{\omega=\omega_{cr}}. \tag{2-39}
\]

An examination of Eqs. (2-31) to (2-34) and (2-39) shows that the critical angle \( \omega_{cr} \) as well as the coefficients \( N_\gamma, N_c \) and \( N_h \) depend not only on the drainage layout and on \( \varphi, \gamma_d \) and \( \bar{H} \), but also on \( \bar{c} \) and
which means that the support pressure $s$ depends non-linearly on $\bar{c}$ and $\bar{h}_0$. This in combination with the large number of influencing parameters renders the elaboration of design nomograms practically impossible. However, it will be shown below that: (i), the number of parameters can be reduced by two ($\gamma_d$ and $H$ disappear from the parameter list) if the upper boundary of the seepage flow domain is not too close to the tunnel crown (specifically, if $T > 5D$); and, (ii), an approximate linearized relationship can be considered instead of Eq. (2-30), which is sufficiently accurate for practical design purposes.
Figure 2.25. Nomograms for the coefficients $F_0$ to $F_3$ in the case of drainage via boreholes from the tunnel face ($l_D/D = 3$; variable: number $n$ of boreholes)

2.4.1.1 Reduction in the number of parameters in the case of $T > 5D$

As $T = \min(H, H_w)$, the condition $T > 5D$ is fulfilled by subaqueous tunnels with an overburden of at least $5D$ and by mountain tunnels at a depth of minimum $5D$ underneath the water table.

The coefficient $e_s$ (Eq. (2-14)) decreases rapidly with increasing values of $T$ and becomes practically zero for $T/D > 5$ even if the friction angle is small and the wedge angle $\omega$ big. For $e_s = 0$, the coefficients become $N_{c\omega} = f(\omega, \phi, \delta_s)$, $N_{c\omega} = f(\omega, \phi, \delta_s)$ and $N_{h\omega} = f(\omega, \phi, \delta_s, I_1, I_3)$ according to Eqs. (2-31) to (2-33), where $\delta_s = f(\omega, \phi, \overline{c}, \overline{h}_0, I_s)$ according to Eqs. (2-27) and (2-29).

The coefficients $I_1$, $I_3$ and $I_s$ appearing in these equations depend on the hydraulic head field and, therefore, on the geometric parameters of the seepage flow domain (tunnel diameter $D$, distance $T$...
between tunnel crown and upper boundary of the seepage flow domain, location, length and diameter of the drainage boreholes; see Eq. (2-36)) and on the hydraulic boundary conditions. At the far-field boundaries and at \( x_3 = T + D/2 \), the hydraulic head is equal to \( h_0 \). At the tunnel face and along the drainage boreholes, the pore pressure is atmospheric and, consequently, the hydraulic head is equal to the elevation \( x_3 \), which varies between \(-0.5D\) (tunnel floor) and \(0.5D\) (tunnel crown), i.e. \( |h| \leq 0.5D \). If \( T > 5D \), the effect of this variation is small relative to \( h_0 (= 0.5D + H_w > 5.5D) \) and, consequently, the head at the face and along the borehole walls can be taken approximately equal to zero. The hydraulic head field then depends linearly on the boundary value \( h_0 \) (due to the linearity of Darcy’s law). In addition, the size of the seepage flow domain above the crown \( T \) is practically negligible for the head distribution if it is more than a few tunnel diameters (Anagnostou and Kovári 1996, Zingg and Anagnostou 2012a).

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**Figure 2.26.** Nomograms for the coefficients \( F_0 \) to \( F_3 \) in the case of drainage via boreholes from the tunnel face \((n = 6; \text{variable: borehole location } r_{dl}/D)\).
Consequently, if \( T/D > 5 \), the hydraulic head at every point \( x_k \) may be expressed in general as
\[ h \approx h_0 \cdot f(\xi_k), \]
which means that the dimensionless coefficients \( I_1 \) and \( I_2 \) depend only on the angle \( \omega \) (see Eqs. (2-10) and (2-11)), while \( I_s \) depends additionally on \( \varphi \) and on \( T/D \) (Eq. (2-26)). The latter appears in the upper integration bound of the third r.h.s. term of Eq. (2-26), but is practically irrelevant if \( T/D > 5 \) because of the rapid decay of the exponential term of the integrand.

Consequently, for given drainage layout, the critical angle \( \omega_{cr} \) as well as the coefficients \( N_f, N_c \) and \( N_h \) are functions of \( \varphi, c \) and \( h_0 \) (see Fig. 2.8 or later Fig 2.32 for an example illustrating the dependency of \( \omega_{cr} \) on \( c \) and \( h_0 \)):

\[
\bar{s} = N_f(\varphi, \bar{c}, \bar{h}_0) - N_c(\varphi, \bar{c}, \bar{h}_0)\bar{c} + N_h(\varphi, \bar{c}, \bar{h}_0)\bar{h}_0. \tag{2-40}
\]
According to this equation, the support pressure depends non-linearly on the cohesion $c$ and on the hydraulic head $h_0$. The non-linearity is negligible in the absence of seepage flow ($h_0 = 0$), but significant when considering a wide range of hydraulic heads $h_0$.

### 2.4.1.2 Linearization

In order to find out whether the functional dependencies of Eq. (2-40) can be simplified to a degree that would allow this equation to be applied in combination with the smallest possible number of nomograms depicting its r.h.s. coefficients, the computational results of a comprehensive parametric study were analysed.
Tables 2.3 and 2.4 summarize the value ranges considered for the ground and drainage parameters, respectively. The geometric parameters for the drainage schemes were chosen in accordance with the results of Section 2.3.

Analysis of the computational results showed that they can be approximated sufficiently accurate by the following equation, which maintains the structure of the rigorous Eq. (2-40), but is linear in $c$ and $h_0$.

$$
\bar{\varphi} = F_0(\varphi) - F_1(\varphi)\bar{\varphi} + (F_2(\varphi) - F_3(\varphi)\bar{\varphi})\overline{h_0}
$$

(2-41)
According to this equation, the coefficients $F_0$ and $F_1$, which correspond to the coefficients $N_\gamma$ and $N_c$ of Eq. (2-40), can be taken as functions only of the friction angle $\phi$ for a given drainage layout, i.e. the dependency of $N_\gamma$ and $N_c$ on $c$ and $h_0$ can be neglected. As can be seen from the last bracketed r.h.s. term, which corresponds to $N_h$, the dependency of $N_h$ on $c$ can be linearized, while the effect of $h_0$ can be neglected. The coefficients $F_2$ and $F_3$ thus depend only on the friction angle and on the drainage layout.
Table 2.3. Geotechnical conditions considered in working out the design nomograms

<table>
<thead>
<tr>
<th>Problem layout</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of cover</td>
<td>$H/D$</td>
</tr>
<tr>
<td>Hydraulic head</td>
<td>$\gamma_h \delta' \gamma' D$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Ground</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective cohesion</td>
<td>$c/\gamma' D$</td>
</tr>
<tr>
<td>Angle of eff. internal friction</td>
<td>$\phi$</td>
</tr>
<tr>
<td>Submerged unit weight</td>
<td>$\gamma' / \gamma_w$</td>
</tr>
<tr>
<td>Dry unit weight</td>
<td>$\gamma_d / \gamma_w$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Coefficients of lateral stress</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Wedge</td>
<td>$\lambda_w$</td>
</tr>
<tr>
<td>Prism</td>
<td>$\lambda_p$</td>
</tr>
</tbody>
</table>

(a) The results are applicable for $T/D > 5$ (Sections 2.4.1 and 2.4.2).
(b) The results are applicable for $2.5 \leq \gamma_h \delta' \gamma' D \leq 30$ (Fig. 2.32).
(c) The results are applicable for $c/\gamma' D \geq 0.2$ (Section 2.4.1.2).
(d) The results are sufficiently accurate for $1 \leq \gamma' / \gamma_w \leq 1.8$.
(e) This parameter is irrelevant for $T/D > 5$ (Section 2.4.1.1).

Table 2.4. Parameters for the drainage schemes considered in working out the design nomograms

<table>
<thead>
<tr>
<th>Drainage measures</th>
<th>$n$</th>
<th>$r_{dp}/D$</th>
<th>$l_{dp}/D$</th>
<th>$d_{dp}/D$</th>
<th>$L_{dp}/D$</th>
<th>$L_{dp}/D$</th>
<th>$a_{dp}/D$</th>
<th>Nomograms for $F_0$ to $F_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>None (only natural drainage action of the tunnel face)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>Fig. 2.24 (n = 0)</td>
</tr>
<tr>
<td>Axis-parallel boreholes (a) through the tunnel face or niches</td>
<td>2 – 10</td>
<td>0.38</td>
<td>1.5</td>
<td>2 – 10</td>
<td>0.38</td>
<td>3</td>
<td>6</td>
<td>0.38, 0.67</td>
</tr>
<tr>
<td>Co-axial pilot tunnel</td>
<td>0</td>
<td>0</td>
<td>0.05 - 0.5</td>
<td>0</td>
<td>0</td>
<td>1.4 - 5.5</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>First tube of twin tunnel</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1.4 - 5.5</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>External pilot tunnel with permeable lining</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Radial boreholes (b) from external pilot tunnel with permeable lining</td>
<td>1 - 6</td>
<td>3</td>
<td>0.3</td>
<td>2</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>Fig. 2.29</td>
</tr>
<tr>
<td>Radial boreholes (b) from external pilot tunnel with impermeable lining</td>
<td>1 - 6</td>
<td>3</td>
<td>0.3</td>
<td>2</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>Fig. 2.30</td>
</tr>
</tbody>
</table>

(a) Borehole diameter: $d_{dp}/D = 0.01$; Borehole location: see Fig. 2.13
(b) Borehole diameter: $d_{dp}/D = 0.01$; Borehole location: see Fig. 2.21
The coefficients $F_0$ to $F_3$ were determined by fitting Eq. (2-41) to the results of a parametric study. Their values were chosen conservatively in order for the proposed design equation to provide an estimate of the required support pressure that is slightly on the safe side for the practically relevant ranges of cohesion and support pressure ($\bar{c} \geq 0.2$, $0 \leq \bar{s} \leq 4$) and up to very high hydraulic head ($\bar{h}_0 \leq 25$). A normalized support pressure $\bar{s}$ of 4 clearly corresponds to extremely heavy face reinforcement, while face stability in a ground with a normalized cohesion $\bar{c}$ lower than 0.2 would anyway require additional improvement measures such as grouting.

Figure 2.31 illustrates the principle of curve fitting. The solid curves show the exact normalized support pressure $\bar{s}$ as a function of the normalized hydraulic head $\bar{h}_0$ for different values of the normalized cohesion $\bar{c}$, a fixed value of the friction angle ($\varphi = 30^\circ$) and a given drainage layout ($n = 4$ as in Fig. 2.13). Within the considered range ($\bar{s} \leq 4$, $\bar{h}_0 \leq 25$, indicated with dotted lines), $\bar{s}$ depends almost linearly on $\bar{h}_0$:

$$\bar{s} \approx \beta_1 + \beta_2 \bar{h}_0, \quad (2-42)$$

where

$$\begin{align*}
\beta_1 &= \frac{\bar{s}}{|_{\bar{h}_0 = 0}}, \\
\beta_2 &= \frac{4 - \bar{s}}{|_{\bar{h}_0 = 0}}, & \text{if } \bar{s}|_{\bar{h}_0 = 0} \geq 0, \\
\beta_1 &= \frac{4 - \bar{s}}{\bar{h}_0|_{\bar{h}_0 = 0} - \bar{h}_0|_{\bar{h}_0 = 4}}, \\
\beta_2 &= \frac{4}{\bar{h}_0|_{\bar{h}_0 = 4} - \bar{h}_0|_{\bar{h}_0 = 0}}, & \text{if } \bar{s}|_{\bar{h}_0 = 0} < 0 \text{ and } \bar{s}|_{\bar{h}_0 = 25} > 4, \\
\beta_1 &= \frac{\bar{s}|_{\bar{h}_0 = 25} - \bar{h}_0|_{\bar{h}_0 = 0}}{25 - \bar{h}_0|_{\bar{h}_0 = 4}}, \\
\beta_2 &= \frac{\bar{s}|_{\bar{h}_0 = 25}}{25 - \bar{h}_0|_{\bar{h}_0 = 0}}, & \text{if } \bar{s}|_{\bar{h}_0 = 0} < 0 \text{ and } \bar{s}|_{\bar{h}_0 = 25} < 4.
\end{align*} \quad (2-43)$$

Figures 2.31b and 2.31c show the coefficients $\beta_1$ and $\beta_2$, respectively, as functions of the normalized cohesion $\bar{c}$. They depend practically linearly on $\bar{c}$:

$$\beta_1 \approx F_0 - F_1 \bar{c}, \quad (2-44)$$

$$\beta_2 \approx F_2 - F_3 \bar{c}, \quad (2-45)$$

where $F_0$ to $F_3$ can be determined by linear regression. As the $\bar{s}(\bar{h}_0)$ curves in Figure 2.31a are nearly parallel, their slopes $\beta_2$ are almost equal ($F_3$ is small), while their intercept $\beta_1$ with the vertical axis depends markedly on the normalized cohesion $\bar{c}$. Eqs. (2-42), (2-44) and (2-45) result in the approximate design equation (2-41), which is represented by the dashed lines in Figure 2.31a.

The linear approximation in Figure 2.31a slightly overestimates the support pressure inside the range considered and may underestimate the support pressure outside this range (i.e. at hydraulic heads that are either higher than the head that necessitates a normalized support pressure of 4, or lower than the head, where the necessary support pressure becomes equal to zero). This is, however, irrelevant from a practical point of view. At hydraulic heads above the range considered (i.e. for $\bar{h}_0 > \bar{h}_0|_{\bar{h}_0 = 4}$) the approximate support pressure is anyway higher than 4, thus indicating that advance drainage is insufficient for stability without ground improvement (remember that a normalized support pressure $\bar{s}$ of 4 corresponds to a very heavy face support). On the other hand, for $\bar{h}_0 > \bar{h}_0|_{\bar{h}_0 = 0}$ the exact support pressure will be “less” negative, but still negative, i.e. the face will anyway be stable without support.
Figure 2.31. Illustration of the curve-fitting procedure ($\phi = 30^\circ$; 4 drainage boreholes of length $l_{dr} = 3D$ with locations according to Fig. 2.13): (a) exact and approximate normalized support pressure $s\overline{\sigma}$ as a function of the normalized hydraulic head $h_0$; (b) coefficient $\beta_1$ (Eq. (2-44)) and, (c), coefficient $\beta_2$ (Eq. (2-45)) as a function of the cohesion $c$

### 2.4.2 Applicability limits of the design nomograms

Due to the underlying fitting procedure, the nomograms are not applicable where there is no seepage flow. In addition as mentioned above, Eq. (2-40) presupposes that the upper boundary of the seepage flow domain is not too close to the tunnel crown. If $T/D < 5$, the hydraulic gradients will be higher than those in the nomograms, which will then consequently underestimate the necessary support pressure. The greater the drainage-induced pore pressure relief (i.e. the lower the factor $F_2$), the greater will be the error.

In order to quantify the error, a shallow tunnel ($T/D = 1.85$) is considered with advance drainage either from the face (via 6 boreholes, see inset in Fig. 2.32a) or via a co-axial pilot tunnel (Fig. 2.32b). The black lines in the diagrams in Figure 2.32 show the exact support pressure $s$ as a function of the initial hydraulic head $h_0$ and the cohesion $c$. The red lines were obtained from the design equation (2-41) using the coefficients $F_0$ to $F_3$ according to Figures 2.25 and 2.27. The difference between the exact and the approximate support pressure clearly increases with the hydraulic head. The error of design equation (2-41) can be compensated by increasing the coefficient $F_2$ by about 15% (see marked points in the diagrams in Fig. 2.32). The correction is satisfactory even for $c = 0$ or very high hydraulic heads ($h_0 > 25$). Several comparative computations with $n = 2$ to 10 boreholes (drainage layout in Fig. 2.25)
Figure 2.32.  Exact and approximate normalized support pressure $\bar{s}$ as a function of the normalized initial hydraulic head $\bar{h}_0$ and of the normalized cohesion $\bar{c}$ for advance drainage, (a), via boreholes from the face or, (b), a co-axial pilot tunnel.

Figure 2.33.  Exact and approximate normalized support pressure $\bar{s}$ as a function of the normalized cohesion $\bar{c}$ and of the normalized borehole diameter $d_{wh}/D$. 

Legend:
- exact computation
- nomograms (Fig. 2.25)
  - $F_1 = 0.162$, $F_2 = 0.207$
  - $F_1 = 1.954$, $F_2 = 0.007$
- nomograms (Fig. 2.25 but with $F_2 = 0.232$)
showed that increasing the coefficient $F_2$ by 10-20% (more specifically: 9% for $n = 2$, 12% for $n = 4-6$, 14% for $n = 8-10$) also makes it possible to assess face stability for $T/D < 5$. As indicated by the results in Figure 2.32b, this rule of thumb also applies to the other drainage layouts.

The nomograms were developed for a normalized borehole diameter $d_{dr}/D = 0.01$. This assumption is adequate for typical cross-sections of traffic tunnels and typical diameters of drainage boreholes (about 10 m and 10 cm, respectively), but leads to an under- or overestimation of the effects of advance drainage (and thus to an over- or underestimation of the necessary support pressure) in the case of smaller or larger tunnel cross-sections.

In order to quantify the error arising with normalized borehole diameters other than $d_{dr}/D = 0.01$, the example of a tunnel with advance drainage via 6 boreholes is considered (see inset in Fig. 2.33). The red solid line in the diagram in Figure 2.33 shows the required support pressure as a function of the cohesion under the nomograms. The black lines show the exact results for several values of the normalized borehole diameter $d_{dr}/D$. If the ratio $d_{dr}/D$ is extremely large (0.05, which would apply, e.g., to a very small tunnel diameter of 2 m and borehole diameter of 0.1 m), then the nomograms overestimate the necessary support pressure by up to $0.5γ'D$ (compare points D and B in Fig. 2.33).

However, the error soon becomes very small for $d_{dr}/D = 0.02$ (maximum $0.2γ'D$). On the other hand, if the ratio $d_{dr}/D$ is smaller than the one assumed by the nomograms, then they will slightly underestimate the necessary support pressure (by less than 4% in the case of $d_{dr}/D = 0.005$, see points A and B in Fig. 2.33). In conclusion, the accuracy of the approximate design equation (2-41) is acceptable in the range $d_{dr}/D = 0.005 - 0.02$, i.e. for most practical purposes (a borehole diameter of 10 cm and tunnel diameters of 5 to 15 m).

### 2.4.2.1 Error of the nomograms

The absolute error $Δ\bar{S}$ in the normalized required support pressure is defined as

$$Δ\bar{S} = \bar{S}_{\text{nomo}} - \bar{S}, \quad \text{for } \bar{S}_{\text{nomo}} > 0 \text{ and } 0 < \bar{S} < 4,$$

(2-46)

where $\bar{S}$ and $\bar{S}_{\text{nomo}}$ denote the normalized exact support pressure (Eq. (2-40)) and the approximate support pressure according to the nomograms (Eq. (2-41)), respectively. The relative error

$$ε_r = \frac{Δ\bar{S}}{\bar{S}}$$

(2-47)

provides additional insight for high support pressures $\bar{S}$, but might misleadingly result in high values at low support pressure.

For a given drainage scheme and fixed values of the coefficients $λ_w$ and $λ_p$ it is

$$Δ\bar{S} = f (ϕ, \bar{c}, \bar{h}_0)$$

(2-48)

(see text before Eq. (2-40)). Figures 2.34 to 2.37 show the absolute error $Δ\bar{S}$ as a function of the normalized head $\bar{h}_0$ for four ground cohesions $\bar{c}$. The relative error $ε_r$ is indicated for selected maxima. Each figure shows three values of angle of internal friction $ϕ$ (dotted, dashed and solid line); (a) to (g) corresponds to the cases of the nomograms in Figures 2.24 to 2.30.

In ground without any cohesion (Fig. 2.34), the nomograms tend to underestimate the support pressure at high normalized hydraulic head $\bar{h}_0$ ($Δ\bar{S} ≈ -0.2$ or $ε_r ≈ -4$% at high hydraulic head $\bar{h}_0$) and thus it is recommend not to use the nomograms. But already in ground with very low cohesion (Fig. 2.35), the
error $\Delta \bar{s}$ decreases with increasing hydraulic head from a slight overestimation ($\Delta \bar{s} \approx 0.2$ or $\varepsilon_r = 3\text{-}10\%$ at about $\bar{h}_0 = 4$) to nearly the exact support pressure ($\Delta \bar{s} \leq 0.05$ at $\bar{h}_0 = 25$). At higher ground cohesion (Fig. 2.36), the overestimation grows and with it the error, distinctively at high hydraulic heads. Drainage layouts of small effectiveness (e.g. $n = 2$ in Fig. 2.36a) overestimate the required support pressure (up to $\Delta \bar{s} = 0.8$ or $\varepsilon_r = 21\%$ at $\bar{h}_0 \approx 9$) due to their non-linearly behaviour in the cohesion $\bar{c}$ especially at low internal angle $\phi$. The error of the other drainage layouts is rather small and error sums up to at most $\Delta \bar{s} = 0.4$ ($\varepsilon_r = 8\text{-}17\%$). At even higher cohesion (Fig. 2.37), the absolute error stays below that range. For high hydraulic heads, the absolute error increases especially for very low friction angle and low drainage effectiveness. Thus for the very rare combination of tunnelling under high hydraulic pressure through high cohesive material of very low friction angle $\phi \leq 20^\circ$ (e.g. graphitic phyllite), the nomograms overestimate the support pressure by about 10% (see $\varepsilon_r$-values in Fig. 2.37), in case of low drainage effectiveness even by 23-40% (Fig. 2.37a).

Keeping in mind practical purposes, an error of $\Delta \bar{s} < 0.4$ can be neglected, as it depicts an un-normalized support pressure of less than 50 kPa for a common road tunnel and ground weight ($D \approx 10$ m, $\gamma' \approx 12$ kN/m$^3$). Thus according to Figures 2.35 to 2.37, the nomograms provide sufficient safe-side accuracy for common ground with at least some marginal cohesion.

### 2.4.3 Use of the nomograms

The use of the nomograms is straightforward: choose the applicable nomogram according to the intended drainage scheme (Figs. 2.24 to 2.30); read out the values of the coefficients $F_0$ to $F_3$; and calculate the required support pressure $s$ by means of design equation (2-37). Safety factors can easily be taken into account by using reduced shear strength parameters ($c, \tan\phi$) or a higher hydraulic head $h_0$.

Consider, for example, the problem of Section 2.3 (Fig. 2.5) with 6 axial drainage boreholes drilled from the face ($d_r/D = 0.01$, $l_r/D = 3$) and the ground parameters $\phi = 30^\circ$, $c = 100$ kPa, $\gamma' = 12$ kN/m$^3$ and $\gamma_w = 10$ kN/m$^3$. The applicable nomograms are given in Figure 2.25. For $\phi = 30^\circ$ and $n = 6$, the coefficients are: $F_0 = 0.15$, $F_1 = 1.9$, $F_2 = 0.21$ and $F_3 = 0.007$. Inserting these values in Eq. (2-37) results in a support pressure of 104 kPa.

The design equation (2-37) can be used in an inverse way to estimate the critical cohesion $c$ (i.e. the cohesion that would render face support measures unnecessary). Solving Eq. (2-37) with respect to $c$ for $s = 0$ results in a critical cohesion of 152 kPa for the example considered. If the cohesion is higher than this value, then advance drainage would suffice for face stability.
Figure 2.34. Error $\Delta \bar{\sigma}$ in the support pressure calculation with the nomograms as a function of the hydraulic head $\bar{h}_0$ for $c = 0$ and $\varphi = 20, 30, 40$ of: (a) Fig. 2.24, (b) Fig. 2.25, (c) Fig. 2.26, (d) Fig. 2.27, (e) Fig. 2.28, (f) Fig. 2.29; (g) Fig. 2.30
Figure 2.35. Error $\Delta s$ in the support pressure calculation with the nomograms as a function of the hydraulic head $\bar{h}_0$ for $c = 0.3$ and $\varphi = 20, 30, 40$ of: (a) Fig. 2.24, (b) Fig. 2.25, (c) Fig. 2.26, (d) Fig. 2.27, (e) Fig. 2.28, (f) Fig. 2.29; (g) Fig. 2.30
Figure 2.36. Error $\Delta \bar{s}$ in the support pressure calculation with the nomograms as a function of the hydraulic head $\bar{h}_0$ for $\bar{c} = 1$ and $\varphi = 20, 30, 40$ of: (a) Fig. 2.24, (b) Fig. 2.25, (c) Fig. 2.26, (d) Fig. 2.27, (e) Fig. 2.28, (f) Fig. 2.29; (g) Fig. 2.30
Figure 2.37. Error $\Delta s$ in the support pressure calculation with the nomograms as a function of the hydraulic head $\bar{h}_0$ for $\bar{c} = 2$ and $\varphi = 20, 30, 40$ of: (a) Fig. 2.24, (b) Fig. 2.25, (c) Fig. 2.26, (d) Fig. 2.27, (e) Fig. 2.28, (f) Fig. 2.29; (g) Fig. 2.30
2.5 Application examples

2.5.1 Albula tunnel

The planned Albula II railway tunnel will run at an axial distance of 30 m parallel to the historic Albula I tunnel, a UNESCO engineering landmark built about 110 years ago in Switzerland (Fig. 2.38). Albula I became famous because of the difficulties experienced during construction through the so-called rauwacke formation (Theiler et al., 2013). The latter consists of cellular dolomite, a weak rock exhibiting a porous, sponge-like structure with pore dimensions in the range of millimetres. Close to the boundary with the next geological unit, the rauwacke formation contains a network of what are probably tubular cavities with fine-grained infillings. The locally almost cohesionless ground in combination with the high water table (initially about 120 m above the alignment) resulted in instabilities with several instances of material inrush and tunnel flooding. Overcoming the rauwacke formation, which was only 110 m long, caused a delay of 11 months in construction.

On account of the previous tunnelling experience, the rauwacke formation is also expected to be challenging in the construction of Albula II. According to recent geological investigations, the infillings of the voids appear – when drained – to be “a stiff to weak, strongly silty sand with some gravel”, but under seepage conditions become flowing slurry (Sieber et al., 2014). Advance drainage therefore represents a construction option, alone or in combination with ground improvement by grouting or freezing, provided of course that the environmental impact of drainage can be accepted.

In the following paragraphs, we investigate the effectiveness of some possible drainage schemes with respect to the stabilization of the tunnel face by using the design equation (2-37). For this purpose, the egg-shaped cross-section of the planned tunnel is approximated by a circular cross-section of same area (44 m², diameter 7.5 m). The shear strength parameters for the ground depend on the degree of disintegration and on the fraction of voids with soft infillings. The friction angle $\phi$ of the rauwacke is in the range of 25-30°, while the cohesion amounts to 5-10 kPa for the infillings and to 500-1000 kPa for the rock (Sieber et al., 2014). The computations were carried out assuming overall values representing a weakly consolidated rauwacke formation with a high fraction of infillings ($\phi = 25^\circ$, $c = 50$ kPa). According to recent measurements, the current piezometric head is about 50 m above the tunnel. Due to the drainage action of Albula I, this value is probably lower than the undisturbed head that prevailed before its construction (estimated to 120 m).

Table 2.5 shows the coefficients $F_0$ to $F_3$ as well as the support pressure $s$ after Eq. (2-37). Without drainage measures the necessary support pressure amounts to 191 kPa (Table 2.5, row 1). Such high support pressure is barely feasible with face bolts: Consider, for example, a dense face reinforcement consisting of fully grouted bolts spaced at 1 m (i.e., a bolt density $n_b$ of 1 bolt/m²) and a wedge with an inclined slip surface forming an angle of $45^\circ-\phi/2 = 32.5^\circ$ to the vertical direction (Fig. 2.39). Where the bolts are sufficiently long, the limiting factor for the support force offered by the reinforcement is the anchorage length of the bolts inside the wedge (Anagnostou and Serafeimidis, 2007). For the wedge under consideration, the average anchorage length amounts to about 2.1 m. Taking the diameter $d_b$ of the grouted bolt boreholes equal to 0.1 m and the bond strength $\tau_m$ of the grout–ground interface equal to maximum 150 - 200 kPa, the support pressure offered by the reinforcement amounts at most to $n_b \cdot \pi \cdot d_b^2 \cdot \tau_m = 100 - 130$ kPa, which is considerably lower than the necessary support pressure.
Figure 2.38. Problem layout of application example Albula

Figure 2.39. Estimate of the support pressure provided by face reinforcement: (a) actual tunnel cross-section; (b) equivalent square cross-section; (c) geometry of the wedge in longitudinal section

Table 2.5. Coefficients $F_0$ to $F_3$ and resulting support pressure $s$ for the application example Albula

<table>
<thead>
<tr>
<th>Drainage layout</th>
<th>$F_0$</th>
<th>$F_1$</th>
<th>$F_2$</th>
<th>$F_3$</th>
<th>$s$ [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 None (Fig. 2.24, $n = 0$)</td>
<td>0.134</td>
<td>2.241</td>
<td>0.552</td>
<td>0.014</td>
<td>191</td>
</tr>
<tr>
<td>2 2 axis-parallel boreholes through the face (Fig. 2.24, $n = 2$, $l_{dr} = 1.5D = 11.3$ m)</td>
<td>0.154</td>
<td>2.145</td>
<td>0.377</td>
<td>0.016</td>
<td>103</td>
</tr>
<tr>
<td>3 4 axis-parallel boreholes through the face (Fig. 2.24, $n = 4$, $l_{dr} = 1.5D = 11.3$ m)</td>
<td>0.189</td>
<td>2.218</td>
<td>0.292</td>
<td>0.016</td>
<td>57</td>
</tr>
<tr>
<td>4 6 axis-parallel boreholes through the face (Fig. 2.24, $n = 6$, $l_{dr} = 1.5D = 11.3$ m)</td>
<td>0.199</td>
<td>2.251</td>
<td>0.268</td>
<td>0.013</td>
<td>43</td>
</tr>
<tr>
<td>5 Co-axial pilot tunnel of 3 m diameter (Fig. 2.27)</td>
<td>0.177</td>
<td>2.231</td>
<td>0.244</td>
<td>0.001</td>
<td>20</td>
</tr>
<tr>
<td>6 Face of 3 m diameter pilot tunnel $d_p$ (Fig. 2.24, $n = 8$, $l_{dr} = 1.5d_p = 4.5$ m)</td>
<td>0.216</td>
<td>2.331</td>
<td>0.232</td>
<td>0.008</td>
<td>4</td>
</tr>
<tr>
<td>7 Drainage curtains spaced at 7.5 m, each having 6 boreholes, executed from Albula I (Fig. 2.30)</td>
<td>0.190</td>
<td>2.391</td>
<td>0.191</td>
<td>0.005</td>
<td>0</td>
</tr>
</tbody>
</table>
According to rows 2 to 4 of Table 2.5, which apply to the case of advance drainage via 2 to 6 boreholes from the tunnel face, four drainage boreholes, each a minimum of 11 m long, would be sufficient to reduce the necessary support pressure to an acceptable level. A considerable reduction in the necessary support pressure could also be achieved by first excavating a 3 m diameter coaxial tunnel (Table 2.5, row 5). Stabilization of the pilot tunnel’s face would, nevertheless, require advance drainage by at least 8 boreholes (Table 2.5, row 6).

Alternatively, drainage of the ground ahead of the excavation face of Albula II tunnel could be carried-out by means of about 30 m long boreholes from the existing Albula I tunnel (Table 2.5, row 7). As the latter is lined by a stonework arch, the necessary support pressure can be estimated with the coefficients $F_0$ to $F_3$ according to Figure 2.29 (permeable lining). In the present case, we used Figure 2.30, which applies to an impermeable lining, because the drainage effect of the existing tunnel is taken into account by considering the current piezometric head (50 m) instead of the initial, undisturbed head. It should be noted that the support pressure computation with the nomograms in Figure 2.30 is sufficiently accurate in spite of the differences between the actual geometry and the geometry underlying the nomograms (smaller tunnel spacing, symmetrically arranged drainage curtains). This was confirmed by a numerical seepage flow analysis that was carried-out considering the actual geometry (Fig. 2.40). According to Table 2.5, row 7, drainage curtains (spaced at 7.5 m and each consisting of six boreholes) are sufficient for face stability. This solution would thus not only avoid the interference between excavation and drainage work, but also render unnecessary any other stabilization measures. On the other hand, the execution of drainage work from the existing tunnel would impose constraints on railway operations.

![Figure 2.40](image-url)  
Figure 2.40. Axonometric projection of the hydraulic head field of application example Albula in the presence of drainage curtains
2.5.2 Lake Mead Intake No. 3 Tunnel

The Lake Mead Intake Tunnel No. 3 belongs to Las Vegas’ water supply scheme. It is 4.7 km long and has a diameter of 7.22 m. Tunnel excavation was recently completed using a convertible hybrid TBM capable of boring in open mode or in closed mode as a slurry shield (McDonald and Burger, 2009). The ground consists of metamorphic and tertiary sedimentary rocks (conglomerates, breccias, sandstones, siltstones and mudstones of very variable quality) with several fault zones probably recharged directly from the lake (Fig. 2.41). The maximum hydrostatic pressure amounts to about 14 bar. Due to the lack of experience with closed-mode operation under such high pressures and the very poor local ground quality, it was necessary to design face stabilisation measures such as advance drainage and pre-excavation grouting that would allow for open mode operation.

Figure 2.41. Geological profile of the Lake Mead Intake No. 3 tunnel (after Feroz et al., 2007)

Figure 2.42. Lake Mead Intake No. 3 tunnel, Ch. 12+60 (point A): Support pressure $s$ as a function of the cohesion $c$
In the first part of the alignment through metamorphic rocks, considerable difficulties were encountered due to the unfavourable combination of high water pressure, extremely high rock permeability ($10^{-4}$ to $10^{-5}$ m/s) and the presence of an unexpected fault zone subparallel to the tunnel (Anagnostou, 2014). The fault, consisting of almost cohesionless material, made it necessary to operate in closed mode at 14 bar for several hundred metres. Attempts to bring the slurry pressure below the hydrostatic pressure resulted in extremely high, barely manageable quantities of water inflow (up to 1100 m$^3$/hour, Nicola et al., 2014). Advance drainage under such conditions was ineffective.

The sedimentary rocks exhibited a medium to low permeability ($10^{-6}$ to $10^{-10}$ m/s) and proved sufficiently stable at least in short term. The TBM was operated mainly in open mode in combination with 3 boreholes drilled through the cutter head for advance probing and drainage. The boreholes were 30-45 m long and overlapped by 10 m. In order to ensure stability during longer standstills (of more than two days), 2 to 3 additional boreholes of at least 10 m were drilled.

The advance drainage in the Lake Mead tunnel has considerably widened the feasibility range of open mode TBM drives and atmospheric interventions in the working chamber for inspection and maintenance. Related studies can be found elsewhere (Anagnostou et al. 2010, 2014). Here, we focus on one peculiarity of shielded TBMs in order to show a limitation of the proposed design nomograms. In a shielded TBM, water inflows occur not only from the tunnel face and the drainage boreholes, but also from the rock around the shield. The shield of the Lake Mead TBM is 14.87 m long, which means that the total area of the seepage face is 337 m$^2$ larger than the tunnel face (41 m$^2$). As larger seepage face areas obviously favour pore pressure relief, the question arises as to how much the nomograms (which assume no flow through the tunnel periphery) underestimate stability in such cases.

Figure 2.42 shows the relationship between the necessary support pressure $s$ and the cohesion $c$ for the following drainage cases: (a) from the tunnel face only (based on the nomograms in Fig. 2.25); (b) from the tunnel face and the shield area (based on a numerical seepage flow analysis taking account of the additional seepage face around the shield); (c) from the tunnel face and from four advance drainage boreholes (based on the nomograms in Fig. 2.25); (d) from the tunnel face, the shield area and four advance drainage boreholes (based on a numerical seepage flow analysis taking account of the additional seepage face around the shield as well as the exact locations, length and diameter of the boreholes according to the inset in Fig. 2.42).

Without the boreholes, drainage through the additional seepage face around the shield reduces the necessary support pressure by about 200 kPa or, for the given support pressure, the necessary cohesion by about 100 kPa (compare a to b in Fig. 2.42). In the case of advance drainage by 4 boreholes, disregarding the drainage in the shield area overestimates the necessary support pressure and cohesion by about 125 kPa and 60 kPa, respectively (compare c to d in Fig. 2.42). This example shows that the nomograms should be used with care where the seepage flow conditions are very different from the ones assumed by the nomograms.
2.6 Conclusions

Advance drainage greatly improves tunnel face stability because it reduces pore pressures and their gradients in the ground ahead of the face. The study in hand quantifies the effects of various advance drainage schemes by means of limit equilibrium computations which take account of the steady-state, three-dimensional seepage flow conditions prevailing in a homogenously permeable ground.

Four to six drainage boreholes (from the tunnel face or a niche) with a minimum length of one and a half tunnel diameters will generally be enough to reduce the necessary face support pressure significantly or even to render support unnecessary. The marginal utility of advance drainage diminishes for more or longer boreholes. The boreholes are more effective if they are located in the upper part and close to the periphery of the tunnel cross-section, but their exact positioning (roof or lateral) is not so important.

The drainage effect of a pilot tunnel (located either inside or outside the main tunnel cross-section) is also considerable. Even a very small diameter coaxial pilot tunnel brings so much pore pressure relief that the face of the main tunnel may be stable without additional auxiliary measures. Sparsely arranged drainage curtains (e.g. spaced at about one diameter intervals along the tunnel and consisting of two to four drainage boreholes each) improve the drainage effectiveness of adjacent tunnels.

In the case of twin tunnels, the drainage action of the first tube is important with respect to face stability of the second tube, even if the distance of the two tubes is relatively large (4-7 tunnel diameters).

A design equation has been developed which makes it possible to assess the stabilizing effect of drainage and study the various options rapidly, thus providing a very valuable design aid. The coefficients that appear in this equation depend on the friction angle and the geometric parameters for the drainage layout. They can be depicted using the dimensionless nomograms worked out by analysing the computational results of a comprehensive parametric study incorporating a wide parameter range and several different advance drainage schemes.

The nomograms provide an estimate of the required support pressure that is slightly on the safe-side for the practically relevant ranges of cohesion and support pressure \( (c/\gamma'D \geq 0.2, 0 \leq s/\gamma'D \leq 4) \) and up to very high hydraulic heads \( (\gamma_w h_w/\gamma'D \leq 30) \).

The nomograms assume that \( T = \min(H, H_w) > 5D \). This condition is fulfilled by subaqueous tunnels \( (H_w > H) \) with an overburden \( H \) of minimum 5 times the tunnel diameter \( D \) and by mountain tunnels \( (H_w < H) \) at a depth \( H_w \) of minimum 5\( D \) underneath the water table. For \( T/D < 5 \), the nomograms underestimate the support pressure. This can be compensated by increasing the coefficient \( F_2 \) roughly by 20\%. Of course, the nomograms are not applicable where there is no seepage flow.

The nomograms were developed for a normalized borehole diameter \( d_b/D = 0.01 \), which is typical for usual borehole and traffic tunnel diameters, but they are sufficiently accurate in the range \( d_b/D = 0.005 \text{–} 0.020 \), i.e. for most practical purposes.
3. Effectiveness of drainage measures for tunnel face stability in ground of non-uniform permeability

3.1 Introduction

Chapter 3 aims to improve understanding of the differences in face stability and drainage effectiveness between ground of uniform and non-uniform permeability, hereinafter referred to as “homogeneous” and “heterogeneous” ground, respectively. It analyses a series of formations, exhibiting heterogeneity at various scales and consisting of alternating horizontal or vertical aquifers and aquitards intersecting or being in close proximity to the tunnel face (Fig. 3.1). The performed analyses provide valuable information about whether and to what extent the required support pressure is higher or lower than in the case of homogeneous ground; which ground structures are critical for face stability and necessitate a higher support pressure; to which extent advance drainage does allow for reduction in support pressure; and where the drainage boreholes have to be arranged in order to be most effective.

Figure 3.1. Formations consisting of alternating aquifers and aquitards: hydraulic heterogeneity due to horizontal (a) and vertical (b) layers; (c) a single fault zone or (d) variation in the longitudinal direction due to intensive folding

Sections 3.2 to 3.5 investigate face stability in a horizontally stratified ground (Fig. 3.1a). Horizontal layers of variable permeability may be present in quaternary formations due to the sedimentation sequence; in sedimentary rocks (e.g. alternating layers of marl and sandstone); or – more rarely – in the case of sub-horizontal faults (such a fault was encountered, e.g. in the Bodio Section of the Gotthard Base Tunnel; Ferrari and Pedrazzini, 2008). A wide range of the heterogeneity scale characterized by layer thicknesses from decimetres to decametres is considered, first by analysing the case of tunnelling close to the horizontal interface of two differently permeable formations (Section 3.2). Then a single layer exhibiting a higher or a lower permeability than the surrounding ground will be considered, paying attention to the effect of its elevation and thickness (Section 3.3) and at variable elevation (Section 3.4). Finally, the case of alternating thin horizontal layers will be discussed, which can be treated as a homogeneous anisotropic medium (Section 3.5).

Sections 3.6 to 3.9 investigate cases of a permeability variation in the horizontal direction, which is caused by a sequence of practically vertical, alternating zones with different lithological or structural characteristics. This situation is encountered typically when approaching the interface of an aquifer with an aquitard (Fig. 3.1b), in fault zones (Fig. 3.1c) or in intensively folded formations (Fig. 3.1d). Single fault zones, consisting of crushed rock or soil-like material of low cohesion, if encountered
suddenly, may result in a large-scale instability and subsequent inundation of a long portion of the tunnel (Fig. 3.2a). For example, during construction of the Albula railway tunnel in the beginning of 20th century in the Swiss Alps, it took almost one year to overcome a 113 m long tunnel section through "a ground consisting of finest dolomite sand" (so-called "running ground") under a water pressure of about 12 bar (Hennings, 1908; Theiler et al., 2013); the water in this zone was infiltrating from all sides and washed out big quantities of sand. Considerable problems with water and mud inrushes occurred also in the Sampuoir section of the Engadine power station, where competent rock alternated with up to 2 m wide zones filled with loose material (Fig. 3.2b, König, 1968; Kovári and Anagnostou, 1996). A recent case of particularly demanding heterogeneous ground is that of the Lake Mead Intake No. 3 tunnel, where a fault striking almost parallel to the tunnel axis was encountered unexpectedly and made it necessary to realign the tunnel (e.g. Brierley Associates, 2011; Anagnostou et al., 2014; Nickerson et al., 2015; Smith, 2015). Faults occur alone or in a group (Fig. 3.3a and b, respectively) and are in some cases accompanied laterally by heavily jointed and fractured rock (referred to as "damage zone" by Faulkner et al., 2010; see Fig. 3.3), while in other cases the transition to the surrounding rock is sharp (Könz, 1968). They often exhibit permeability contrasts of several

**Figure 3.2.** (a) Collapse in water-bearing fault zone; (b) Water and mud inflows during construction of a tunnel for the Engadin power station at Sampuoir

**Figure 3.3.** Permeability distribution in a fault zone, (a), with a single core and, (b), with multiple cores (Faulkner et al., 2010)
orders of magnitude \((10^{-10^4}, \text{cf.}, \text{e.g.,} \ Evans \ et \ al., \ 1997)\), which result in considerable anomalies in the pore pressure distribution (Micarelli \ et \ al., \ 2006; Ganarod \ et \ al., \ 2008, Faulkner \ et \ al., \ 2010). In the case of faults in hard brittle rocks, the fault zones are often blocky, brecciated and due to the fracture connectivity more permeable than the surrounding rock. On the other hand, in intensively sheared and weathered rocks, the fault core will be mostly fine-grained (clayey or silty) and less permeable than its surroundings. Therefore, both low- and high-permeability fault zones will be investigated. Section 3.6 starts with an investigation of tunnelling close to the vertical interface of two differently permeable formations. An example of encountering a single vertical fault will be discussed in Section 3.7, while Section 3.8 will focus on face stability when entering the fault zone. Finally, the case of multiple vertical layers (including that of thin alternating layers, which can be modelled as a homogeneous, anisotropic medium) will be investigated (Section 3.9).

For all geological situations mentioned above, the effect of permeability heterogeneity on face stability will be quantified in terms of the support pressure that is required for stabilizing the face. The example of a subaqueous cylindrical tunnel will be studied, either without or with advance drainage measures according to Figure 3.4. The considered drainage layout of six axial boreholes was proved to be the most effective in homogeneous ground (Section 2.3.3.2).

As in Chapter 2, the estimation of the required face support pressure is based upon the model of Anagnostou and Kovári (1996). The seepage forces, which have to be introduced into the equilibrium equations, are determined by numerical three-dimensional, steady-state seepage flow analyses taking account of both the heterogeneity of the ground and the presence of the drainage boreholes, assuming that their hydraulic capacity allows water discharge under atmospheric pressure. The tunnel lining is taken impervious; no drawdown of water table is considered. For simplicity and as this Thesis focuses on hydraulic effects, uniform shear strength is assumed\(^4\) (with one exception in Section 3.8.2); keeping in mind that layers of different permeability may exhibit different shear strength as well. Table 3.1 summarizes the parameters assumed for the seepage flow and limit equilibrium analyses.

---

\(^4\) For formations with horizontal layers, also failure mechanisms involving only an individual layer were considered. However, as the shear strength of the ground is taken uniform, the failure mechanism that comprises the entire face requires the highest support pressure and thus was decisive.
### Table 3.1. Parameters for the comparative analyses

<table>
<thead>
<tr>
<th>Problem layout</th>
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<tbody>
<tr>
<td>Depth of cover</td>
<td>( H )</td>
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<tr>
<td>Elevation of water table</td>
<td>( H_w )</td>
</tr>
<tr>
<td>Tunnel diameter</td>
<td>( D )</td>
</tr>
<tr>
<td>Thickness of layer or zone</td>
<td>( d_L )</td>
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<table>
<thead>
<tr>
<th>Ground</th>
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<tr>
<td>Effective cohesion</td>
<td>( c )</td>
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<tr>
<td>Angle of eff. internal friction</td>
<td>( \varphi )</td>
</tr>
<tr>
<td>Submerged unit weight</td>
<td>( \gamma' )</td>
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<tr>
<td>Unit weight water</td>
<td>( \gamma_w )</td>
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</table>

<table>
<thead>
<tr>
<th>Shear resistance of the vertical slip surfaces</th>
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<td>Coefficient of lateral stress in wedge</td>
<td>( \lambda_w )</td>
</tr>
<tr>
<td>Coefficient of lateral stress in prism</td>
<td>( \lambda_p )</td>
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</table>

<table>
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<tr>
<td>Diameter</td>
<td>( d_{dr} )</td>
</tr>
<tr>
<td>Length</td>
<td>( l_{dr} )</td>
</tr>
<tr>
<td>Number</td>
<td>( n )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Permeability of the ground</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Permeability ratio (layer to host rock)</td>
<td>( k_i/k )</td>
</tr>
<tr>
<td>Degree of anisotropy</td>
<td>( k_p/k_n )</td>
</tr>
</tbody>
</table>

### 3.2 Tunnelling close to the horizontal interface of aquitard and aquifer

Assume tunnelling close to a horizontal permeability interface (Fig. 3.5a) when considering no or six advance drainage boreholes (red and blue lines, respectively). Figure 3.5b shows the support pressure, which is required for face stability, as a function of the distance between the tunnel axis and the interface of the two zones. The permeability contrast was taken equal to 100; the reference case of uniform permeability is also included in the diagram. For simplicity, the figures discussed in detail assume ground without any cohesion (denoted with support pressure \( s_0 \)). An increase in ground cohesion \( c \) might be approximated by the equations inside the diagram for support pressures \( s \). The roughly linear shift downwards of the curves is confirmed by Figure 3.6, which shows the results of a parametric study into the effect of the cohesion for the problem of Figure 3.5.

The heterogeneous structure of the ground has a remarkable effect on face stability, if the tunnel is closer than about to diameters to the interface of aquifer to aquitard. Two potentially critical situations are indicated with point A and B in Figure 3.5b. The corresponding hydraulic head fields are given in Figure 3.7.

In both cases A and B, the tunnel is located completely within the aquitard, but very close to the aquifer. In case A (aquitard above of aquifer), the interface of the two formations is at the tunnel floor (case A in Fig. 3.7). In case B (aquifer above of aquitard), the interface of the two formations is located at the tunnel crown (case B in Fig. 3.7). As the tunnel does not intersect the aquifer and the
Figure 3.5. (a) Two differently permeable zones with horizontal interface and (b) required support pressure $s_0$ as a function of the distance $z_I$ in ground without any cohesion $c$ (permeability contrast $k_{\text{upper}}/k_{\text{lower}} = 0.01, 1, 100$; blue: no drainage measure; red: six axial boreholes according to Fig. 3.4; other parameters according to Table 3.1)

Figure 3.6. Results of a parametric study into the effect of ground cohesion $c$ in the problem of Figure 3.5
Figure 3.7. (a) Distribution of the hydraulic head $h$ along two lines above and ahead of the tunnel face for two horizontal zones of different permeability (cases A and B of Fig. 3.5). Belonging surface plots of the hydraulic head field as well as required support pressure $s$ (b) without advance drainage measures and (c) with six axial drainage boreholes ($c = 50$ kPa, other parameters according to Fig. 3.4 and Table 3.1)

latter is by factor 100 more permeable than the aquitard; the pore pressure within the aquifer remains practically hydrostatic in spite of the drainage action of the tunnel face and of the boreholes (compare head fields in Fig. 3.7b and Fig. 3.7c, respectively). Consequently, the hydraulic head at the interface of the two zones is practically equal to the initial head and since the interface is located close to the tunnel, the pore pressure inside the aquitard and around the face are higher than in the case of homogeneous ground (compare also solid to dashed lines in Fig. 3.7a).

The distance between the red and blue lines in Figures 3.5 and 3.7 represents a measure of the effectiveness of the drainage boreholes in terms of support pressure and pore pressure relief, respectively. In homogeneous ground, the drainage boreholes would cause a pore pressure relief by about 75% in front of the face and reduce also the hydraulic head gradient above the tunnel (Fig. 3.7a), which results in a decrease in the required support pressure by about 450 kPa (Fig. 3.5). In heterogeneous ground, the drainage boreholes reduce the required support pressure by 250 - 500 kPa, i.e. as much as an increase in cohesion by 100 – 200 kPa (e.g. by grouting; Fig. 3.5 and Fig. 3.6). However, their effectiveness is lower if they are drilled within the aquitard and the aquifer is either just above the tunnel (point B in Fig. 3.5) or just underneath the boreholes (between point A and C in Fig. 3.5). Boreholes reaching the aquifer would be significantly more effective (see later Section 3.4.2.3).
3.3 A single, horizontal aquifer or aquitard symmetric to the tunnel axis

Complexity of permeability heterogeneity increases when considering one high- or low-permeability layer of variable thickness on the tunnel axis (Fig. 3.8a). Figure 3.8b shows the required support pressure $s_0$ for face stability as a function of the normalized layer thickness $d_L$ in ground without any cohesion; the roughly linear shift for increasing ground cohesion $c$ is shown in Figure 3.9.

A single high- or low-permeability layer has a considerable effect on the required face support pressure even if it is relatively thick (Fig. 3.8b): the support pressure tends to the value of homogeneous ground only for very large layers ($d_L/D > 7$). The heterogeneity effect is biggest when the layer thickness is equal to the tunnel diameter (cases A and B in Fig. 3.8; the corresponding hydraulic head fields are given in Fig. 3.10).

Overall, a low-permeability layer increases support demand compared to homogeneous ground, while a high-permeability layer is more favourable for stability. The aquitard acts as a hydraulic barrier and the hydraulic head is higher than in homogeneous ground both ahead and above of the tunnel face (compare dashed to solid lines in Fig. 3.10a and surface plots of case A in Fig. 3.10b and c). The aquifer acts as a natural advance drainage, which reduces the hydraulic head in relation to homogeneous ground both ahead and above of the tunnel face (compare dash-dotted to solid lines in Fig. 3.10a and surface plots of case B in Fig. 3.10b and c). It is remarkable that in the absence of drainage boreholes, even a very thin high-permeability layer results in a significant reduction of the required support pressure (case C in Fig. 3.8b).

Drainage boreholes reduce the required face support pressure by 300 – 500 kPa depending on the layer thickness (compare red to blue lines in Fig. 3.8). In case of a low-permeability layer, the drainage boreholes cause an additional pore pressure relief only inside the layer (case A in Fig. 3.10c). The hydraulic head in the surrounding aquifer is not affected by the drainage boreholes (compare red and blue dashed lines in l.h.s. of Fig 3.10a) and is clearly higher than in homogeneous ground (compare dashed to solid blue lines in Fig. 3.10a). Therefore, the support pressure is very high (upper blue line in Fig. 3.8b). Note that another arrangement of boreholes (such as they reach the aquifer) would increase effectiveness of the drainage measure (see later Section 3.4.2.3).

In the presence of a high-permeability layer, the additional pore pressure relief and support pressure reduction due to the drainage boreholes is smaller than in homogeneous ground, because the aquifer layer acts as a natural advance drainage, which anyway reduces pore pressures and face support pressure (compare case B in Fig. 3.10b to 3.10c).
Figure 3.8. (a) Single horizontal layer coaxial to the tunnel axis and (b) required support pressure $s_0$ as a function of the thickness of the layer $d_L$ in ground without any cohesion $c$ (permeability contrast $k_L/k = 0.01, 1, 100$; blue: no drainage measure; red: six axial boreholes according to Fig. 3.4; other parameters according to Table 3.1)

Figure 3.9. Results of a parametric study into the effect of ground cohesion $c$ in the problem of Figure 3.8
3.4 A single horizontal layer of variable elevation and thickness

3.4.1 Support pressure and hydraulic head field

Permeability heterogeneity of a single horizontal layer is characterized by the permeability ratio \( k_L/k \), the layer thickness \( d_L \) and elevation \( z_b \) (Fig. 3.11a). Figure 3.11 shows the results of a parametric study into these effects on the required support pressure as a function of the permeability ratio \( k_L/k \) for a 2 m and a 5 m thick layer, respectively (for ground with some cohesion see Figure 3.12). The markers indicate different ground models of layer elevation from tunnel invert to slightly above the roof (layers located outside that range have a negligible influence on face stability; see also Fig. 3.5 or Fig. 3.8). The ground models, which can be summarized as unfavourable (A, B) and favourable (C, D) concerning face stability, are sketched for each extremal permeability ratio in Figure 3.11b.

In case of a low-permeability layer (\( k_L/k < 1 \)), all ground models require roughly the same support as in homogeneous ground. Distinctive exception is the case of advance drainage with a barrier layer at...
the roof, where a considerably higher support pressure is necessary (compare blue A to E in Fig. 3.11c and d). In case of a high-permeability layer \( (k_L/k > 1) \), a clearly lower support pressure than in homogeneous ground is required in case where the layer is within the tunnel face (compare e.g. D to E in Fig. 3.11c and d). On the other hand, remarkable more support pressure than in homogeneous ground is required in case of lacking connection of the layer to a seepage face (compare e.g. B to E in Fig. 3.11). Assuming for example a ground of cohesion \( c = 150 \text{kPa} \) and considering 6 drainage boreholes, the tunnel face in homogeneous ground would be stable, while in case of a 2 m thick high-permeability layer at the tunnel roof, still 200 kPa support pressure would be required (Fig. 3.12e).

The hydraulic head field of the (un-)favourable cases concerning support pressure (A to E in Fig. 3.11) are given in Figures 3.13 and 3.14. As the typical characteristics do not change with increasing layer thickness (the thicker the layer, the more pronounced is the effect of permeability contrast; compare Fig. 3.11c to d), we limit our discussion below to a 2 m thick layer.

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**Figure 3.11.** (a) Parametric study of a single horizontal layer of variable elevation and thickness. (b) Unfavourable and favourable ground models concerning face stability. Required support pressure \( s_0 \) as a function of the permeability contrast \( k_L/k \) in ground without any cohesion \( c \) for (c) a 2 m thick layer and (d) a 5 m thick layer (blue: no drainage measure; red: six axial boreholes according to Fig. 3.4; other parameters according to Table 3.1)
Figure 3.12. Results of a parametric study into the effect of ground cohesion $c$ in the problem of Figure 3.11
### 3.4.1.1 Layer at the tunnel roof

A low-permeability layer at the tunnel roof \( (k_L/k < 1) \) hinders pressure relief above the tunnel face (compare dashed to solid lines in Fig. 3.13a). In case without advance drainage, the barrier-effect of the low-permeability layer is of subordinate importance with respect to the required support pressure, because the hydraulic head distribution ahead of the face is practically the same as in homogeneous ground (case A and E in Fig. 3.13b). In case with advance drainage boreholes however, the required support pressure is significantly higher than in homogeneous ground, because the boreholes are not able to relieve the pressure above of the aquitard (case A and E in Fig. 3.13c).

A high-permeability layer at the tunnel roof \( (k_L/k < 1) \) acts as natural drainage and is favourable for both cases with and without drainage. Especially compared to no advance drainage measure, the drainage boreholes allow for a drastic reduction of the required support pressure by about 85% (from 403 to 58 kPa; case D in Fig. 3.13b and c).

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**Figure 3.13.** (a) Distribution of the hydraulic head \( h \) along two lines above and ahead of the tunnel face in the case of a 2 m thick layer at the tunnel roof (cases A, D and E of Fig. 3.11c). Belonging surface plots of the hydraulic head field as well as required support pressure \( s \) (b) without advance drainage measures and (c) with six axial drainage boreholes \( (c = 50 \text{ kPa}, \text{ other parameters according to Fig. 3.4 and Table 3.1}) \).
3.4.1.2 Layer just above the tunnel roof

Compared to the hydraulic head field in homogeneous ground, a low-permeability layer above the tunnel roof \((k_L/k < 1)\) is favourable ahead, but not above of the tunnel face (compare dashed to solid lines in Fig. 3.14a). The pore pressure dissipates mostly within the layer and the hydraulic head above the aquitard is practically equal to the initial hydraulic head. However for face stability considerations, the distance of the hydraulic gradients to the tunnel face is large enough and the support pressures required are similar to the ones required in homogeneous ground (compare cases C to E in Fig. 3.14).

A high-permeability layer \((k_L/k > 1)\) just above the tunnel roof cannot act as natural drainage due to the lack of connection to a seepage face, be that the drainage boreholes or the tunnel face (case B in Fig. 3.14). The hydraulic head is higher than in homogeneous ground both ahead and above the tunnel face (compare dash-dotted to solid lines in Fig. 3.14a) and thus significant more support pressure is necessary for face stability (case B in Fig. 3.11).

![Figure 3.14.](image)

(a) Distribution of the hydraulic head \(h\) along two lines above and ahead of the tunnel face in the case of a 2 m thick layer just above the tunnel roof (cases B, C and E of Fig. 3.11c). Belonging surface plots of the hydraulic head field as well as required support pressure \(s\) (b) without advance drainage measures and (c) with six axial drainage boreholes \((c = 50\) kPa, other parameters according to Fig. 3.4 and Table 3.1)
3.4.2 Optimizing the drainage borehole layout

The effectiveness of advance drainage can be increased by optimizing the borehole layout such as at least some boreholes intersect the more permeable layer. In fact, by shifting two of the lower boreholes upwards into the overlying aquifer (by drilling from a niche as in Fig. 3.15b, or by drilling steeply inclined boreholes as in Fig. 3.15c), the hydraulic barrier effect of the aquitard can be defused. In case of a low-permeability layer in the tunnel face (central column in Fig. 3.15; corresponding to case A in Fig. 3.11c and Fig. 3.13c), about half the support pressure of the standard drainage layout is required (216-239 instead of 413 kPa; Fig. 3.15). In case of a high-permeability layer without connection to a seepage face (right column in Fig. 3.15; corresponding to case B in Fig. 3.11c and Fig. 3.13c), the support pressure decreases even more (from 415 to 69-127 kPa; Fig. 3.15) and is lower than the support pressure required in homogeneous ground (of 191 kPa). Thus for both unfavourably situated low- and high-permeability layer, a wise borehole arrangement levels the support pressure needed to about the values required in homogeneous ground.

Figure 3.15. Surface plots of the hydraulic head field as well as required support pressure \( s \) for the cases A and B of Figure 3.11c (a) with the standard drainage layout, (b) with drainage from a niche (c) with inclined drainage boreholes (other parameters according to Fig. 3.4 and Table 3.1)
3.4.3 Application example

The practical significance of these results is discussed for an exemplary tunnel in ground of very low cohesion \( (c = 50 \text{ kPa}) \). A marginally inclined, 2 m thick layer crosses the alignment such as the previously discussed ground models apply with sufficient accuracy (longitudinal section in Fig. 3.16a). The required support pressure \( s \) for face stability when approaching a high-permeability layer is shown in Figure 3.16b; approaching a low-permeability layer is plotted in Figure 3.16c. Each figure shows the support pressure required when considering no advance drainage measure (red line), advance drainage with six axial boreholes (blue) and advance drainage with optimized borehole location (green; two boreholes arranged in the more permeable ground as discussed in Section 3.4.2).

![Figure 3.16](image)

When crossing a high-permeability layer without advance drainage measures, an overall very high support pressure is required, which shows high sensitivity to the variability of the ground \( (347 - 769 \text{ kPa}; \text{red in Fig. 3.16b}) \). As a face support pressure of about 200 kPa would yet necessitate heavy face reinforcement (grey area in Fig. 3.16b) and pressures of more than 300 kPa cannot be materialized (Section 2.5.1), auxiliary measures such as grouting or freezing would be necessary to

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Figure 3.16. (a) Longitudinal section of a tunnel crossing a marginally inclined, 2 m thick layer in ground of cohesion \( c = 50 \text{ kPa} \). Required support pressure \( s \) (b) when approaching a high-permeability layer and (c) when approaching a low-permeability layer (blue: no drainage measure; red: six axial boreholes according to Fig. 3.4; green: optimized drainage layout with two boreholes shifted in the more permeable ground; other parameters according to Table 3.1)
stabilize the face. Advance drainage by axial boreholes from the face would reduce the required support pressure mostly to a technically manageable level (58 - 415 kPa; blue in Fig. 3.16b). Optimizing the borehole arrangement by drilling two boreholes in the more permeable ground would finally decrease the support overall to a feasible range (57 – 243 kPa; green in Fig. 3.16b).

The support pressure when crossing a low-permeability layer without advance drainage measures is less variable, but yet too high for conventional tunnelling (527 - 673 kPa; red in Fig. 3.16c). Advance drainage with axial boreholes decreases the required support pressure down to the feasible range of face bolts with only exception of the previously discussed situation of a barrier layer in the tunnel face (141 - 413 kPa; blue in Fig. 3.16c). Optimizing the borehole arrangement and dewatering the permeable ground above the layer, also these pressures would be reduced to a technically manageable level (59 – 191 kPa; green in Fig. 3.16c).

### 3.5 Thinly interbedded horizontal aquifers and aquitards

#### 3.5.1 Homogenisation to an equivalent anisotropic model

Sedimentary deposits (heterogeneous quaternary soils or sedimentary rocks) have often a considerably higher permeability in the horizontal than in the vertical direction. If the layer thickness is small in relation to the size of the tunnel cross-section, the ground can be considered as a homogeneous medium of anisotropic permeability. The equivalent permeability parallel and normal to the strata are (e.g. Bear et al., 1968):

\[
\begin{align*}
 k_p &= a_1 k_1 + a_2 k_2 , \\
 k_n &= \left( \frac{a_1 + a_2}{k_1 k_2} \right)^{-1},
\end{align*}
\]

where \( a_i \) and \( k_i \) denote the fraction and the permeability of the layer \( i \), and are considered in an orthotropic permeability matrix. The steady state hydraulic head field depends on the degree of anisotropy, expressed by the ratio

\[
\frac{k_p}{k_n} = 1 + a_i (1 - a_i) \left( \frac{k_1}{k_2} + \frac{k_2}{k_1} - 2 \right),
\]

but not on the individual values of \( k_p \) and \( k_n \).

#### 3.5.2 Maximum layer thickness

The equivalent homogeneous anisotropy model allows neglecting numerical expensive, discretely depicted layers in an FE-model. In order to determine the maximum layer thickness for which homogenisation to an equivalent homogeneous anisotropy model is possible, Figure 3.17 shows the required face support pressure \( s \) as a function of the normalized layer thickness \( d/D \) (ground without any cohesion). The results of stratified media consisting of discretely modelled layers are indicated as black crosses (inset in Fig. 3.17a); the equivalent anisotropic medium is shown in red.

The range of required support pressure (which is due to the upmost layer at the face acting as a hydraulic barrier or as a natural drainage) decreases with decreasing layer thickness (highlighted in
grey in Fig. 3.17b). The face support needed in an equivalent anisotropic medium \((s = 555 \text{ kPa})\) is nearly identical to the one required when modelling discrete layers of adequately thin strata \((s = 514 - 610 \text{ kPa} \text{ for } d/D = 0.2; \text{ Fig. 3.17b})\). The support pressure for thicker layers deviates more than 10\% from the pressure required considering the equivalent anisotropic permeability model and is not safe-side.

![Figure 3.17](image)

**Figure 3.17.** (a) Multiple horizontal layers and (b) required support pressure \(s\) as a function of the layer thickness \(d\) in ground without any cohesion \(c\) (black: discretely modelled layers of permeability contrast \(k_L/k = 0.01 \text{ and } 100\); red: equivalent anisotropic model; other parameters according to Table 3.1)

### 3.5.3 Effect of permeability anisotropy

The effect of permeability anisotropy on face stability is discussed for an increasing degree of anisotropy (Fig. 3.18a). Figure 3.18b shows the required support pressure \(s_0\) as a function of the permeability ratio \(k_1/k_2\) and the degree of anisotropy \(k_p/k_n\) for cohesionless ground (Fig. 3.19 shows the results of a parametric study into the effect of the cohesion in the problem of Fig. 3.18). The solid red line applies to no advance drainage measures, the blue to advance drainage with six axial boreholes. The dotted lines indicate the support pressure required in isotropic ground. Figure 3.18c shows the belonging distribution of the hydraulic head \(h\) along two lines above and ahead of the tunnel face.

#### 3.5.3.1 Without advance drainage measure

The required support pressure decreases with increasing degree of anisotropy (red in Fig. 3.18b). An increase of \(k_p/k_n\) from 1 to 10 (which corresponds to a permeability ratio of \(k_1/k_2 = 38\) in a thin-layered ground) leads to a support pressure reduced by \(\Delta s = 178 \text{ kPa}\). However, the required support pressure is far above the feasible range of about 200 kPa (cf. Section 2.5.1). Yet in a ground of higher cohesion \((e.g. \ c = 250 \text{ kPa}; \text{ Figure 3.19})\), no face support would be required due to this moderate permeability anisotropy.
Figure 3.18. (a) Permeability anisotropy of the equivalent homogeneous model representing very thin horizontal layers. (b) Required support pressure $s$ as a function of the permeability ratio $k_1/k_2$ and the degree of anisotropy $k_p/k_n$ at logarithmic scale in ground without any cohesion $c$ and (c) belonging distribution of the hydraulic head $h$ along two lines above and ahead of the tunnel face (blue: no drainage measure; red: six axial boreholes according to Fig. 3.4; other parameters according to Table 3.1)

The favourable effect of permeability anisotropy is due to the pore pressure dissipating mainly in the lower permeable direction. Indeed, the hydraulic head above of the tunnel is higher than in the isotropic case (compare red lines in l.h.s. of Fig. 3.18c). But the seepage area of the tunnel face, perpendicular to the more permeable direction, favours the pore pressure relief ahead of the face (compare red lines in r.h.s. of Fig. 3.18c) and leads to lower support pressures.

3.5.3.2 With advance drainage boreholes

The support pressure for face stability slightly increases with increasing degree of anisotropy (blue in Fig. 3.18b). However, the increase is small ($\Delta s = 32$ kPa for an increase of $k_p/k_n$ from 1 to 10) and the required support pressure is still remarkably lower than without boreholes (compare blue to red lines in Fig. 3.18b).
When considering anisotropic permeability, the hydraulic head above of the tunnel is higher than in the isotropic case (blue lines in l.h.s. of Fig. 3.18c). But compared to without drainage measure, trend reverses ahead of the tunnel, where the hydraulic head is also higher as the isotropic case (blue lines in r.h.s. of Fig. 3.18c). Due to the lower permeability in vertical direction, the horizontal seepage faces of the drainage boreholes are less efficient than in isotropic ground.

Figure 3.19. Results of a parametric study into the effect of ground cohesion $c$ in the problem of Figure 3.18

3.6 Tunnelling close to the vertical interface of an aquitard or an aquifer

Assume tunnelling close to a vertical permeability interface of an aquifer and an aquitard (Fig. 3.20a). Figure 3.20b evaluates the required support pressure $s_0$ as function of the distance of permeability-interface to tunnel face $x_f$ in cohesionless ground (for ground of higher cohesion see Figure 3.21). Again, the permeability contrast was taken equal to 100; the case of uniform permeability is added for comparison.

Without advance drainage, the required support pressure is highly sensitive to permeability heterogeneity if the tunnel face is close to the permeability-interface ($3 \geq x_f/D \geq -1$ for red line in Fig. 3.20b). Approaching an aquifer requires a distinctive higher support pressure (e.g. point A in Fig. 3.20b) than approaching an aquitard (point B in Fig. 3.20b). The maximum support pressure of 1.6 times the value required in homogeneous ground is necessary if the aquifer is very close the tunnel face ($x_f/D = 0.1$ in Fig. 3.20b).
Figure 3.20. (a) Two differently permeable zones with vertical interface and (b) required support pressure \( s_0 \) as a function of the distance \( x_f \) in ground without any cohesion \( c \) (permeability contrast \( k_z / k = 0.01, 1, 100 \); blue: no drainage measure; red: six axial boreholes according to Fig. 3.4; other parameters according to Table 3.1)

Comparison of support pressure:

- \( n = 0 \) (a):
  - approximation \( s = s_0 - 2.5 c \)
  - exact values

- \( n = 6 \) (b):
  - approximation \( s = s_0 - 2.0 c \)
  - exact values

Figure 3.21. Results of a parametric study into the effect of ground cohesion \( c \) in the problem of Figure 3.20
Advance drainage boreholes reduce the sensitivity to variability of the ground remarkably (blue lines in Fig. 3.20b). The required support pressure is nearly the same as in homogeneous ground and has small peaks only when the interface is at or just behind the tunnel face ($0 \leq x_{ff}/D \leq -1$).

The hydraulic head field of the potential critical situations indicated with A and B in Figure 3.20 are given in Figure 3.22.

When the tunnel face without advance drainage gets close to an aquifer, the face itself is located within the lower permeable zone, where most of the pore pressure dissipates (case A in Fig. 3.22b). The hydraulic head at the interface of the two zones is virtually equal to the initial head (red dashed line in r.h.s. of Fig. 3.22a). This leads to unfavourable hydraulic gradients and a pronounced higher required support pressure than in homogeneous ground. The hydraulic head distribution is by far more favourable when the tunnel face gets close to an aquitard (case B in Fig. 3.22b), where the pore pressure dissipates mainly within the aquitard itself and the hydraulic head is lower above and particularly ahead of the tunnel face (compare dash-dotted with solid red lines in Fig. 3.22a).
Because the advance drainage boreholes pierce both zones of different permeability, they lead to a uniformly favourable hydraulic head field in vicinity of the tunnel face, independently of approaching and aquifer or an aquitard (Fig. 3.22c). The hydraulic head distribution both ahead and above the tunnel face is about the same for cases A and B or homogeneous ground (compare blue lines in Fig. 3.22a) and thus the required support pressure is hardly affected by permeability heterogeneity.

3.7 Encountering a vertical fault zone

As the main findings of encountering a fault zone are similar to the previously discussed case of approaching an aquifer or an aquitard (Section 3.6), we limit ourselves to a representative example. Consider a 10 m thick fault zone in 15 m distance to the tunnel face when tunnelling without and with advance drainage measures (red and blue lines in Fig. 3.23a, respectively). Figure 3.23a shows the distribution of the hydraulic head \( h \) along two lines above and ahead of the tunnel face for a permeability contrast of 100 and the reference case of uniform permeability. Figures 3.23b and c show the surface plots of the hydraulic head field as well as the necessary face support pressure \( s \) when assuming ground cohesion of 50 kPa.

Figure 3.23. (a) Distribution of the hydraulic head \( h \) along two lines above and ahead of the tunnel face when encountering a highly/poorly permeable fault zone. Belonging surface plots of the hydraulic head field as well as required support pressure \( s \) (b) without advance drainage measures and (c) with six axial drainage boreholes \((c = 50 \text{ kPa}, \text{other parameters according to Fig. 3.4 and Table 3.1})\)
When encountering a low-permeability fault \((k_L/k = 0.01)\) without advance drainage measures, nearly 30% of the pore pressure relief takes place within the fault itself (compare dashed to solid red lines r.h.s. of Fig. 3.23a). The hydraulic head distribution both above and ahead of the face is favourable compared to homogenous ground and thus a lower support pressure is required (547 to 635 kPa in Fig. 3.23b). On the other hand, when encountering a high-permeability fault \((k_L/k = 100)\), pore pressure relief takes place between fault and face; the hydraulic head is in the fault itself equal to the initial value (compare dash-dotted red lines r.h.s. of Fig. 3.23a). The required support pressure is thus higher than in homogeneous ground (736 to 635 kPa in Fig. 3.23b; note that the value of the support pressure depends on the distance to the permeability-boundary, cf. Section 3.6.1).

Because the advance drainage boreholes pierce both fault zone and host rock, a uniformly head field over both formations develops (Fig. 3.23c). The hydraulic head distribution both ahead and above the tunnel face is similar for a low- and high-permeability fault as well as for homogeneous ground (compare blue lines in Fig. 3.23a) and therefore, the required support pressure is nearly constant (187 - 200 kPa in Fig. 3.23c).

3.8 Entering a single vertical zone

3.8.1 Hydraulic head field

When entering a single vertical fault zone, the hydraulic head field depends on the zone thickness \(d_L\), the ratio of permeability of zone to host rock \(k_L/k\) and on the drainage measures, especially on the length of the boreholes \(l_{dr}\) (Fig. 3.24a). The distribution of the hydraulic head is discussed in Figure 3.24, considering a representative example of entering a 10 m thick fault zone without drainage measures (red lines) and with six advance drainage boreholes of variable length (blue lines in Fig. 3.24a).

3.8.1.1 Without advance drainage measures

As the head difference between the far field and tunnel face dissipates mainly within the less permeable zone, a low-permeability fault (red dashed line in Fig. 3.24a) “attracts” a higher head gradient than a high-permeability fault (red dash-dotted line in Fig. 3.24a). The narrower the lower-permeability zone, the higher the gradient and the more adverse will be the situation.

For the example a 10 m thick low-permeability zone \((k_L/k = 0.01)\) in Fig. 3.24b), the average head gradient in the fault is by 30% higher than in homogenous ground (or in terms of required support pressure to 836 instead of 635 kPa; Fig. 3.24b). In case of the high-permeability zone \((k_L/k = 100)\) in Fig. 3.24b), pore pressure relief takes places mainly ahead of the fault zone, the average gradient in the fault is by 40% lower than in homogenous ground and hence favourable in terms of support pressure (332 to 635 kPa in Fig. 3.24b).

3.8.1.2 With advance drainage boreholes

In case of a low-permeability zone (blue dashed lines in Fig. 3.24a), the drainage boreholes must reach at least 1-2 m in the rock behind the fault in order to reduce the water pressure acting on the zone. If they do not intersect the interface to the more permeable rock, they have nearly no effect (compare dark to light blue lines in Fig. 3.24a and l.h.s. of Fig. 3.24c to d and e).

In the case of a high-permeability zone (blue dash-dotted lines in Fig. 3.24a), the pore pressure dissipates mainly within the rock ahead of the fault. Therefore, the borehole length is less important.
Figure 3.24. (a) Distribution of the hydraulic head $h$ along two lines above and ahead of the tunnel face when entering in a fault zone of different permeability. Belonging surface plots of the hydraulic head field as well as required support pressure $s$ (b) without advance drainage measures; with six axial drainage boreholes of length (c) $l_{dr} = 30$ m; (d) $l_{dr} = 10$ m and (e) $l_{dr} = 5$ m ($c = 50$ kPa, other parameters according to Fig. 3.4 and Table 3.1)
(and partially superimposed by the drainage effect of the tunnel face). The boreholes should intersect the interface of the fault zone, but they do not offer any value for face stability if extending deeper into the surrounding (compare r.h.s. of Fig. 3.24c to d and e).

Note that even in homogeneous ground, a minimum drainage length (of about 1.5 times the tunnel diameter; compare central column of Fig. 3.24c to d and e and cf. Section 2.3.3.3) should be provided at any time in order to avoid face instabilities due to an insufficient shift of the maximum hydraulic gradients into the ground.

### 3.8.2 Support pressure

#### 3.8.2.1 Unstable or stable neighbouring rock

In contrast to the previous sections, we investigate both cases of neighbouring stable and unstable rock when entering a vertical fault zone. Thus the stability conditions are different from those prevailing in homogeneous ground not only due to the anomaly of the hydraulic head distribution that is induced by the permeability variation, but also because the extent of the potentially unstable region ahead of the face might be limited by the thickness of the zone.

The effect of the thickness of the zone $d_L$ is explained by means of Figure 3.25 showing the support pressure $s$ needed in the reference case of homogeneously permeable ground ($c = 0$; other ground parameters see Table 3.1).

![Figure 3.25. Required support pressure $s$ as a function of the vertical layer thickness $d_L$ in ground without any cohesion $c$ (homogeneous permeability; no drainage measure; other parameters according to Fig. 3.4 and Table 3.1)](image)

**3.8.2.1.1 Unstable neighbouring rock**

Entering a fine-grained fault zone in a highly fractured rock might be considered as homogeneous ground from mechanical point of view, as both rock and fault have the same ground parameters $c, \varphi$.

The neighbouring rock is unstable and the failure angle $\omega$ can develop over both fault zone and rock (upper insets in Fig. 3.25). The required support pressure always reaches its maximum value $s_{\text{max}}$ as both rock and fault zone fail (solid line in Fig. 3.25).
3.8.2.1.2  Stable neighbouring rock

Entering a fault surrounded by stable rock, the required support pressure for face stability is not only determined by the hydraulic head distribution, but also by the geometry of the fault (e.g. entering a brecciated fault zone in a competent, solid rock with different ground parameters $c$, $\varphi$; lower insets in Fig. 3.25). The wedge angle $\omega$ is limited by the thickness of the fault zone ($\omega \leq \arctan(d_L/D)$) and thus the support pressure cannot reach its maximum $s_{\text{max}}$ (dotted line in Fig. 3.25). However this is true only if the fault zone is narrower than a critical thickness (indicated by the cross symbol in Fig. 3.25). For thicker zones, the thickness ceases to play a role and the support pressure reaches the maximum value $s_{\text{max}}$.

3.8.2.2  Parametric study

Figure 3.26 shows the results of a parametric study into the effects of permeability heterogeneity and stable or unstable neighbouring rock on the required face support $s_0$ when entering a single vertical zone of variable thickness $d_L$ in cohesionless ground (Figure 3.27 shows the results of a parametric investigation into the effect of the cohesion). The cases of unstable and stable neighbouring rock are plotted with solid and dashed lines, respectively. Four permeability ratios are considered (for better readability, the ratios $k_L/k = 0.1$ and 10 are plotted in light colour) in addition to the reference case of uniform permeability.

![Diagram](image)

Figure 3.26. (a) Parametric study of a single vertical layer of variable thickness. (b) Required support pressure $s_0$ as a function of the layer thickness $d_L$ in ground without any cohesion $c$ (permeability contrast $k_L/k = 0.01$-100; blue: no drainage measure; red: six axial boreholes according to Fig. 3.4; other parameters according to Table 3.1)
3.8.2.2.1 Without drainage measure

The required support pressure is sensitive to permeability heterogeneity, but does not increase linearly with growing permeability ratio (compare red lines for $k_i/k = 1$ to $k_i/k = 0.1$ or $0.01$ in Fig. 3.26b). Face support in large faults is similar to the one required in homogeneous ground (asymptotically converging curves for $d_L/D \geq 2.5$), but clearly different for low fault thicknesses $d_L/D < 1\text{-}1.5$.

In unstable neighbouring rock (solid red lines in Fig. 3.26b), all curves would start at the value of homogeneous ground for $d_L/D = 0$ (which is not pictured for the sake of simplicity), but then rapidly diverge. In a narrow low-permeability zone ($k_i/k < 1$), considerably higher support pressures than in homogeneous ground are required for face stability. The fault zone itself acts as a hydraulic barrier, within which unfavourable high hydraulic gradients develop. The gradients and thus the required support pressure decrease with increasing zone thickness. In case of a very narrow high-permeability zone ($k_i/k > 1$), high gradients develop immediately after the zone, but still within the potentially unstable wedge (which is e.g. $\omega = 65^\circ$ for $d_L/D = 0.1$). At a fault thickness of about half the tunnel face ($d_L/D = 0.5$), the draining action of the tunnel face leads to a favourably reduced pore pressure distribution all-over the zone such as the support pressure decreases to a local minimum, before increasing again with increasing zone thickness.

In stable neighbouring rock, the support pressure for narrow faults is lower than for unstable rock due to the limited extent of failure (compare dashed to solid red lines in Fig. 3.26b). In case of a low-permeability zone ($k_i/k < 1$), the support over the thickness exhibits a flat maximum due to the competing effects of permeability heterogeneity (the required support pressure decreases with increasing thickness) and geometry (the support increases with the thickness). In case of a high-permeability zone ($k_i/k > 1$), the support pressure is lower compared to homogeneous ground due to the favourable hydraulic head field.

3.8.2.2.2 With advance drainage boreholes

On account of the structural complexity of geological faults, permeability estimates are highly uncertain (Faulkner et al., 2010). Figure 3.26b shows that advance drainage substantially reduces the sensitivity of the support pressure with respect to permeability contrasts: the differences between the five permeability cases are considerably smaller in the presence of drainage boreholes than they are without advance drainage (compare blue to red lines in Fig. 3.26b especially for narrow faults).

In unstable neighbouring rock (solid blue lines in Fig. 3.26b), the extrema observed when considering no drainage measure (solid red lines in Fig. 3.26b) disappear in the presence of advance drainage. Sufficiently long advance drainage boreholes eliminate especially the maxima predicted for low-permeability faults, because it uniformly relieves the pore pressure both in rock and fault zone (see also l.h.s. of Fig. 3.24c).

Also in stable neighbouring rock (dashed blue lines in Fig. 3.26b), less support pressure is required when considering zones of a thickness up to about one tunnel diameter. For thicker zones, the support pressures coincide with the ones required assuming unstable neighbouring rock.
3.8.3 Application example

The practical significance of these findings is discussed for an exemplary tunnel in a vertically stratified ground of uniform, very low cohesion \( c = 50 \text{ kPa} \); other parameters see Table 3.1). The tunnel crosses 5 m thick layers oriented perpendicular to the tunnel axis (Fig. 3.28a). The permeability contrast of 100 leads to distinctively changing water inflows, but also to variable support pressure required to stabilize the tunnel face. The support pressure \( s \) when repeatedly crossing high- and low-permeability layers is shown in Figure 3.28b as a function of the position of the tunnel face \( x_f \). Face stability considering no advance drainage measure is marked as red line; advance drainage with six axial boreholes in blue (borehole layout see Fig. 3.4).

Without drainage measure, face stability is highly sensitive to both permeability heterogeneity and location of the tunnel face within the layers. In conventional tunnelling, the tunnel face would collapse without an extremely heavy and practically unfeasible additional support. Entering a high-
permeability zone requires lower support pressure than homogeneous ground (compare dotted to solid red line in Fig. 3.28b at $x_f = 0$). The closer the tunnel faces gets to the low-permeability layer ($0 \leq x_f \leq 5$ m), the more increases the support pressure. This is due to high hydraulic gradients developing in the upcoming low-permeability layer, which are still within the potentially unstable wedge (cf. Section 3.8.2.2.1). Due to the hydraulic barrier effect of the low-permeability zone, the highest support pressure is required immediately before leaving the low-permeability layer (1086 kPa at $x_f = 8.75$ m in Fig. 3.28b).

With advance drainage boreholes, the sensitivity to both permeability heterogeneity and location of the tunnel face within the layer is substantially reduced (blue solid line in Fig. 3.28b). The support pressure required within the high-permeability layer is lower than the one within the low-permeability layer, which is due to the preferential steeper hydraulic gradients within the aquitard. Overall, the support pressures required are within the feasible range of face bolting ($s \leq 200$ kPa) and thus prove feasibility of the tunnel example only by help of sufficiently long advance drainage boreholes.

![Figure 3.28](image_url)

Figure 3.28. (a) Longitudinal section of a tunnel crossing 5 m thick vertical layers of permeability contrast $k_1/k_2 = 100$. (b) Required support pressure $s$ as a function of the position of the tunnel face $x_f$ in ground of cohesion $c = 50$ kPa (blue: no drainage measure; red: six axial boreholes according to Fig. 3.4; other parameters according to Table 3.1)
3.9 Thinly interbedded vertical aquifers and aquitards

3.9.1 Homogenisation to an equivalent anisotropic model

In cases where sedimentary deposits undergo intense folding, thin aquifers and aquitards might become vertical (Fig. 3.2d). As explained in Section 3.5.1, the ground consisting of sufficiently thin layers can be considered as homogeneous medium of orthotropic permeability. The equivalent permeability is then calculated according to Eqs. (3-1), (3-2) and the hydraulic head field of the equivalent anisotropic model is governed by the degree of anisotropy (Eq. (3-3)).

3.9.2 Maximum layer thickness

The maximum layer thickness for which homogenisation is possible is determined by Figure 3.29. It shows the required face support pressure \( s \) as a function of the normalized layer thickness \( d/D \) (\( c = 0 \), other ground parameters see Table 3.1). The support pressure\(^5\) needed when considering stratified media consisting of discretely modelled layers are indicated as black crosses (inset in Fig. 3.29a); the equivalent anisotropic medium is shown in red.

![Figure 3.29](image)

(a) Multiple vertical layers and (b) required support pressure \( s \) as a function of the layer thickness \( d \) in ground without any cohesion \( c \) (black: discretely modelled layers of permeability contrast \( k_t/k = 0.01 \) and 100; red: equivalent anisotropic model; other parameters according to Table 3.1)

The difference in required support pressure of each layer thickness (highlighted in grey in Fig. 3.29b) originates from a high- or a low-permeability layer immediately at the tunnel face. The thinner the

\(^5\) Please note that the present study considers the failure mechanism of wedge and prism (Fig. 2.2), but does not consider potential failure of individual layers such as spalling or buckling.
discretely modelled strata, the smaller are the ranges and the higher are the resulting support pressures (barrier effect of a low-permeability layer close to the face, cf. Section 3.8.2.2.1). The required face support pressure when modelling discrete, thin layers of \( \frac{d}{D} \leq 0.05 \) (\( s = 1039 - 1144 \); Fig. 3.29b) is within 10\% accuracy of the face support needed in an equivalent anisotropic medium (\( s = 1135 \) kPa). For thicker layers, the support pressure considering the equivalent anisotropic permeability model overestimates the face support needed.

### 3.9.3 Effect of permeability anisotropy

The effect of permeability anisotropy representing very thin vertical layers on face stability is discussed in Figure 3.30. Figure 3.30b shows the required support pressure \( s_0 \) as a function of the degree of anisotropy \( \frac{k_p}{k_n} \) (and the permeability ratio \( \frac{k_1}{k_2} \)) when assuming cohesionless ground (Figure 3.31 shows the results of a parametric study into the effect of cohesion). The solid red line

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**Figure 3.30.**

(a) Permeability anisotropy of the equivalent homogeneous model representing very thin vertical layers.

(b) Required support pressure \( s \) as a function of the permeability ratio \( k_1/k_2 \) and the degree of anisotropy \( k_p/k_n \) at logarithmic scale in ground without any cohesion \( c \) and (c) belonging distribution of the hydraulic head \( h \) along two lines above and ahead of the tunnel face (blue: no drainage measure; red: six axial boreholes according to Fig. 3.4; other parameters according to Table 3.1)
applies to no advance drainage measures, the blue line to six axial advance drainage boreholes. The dotted lines indicate the support pressure required in isotropic ground. Figure 3.30c shows the belonging distribution of the hydraulic head $h$ along two lines above and ahead of the tunnel face.

### 3.9.3.1 Without advance drainage measure

The required support pressure increases with increasing degree of anisotropy (red in Fig. 3.30b) and is overall far above the feasible range of about 200 kPa (Section 2.5.1). An increase of $k_p/k_n$ from 1 to 10 (which corresponds to a permeability ratio of $k_1/k_2 = 38$ in a thin-layered strata) requires an additional support pressure of $\Delta s = 295$ kPa.

The hydraulic head distribution above of the face is unaffected of permeability anisotropy (compare red lines in l.h.s. of Fig. 3.30c). Ahead of the face, the open tunnel face reliefs pore pressure not as effective as in isotropic ground, because the seepage face is perpendicular to the lower permeable direction and high hydraulic gradients result (compare red lines in r.h.s. of Fig. 3.30c).

### 3.9.3.2 With advance drainage boreholes

The support pressure for face stability only marginally increases with increasing degree of anisotropy (blue; $\Delta s = 29$ kPa for an increase of $k_p/k_n$ from 1 to 10 in Fig. 3.30b).

Again, the hydraulic head distribution above of the face is nearly unaffected of permeability anisotropy (compare blue lines in l.h.s. of Fig. 3.30c). Immediately ahead of the tunnel face, the drainage boreholes are not able to reduce the hydraulic head as effective as in isotropic ground (due to the diminished seepage action of the tunnel face). However, in some distance of the tunnel face, the drainage boreholes are even more effective than in isotropic ground due to their seepage faces being perpendicular to the more permeable direction (blue lines in r.h.s. of Fig. 3.30c).

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**Figure 3.31.** Results of a parametric study into the effect of ground cohesion $c$ in the problem of Figure 3.30
3.10 Conclusions

In water-bearing ground of non-uniform permeability, the hydraulic head distribution is decisive for face stability. The head distribution depends on the permeability ratio of layer and surrounding rock; the orientation, elevation and thickness of the layer and the arrangement of the advance drainage boreholes.

Advance drainage boreholes may suffice to lower the pore water pressure such that both the risk of face instability and the sensitivity to permeability contrast is clearly reduced. Furthermore, the boreholes may serve as hydraulic exploration of the ground by capturing changes in water ingress at different borehole depth during drilling (of course protected against high water pressures by means of a so-called "preventer") and thus provides useful information about the permeability distribution of the upcoming ground.

When tunnelling in a ground containing horizontal layer(s), less support pressure than in the reference case of homogeneous ground is required when a high-permeability layer intersects a seepage face and thus acts as natural advance drainage. Unfavourable are layers acting as a hydraulic barrier and thus hindering pore pressure relief above of the tunnel face (e.g. a low-permeability layer in the upper part of the tunnel face or a high-permeability layer above the tunnel roof, i.e. without connection to a seepage face). Six axial advance drainage boreholes in the upper part of the tunnel face reduce the required support pressures by about 60%. The arrangement of the boreholes should aim to reduce the pore pressure as widely as possible above and ahead of the tunnel face. This can be achieved by wisely arranging the boreholes such as they reach into the permeable rock above or below a low-permeability layer (e.g. by drilling two horizontal boreholes from a roof niche or by drilling them inclined from the tunnel face).

When tunnelling in a ground containing vertical fault zone(s), less support pressure than in the reference case of homogeneous ground is required when encountering an aquifer connected to a seepage face (i.e., the tunnel face or drainage boreholes; of course at the expense of potentially high water inflow). Highly unfavourable concerning face stability is a low-permeability zone at or around the tunnel face acting as hydraulic barrier and “attracting” high hydraulic gradients. Six axial advance drainage boreholes improve stability remarkably if the boreholes reach into the rock ahead of the fault zone. Moreover, by uniformly lowering the hydraulic head both in fault zone and surrounding ground, the risk of other possible failure mechanism such as punching decreases.

In case of adequately thin-layered ground relatively to the tunnel diameter ($d/D \leq 0.2$ for horizontal stratified ground, $d/D \leq 0.05$ for vertical strata), it is suitable to use an equivalent homogeneous anisotropic model for calculating the seepage flow condition. This is numerically faster than to model discretely layered ground and yet provides face-stability results of at least 10% accuracy to the equivalent anisotropic medium. Again, advance drainage boreholes reduce sensitivity to permeability heterogeneity clearly.

Permeability heterogeneity in ground may be anthropogenic when bear in mind e.g. a sealing grouting body. In cases where the low-permeability grouting body comprises the entire potential failure volume around the tunnel face, a hydraulic head field comparable to homogeneous ground develops. But in cases where limiting accessibility render impossible executing a full grouting body, horizontal low-permeability layer(s) may results and unfavourable hydraulic head distributions might develop similarly to the ones discussed.
4. Effect of limited capacity of drainage boreholes on tunnel face stability

4.1 Introduction

Chapter 4 focuses on the limited effectiveness of drainage measures with respect to face stability in cases where the drainage borehole walls do not represent seepage faces under atmospheric pressure and studies: (i), the capacity of the drainage boreholes hindering full pressure relief in highly permeable ground under a high water table; and (ii), casings required for stabilizing the borehole, but which in turn restrict pore pressure relief to small openings (cf. Table 1.1).

The limited capacity of a drainage borehole may become significant in highly permeable ground and at great elevation of the water table. Large inflow may change the flow regime within the borehole from open-channel (free surface or gravity flow; Fig. 4.1b and c) to pipe flow (pressurized flow, Fig. 4.1a). The pressure developing within the borehole results in reduced pore pressure relief in the surrounding ground. Literature about the limited capacity of advance drainage boreholes is very scanty and to the author’s knowledge there are only Hong et al. (2007) who calculated for a simplified example the maximum borehole length which provides full pipe flow capacity (but without reference to face stability). Much more attention was paid to hydraulic capacity of boreholes in related fields such as Karst, well or petroleum engineering, but mainly focusing on large scale. Reference here is restricted to works, supporting the author’s approach described in Section 4.2.2, where it was shown that turbulent flow hydraulics formulae are appropriate for approximating flow in Karst conduits.

Figure 4.1. (a) Pressurized flow from a borehole during construction of Olafsfjödur street tunnel, Island (courtesy of Karl Gunnar Holter) and outflow of drainage borehole (Lake Mead Intake No. 3) of (b) discharge of about 0.25m³/h and (c) dripping only
(e.g. Louis, 1967; Atkinson, 1977; Jeannin, 2001) and where the suitability of equivalent permeability models for considering pipe flow was shown: Halford (2000) simulated laminar and turbulent flow in and towards a well by considering an “equivalent hydraulic conductivity” which depends on the Reynolds number and – for turbulent flow – on the pipe friction factors. Shoemaker et al. (2008a, 2008b) slightly modified the turbulent equivalent hydraulic conductivity in order only to depend on the Reynolds-Number (but not on pipe friction data) and pointed out the importance of considering turbulent flow when modelling preferential groundwater flow layers. Birch et al. (2007) assigned “values of an equivalent hydraulic conductivity along the length of the well screen that would simulate the head losses associated with pipe flow” for simulating flow rates in horizontal wells. Chen et al. (2003) focused on a single horizontal well, where an equivalent hydraulic conductivity represented laminar and turbulent flow, and found good agreement of his laboratory study with the numerical findings. Wang and Zhang (2007) continued numerical investigations similar to Chen et al. (2003), but with a slightly different coefficient of friction and Li et al. (2012) applied the same approach to an example of drain pipes below a concrete channel. Section 4.2 investigates an equivalent permeability model considering pipe- and open-channel flow equations for determining the hydraulic capacity of drainage boreholes and revolves around the following leading questions: How does tunnel face stability change when the limited flow capacity of the drainage boreholes is considered? What is the maximum ground permeability for which it is safe to assume a seepage face at the borehole wall?

Drainage effectiveness may also be limited by borehole casings becoming necessary when the borehole walls are unstable. These casings typically restrict the passage of water to small openings and therefore impede pore pressure relief around the boreholes, which in turn reduces the effectiveness of the drainage measure with respect to face stability (Section 4.3).

4.2 Flow capacity of the drainage boreholes

4.2.1 Introduction

The goal of this Section is to investigate face stability taking account of the effect of the limited flow capacity of the boreholes on the pore pressure relief. The detailed flow regime within the borehole can be neglected, as focus is on pore pressure relief in the surrounding ground. Instead, considering averaged coaxial flow velocities within the borehole at steady state is sufficient (no need for computational fluid dynamics, CFD). This reduces the complexity considerably and allows making use of empirical equations of pipe flow and open-channel flow hydraulics.

Section 4.2.2 shows how the pipe and open-channel flow within the boreholes can be considered as flow through a porous medium of equivalent permeability (also known as “equivalent hydraulic conductivity”; here used interchangeably). Section 4.2.3 considers the simplest possible borehole-ground seepage flow interaction problem (a single drainage borehole) and elaborates the FE-model. Comparison of the results considering pipe- and open-channel flow to the results taking account of pipe-flow only will show that the latter is sufficiently accurate for the purpose of investigating face stability. The section further presents some characteristic results on the effect of ground permeability, initial water head and surface roughness of the single borehole. Finally, a simplified analytical solution for the single borehole - ground interaction problem is derived allowing validation of the FE-results. Section 4.2.4 quantifies the reduced drainage effectiveness with respect to tunnel face stability considering a common advance drainage layout (six axial boreholes drilled from the face) and discusses the applicability of the design nomograms (Chapter 2).
4.2.2 Equivalent permeability

4.2.2.1 Borehole with pressurized pipe flow

Drainage boreholes are straight pipes of circular cross-section. Within the pipe, laminar or turbulent flow develops, depending on the Reynolds number

\[ \text{Re} = \frac{v_x \cdot d_{dr}}{v} \]  

relating the product of pipe diameter \( d_{dr} \) and axial flow velocity \( v_x \) (averaged over the pipe cross-section) to the kinematic viscosity of the water \( v \). Laminar flow usually evolves in small diameter pipes and for low flow velocities (Re < 2300). For higher Reynolds numbers, first transitional then fully turbulent flow develops. Considering a typical borehole diameter of \( d_{dr} = 0.1 \) m and the kinematic viscosity of water \( v = 1.307 \cdot 10^{-6} \) m\(^2\)/s at 10°C, turbulent flow develops already for flow velocities \( v_x \) superior to about 0.03 m/s. As it will be shown later (Section 4.2.3.2), these flow velocities are easily reached within drainage pipes, even in low-permeability ground. Therefore, turbulent flow only is considered below.

In a drainage borehole, water inflow takes place along its entire shell surface. Here we consider the borehole as a pipe consisting of sequential segments, each of section length \( \Delta x \), and assume inflow taking place only at the connection points (i.e. the nodes).

Figure 4.2. Pipe flow

The flow in a completely filled (pressure) pipe of section length \( \Delta x \) and of constant diameter \( d_{dr} \) is assumed as incompressible, steady and turbulent between locations \( A, B \) (Fig. 4.2). According to the conservation of energy, energy head at point \( A \) \( (h_{e,A}) \) has to be equal to the sum of stored energy at point \( B \) \( (h_{e,B}) \) and the head loss \( h_V \) due to friction:

\[ h_{e,A} = h_{e,B} + h_V \]  

(4-2)
After Bernoulli, the energy head consists of velocity-, pressure- and geodetic head (the latter also referred to as elevation or potential head; Fig. 4.2):

$$\frac{v_A^2}{2g} + \frac{p_A}{\rho \cdot g} + z_A = \frac{v_B^2}{2g} + \frac{p_B}{\rho \cdot g} + z_B + h_v \quad , \quad (4-3)$$

where \( g \) denotes the gravity acceleration, \( \rho \) the density of the fluid, \( v \) the average flow velocity over the pipe cross-section in axial pipe direction, \( p \) the pressure and \( z \) the geodetic height. Due to mass conservation, the discharge at point \( A \) is equal to the discharge at point \( B \). As the pipe diameter is constant, the average axial flow velocity within the pipe is constant (\( v_A = v_B = v_x \)) and Eq. (4-3) simplifies to

$$h_v = \frac{P_A - P_B}{\rho \cdot g} + z_A - z_B \quad , \quad (4-4)$$

or, noting that the r.h.s. term is equal to the hydraulic head difference in a porous medium,

$$h_v = I_x \Delta x \quad , \quad (4-5)$$

where \( I_x \) is the hydraulic head gradient. According to the empirical equation of Darcy-Weisbach (see e.g. Bollrich, 2000), the head loss is

$$h_v = \frac{\lambda_h}{d_{dr}} \Delta x \cdot \frac{v_x^2}{2g} \quad , \quad (4-6)$$

where \( \lambda_h \) represents an empirical friction coefficient. Suggested by Colebrook-White (1937) and based upon the previous works of Prandtl, von Karman and Nikuradse, it can be expressed as follows:

$$\frac{1}{\sqrt{\lambda_h}} = -2 \cdot \log_{10} \left( \frac{2.51}{Re} \cdot \sqrt{\lambda_h} + \frac{k_{equiv}}{d_{dr}} \cdot \frac{3.71}{Re} \right) \quad , \quad (4-7)$$

where \( k_{equiv} \) denotes the equivalent sand roughness. The empirical coefficient \( \lambda_h \) depends thus only on the Reynolds number and on the ratio of \( k_{equiv} \) to pipe diameter \( d_{dr} \) (the so-called relative roughness of the pipe; Moody, 1944).

The equivalent sand roughness \( k_{equiv} \) quantifies roughness and texture of the borehole wall and amounts from \( k_{equiv} = 15 \text{ mm} \) (rock excavation; “hydraulically very rough”) to \( 5 \text{ mm} \) (smooth rock excavation or highly incrusted steel pipe; “hydraulically rough”) and for comparison to \( 0.05 \text{ mm} \) (a surface smooth as a newly rolled steel pipe; “hydraulically smooth”; Bollrich, 2000). Eq. (4-7) covers both smooth and rough pipe walls: The l.h.s. bracketed term of Eq. (4-7) represents a hydraulically smooth pipe, where the friction coefficient depends on the viscosity, i.e. on Re only (smooth: \( Re \cdot k_{equiv}/d_{dr} < 65 \)); the r.h.s. bracketed term represents hydraulically rough pipes where the wall roughness dominates the flow behaviour (rough: \( Re \cdot k_{equiv}/d_{dr} > 1300 \)).
From Eqs. (4-1), (4-6), (4-5) and (4-7), we obtain the averaged flow formula for turbulent pipe flow

\[ v_x = -K_x I_x , \quad (4-8) \]

with

\[ K_x = K_{x,\text{pipe}} = 2 \cdot \log_{10} \left( \frac{2.51 \nu}{d_{dr} \cdot \sqrt{2g \cdot d_{dr} \cdot I_x}} + \frac{k_{x,eq}}{3.71 d_{dr}} \right) \sqrt{\frac{2g \cdot d_{dr}}{I_x}} . \quad (4-9) \]

Eq. (4-8) is formally the same as a non-linear Darcy's law. This allows modelling the pipe as an equivalent porous medium with head gradient-dependent hydraulic conductivity \( K_{x,\text{pipe}} \).

![Figure 4.3. Open-channel flow](image)

### 4.2.2.2 Borehole with open-channel flow

Open-channel flow must be considered if a borehole is not filled completely (Fig. 4.3). Slightly non-uniform open-channel flow (the free surface slightly more inclined than the pipe itself) can be approximated with reasonable accuracy by small modifications of the pipe flow formula (Eq. (4-8); Jirka and Lang, 2009). Specifically, considering that friction develops only along the contact surface between pipe wall and fluid (“wetted perimeter” \( U_{\text{wet}} \)), the diameter \( d_{dr} \) in Eq. (4-9) has to be replaced by the hydraulic diameter \( d_{hy} \). The average flow velocity (smeared over the cross-section) is thus given again by Eq. (4-8) with the equivalent permeability

\[ K_x = K_{x,\text{open}} = 2F \cdot \log_{10} \left( \frac{2.51 \nu}{d_{hy} \cdot \sqrt{2g \cdot d_{hy} \cdot I_x}} + \frac{k_{x,eq}}{3.71 d_{hy}} \right) \sqrt{\frac{2g \cdot d_{hy}}{I_x}} . \quad (4-10) \]

where

\[ F = \frac{1}{\pi} \left( \arccos \left( 1 - 2 \frac{t_{dr}}{d_{dr}} \right) - 2 \left( 1 - 2 \frac{t_{dr}}{d_{dr}} \right) \sqrt{\frac{t_{dr}}{d_{dr}} \left( 1 - \frac{t_{dr}}{d_{dr}} \right)} \right) . \quad (4-11) \]
with $t_{dr}$ denoting the fill-level (Fig. 4.3) and $F$ represents the fraction of fluid area $A_{\text{fluid}}$ to the pipe cross-section area ("filling ratio"). For the circular pipe cross-section it is

$$d_{hy} = 4r_{hy} = \frac{4A_{\text{fluid}}}{U_{\text{wet}}} = F \cdot \frac{d_{dr} \cdot \pi}{\arccos \left( 1 - 2 \frac{t_{dr}}{d_{dr}} \right)}, \quad (4-12)$$

where $r_{hy}$ denotes the hydraulic radius, i.e. the ratio of fluid area to the wetted perimeter in the pipe cross-section.

Modelling of the flow along the borehole as seepage flow through an equivalent porous medium (Eq. (4-8)) requires a criterion for the selection of the appropriate equivalent permeability (Eq. (4-9) or (4-10)), depending on whether pipe or open-channel flow takes place. It has to be formulated in terms of the pressure $p$ (the only state variable in a seepage flow analysis). As criterion for the transition from open-channel to pipe flow, the average pipe pressure can be considered (Fig. 4.4a):

$$p_{tr} = \frac{1}{2} \cdot d_{dr} \cdot \gamma_w. \quad (4-13)$$

Eq. (4-13) is hereafter referred to as “transition pressure” and represents a material constant of the equivalent porous medium. In the case of open-channel flow ($p < p_{tr}$ or $t_{dr}/d_{dr} < 1$), the equivalent permeability does not depend only on the head gradient, but also on the relative fill level $t_{dr}/d_{dr}$ (Eq. (4-11)-(4-12)), which for the same reasons as above has to be expressed as a function of the average pressure $p$ (Fig. 4.4a):

$$\frac{t_{dr}}{d_{dr}} = \frac{p}{p_{tr}}. \quad (4-14)$$

Figure 4.4.  (a) Average water pressure $p$ in a borehole cross-section of diameter $d_{dr}$ at fill-level $t_{dr}$; transition pressure $p_{tr}$ at $t_{dr}/d_{dr} = 1$. (b) Open-channel discharge $Q_{\text{open}}$ from a single borehole normalized by maximum discharge $Q_{\text{full}}$ as a function of the relative fill-level $t_{dr}/d_{dr}$. (c) Equivalent permeability $K_x$ on logarithmic scale as a function of the hydraulic gradient $I_x$ for several average pressures $p$ ($K_x$ of Eqs. (4-9), (4-10) with $d_{dr} = 0.1$ m, $p_{tr} = 500$ Pa, $k_{eq} = 5$ mm, $\nu = 1.307 \cdot 10^{-6}$ m$^2$/s)
Summing up, the flow in the boreholes can be considered as seepage flow in an equivalent porous medium obeying a non-linear Darcy law (Eq. (4-8)). The non-linearity is due to the dependency of the equivalent permeability $K_x$ on the head gradient $I_x$ and on the average pressure $p$: If $p/p_{tr} < 1$, then the equivalent permeability is given by Eq. (4-10) with $F$, $d_h$, and $t_{dr}/d_{dr}$ after Eqs. (4-11), (4-12) and (4-14), respectively; else (if $p/p_{tr} \geq 1$), the permeability is given by Eq. (4-9).

Figure 4.4c shows the equivalent permeability $K_x$ as a function of the hydraulic head gradient $I_x$ and of the average pressure $p$ for an exemplary pipe (with $d_{dr} = 10 \, \text{cm}$, $p_{tr} = 500 \, \text{Pa}$, $k_{s,eq} = 5 \, \text{mm}$ and $\nu = 1.307 \times 10^{-6} \, \text{m}^2/\text{s}$). It shows a non-linear decrease of permeability with increasing gradient. The equivalent permeability of a partially filled pipe is smaller than the one of the completely filled pipe (Eq. (4-9); red line in Fig. 4.4c), except for the nearly full pipe ($p = 460 \, \text{Pa}$ or $t_{dr}/d_{dr} = 0.92$ in Fig. 4.4c; (Eq. (4-10)). The reason can be explained based upon the “filling-curve” shown in Figure 4.4b, i.e. the relative discharge $Q_{\text{open}}/Q_{\text{full}}$ as a function of the relative fill-level $t_{dr}/d_{dr}$ ($Q_{\text{full}}$ denoting the discharge of a just fully filled pipe, i.e. at $t_{dr}/d_{dr} = 1$). Because of the shorter wetted circumference inducing friction, a nearly filled pipe has a higher discharge than a fully-filled pipe (maximum in Fig. 4.4b), thus also a higher flow velocity or a slightly higher equivalent permeability.

For $I_x = 0$, the permeability according to Eqs. (4-9) and (4-10) becomes infinite, but this is irrelevant as there is no seepage flow in this case. To avoid division by zero, an arbitrary value (e.g. $K_x = 100 \, \text{m/s}$) can be assigned to the permeability if $I_x = 0$.

4.2.2.3 Transverse permeability

The equations derived above describe flow (as seepage flow through an equivalent porous medium) in the axial pipe direction only. In the absence of ground - pipe interaction it would be sufficient to
model the boreholes by linear (one-dimensional) elements. Seepage flow in the ground around the boreholes (and water discharge into the boreholes) depends however on the borehole radius. Therefore the boreholes have to be modelled as three-dimensional objects (Fig. 4.5a), which necessitates an assumption about the permeability of the equivalent porous medium in the plane of the borehole cross-section (i.e. perpendicular to the borehole axis). The transverse permeability is taken here 100 times higher than the ground permeability $K_g$, ensuring a uniform pressure distribution in the cross-section plane of the borehole. Comparative computations showed that this assumption is not essential. Consider for example the numerical results of Figure 4.5, which were obtained taking the transverse permeability either equal to the ground permeability $K_g$, or 100 times higher than $K_g$: in both cases, the velocity distribution is approximately uniform over the borehole cross-section; the hydraulic head field in the ground around the borehole does not depend on the transverse permeability.

4.2.3 Ground - single drainage borehole interaction

4.2.3.1 FE simulation

4.2.3.1.1 Computational model

A single, 30 m long, 0.1 m diameter drainage borehole is considered (Fig. 4.6). The seepage flow domain extends up to the 100 m ahead and around the borehole. The hydraulic head at the far-field boundaries is taken equal to the initial hydraulic head $h_0$ (100 m). The outlet of the drainage borehole represents the only seepage face under atmospheric pressure. The interior of the borehole is modelled as an equivalent porous medium (Section 4.2.2). The ground is considered as isotropic porous medium obeying Darcy’s law with the permeability $K_g$. Table 4.1 summarizes the assumed parameters.

Note that in highly permeable ground well below the water table, turbulent flow conditions may prevail even in the ground itself: during construction of Intake No. 3 at Lake Mead, 3.7 m$^3$/s water discharge was measured collected from a seepage area of about 360 m$^2$ (Nicola et al., 2014). The average inflow velocity of 0.01 m/s indicates clearly turbulent seepage flow within the pores of the ground (turbulent flow for $Re > 1$). However, assuming Darcy’s law is an assumption on the safe side for flow in highly permeable ground, because the linear increase of filter velocity with hydraulic gradient overestimates both filter velocity and water inflow into the boreholes.

Table 4.1. Parameters for the comparative analyses of the ground - single drainage borehole interaction

<table>
<thead>
<tr>
<th>Problem layout</th>
<th></th>
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</thead>
<tbody>
<tr>
<td><strong>Depth of cover</strong></td>
<td>$H$</td>
</tr>
<tr>
<td><strong>Initial hydraulic head</strong></td>
<td>$h_0$</td>
</tr>
<tr>
<td><strong>Borehole diameter</strong></td>
<td>$d_{dr}$</td>
</tr>
<tr>
<td><strong>Borehole length</strong></td>
<td>$l_{dr}$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Ground</th>
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</thead>
<tbody>
<tr>
<td><strong>Permeability</strong></td>
<td>$K_g$</td>
</tr>
</tbody>
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<table>
<thead>
<tr>
<th>Borehole</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Equivalent permeability</strong></td>
<td>$K_x$</td>
</tr>
<tr>
<td><strong>Transition pressure</strong></td>
<td>$p_t$</td>
</tr>
<tr>
<td><strong>Kinematic viscosity water</strong></td>
<td>$\nu$</td>
</tr>
<tr>
<td><strong>Equivalent sand roughness</strong></td>
<td>$k_{s,eq}$</td>
</tr>
</tbody>
</table>
Figure 4.6. Problem setup for the comparative analysis of a single drainage borehole

![Figure 4.6](image)

boundary conditions:
abcd: hydraulic head $h_0$
ef: seepage face ($p_{atm}$)
de, fa: no flow

Figure 4.7. Distribution of, (a) pressure $p$, (b) axial velocity $v_x$ and, (c), transverse velocities $v_y, v_z$ along the borehole axis $x$ ($k_{eq} = 5 \text{ mm}, h_0 = 100 \text{ m}, K_g = 2 \cdot 10^{-7} \text{ m/s}$ other parameters according to Table 4.1)

![Figure 4.7](image)
4.2.3.1.2 Model behaviour

Figure 4.7a shows the numerically calculated distributions of the pressure $p$ along the borehole. The pressure increases with the distance from the drained boundary and reaches at $x = 9$ m the transition pressure $p_{tr}$ (Eq. (4-13)), which means that the flow regime changes from open-channel to pipe flow. The pressure variation in the transverse direction is small and corresponds to the increase of geodetic head within the borehole. The axial Darcy velocity $v_x$ increases close to the borehole outlet at $x = 0$ (Fig. 4.7b). The lowest axial velocity ($v_x = 0.05$ m/s at $x = 30$ m) confirms the assumption of turbulent flow regime ($v_x > v_{c,turb} = 0.03$ m/s; Section 4.2.2.1). The transverse Darcy velocities $v_y$ and $v_z$ are several orders of magnitude smaller than the axial velocity (compare Fig. 4.7c to b).

4.2.3.1.3 Simplification for open-channel flow

In the case of open-channel flow, the water pressure at the borehole walls is between zero and maximum $d_w \gamma \rho_w$, which is by orders of magnitude lower than the far-field pore pressures in the ground. This suggests that the explicit consideration of open-channel flow (Eq. (4-9)) is irrelevant for the pore pressure field in the surrounding ground. It will be shown below that the hydraulic head field can be determined with sufficient accuracy by modelling open-channel flow considering a completely filled borehole, i.e., by disregarding the effect of $p$ on equivalent permeability and using only Eq. (4-9).

Figure 4.8 shows the exact (black solid lines) as well as the approximate (red dashed lines) pressure distribution along the borehole axis (Fig. 4.8a-4.8c) and along the horizontal direction perpendicular to the borehole axis (Fig. 4.8d-4.8f). The latter serves as measure of the pore pressure relief in the ground, which is decisive for the stability computations in the upcoming sections, and is evaluated at two borehole depths ($x = 2$ and $29$ m).

For a highly permeable ground ($K_g = 1 \cdot 10^{-5}$ m/s; Fig. 4.8a), pipe-flow only develops and all the curves coincide. The normalized pressure distribution perpendicular to the borehole remains constant within the borehole (i.e., up to $y = 0.05$ m) and then increases for both the exact and approximated solution (Fig. 4.8d).

For moderately permeable ground ($K_g = 1 \cdot 10^{-6}$ m/s; Fig. 4.8b), flow changes from pipe- to open-channel. For the exact solution (solid line in Fig. 4.8b) open-channel flow occurs over the first 1.2 meters of the drainage borehole (i.e., $p < p_{tr}$), after which the pipe-flow pressure increases to about 5 kPa at the end of the borehole (point A in Fig. 4.8b). In case of the approximated solution, the first two meters of the drainage borehole are in open-channel flow and the pressure reaches a value of 3 kPa (point B in Fig. 4.8b). However, the maximum head difference at the rear end of the drainage borehole is very small (< 20 cm) and all pressure difference disappears within less than 30 cm of horizontal distance to the borehole (Fig. 4.8e).

For less permeable ground ($K_g = 1 \cdot 10^{-7}$ m/s; Fig. 4.8c) solely open-channel flow occurs ($p < p_{tr}$ along the entire borehole). The difference between the approximation and the exact solution is equal to a water head difference of less than 1 cm (compare point A to B in Fig. 4.8c), which is hardly visible in the pore pressure distribution normal to the borehole (Fig. 4.8f).

The exact solution needs at least 10 times more computation time than the approximate solution, without significant increase in accuracy. Therefore, it is reasonable to neglect the detailed development of open-channel flow. The computations of the following Sections were performed for the simplified model, i.e., disregarding the pressure dependency of the equivalent permeability and calculating the latter according to Eq. (4-9).
Figure 4.8. Top row: Sketches of flow condition. (a) to (c): Pressure distribution \( p \) along the borehole axis \( x \); (d) to (f): Normalized pressure distribution in \( y \)-direction for several ground permeabilities \( K_g \) \((k_{eq} = 5 \text{ mm}, p_0 = 1000 \text{ kPa}, \text{other parameters according to Table 4.1})\)

4.2.3.1.4 Characteristic results

Figures 4.9a and b show the pressure \( p \) (normalized by the initial pressure \( p_0 \)) along the borehole axis \( x \) and the hydraulic head field, respectively, for several values of ground permeability \( K_g \). In a highly permeable ground \((K_g \geq 1 \cdot 10^{-4} \text{ m/s})\), pore pressure relief is limited to the vicinity of the outlet. The length of the borehole becomes irrelevant as it is not able to induce any pressure reduction deep into the ground (compare head fields of \( K_g \leq 1 \cdot 10^{-4} \) to \( K_g > 1 \cdot 10^{-4} \) in Fig. 4.9b). In moderately permeable ground, advance drainage reduces the pressure to less than 10% of its initial value along the borehole axis \( x \) as well as in its vicinity \((K_g = 1 \cdot 10^{-5} \text{ m/s})\). For lower permeability, the drainage borehole lowers the initial pressure to nearly zero \((p/p_0 \leq 0.003 \text{ for } K_g \leq 1 \cdot 10^{-6} \text{ m/s})\).

The higher the initial head \( h_0 \), the higher is the pressure \( p \) developing in the borehole (Fig. 4.10). But it is the ground permeability which mainly determines the flow regime within the borehole: flow transition from pipe to open-channel flow (transition points \( p/p_r = 1 \) added for orientation in Fig. 4.10) takes place in moderately permeable ground and the lower the initial head, the longer is the section of open-channel flow (Fig. 4.10b). In highly permeable ground only pipe-flow develops (Fig. 4.10a).
The roughness of the borehole walls is captured by the value of equivalent sand roughness $k_{s,eq}$. Figure 4.11 shows the pressure distribution along the borehole axis for rough ($k_{s,eq} = 5-15$ mm; “hydraulically rough behaviour”) and smooth borehole walls ($k_{s,eq} = 0.05$ mm; “hydraulically smooth behaviour”). The latter is plotted for comparative purposes only, because such smooth surfaces are achievable only by using borehole casings, which however reduce inflow remarkably (cf. Section 4.3). The rougher the borehole wall (i.e. the higher $k_{s,eq}$), the higher is the pressure developing within the borehole. However, the effect of surface roughness is of secondary importance compared to that of the
ground permeability (compare pressure distributions for $k_{s,eq} = 5$ mm to $k_{s,eq} = 15$ mm in both Fig. 4.11a,b).

Figure 4.12 shows the water discharge $Q$ from the borehole as a function of the permeability of the ground $K_g$ in a double logarithmic scale. At low permeabilities, where the water inflows to the borehole are so small that pressure in the borehole is practically atmospheric, $Q$ increases linearly with $K_g$. At high $K_g$, pressure develops within the borehole and therefore $Q$ increases sub-linearly with $K_g$. Discharge is lower than when assuming sufficient drainage capacity (i.e. atmospheric boundary condition prevailing at the borehole wall; dotted in Fig. 4.12).

**Figure 4.11.** Pressure $p$ normalized by the initial pressure $p_0$ along the borehole axis $x$ for variable surface roughness $k_{s,eq}$ of the borehole in (a) highly permeable and (b) medium permeable ground ($h_0 = 100$ m, other parameters according to Table 4.1)

**Figure 4.12.** Discharge $Q$ from a single borehole as a function of the permeability of the ground $K_g$ for variable borehole surface roughness $k_{s,eq}$ ($h_0 = 100$ m, other parameters according to Table 4.1)
4.2.3.2 Analytical treatment and FE validation

An analytical solution for the ground - single borehole interaction will be derived below, which serves as validation of the numerical results and illustrates the effect of the problem parameters. Figure 4.13 shows the problem layout: We assume rotational symmetry and calculate the radial inflow velocity \( q_r \) in a borehole of radius \( r \) within a ground of permeability \( K_g \) (Fig. 4.13a) with boundary condition of atmospheric pressure at the borehole wall and the initial head \( h_0 \) acting at the far-field boundary, which is assumed in a radial distance of \( r = R \). The incremental borehole section \( dx \) is shown in Figure 4.13b, where the radial inflow into a circular pipe takes place along the section shell and leads to an increase in axial velocity \( q_x \). The analytical solution is based upon: (i) the simplifying assumption that seepage flow in the ground occurs only in the radial direction and obeys rotational symmetry; (ii) the simplifying assumption of a hydraulically rough pipe with flow in the axial direction according to Section 4.2.2.1; and, (iii), the mass conservation equation in the borehole.

![Figure 4.13](image)

**Figure 4.13.** Simplified borehole problem: (a) radial inflow into a borehole of radius \( r \) from ground with permeability \( K_g \); (b) conservation of mass at a circular borehole with diameter \( d_r \) and infinitesimal length \( dx \) when considering radial inflow \( q_r \) at its shell surface

4.2.3.2.1 Governing equations

Mass continuity and Darcy's law result in the following expression for the radial inflow velocity \( q_r \) at location \( x \) of a borehole of diameter \( d_r \) (e.g. Matthews and Russell, 1967):

\[
q_r(x) = \alpha_i \frac{2K_g h_0}{d_r} \left(1 - \bar{h}(x)\right), \quad (4-15)
\]

where \( K_g \) and \( h_0 \) denote the ground permeability and the far-field hydraulic head, respectively; \( \bar{h}(x) \) is the normalized hydraulic head

\[
\bar{h}(x) = \frac{h(x)}{h_0}; \quad (4-16)
\]
\[ \alpha_1 = \left( \ln \frac{2h_0}{d_{dr}} \right)^{-1}. \]  

(4-17)

For a pipe completely filled with an incompressible fluid, mass conservation results in the following relationship between radial inflow and axial velocity \( q_x \) (Fig. 4.13b)

\[ \frac{dq_x}{dx} = -\frac{4}{d_{dr}} q_r. \]  

(4-18)

Considering the radial inflow according to Eq. (4-15) and introducing the normalized co-ordinate

\[ \xi = \frac{x}{l_{dr}}, \]  

(4-19)

and the normalized axial velocity

\[ \overline{q}_x = \frac{q_x}{\alpha_2}, \]  

(4-20)

where \( l_{dr} \) denotes the borehole length and

\[ \alpha_2 = \alpha_1 K_x 8 \frac{l_{dr}}{d_{dr}} \frac{h_0}{d_{dr}}, \]  

(4-21)

Eq. (4-18) can be written as follows:

\[ \frac{d\overline{q}_x}{d\xi} = \overline{h} - 1. \]  

(4-22)

The axial pipe flow velocity is given by Eq. (4-8) with \( K_x \) after Eq. (4-9). Neglecting the viscosity term in Eq. (4-9) (which is a reasonable assumption for hydraulically rough pipes) and considering Eqs. (4-16), (4-19), (4-20) and (4-21), results in the following differential equation for the axial pipe flow:

\[ \frac{d\overline{h}}{d\xi} = \alpha_3 \overline{q}_x^2, \]  

(4-23)

with the dimensionless coefficient

\[ \alpha_3 = 8 \frac{K_x^2}{gh_0} \left( \frac{l_{dr}}{d_{dr}} \right)^3 \left( \frac{h_0}{d_{dr}} \right)^2 \left( \log_{10} \frac{k_{s,eq}}{3.71d_{dr}} \right)^2. \]  

(4-24)

Eqs. (4-22) and (4-23) represent a system of two differential equations for the normalized axial flow velocity and the normalized hydraulic head, to be solved for the boundary conditions of no axial inflow into the borehole from the ground and zero hydraulic head at the outlet:

\[ \overline{q}_x (1) = 0, \quad \overline{h} (0) = 0. \]  

(4-25)
It is evident from Eqs. (4-22) and (4-23) that the hydraulic head distribution depends solely on the dimensionless coefficient \( \alpha_3 \), which combines all influencing parameters: ground permeability and far-field hydraulic head; borehole diameter, length and roughness.

For the latter derivations, it is valuable to compute the average hydraulic head as well as the maximum head gradient in the borehole. Taking account of Eq. (4-22), the average hydraulic head over the borehole interval \((0, \zeta)\) reads as follows;

\[
\bar{h}_{av}(\zeta) = \frac{1}{\zeta} \int_0^\zeta \bar{h} d\zeta = 1 + \frac{\bar{q}_\alpha(\zeta) - \bar{q}_\alpha(0)}{\zeta},
\]

which for \( \zeta = 1 \) and considering that \( \bar{q}_\alpha(1) = 0 \) provides the average head over the entire borehole:

\[
\bar{h}_{av} = 1 - \bar{q}_\alpha(0).
\]

The maximum hydraulic gradient occurs at the outlet of the borehole \((\zeta = 0)\) and results from Eqs. (4-23) and (4-27) in:

\[
\frac{d\bar{h}}{d\zeta} \bigg|_{\zeta=0} = \alpha_3 \left(1 - \bar{h}_{av}(1)\right)^2,
\]

i.e., the average hydraulic head provides an indication of the maximum hydraulic gradient.

In spite of its apparent simplicity, the system of Eqs. (4-22) and (4-23) cannot be solved analytically. It can be solved, however, by a standard differential equation solver (the linearly implicit Euler method in Wolfram Mathematica's NDSolve subroutine), thus allowing an independent validation of the numerical results obtained by the finite element method. In addition, an approximate solution of the system of Eqs. (4-22) and (4-23) can be obtained analytically, taking the radial inflow constant over the entire borehole considering the average hydraulic head (see next section).

4.2.3.2.2 Approximate solution

Taking the radial inflow rate constant and according to the average hydraulic head \( \bar{h}_{av}(1) \), Eq. (4-22) becomes:

\[
\frac{d\bar{q}_\alpha}{d\zeta} = \bar{h}_{av} - 1.
\]

Integrating this equation for the boundary condition \( q_\alpha(1) = 0 \) results in

\[
\bar{q}_\alpha = \left(\bar{h}_{av}(1) - 1\right)(\zeta - 1),
\]

which inserted into Eq. (4-23) yields a differential equation for the hydraulic head,

\[
\frac{d\bar{h}}{d\zeta} = \alpha_3 \left(\bar{h}_{av}(1) - 1\right)^2 \left(\zeta - 1\right)^2,
\]
the integration of which provides the distribution of the hydraulic head:

\[ \bar{h}(\xi) = \alpha_3 \left( \bar{h}_w(1) - 1 \right)^2 \left( \frac{\xi^3}{3} - \xi^2 + \xi \right). \]  

(4-32)

Integrating the r.h.s. of this equation on the interval (0, 1) yields an algebraic equation for the average head,

\[ \bar{h}_w(1) = \frac{1}{\xi^3} \int_0^1 \bar{h}_w \xi = \frac{\alpha_3}{4} \left( \bar{h}_w(1) - 1 \right)^2, \]  

(4-33)

the solution of which for \( \bar{h}_w < 1 \) reads as follows:

\[ \bar{h}_w = 1 + \frac{2}{\alpha_3} \sqrt{\left( 1 + \frac{2}{\alpha_3} \right)^2 - 1}. \]  

(4-34)

Inserting Eq. (4-33) into Eq. (4-32) gives the head distribution

\[ \bar{h} = 4\bar{h}_w \left( \frac{\xi^3}{3} - \xi^2 + \xi \right). \]  

(4-35)

The maximum head and the maximum head gradient occur at the end (\( \xi = 1 \)) and at the outlet (\( \xi = 0 \)) of the borehole, respectively, and read as follows:

\[ \bar{h}_{\text{max}} = \bar{h}(1) = \frac{4}{3} \bar{h}_w, \]  

(4-36)

\[ \left. \frac{d\bar{h}}{d\xi} \right|_{\xi=0} = 4\bar{h}_w. \]  

(4-37)

4.2.3.2.3 Computational results and validation of the FE-solution

Figure 4.14 shows the distribution of the normalized hydraulic head \( \bar{h} \) according to the approximate equation (4-35) (dashed lines) and the numerical solution of Eqs. (4-22) and (4-23) (solid lines). The hydraulic head increases along the borehole and with the value of coefficient \( \alpha_3 \) (i.e. according to Eq. (4-24): with increasing ground permeability \( K_g \) for given initial hydraulic head \( h_0 \), borehole length \( l_d \), diameter \( d_d \) and roughness \( k_{s,eq} \)).

The error of the approximate solution increases with the coefficient \( \alpha_3 \). For \( \alpha_3 \leq 0.1 \), however, the head in the first third of the borehole (which would be decisive in a face stability analysis) is less than 1% of the initial head \( h_0 \) and both the approximate and the numerical solution coincide. For \( \alpha_3 = 1 \), the approximate solution slightly overestimates the head (error in the maximum hydraulic gradient 1.8%).

Up to \( \alpha_3 \leq 2 \), the approximate solution is sufficiently accurate or slightly conservative and the error in the maximum hydraulic gradient is less than 5%. For higher \( \alpha_3 \)-values, the error in the maximum gradient increases (10, 16 and 45% for \( \alpha_3 = 5, 10 \) and 100). The reason can be explained by comparing the results for \( \alpha_3 = 1 \) and 10 in terms of the normalized hydraulic head \( \bar{h} \) (Fig. 4.15a and b) and axial flow velocity \( \bar{q}_a \) (Fig. 4.15c and d). The approximate solution assumes a linear velocity distribution
along the borehole, which coincides with the numerical solution only for small values of \( \alpha_3 \) (i.e., for moderate permeability of \( K_g = 3.06 \times 10^{-5} \text{ m/s} \) in the example shown in Fig. 4.15c), but deviates for higher \( \alpha_3 \) (\( K_g = 9.68 \times 10^{-5} \text{ m/s} \); Fig. 4.15d). However even for \( \alpha_3 = 10 \), the deviation in hydraulic head distribution appears mainly in the rear borehole, while the average head in the first third of the borehole is sufficiently accurate (Fig. 4.15b).

Figure 4.14. Simplified borehole problem: Normalized hydraulic head \( \bar{h} \) as a function of the normalized borehole length \( \xi \) for several values \( \alpha_3 \)

Figure 4.15. Simplified borehole problem: Comparison of the numerical and the simplified solution in terms of (a,b) the normalized hydraulic head \( \bar{h} \) and (c,d) the normalized flow velocity \( \bar{q}_x \) as a function of the normalized borehole length \( \xi \)
Figure 4.16. Simplified borehole problem: Comparison of the FEM and the numerical solution in terms of the normalized hydraulic head $\tilde{h}$ as a function of the normalized borehole length $\xi$ for several ground permeability $K_g$ (one single borehole with $h_0 = 100$ m, $l_d = 30$ m, $d_0 = 0.1$ m, $k_s,eq = 5$ mm).

Figure 4.16 shows that the normalized hydraulic head according to the FE-method (solid lines in Fig. 4.16; taken from Fig. 4.9a) agrees well with the numerical solution (dash-dotted lines in Fig. 4.16; according to Eqs. (4-22) and (4-23)). More specifically, the solutions are practically identical in the case of a highly permeable ground ($K_g = 10^{-3}$ m/s). For $K_g = 10^{-4}$ m/s, the numerical solution underestimates the hydraulic head at the rear borehole by 8% compared to the FEM-results (due to the assumption of radial inflow, which does not take account of the axial flow velocity). The underestimation is larger in a lower-permeability ground ($K_g \leq 1 \cdot 10^{-5}$ m/s), but both results are within the same range of hydraulic head reduction (of less than 10% of initial head for $K_g = 1 \cdot 10^{-4}$ m/s and less than 0.3% for $K_g = 1 \cdot 10^{-6}$ m/s).

Figure 4.17. Problem setup for the comparative analysis of the tunnel example
4.2.4 Face stability

4.2.4.1 Computational model

Face stability is analysed for a tunnel example considering six axial drainage boreholes drilled from the face (Fig. 4.17; limit equilibrium model described in Section 2.2). The numerical model contains the boreholes as porous material of equivalent permeability according to Eq. (4-9); the permeability orthogonal to the borehole axis is taken 100 times higher than that of the ground $K_g$. The tunnel lining is considered as waterproof up to the face (no-flow boundary condition); the tunnel face including the borehole outlets are considered as seepage faces (atmospheric pressure). The water table is assumed to remain constant in spite of the drainage action of the tunnel (no drawdown). Table 4.2 summarizes the parameters assumed for the seepage flow and limit equilibrium analyses.

<table>
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<th>Table 4.2. Parameters for the comparative analyses of the hydraulic capacity in the tunnel example</th>
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<td><strong>Problem layout</strong></td>
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<td>Tunnel diameter</td>
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<td><strong>Ground</strong></td>
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<td>Unit weight water</td>
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<td>Coeff. of lateral stress prism</td>
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<td><strong>Drainage boreholes</strong></td>
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<td>Equivalent permeability</td>
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<td>Transition pressure</td>
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<td>Kinematic viscosity water</td>
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<td>Equivalent sand roughness</td>
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</table>

4.2.4.2 Characteristic results

The influence of the hydraulic capacity of the boreholes is evaluated in terms of the face support pressure that is needed for stability. For the purposes of comparison the following two borderline cases are considered: (i) no drainage measures, i.e. pore pressure relief only due to the natural drainage action of the tunnel face; and (ii), boreholes of sufficient capacity (which means that the borehole walls represent seepage faces under atmospheric pressure). These two borderline cases bound the range of face support pressures that would be needed if pressure develops inside the boreholes.
Figure 4.18. Face stability and water discharge of the tunnel example ($k_s,_{eq} = 5$ mm, other parameters according to Table 4.2): (a) required support pressure $s$ as a function of failure angle $\omega$ and (b) as a function of the permeability of the ground $K_g$; (c) water discharge $Q$ as a function of the permeability of the ground $K_g$ on a logarithmic scale.

Figure 4.18a shows the support pressure $s$ required for face stability in a cohesionless ground as a function of the wedge angle $\omega$ for several ground permeabilities $K_g$. The support pressures for the two above-mentioned borderline cases are marked by crosses and triangles. A high ground permeability leads to a considerably higher required support pressure ($s = 702$ vs. 300 kPa for $K_g = 1 \cdot 10^{-3}$ vs. $K_g \leq 1 \cdot 10^{-5}$ m/s) and to a more extended unstable region (critical angle $\omega_{cr} = 62^\circ$ vs. $35^\circ$). In medium or low permeability ground ($K_g \leq 1 \cdot 10^{-5}$ m/s), the necessary support pressure is identical to that obtained assuming that the borehole walls represent seepage faces. Figure 4.18b shows that this is also true for higher ground cohesion $c$.

Figure 4.18c shows the relationship between discharge $Q$ and ground permeability $K_g$ in a double logarithmic scale. The solid lines indicate the discharge when considering the hydraulic capacity; the dotted line is added for orientation and indicates the total discharge when assuming sufficient capacity in the boreholes. Up to a permeability of $K_g = 1 \cdot 10^{-5}$ m/s, most of the discharge accumulates from the
drainage boreholes (ratio $Q_{dr}/Q_{tot} ≈ 70\%$ in Fig. 4.18c, where “$Q_{tot}$” denotes the total discharge and “$Q_{dr}$” indicates the sum of discharge of all six drainage boreholes). For higher ground permeability, the effectiveness of the drainage boreholes with respect to pore pressure relief decreases, which results in less discharge from the boreholes. In extremely permeable ground ($K_g = 1\cdot10^{-3} \text{ m/s}$), inflow from the boreholes is only 23% of the total inflow, while large discharge from the tunnel face is predicted ($Q_{face} = 3.3 \text{ m}^3/\text{s}$ in Fig. 4.18c; keeping in mind the possible overestimation of $Q_{face}$ due the assumption of laminar flow in the ground may, cf. Section 4.2.3.1.1). In conventional tunnelling practice, such a large discharge is probably too high and additional sealing measures such as grouting would be required. The latter would reduce ground permeability and thus result in lower borehole pressures or even change in the flow regime within the boreholes.

Chapter 2 provides nomograms for calculating the required support pressure under the assumption of sufficient capacity of the drainage boreholes. Question arises about the applicability of these

<Figure 4.19. Required support pressure $s$ normalized by the support pressure $s_{nomo}$ (according to the nomograms when assuming sufficient capacity, i.e. boreholes under atmospheric pressure) as a function of the permeability of the ground $K_g$ for (a) variable surface roughness $k_{s,eq}$ of the borehole, (b) ground cohesion $c$, (c) elevation in water table $H_w$ and (d) borehole length $l_{b}$ (other parameters according to Table 4.2)>
nomograms in the case of pressure developing in the drainage boreholes. Figure 4.19 shows the required support pressure \( s \) considering hydraulic capacity as a ratio of the support pressure according to the nomograms \( s_{nomo} \), plotted as a function of the ground permeability \( K_g \) when considering several values of roughness of the borehole wall \( k_{s,eq} \) (Fig. 4.19a), of the ground cohesion \( c \) (Fig. 4.19b), of the elevation of water table \( H_w \) (Fig. 4.19c) and of the drainage borehole length \( l_{dr} \) (Fig. 4.19d). Overall for \( K_g \leq 1 \cdot 10^{-5} \text{ m/s} \), the same face support is needed as when assuming sufficient hydraulic capacity in the boreholes. But in extremely permeable ground (\( K_g = 1 \cdot 10^{-3} \text{ m/s} \)), the hydraulic capacity of the boreholes limits their effectiveness considerably. More specifically, the deviation in support pressure increases with the roughness of the borehole walls, but this effect is of secondary importance (Fig. 4.19a). Crucial is however the deviation in support pressure with increasing ground cohesion (e.g., 5 times more support pressure is required when considering hydraulic capacity in ground of \( c = 100 \text{ kPa} \) and \( K_g = 1 \cdot 10^{-3} \text{ m/s} \) in Fig. 4.19b). In ground of cohesion \( c = 150 \text{ kPa} \), there is even no additional face support required according to the nomograms (thus it is not any more possible to determine a ratio \( s/s_{nomo} \)), but considering hydraulic capacity results in a support pressure of about 300 kPa (Fig. 4.18b). Further and again of secondary importance, the deviation in support pressure slightly increases with increasing elevation of the initial water table (Fig. 4.19c) as well as with increasing borehole length (Fig. 4.19d).

4.2.4.3 Applicability of nomograms

The applicability range of the nomograms (Chapter 2) can also be determined using the approximate analytical solution of the ground - single borehole interaction problem (cf. Section 4.2.3.2).

4.2.4.3.1 Radial inflow from the ground into several boreholes

In order to make a reasonably conservative statement about the applicability range, the upper limit of the quantity of radial inflow into the boreholes is considered. The maximum quantity of water inflow would occur in the case of ideal drainage (i.e., atmospheric pressure) of the entire tunnel cross-section up to the length \( l_{dr} \) ahead of the face (Fig. 4.20). Neglecting flow in the axial direction, the total quantity of inflow in a cylindrical drained zone of radius \( a \) (the tunnel radius) and length \( l_{dr} \) (the borehole length), reads as follows (Eq. (4-15)):

\[
Q = 2\pi al_{dr} \frac{K_g h_0}{a \ln \frac{h_0}{l_{dr}}} (1 - \bar{h}) .
\]  

(4-38)

Figure 4.20. Simplified borehole problem: (a) ideal drainage up to a length \( l_{dr} \) ahead of the tunnel of radius \( a \) and assumption of the same discharge \( Q \) being collected via (b) \( n \) drainage boreholes.
As the quantity of inflow into the outermost boreholes is negligibly higher than the quantity of inflow into the central boreholes, it is reasonable to assume that the total quantity of inflow is distributed uniformly among all drainage boreholes. The average radial inflow per borehole is thus equal to the total quantity of inflow divided by the number and the borehole shell surface (Fig. 4.20):

\[
q_r = \frac{1}{n} \frac{2K_{rh}h_0}{d_{dr} \ln \frac{h_0}{a}} (1 - \bar{h}).
\]  

(4-39)

Thus the radial flow from the ground can be written as previously

\[
q_r = \alpha_i \frac{2K_{rh}h_0}{d_{dr}} (1 - \bar{h}).
\]  

(4-15)

the only difference being that the coefficient \( \alpha_i \) (rather than given by Eq. (4-17)) reads here as follows:

\[
\alpha_i = \left( n \ln \frac{h_0}{a} \right)^{-1} = \left( n \ln \frac{2h_0}{D} \right)^{-1}.
\]  

(4-40)

4.2.4.3.2 **Criterion of admissible maximum hydraulic gradient**

The nomograms of Chapter 2 can be considered as applicable if the maximum hydraulic gradient is less than a certain value \( i_{adm} \) (hereafter referred to as "admissible maximum hydraulic gradient"):

\[
\left. \frac{d \bar{h}}{dx} \right|_{x=0} = \frac{h_0 d \bar{h}}{l_{dr} d \bar{h}} \leq i_{adm}.
\]  

(4-41)

which leads considering Eqs. (4-37) and (4-34) to:

\[
4 \left( 1 + \frac{2}{\alpha_3} \right) \sqrt{\left( 1 + \frac{2}{\alpha_3} \right)^2 - 1} \leq \frac{l_{dr}}{h_0} i_{adm}.
\]  

(4-42)

The normalized maximum gradient (l.h.s. of Eq. (4-42)) is always smaller than \( \alpha_3 \). If we assume for \( \alpha_3 < 2 \) that

\[
\alpha_3 = \frac{l_{dr}}{h_0} i_{adm}
\]  

(4-43)

(an assumption, which is slightly on the safe-side, but not too conservative in the range for \( \alpha_3 < 2 \), then the normalized maximum gradient is always smaller than the r.h.s. of Eq. (4-43).
According to the range of applicability given in Chapter 2 (see there Table 2.3: \( T/D > 5 \) which in terms of hydraulic head is about \( h_0/D > 5 \); and the drainage length of axial boreholes of \( l_{dr}/D = 1.5 \) and \( 3 \) given in Figs. 2.24 and 2.25), Eq. (4-43) becomes

\[
\alpha_s = \frac{l_{dr}}{h_0} \frac{D}{i_{adm}} = \frac{3}{5} i_{adm} < 2 ,
\]  

(4-44)

i.e. the assumed range of \( \alpha_s < 2 \) for Eq. (4-43) corresponds to a range up to \( i_{adm} = 3.3 \).

4.2.4.3.3 Admissible ground permeability

Inserting Eq. (4-24) (which considers the inflow into several boreholes according to Eq. (4-40)) into Eq. (4-43) leads to a simple relationship for the admissible ground permeability

\[
K_g = n \left( \frac{i_{adm}}{8} \frac{gD}{\alpha_s} \right)^{0.5} \left( \frac{l_{dr}}{D} \right)^{-1} \left( \frac{d_{dr}}{D} \right)^{2.5} \left( \frac{h_0}{D} \right)^{1} \ln \frac{2h_0}{D} \log_{10} \frac{3.71}{k_{s,eq}} \frac{d_{dr}}{D} .
\]

(4-45)

The higher the admissible permeability of the ground, the larger is the range of applicability of the nomograms provided in Chapter 2. The admissible permeability of the ground \( K_g \) decreases and the situation becomes less favourable with

- increasing normalized initial head \( h_0/D \);
- decreasing number of boreholes \( n \);
- increasing normalized borehole length \( l_{dr}/D \);
- decreasing borehole diameter \( d_{dr} \);
- increasing roughness \( k_{s,eq} \);
- increasing tunnel diameter \( D \);
- decreasing admissible gradient \( i_{adm} \).

Figure 4.21 shows the permeability of the ground \( K_g \) as a function of the admissible hydraulic gradient \( i_{adm} \). Three cases of normal, favourable and adverse drainage situations are considered (Table 4.3). The highlighted area indicates the range of admissible permeability of each drainage case when considering the upper and lower bound of normalized hydraulic head \( h_0/D = 5 \) and \( 40 \), respectively. In the normal drainage case, ground permeabilities \( K_g = 1 \) to \( 3 \cdot 10^{-5} \) m/s are admissible even for low hydraulic gradient \( i_{adm} = 0.1 \) (which is in accordance with the FEM-results previously discussed in Fig. 4.18). In the favourable drainage case, the admissible ground permeability increase to \( K_g = 1 \cdot 10^{-4} \) m/s, while in the adverse case only \( K_g = 1 \cdot 10^{-7} \) m/s are admissible (\( i_{adm} = 0.1 \)).

In conclusion, the provided nomograms of Chapter 2 can be used in normal drainage cases (Table 4.3) up to moderately permeable ground \( (K_g \leq 1 \cdot 10^{-4} \) m/s). In that wide range, the nomograms are valid even when considering the hydraulic flow capacity of the drainage boreholes.
Figure 4.21. Admissible permeability of the ground $K_g$ as a function of the admissible hydraulic gradient $i_{adm}$ providing sufficient capacity in the drainage boreholes (drainage cases see Table 4.3)

Table 4.3. Values of the considered drainage cases

<table>
<thead>
<tr>
<th>Parameters of drainage cases:</th>
<th>normal</th>
<th>adverse</th>
<th>favourable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of drainage boreholes</td>
<td>$n$ [-]</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>Diameter of drainage boreholes</td>
<td>$d_{dr}$ [m]</td>
<td>0.1</td>
<td>0.06</td>
</tr>
<tr>
<td>Equivalent sand roughness</td>
<td>$k_{se}$ [m]</td>
<td>0.05</td>
<td>0.015</td>
</tr>
<tr>
<td>Tunnel diameter</td>
<td>$D$ [m]</td>
<td>10</td>
<td>15</td>
</tr>
<tr>
<td>Length of drainage boreholes</td>
<td>$l_{dr}/D$ [-]</td>
<td>1.5</td>
<td>3</td>
</tr>
<tr>
<td>Initial hydraulic head</td>
<td>$h_0/D$ [-]</td>
<td>5, 40</td>
<td>5, 40</td>
</tr>
</tbody>
</table>
4.3 Borehole casings

A common measure when encountering unstable drainage borehole walls is using casings (Fig. 4.22). The latter may cause difficulties in drilling (e.g. jamming of the casing or failure of the joints of two casing segments due to high friction at the borehole as shown in Fig. 4.22d) and thus trial drilling may be necessary. In terms of hydraulic head field, the casing screens reduce the effectiveness of the drainage measures as they impede pore pressure relief around the boreholes due to the restricted passage of water to small openings. The present section quantifies that effect for the borehole screens sketched in Figure 4.23a representing fairly sealed casings (compare to Fig. 4.22).

4.3.1 Computational model

Face stability is analysed for the tunnel example of Figure 4.17 and according to the limit equilibrium model described in Section 2.2. The water table is assumed to remain constant in spite of the drainage action of the tunnel (no drawdown). The tunnel lining is considered as waterproof up to the face (no-flow boundary condition); the tunnel face is considered as seepage faces (atmospheric pressure). The casing is taken as an impervious boundary with the exception of its openings (Fig. 4.23a), which are considered as seepage faces under atmospheric pressure. Possible local losses in hydraulic potential due to water entering the openings are neglected (for consideration of local losses at well screens see for example Siwoń, 1987; Ouyang et al., 1998; Clerno, 2006).

Table 4.4 summarizes the parameters assumed for the seepage flow and limit equilibrium analyses.

![Figure 4.22. Exemplary screens of drainage casings: (a) perforated, (b) slotted (courtesy of Baosu Pipe), (c) oblong slotted (courtesy of Pancera Tubi e Filtri S.r.l.). (d) Failure of the screw thread at the joint of the segmental casings](image-url)
Table 4.4. Parameters for the comparative analyses of cased boreholes in the tunnel example

<table>
<thead>
<tr>
<th>Problem layout</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of cover</td>
<td>$H$</td>
</tr>
<tr>
<td>Elevation of water table</td>
<td>$H_{w}$</td>
</tr>
<tr>
<td>Tunnel diameter</td>
<td>$D$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Ground</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective cohesion</td>
<td>$c$</td>
</tr>
<tr>
<td>Angle of eff. internal friction</td>
<td>$\phi$</td>
</tr>
<tr>
<td>Submerged unit weight</td>
<td>$\gamma'$</td>
</tr>
<tr>
<td>Unit weight water</td>
<td>$\gamma_w$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Shear resistance of the vertical slip surfaces</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Coeff. of lateral stress wedge</td>
<td>$\lambda_w$</td>
</tr>
<tr>
<td>Coeff. of lateral stress prism</td>
<td>$\lambda_p$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Drainage boreholes</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>$d_{dr}$</td>
</tr>
<tr>
<td>Length</td>
<td>$l_{dr}$</td>
</tr>
<tr>
<td>Number</td>
<td>$n$</td>
</tr>
</tbody>
</table>

4.3.2 A single cased drainage borehole

Figure 4.23b shows the distribution of the pressure $p$ (normalized by the initial pressure $p_0$) along the cased borehole wall of the boreholes screens according to Figure 4.23a. The pressure minimum and maximum occur at the opening of the casings and at the midpoint between the openings, respectively. The larger the spacing $x_s$ of the openings and the lower the opening ratio $R_s$, the higher is the intermediate pressure. Increasing the opening ratio (compare e.g. black to blue dashed lines in for screen C and G in Fig. 4.23b: twice the draining area leads to 1.5 times lower intermediate pressures) has about the same effect as decreasing the spacing (compare e.g. screen D to G: half the spacing leads to a 1.5 times lower intermediate pressures). A pressure reduction to about $p/p_0 \approx 0.1$ is possible when using fairly dense slotted or perforated borehole casings (e.g. slotted borehole screen A, B; e.g. perforated borehole screen E of Fig. 4.23a).

4.3.3 Face stability

Face stability is analysed for the most efficient screens A, B, C, E, F and G of Figure 4.23a and compared to the support pressure required for face stability when considering no casings (i.e. seepage faces under atmospheric pressure at all borehole wall).

The effect of the borehole casings is evaluated in terms of the face support pressure $s$ that is needed for stability as a function of the ground cohesion $c$ (Fig. 4.23c). In case using a very densely slotted casing (screen A of Fig. 4.23), the same support pressure is required as when considering uncased boreholes (red dashed line in Fig. 4.23c). But the required support pressure increases fast for more sealed casings (compare e.g. slotted screens A to B to C in Fig. 4.23c). In ground of cohesion $c = 150$ kPa, the more sealed casings require additional face support, while with the densely slotted screen A the tunnel face is stable without additional support.
Thus the use of the nomograms provided in Chapter 2 is recommended for densely slotted or perforated screens only (e.g. slotted screen A of Fig. 4.23a: spacing $x_s \leq d_s/4$, opening ratio $R_s \geq 12\%$; the same effect results when using a screen with about double the perforations of screen E).

![Diagram showing slotted borehole screen and perforated borehole screens](image)

<table>
<thead>
<tr>
<th>screen</th>
<th>$d_s/x_s$ [%]</th>
<th>$R_s$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>screen A</td>
<td>4</td>
<td>12.7</td>
</tr>
<tr>
<td>screen B</td>
<td>3</td>
<td>9.6</td>
</tr>
<tr>
<td>screen C</td>
<td>2</td>
<td>6.4</td>
</tr>
<tr>
<td>screen D</td>
<td>1</td>
<td>3.2</td>
</tr>
<tr>
<td>screen E</td>
<td>4</td>
<td>6.0</td>
</tr>
<tr>
<td>screen F</td>
<td>3</td>
<td>4.5</td>
</tr>
<tr>
<td>screen G</td>
<td>2</td>
<td>3.0</td>
</tr>
<tr>
<td>screen H</td>
<td>1</td>
<td>1.5</td>
</tr>
</tbody>
</table>

$* R_s = \text{seepage area} / \text{borehole shell surface}$

**Figure 4.23.** (a) Screens of the considered drainage casings. (b) Pressure $p$ normalized by the initial pressure $p_0$ along the borehole axis $x$ for the single cased borehole. (c) Required support pressure $s$ as a function of the ground cohesion $c$ for the tunnel example (parameters according to Table 4.4)
4.4 Conclusions

Limited flow capacity of the drainage boreholes may be expressed as flow in porous media according to Darcy’s law of a non-linear, equivalent permeability derived by considering pipe-flow equations. It allows for numerical determination of the hydraulic head field when considering the interaction between seepage flow in the ground and turbulent pipe flow in the borehole. The FEM-results are in good agreement with an analytical solution derived in the thesis.

For ground of permeability $K_g \leq 1 \cdot 10^{-5}$ m/s, flow capacity does not limit drainage effectiveness, independently of the initial hydraulic head, the surface roughness of the borehole wall, the drainage borehole length or the ground cohesion. It is thus safe to assume sufficient drainage capacity (i.e. atmospheric pressure acting at the borehole walls in numerical modelling) and the design aids for evaluating face stability provided in Chapter 2 are applicable. For higher ground permeabilities however, flow capacity forces a several times higher support pressure for face stability, tending to the values required without any drainage measures. Due to the high water discharge from the open tunnel face in such permeable ground, probably additional sealing measures such as grouting would be needed to provide safe work-conditions at the tunnel face. Obviously, permeability would thus change to more favourable conditions again.

Drainage success may also be limited by borehole casings. Densely slotted or perforated screens do not limit drainage effectiveness in terms of face stability. More sealing casings (i.e. spacing $x_s > d_b/4$, ratio of seepage area $R_s < 13%$; cf. Fig. 4.23) should be handled with particular caution as they necessitate an increase of the support pressures given in the nomograms of Chapter 2.
5. Other operational and environmental factors limiting the effectiveness of advance drainage measures for face stability

5.1 Introduction

Chapter 5 successively studies the operational and environmental factors of Table 1.1, which may reduce the effectiveness of the drainage measures with respect to face stability: (i) the lead-time in poorly permeable ground, where pore pressure relief by advance drainage may take a prohibitively long time to work; (ii) environmental constraints with respect to the drawdown of the water table, or (iii) the magnitude of settlements, which may impose limits on the amount of admissible pore pressure relief, and finally (iv) the quantity of water inflow, which may be restricted due to the pumping capacity on site.

In ground of low permeability, pore pressure relief by advance drainage boreholes may take a prohibitively long time to occur. The lead-time for reaching practically stationary condition in a Darcy-material is discussed in literature e.g. for drawdown of the water table due to wells, but not when considering drainage measures for increasing the stability of a tunnel face. Section 5.2 closes this knowledge gap and quantifies the time required until the hydraulic head is lowered sufficiently to be taken into account for face stability considerations according to Chapter 2.

Drainage-induced pore pressure relief may be undesirable for environmental reasons such as the disturbance of the hydrogeological conditions due to the drawdown in water table or the admissible subsidence of ground. Operational constraints such as the pumping capacity available on site may additionally limit the amount of admissible water inflow (e.g. as encountered during construction of Lake Mead Intake No. 3; Nicola et al., 2014). Water inflow to wells is widely discussed in tunnelling literature (Theis, 1932; see e.g. Coli and Pinzani, 2014, for a recent state of the art), but mostly limited to tunnel cross-sections far away of the face (i.e. two-dimensional consideration only). Heuer (1995; 2005) estimated the inflow at the tunnel face (which needs to be considered in three dimensions) to about 1-5 times higher as the cross-sectional discharge. This is in good agreement with the inflow calculated by Anagnostou (1995), where additionally the groundwater drawdown due to tunnelling with an open tunnel face was quantified. Atwa et al. (2000) computed the drawdown of the water table for a single shallow tunnel example with advance drainage boreholes of unusually large drainage diameters, which probably overestimate the drainage effect (diameter of boreholes 0.2 m, diameter of tunnel 5 m).

In Section 5.3, the additional groundwater drawdown caused by axial boreholes from the tunnel face is quantified. Then, the settlement induced by pore water relief resulting from drainage measures is discussed for both situations of continuous groundwater recharge and groundwater drawdown (Section 5.4). Finally, the water inflow arising from drainage measures is analysed in Section 5.5.
5.2 Time dependency

5.2.1 Problem

Previous investigations considered stationary conditions of the hydraulic head field. But in ground of low permeability, achieving this state might require inadmissible time during the construction process. Imagine tunnelling in competent rock and approaching a weak zone (of equal permeability as the competent rock), where an advance drainage measure is required for ensuring face stability (Fig. 5.1a). In order to relief the destabilizing gradients (temporally and spatially) sufficiently ahead of the tunnel face, drainage boreholes are drilled in some distance to the weak rock (Fig. 5.1b). Then, excavation proceeds by turns with the drilling operations of the drainage boreholes (No.1 and 2, respectively, in Fig. 5.1c and 5.1d).

Before drilling the boreholes into the weak rock, the pore pressure ahead of the tunnel face is at least partially relieved due to the previous drainage stage (indicated with dotted line in Fig. 5.1c and 5.1d). However and with respect to the lead-time required until a drainage measure reaches practically stationary conditions, the most critical situation would be drilling the drainage boreholes and neglecting the pore pressure relief resulting from the previous drainage stage. Thus the section in hand assumes a standstill long enough to lead to a virtually steady state head field around the open tunnel face. Then the drainage boreholes are immediately enabled and the time-dependent relief of pore water pressure is analysed. Two drainage schemes are considered: the borderline case of ideal drainage (Fig. 5.2b) and a common drainage scheme of axial drainage boreholes drilled from the tunnel face (Fig. 5.2c).

Figure 5.1. Sketches of the construction process of tunnel excavation (No. 1) and subsequent drilling works of drainage boreholes (No. 2) when approaching a weak rock requiring drainage measures for face stability

5.2.2 Computational model

The seepage flow analyses are performed for the example of a deep cylindrical tunnel (Fig. 5.2a). The hydraulic head at the far-field boundaries is taken equal to the initial hydraulic head \( h_0 \) (i.e. equal to the elevation of the water table above the tunnel axis). The water table is assumed to remain constant in spite of the drainage action of tunnel face and boreholes (no drawdown). The tunnel lining is considered as impervious up to the face (no-flow boundary condition). The borehole walls are considered as seepage faces under atmospheric pressure (which presupposes uncased boreholes of sufficient hydraulic capacity).
The time-development of the hydraulic head field is determined by numerical, three-dimensional analyses assuming Darcy’s law for the following cases: ideal advance drainage (complete pore pressure relief in the ground ahead of the tunnel face, Fig. 5.2b) and advance drainage via axial boreholes from the tunnel face (Fig. 5.2c). The seepage flow analysis consists of two steps: in the first step, the steady state hydraulic head field is computed considering the tunnel face as a seepage face. In the second step, the drainage measure is activated and a transient analysis starting from the hydraulic head field prevailing after step 1 is carried out.

Table 5.1 summarizes the assumed parameter values.

**Figure 5.2.** (a) Problem setup with drainage schemes considered of (a) no advance drainage, (b) ideal advance drainage by means of complete pore pressure relief in the ground ahead of the tunnel face and (c) advance drainage by means of axial boreholes from the face

<table>
<thead>
<tr>
<th>Table 5.1. Parameters for the comparative analyses of degree of pore pressure relief</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Problem layout</strong></td>
</tr>
<tr>
<td>Distance ( T = \min(H, H_w) ) ( T ) \ 50-400 m</td>
</tr>
<tr>
<td>Tunnel diameter ( D ) \ 10 m</td>
</tr>
<tr>
<td><strong>Hydraulic properties of the ground</strong></td>
</tr>
<tr>
<td>Ratio permeability/storage ( K_g/S_s ) ( 10^7-10^9) m²/s</td>
</tr>
<tr>
<td><strong>Drainage boreholes</strong></td>
</tr>
<tr>
<td>Diameter ( d_{dr} ) \ 0.1 m</td>
</tr>
<tr>
<td>Length ( l_{dr} ) \ 30 m</td>
</tr>
<tr>
<td>Number ( n ) \ 2, 6, 12</td>
</tr>
</tbody>
</table>
5.2.3 Characteristics of time-dependent behaviour

Figure 5.3a illustrates the typical time-development of the hydraulic head field \( h \) (normalized by the initial head \( h_0 \)) considering the borderline case of ideal drainage ahead of a subsea tunnel (inset in Fig. 5.3a). The initial state is marked with \( t = 0 \). As time passes (\( t = 0 \) to 27 h in Fig. 5.3a), the hydraulic head field approaches the stationary distribution (\( t = \infty \)). The average hydraulic head above the tunnel face

\[
B(t) = \frac{1}{H} \int_{D/2}^{H+D/2} h(t, x_3) \, dx_3
\]

serves as time-dependent measurement of drainage progress when considering a specific drainage scheme. The degree of pore pressure relief of a drainage scheme at any specific time \( t \) is evaluated as

\[
M_{\text{eff}}(t) = \frac{B(t) - B(0)}{B(\infty) - B(0)}
\]

and runs from zero (no pore pressure relief for initial state at \( t = 0 \)) to one (full pressure relief for stationary conditions at \( t = \infty \)).

For a homogeneous, isotropic porous media obeying Darcy’s law, the time-dependent hydraulic head field \( h(t,x_3) \) appearing in Eq. (5-1) is governed by the diffusion equation

\[
S_s \frac{\partial h}{\partial t} = K_g \nabla^2 h ,
\]

according to which the ratio of conductivity \( K_g \) to specific storage \( S_s \) is decisive. The latter is defined as volume of water that a unit volume of a (confined) aquifer releases from storage under a unit decline in hydraulic head and is a function of the deformability of the porous medium and the compressibility of water \( c_w \). For linearly-elastic, isotropic ground, the specific storage is

\[
S_s = \gamma_w \left( \frac{3(1-2\nu)}{E} + n g \cdot c_w \right),
\]

where \( \gamma_w \), \( n_g \), \( \nu \) and \( E \) denotes the unit weight of water, the porosity of the saturated ground, its Poisson’s ratio and its modulus of elasticity, respectively (cf. Anagnostou, 1995). The lower both permeability and stiffness of the ground, the more lead-time a drainage measure requires (Eqs. (5-4) and (5-3)). Table 5.2 lists the characteristic values of specific storage \( S_s \), permeability \( K_g \) as well as the resulting ratio \( K_g/S_s \) for typical lithologies.

<table>
<thead>
<tr>
<th>Typical lithologies</th>
<th>( S_s ) [m(^{-1})]</th>
<th>( K_g ) [m/s]</th>
<th>( K_g/S_s ) [m(^2)/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>( 10^{-2} )</td>
<td>( 10^{-9} )</td>
<td>( 10^{-7} )</td>
</tr>
<tr>
<td>Sand (fine)</td>
<td>( 10^{-3} )</td>
<td>( 10^{-6} )</td>
<td>( 10^{-4} )</td>
</tr>
<tr>
<td>Gravel (medium)</td>
<td>( 10^{-4} )</td>
<td>( 10^{-2} )</td>
<td>( 10^{-3} )</td>
</tr>
<tr>
<td>Rock (highly fissured)</td>
<td>( 10^{-4} )</td>
<td>( 10^{-4} )</td>
<td>( 10^{-4} )</td>
</tr>
<tr>
<td>Rock (unfissured)</td>
<td>( 10^{-4} )</td>
<td>( 10^{-10} )</td>
<td>( 10^{-2} )</td>
</tr>
</tbody>
</table>
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Figure 5.3. Time-dependent pore pressure relief when considering ideal advance drainage: (a) Normalized hydraulic head distribution $h/h_0$ above the tunnel face starting at initial condition ($t = 0$) at several time steps up to stationary conditions ($t = \infty$; $K_g/S_s = 0.1$) and (b) degree of pore pressure relief $M_{eff}$ as a function of the time $t$ on a logarithmic scale for several ratios of permeability to specific storage $K_g/S_s$.

The influence of the ratio $K_g/S_s$ is discussed by means of Figure 5.3b showing the degree of pore pressure relief $M_{eff}$ as a function of time $t$ (using a logarithmic scale) for several ratios $K_g/S_s$ (again considering ideal drainage of a subsea tunnel, inset in Fig. 5.3a). The curves equally decrease over time and a 10 times lower ratio $K_g/S_s$ takes 10 times longer to reach the same degree of pore pressure relief (compare point B to D in Fig. 5.3b and see Eq. (5-3)).

Consider as an example a weak rock ($E = 1$ GPa, $\nu = 0.25$, $n_g = 0.2$ with $c_w = 4.8 \cdot 10^{-10}$ Pa$^{-1}$, $\gamma_w = 10$ kN/m$^3$ inserted in Eq. (5-4)), where the specific storage becomes $S_s = 1.6 \cdot 10^{-5}$ m$^{-1}$. In case this rock is highly fissured ($K_g = 1 \cdot 10^{-4}$ m/s), a ratio of $K_g/S_s \approx 10$ results. Practically steady conditions ($M_{eff} = 99.9\%$) are reached after only 0.17 h (point A in Fig. 5.3b). Only if the weak rock appears in unfavourable combination with low permeability (e.g. $K_g = 1 \cdot 10^{-8}$ m/s due to fissures filled with clayey silt), a considerable lead-time of more than 1000 h is required (point B in Fig. 5.3b).

More generally, lead-times of less than 13 h are necessary for ratios $K_g/S_s \geq 0.1$ (point C in Fig. 5.3b). This range covers the typical permeable lithologies, i.e. is valid for permeability of up to $K_g = 10^{-6}$ m/s in a ground of average specific storage ($S_s \approx 10^{-5}$ m$^{-1}$; see Table 5.2). However, time-dependency becomes decisive in clayey formations, where a very long lead-time would be necessary in order to reach a practically steady head field (e.g. for point D in Fig. 5.3b nearly two years). Note, that in such low-permeability ground presumably vacuum lances are installed which in turn accelerate pore pressure relief. Furthermore in unfissured rock, permeability is obviously lower as well (see Table 5.2), but as in sound rock face stability is no issue, this case is of no interest for our concern.
5.2.4 Effect of axial drainage arrangements

5.2.4.1 Degree of pore pressure relief

A common drainage scheme in tunnelling is drilling axial boreholes from the face (Fig. 5.2c with drainage borehole number $n$, length $l_{dr}$, and diameter $d_{dr}$). Dimensional analysis and considering the structure of Eq. (5-3) shows that the degree of pore pressure relief $M_{eff}$ (Eq. (5-2)) may be non-dimensionally expressed as

$$M_{eff} = f \left( \frac{H_w}{D}, \frac{H}{D}, \frac{tK_g}{S_s D^2}, \frac{l_{dr}}{D}, \frac{d_{dr}}{D}, n \right). \quad (5-5)$$

The seepage flow domain extends either up to the ground surface $H$ (subaqueous tunnels) or up to the groundwater table $H_w$. The upper boundary of the numerical model is thus located at distance $T = \min(H, H_w)$ above the tunnel crown and Eq. (5-5) simplifies for a given drainage scheme to

$$M_{eff} = f \left( \frac{tK_g}{S_s D^2} \right). \quad (5-6)$$

Figure 5.4 shows the degree of pore pressure relief $M_{eff}$ as a function of the dimensionless time-factor $tK_g/(S_s D^2)$ on a logarithmic scale for the example of a subsea tunnel and considering $n = 2, 6$ and 12 boreholes of fixed length and diameter (inset in Fig. 5.4). The curves for all axial drainage arrangements nearly coincide, despite the different seepage area, and require a lead-time of $tK_g/(S_s D^2) \approx 100$ (“$n = 2, 6, 12$” in Fig. 5.4). The pore pressure relief caused by ideal drainage measure is faster and requires less lead-time ($\Delta tK_g/(S_s D^2) \approx 50$ for $M_{eff} \approx 1$ in Fig. 5.4).

![Figure 5.4. Degree of pore pressure relief $M_{eff}$ as a function of the dimensionless time-factor $tK_g/(S_s D^2)$ on a logarithmic scale (drainage schemes see Fig. 5.2c)](image-url)
5.2.4.2 Lead time for face stability

Regarding face stability at a given support pressure, a drainage measure should reach nearly stationary conditions at least in the vicinity of the tunnel face, while far away, the hydraulic head may still decrease with time. The left-hand part of Figure 5.5 shows the hydraulic head field in the vicinity of a tunnel example (Fig. 5.5a). The equipotential line highlighted in white is practically at steady-state conditions for high degree of pore pressure relief $M_{\text{eff}}$ (compare Fig. 5.5c to b). The destabilizing gradients increase, i.e. the line gets closer to the face, with decreasing $M_{\text{eff}}$ (Fig. 5.5d). Figure 5.5e quantifies the face support pressure $s$ that is needed for stability as a function of the ground cohesion $c$ for several $M_{\text{eff}}$. The support pressure is calculated as described in Section 2.2, but with introducing the pore pressure field at a time $t$ into the limit equilibrium equations (an approximation, which is

![Figure 5.5](image)

**Figure 5.5.** (a) Tunnel example, (b-d) hydraulic head field and (e) required face support pressure $s$ as a function of the ground cohesion $c$ for selected degree of pore pressure relief $M_{\text{eff}}$ (other parameter according to Table 5.1)

![Figure 5.6](image)

**Figure 5.6.** Time-factor $K_{p}/(S,D^2)$ as a function of normalized distance $T/D$ when considering $n = 2, 6$ and 12 axial drainage boreholes (Fig. 5.2c) for a degree of pore pressure relief $M_{\text{eff}} = 95$ (a) and 90% (b)
sufficiently accurate for our purpose of comparison; for more details see e.g. Eisenstein and Samarasekera, 1992). At a degree of pore pressure relief of $M_{\text{eff}} \geq 90\%$, virtually the same support as at steady state is required (less than 3\% deviation; Fig. 5.5e) and the design nomograms provided in Chapter 2 are applicable. Therefore, we target such degree of pore pressure relief.

For $n = 2, 6$ and 12 boreholes from the face, Figure 5.6 shows the dimensionless time $tK_{g}/(S_sD^2)$ that must elapse in order that pore pressure relief reaches $M_{\text{eff}}$ of 90 or 95\%, and plots it as a function of dimensionless distance $T/D$ (see inset in Fig. 5.6a).

Pore pressure relief requires more time with larger seepage flow domain (compare point A to C in Fig. 5.6a), however for distance $T/D \geq 20$ the increase is small (compare point B to C in Fig. 5.6a). The smaller the seepage area of the drainage measure, the more time is required reaching the target degree of pore pressure relief; though differences within the drainage schemes are – if any – rather small (compare point C to D in Fig. 5.6a). For lower degree of pore pressure relief, the results nearly coincide for all borehole numbers (Fig. 5.6b). Summarizing, a time-factor of about $tK_{g}/(S_sD^2) \geq 58$ provides the target degree of pore pressure relief of 90\% independently of the axial drainage arrangements and cover or water head (Fig. 5.6b; for $M_{\text{eff}} = 95\%$ it is about $tK_{g}/(S_sD^2) \geq 80$ in Fig. 5.6a).

5.2.4.3 Application example

Consider as an example excavating a circular tunnel in frequently fissured rock with silty infillings after having drilled six advance drainage boreholes (Table 5.3) and assume that we know the face support required for stability at steady-state conditions in the fault zone soon to be encountered. We want to determine the required lead-time which provides a degree of pore pressure relief of 90\%, i.e. allows us to stay with our known face support. Entering Figure 5.6 for the target degree of pore pressure relief and distance of seepage flow domain ($M_{\text{eff}} = 90\%$; $T/D = 16.4$) results in $tK_{g}/(S_sD^2) = 48$ (point A in Fig. 5.6b). Thus $t = 48 S_sD^2/K_g = 2.1$ days after having drilled the drainage boreholes, the hydraulic head in the vicinity of the tunnel face reaches nearly stationary conditions. (Please note that this corresponds with the exact numerical computation for the tunnel example of Table 5.3, which results in a lead-time of 2 days. Thus Figure 5.6 is valid for a wide range of tunnel diameters as long as the drainage geometry is according to the inset (i.e. $l_d/D = 3$ and $d_d/D = 0.01$; see also Eq. (5-5)).

<table>
<thead>
<tr>
<th>Table 5.3. Parameters for the application example</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Problem layout</strong></td>
</tr>
<tr>
<td>Depth of cover $H$</td>
</tr>
<tr>
<td>Elevation of water table $H_w$</td>
</tr>
<tr>
<td>Tunnel diameter $D$</td>
</tr>
<tr>
<td><strong>Hydraulic properties of the ground</strong></td>
</tr>
<tr>
<td>Permeability $K_g$</td>
</tr>
<tr>
<td>Specific storage $S_s$</td>
</tr>
<tr>
<td><strong>Drainage boreholes</strong></td>
</tr>
<tr>
<td>Diameter $d_d$</td>
</tr>
<tr>
<td>Length $l_d$</td>
</tr>
<tr>
<td>Number $n$</td>
</tr>
<tr>
<td>--------------------------------------------------</td>
</tr>
<tr>
<td>82 m</td>
</tr>
<tr>
<td>100 m</td>
</tr>
<tr>
<td>5 m</td>
</tr>
<tr>
<td>$2 \cdot 10^{-7}$ m/s</td>
</tr>
<tr>
<td>$3 \cdot 10^{-5}$ m/l</td>
</tr>
<tr>
<td>0.05 m</td>
</tr>
<tr>
<td>15 m</td>
</tr>
<tr>
<td>6</td>
</tr>
</tbody>
</table>
This seemingly long period must be put into perspective of tunnelling practice: The first drainage boreholes (here: length of 15 m) are drilled in safe distance of the fault zone and require about 1 day of drilling operation. Assuming a daily advance of 2 m in conventional tunnelling, it takes about 4 days until the recommended minimum borehole length of 7 m is reached (cf. Section 2.3.3.3: minimum borehole length of about 1.5D). The drilling operation of the next stage of drainage boreholes requires another day, which gives a sum of 7 days and the required lead-time has passed before entering into the fault zone.

5.3 Groundwater drawdown

5.3.1 Problem

In case of sensitive hydrogeological conditions, the maximum drawdown in water table might be limited. Drainage boreholes increase the seepage area and therefore increase the drawdown in water table. Thus the maximum admissible groundwater drawdown may limit the number and/or length of drainage boreholes, which in turn necessitates higher support pressures for face stability (cf. Chapter 2). On the other hand and if there are no restrictions to groundwater drawdown, the lowered hydraulic head favours face stability and less support pressure is required. This section analyses the additional groundwater drawdown caused by axial drainage boreholes drilled from the tunnel face at steady-state conditions.

5.3.2 Computational model

The seepage flow analyses are performed for the example of a cylindrical tunnel (Fig. 5.7) in a homogeneous, isotropic porous medium obeying Darcy’s law. The tunnel lining is impervious (no-flow boundary condition); both face and drainage boreholes are considered as seepage faces under atmospheric pressure (drainage boreholes of sufficient capacity). The distance \( L \) is chosen large enough not to affect the amount of drawdown in the vicinity of the tunnel face (cf. Arn, 1989). The hydraulic head at the far-field boundaries is taken equal to the initial water table \( H_w \), except for the initial groundwater surface (abcd in Fig. 5.7), where a no-flow condition is assigned. The seepage flow domain comprises both the lower, saturated region underneath the water table and the overlying, unsaturated ground. The permeability is assumed to drop sharply at \( p = 0 \) (to 1/100 of the saturated conductivity according to the residual flow method of Desai and Li, 1983; cf. Anagnostou, 1995; Bear et al., 1968) and the free surface is defined as the surface on which pore pressure is equal to atmospheric pressure (\( p = 0 \)). The maximum drawdown \( \Delta H_w \) ahead of the tunnel face is evaluated (Fig. 5.7).

Figure 5.7. Problem setup for the comparative analysis of the tunnel example
The hydraulic head field is determined by numerical, three-dimensional steady-state seepage analyses for the following drainage schemes: no advance drainage (Fig. 5.2a), ideal advance drainage (Fig. 5.2b) and advance drainage via axial boreholes from the tunnel face (Fig. 5.2c).

Table 5.4 summarizes the parameters.

<table>
<thead>
<tr>
<th>Problem layout</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elevation of water table</td>
</tr>
<tr>
<td>Tunnel diameter</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Ideal drainage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length ( l_{dr} )</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Drainage boreholes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter ( d_{dr} )</td>
</tr>
<tr>
<td>Length ( l_{dr} )</td>
</tr>
<tr>
<td>Number ( n )</td>
</tr>
</tbody>
</table>

5.3.3 Borderline cases

The relative drawdown in water table is minimal, if drainage occurs solely via the tunnel face (Fig. 5.2a) and maximal for ideal advance drainage (Fig. 5.2b). For dimensional reasons, the drawdown \( \Delta H_w \) for a given drainage scheme may be expressed as

\[
\frac{\Delta H_w}{H_w} = f \left( \frac{H_w}{D} \right). \tag{5-7}
\]

In case of ideal advance drainage (Fig. 5.2b), the extent \( l_{dr}/D \) of the drained area ahead of the face appears as additional dimensionless parameter. The relative drawdown decreases with increasing initial water head (compare point A to B in Fig. 5.8, see also e.g. Arn, 1989).

The results agree well with Anagnostou (1995) who quantified the drawdown due to the seepage area of the tunnel face only (compare line \( l_{dr} = 0 \) in Fig. 5.8 to the grey crosses taken from Fig. 13 of Anagnostou, 1995). For low initial water head, groundwater level decreases below the tunnel roof due to the drainage action of the face (\( \Delta H_w/H_w > 1 \)). For a high initial water head of 10 times the tunnel diameter, the groundwater level decreases to \( \Delta H_w/H_w = 0.2 \) in case of very long ideal drainage area (point C in Fig. 5.8), and is thus five times higher than for minimum seepage area (point B for no advance drainage in Fig. 5.8).

5.3.4 Effect of distinct drainage boreholes

Figure 5.9 shows the groundwater drawdown in the presence of 2, 6 and 12 axial drainage boreholes (locations after Fig. 5.2c) as a function of the initial water head. The groundwater drawdown is bounded by the borderline cases of no boreholes and ideal drainage up to \( l_{dr} = 3D \) ahead of the face.

The increased seepage area due to the axial drainage boreholes leads to a more pronounced drawdown of groundwater; however does not decrease the water table as much as ideal drainage. Doubling the
Figure 5.8. Relative groundwater drawdown $\Delta H_w / H_w$ as a function of the initial water table $H_w/D$ when considering ideal drainage areas of variable length $l_{dr}$ (parameters according to Table 5.4)

Figure 5.9. Relative groundwater drawdown $\Delta H_w / H_w$ as a function of the initial water table $H_w/D$ when considering several axial advance drainage borehole schemes ($n = 2, 6$ and $12$ boreholes at location of Fig. 5.2c; other parameters according to Table 5.4)

The borehole number does not affect the head as much as starting with advance drainage at all (e.g. for $H_w/D = 4$ in Fig. 5.9: compare point A to B vs. C to D). However, the additional drawdown of water table due to the axial boreholes is substantial. For an example of $H_w/D = 4$, the drawdown in water table is about three times larger with advance drainage boreholes than without (compare point D to A in Fig. 5.9). The drawdown decreases clearly with increasing initial head, while the share of the advance drainage stays about the same (compare ratios of E over F to D over A in Fig. 5.9). For initial head $H_w/D > 10$, the groundwater drawdown is less than 4-12% of the initial water table for all considered drainage schemes.
In case a specific drawdown of the water table is not admissible, the number of drainage boreholes has to be reduced (which in turn of course requires a higher support pressure for face stability, see nomograms of Chapter 2) or the pore pressure relief has to be limited considering other measures such as sealing grouting or ground freezing.

5.4 Settlements

5.4.1 Problem and approach

Water table drawdown may be inadmissible (or limited) not only due to environmental reasons, but also due to potential settlement of the ground surface. We limit ourselves here to a rough assessment of maximum settlement by means of the constrained modulus\(^6\) \(E_S\) (e.g. El Tani, 2001). When assuming linear-elastic behaviour of the ground (Hooke’s law) and considering the change of effective stresses \(\Delta \sigma'\) equal to the change in pore water pressure \(\Delta p\) (Terzaghi and Jelinek, 1954), the settlement \(u_S\) at the ground surface is

\[
 u_S = \int \varepsilon \, dx_3 = \int \frac{\Delta \sigma'}{E_S} \, dx_3 = \int \frac{\Delta p}{E_S} \, dx_3 ,
\]  

(5-8)

where \(\varepsilon\) and \(x_3\) denote the strain of the ground and the vertical coordinate, respectively. The settlement is calculated at the location of the maximum drawdown of water table (see Section 5.3.2) and considers the change in pressure from ground surface down to well below the tunnel invert, where pressure distribution approximates again to the initial, hydrostatic distribution (compare solid to dashed lines and integration area \(\Delta p\) highlighted in grey in Fig. 5.10).

---

\(\Delta\) The constrained modulus \(E_S\) is found directly from one-dimensional consolidation test (oedometer) and may be expressed related to the Young’s Modulus \(E\) and the Poisson’s ratio \(\nu\) by \(E_S = E(1-\nu)/((1+\nu)(1-2\nu))\).
5.4.2 Effect of drainage boreholes

Figure 5.11 shows the surface settlement $u_S$ (multiplied by the constrained modulus $E_S$ of the ground) as a function of the normalized water head for the axial drainage borehole layouts ($n = 2, 6$ and $12$ boreholes at location of Fig. 5.2c) as well as the reference cases (“none” and “ideal”). The solid lines indicate drawdown of water table, the dashed lines no drawdown (see later Section 5.4.2.1). Expectedly, the settlement-factor increases with increasing initial head and with increasing number of advance drainage boreholes.

Consider as an example a deep tunnel in very weak rock ($D = 10$ m, $H_W = 100$ m, $E_S = 500$ MPa). With six axial drainage boreholes, considerable surface settlement of $6.7$ cm would have to be expected (point A in Fig. 5.11a). But as the draining action of the tunnel face causes already an extensive deformation of $4$ cm (point B in Fig. 5.11a), the drainage boreholes trigger relatively small additional settlements, which might be acceptable for the benefit of face stability. In particular, the required face support pressure with advance drainage is only $8$ kPa (assuming ground cohesion of $100$ kPa, additional ground parameters see Table 4.4; nomograms provided in Chapter 2 when taking account of the water drawdown according to Fig. 5.9). But without advance drainage, a high support pressure of $314$ kPa is necessary. Such values are barely feasible in conventional tunnelling even by heavy face bolting (cf. Section 2.5.1). Thus advance drainage proves feasibility of that tunnelling example provided that considerable surface settlement is accepted (else, ground improvement e.g. by grouting would be required).

5.4.2.1 Comparison to no drawdown of groundwater table

In case where water is recharged e.g. by a direct link to an adjacent lake, no groundwater drawdown takes place. To draw a comparison, the computational model described in Section 5.3.2.1 was used but with fixed initial hydraulic head as boundary condition at ground surface (i.e., at abcd in Fig. 5.7).
Without drawdown, pore pressure relief takes place only in the vicinity of the tunnel face. Thus the surface settlements decrease compared to the case without groundwater drawdown (compare dashed to solid lines in Fig. 5.11). The difference in drawdown slightly increases with increasing hydraulic head due to the larger area of influence (compare e.g. points E/F = 1.09 to C/D = 1.14 in Fig. 5.11).

When considering the previous tunnel example (Section 5.4.2), surface settlements considering six axial drainage boreholes decrease to 5.8 cm, while the draining action of the tunnel face causes deformation of 3.7 cm (Fig. 5.11a). On the other hand, the required face support pressure slightly increases to 30 kPa with advance drainage and to 340 kPa without drainage boreholes (nomograms provided in Chapter 2).

5.4.3 Comparison to FE-model

Calculating the settlements by Eq. (5-8) provides an estimate of the ground settlement on the safe side. It considers neither the spatial pore pressure relief nor the spatial distribution of the settlements. For comparison and considering the reference case of tunnelling without drainage boreholes at constant water table, comparative numerical calculations of settlement are carried out by means of the built-in routine for steady-state consolidation in the FE-code Abaqus.

The ground is considered as a linear-elastic porous medium obeying the principle of effective stresses and Darcy’s law. Poisson’s ratio is assumed as \( \nu = 0.3 \) and the Young’s Modulus \( E \) is chosen such as it relates to the constrained modulus \( E_s \) (see footnote 6). The tunnel lining, the lateral far-field boundaries and the lower far-field boundary (located three tunnel diameters below the invert according to Fig. 5.10) do not allow for displacements normal to the respective boundary. Focus is placed only on drainage-induced displacement (i.e. no need to consider the initial stress state, which would be required analysing the excavation-induced settlements). In the first step of calculation, the hydraulic head at the tunnel face is equal to the initial head acting at the far-field boundaries (i.e. hydrostatic stress condition). In the second step, the pore water pressure at the tunnel face is immediately lowered to the atmospheric value, while still the initial head is assumed at the far-field boundaries (no drawdown in water table), and the maximum displacements at the ground surface are calculated.

The results are marked with crosses in Figure 5.11 and are expectedly lower than the values calculated by means of Eq. (5-8), however showing the same trend (compare crosses to grey dashed line in Fig. 5.11). The overestimation increases with increasing hydraulic head (from factor 6.5 to 10.9 at \( H_w/D = 2 \) to 10, respectively). The previously discussed settlements of Figure 5.11 are thus estimates distinct on the safe side, but still provide a quantitative insight of the additional settlements due to drainage measures in comparison to tunnelling without advance drainage.

5.5 Water discharge

5.5.1 Problem

In cases of a highly permeable ground, water inflow from the tunnel face and the boreholes is substantial and appropriate pumping capacity needs to be installed on construction site. The admissible pumping rate may limit the number and/or length of the drainage boreholes, which in turn necessitates higher support pressures for face stability (cf. Chapter 2) or requires additional measures such as sealing grouting. This section analyses steady-state inflow from both tunnel face and axial drainage boreholes assuming sufficient capacity (computational model see Section 5.3.2).
5.5.2 Effect of drainage boreholes

We first consider the favourable case of groundwater drawdown (drainage schemes see Fig. 4.25). Figure 5.12 shows the cumulated discharge\(^7\) of all seepage area on logarithmic scale for the considered reference cases and axial advance drainage schemes (the solid lines indicate drawdown of water table, the dashed lines no drawdown; see later Section 5.5.2.1). Water inflow increases with increasing initial head and proves adequate accordance with previous investigations (grey crosses in Fig. 5.12 with values taken from Fig. 9 of Anagnostou, 1995. The values of the previous investigations are slightly higher, which is probably due to the smaller model size leading to higher gradients.).

Drilling six advance drainage boreholes creates nearly twice the seepage area compared to no advance drainage (135.1 to 78.5 m\(^2\), respectively). Due to the spatial distribution of this seepage area, the discharge collected from face and boreholes is even multiplied by a factor of 2.6 (compare point A to B in Fig. 5.12). The total water discharge increases with increasing number of boreholes and may necessitate larger pumping capacity than without advance drainage boreholes. On the other hand, advance drainage reduces the discharge at the tunnel face by over 60% (compare C to B in Fig. 5.12, where point C indicates discharge at the tunnel face when considering six boreholes), thus improves working conditions at the face provided that the water is discharged immediately from the drainage boreholes.

Figure 5.12. Water discharge \(Q\) divided by the permeability of the ground \(K_g\) (on a logarithmic scale) as a function of normalized initial water table \(H_w/D\) when considering several axial advance drainage borehole arrangements \((n = 2, 6 and 12\) boreholes at location of Fig. 5.2c; other parameters according to Table 5.4)

5.5.2.1 Comparison to no drawdown of groundwater table

In case where water is recharged e.g. by direct connection to an adjacent lake, no groundwater drawdown takes place and larger inflow of water is expected. To draw a comparison, the

\(^7\) Please note that the values of cumulated discharge are conservative estimates and presumably higher than the discharge values measured on site, since the computation does not consider any excavation-induced permeability reduction in the vicinity of the tunnel (cf. Heuer, 1995; Moon and Fernandez, 2010a).
computational model described in Section 5.3.2 was used but with fixed initial hydraulic head as boundary condition at ground surface (i.e., at abcd in Fig. 5.7).

Without drawdown, water discharge is higher due to the higher gradients prevailing (compare dashed to solid lines in Fig. 5.12). The smaller the initial water head, the larger is the difference (e.g. for two axial boreholes 16-32%, compare D/E = 1.32 to F/G = 1.16 in Fig. 5.12). The larger the seepage area (i.e. the number of boreholes), the larger is the difference in discharge between the cases with and without groundwater drawdown.

5.6 Conclusions

The lead-time, i.e. the time needed until practically steady hydraulic conditions are reached after having installed a drainage arrangement, is evaluated as time-factor $tK_g/(S_sD^2)$ (depending on the hydraulic ground properties of specific storage $S_0$ and permeability $K_g$ as well as on the tunnel diameter $D$ and the time $t$). The considered drainage arrangements of several axial boreholes drilled from the face show similar time-dependent behaviour and reach after $tK_g/(S_sD^2) \geq 58$ a pore pressure relief, which is so close to stationary conditions that the nomograms of Chapter 2 can be used to assess face stability. From a practical point of view, the lead-times for most of the permeable lithologies are short enough that they may not be decisive, because drilling operation for the boreholes will probably take more time. Only in clayey formations ($K_g \ll 10^{-6}$ m/s) operationally decisive lead-times are required (note that this is typically the range where vacuum lances are installed in order to increase drainage effectiveness; i.e. the lead-time decreases).

Care should be taken concerning the hydraulic parameters of the ground, where ratio of permeability $K_g$ to specific storage $S_s$ is decisive for the lead-time of a drainage measure (a decrease by factor of 10 necessitates a 10 times longer lead-time). The specific storage $S_s$ may be expressed as a function of the modulus of elasticity of the ground (i.e. the lower the stiffness of the ground, the more lead-time is required). In literature, its value varies only marginal within the common permeable lithologies for tunnelling ($S_s \approx 10^{-5}$ m$^{-1}$). But permeability $K_g$ may vary heavily even in small space by several decimal power and should therefore thoroughly be determined.

Groundwater drawdown due to advance drainage measures increases with increasing seepage area (i.e. with number and length of boreholes) and decreases with increasing initial water head. The considered drainage arrangements of 2 to 12 axial drainage boreholes multiplies groundwater drawdown by a factor of 2 to 3 compared to the drainage action of only the tunnel face.

The potential settlement of the ground surface is estimated assuming linear-elastic ground behaviour and considering the pressure reduction induced by the drainage measure. It shows the considerable settlement due to drainage measures, which however might be acceptable if settlement due to open-face tunnelling is acceptable at the first place. The settlement due to groundwater drawdown is 10-15% larger than the settlement due to local pore water pressure relief only (i.e. no groundwater drawdown).

The total amount of water inflow increases substantially due to advance drainage boreholes and is obviously even higher when assuming no groundwater drawdown. However, advance drainage reduces the inflow at the face, thus improves working conditions at the tunnel face provided that the water is discharged immediately from the drainage boreholes.
6. On the stabilizing effect of drainage on tunnel support in grouted fault zones

6.1 Introduction

Previous investigations of Anagnostou and Kovári (2003) considered two borderline cases of drainage measures of grouting bodies: a “grouting body without drainage” and an “ideally drained grouting body”. The grouting body without drainage refers to the case of a low-permeability grouting body, where the tunnel excavation does virtually not affect the hydraulic head in the untreated ground (Fig. 6.1a). The hydraulic head difference between the in situ head and the excavation boundary is thus dissipated mainly within the grouted body leading to seepage forces $f_s$, which are high if the thickness of the grouted zone is small and the tunnel is located well below the water table. The ideally drained grouting body refers to the assumption of complete pore pressure relief within the grouting body (Fig. 6.1b) thus the seepage forces develop in the untreated ground in safe distance from grouting body and tunnel. However, large inflows may be encountered due to the abandoned sealing effect of the grouting body.

The chapter in hand extends the investigation of Anagnostou and Kovári (2003) with point to following aspects of drainage measures: (i) the effect of ideal drainage of only the inner part of the grouting body; (ii) the effect induced by ideal drainage of an area larger than the subsequent constructed grouting body, and (iii) the consideration of the pore pressure relief resulting from specific arrangements of drainage boreholes instead of assuming ideal drainage.

Radial drainage boreholes arranged in the inner part of the grouting body (Fig. 6.1c) relieve the pore pressure close to the excavation boundary and decrease both the deformation and the risk of inner erosion. They thus may be the method of choice in cases where the grouting body is intended to

![Figure 6.1](image)

*Figure 6.1.* Hydraulic head field when considering (a) the grouting body without drainage, (b) the ideally drained grouting body (c), radial drainage of the inner part of the grouting body and (d) coaxial drainage of the grouting body
maintain its outer sealing effect, but where possible poor injection works in the inner part of the grouting body require a controlled pressure relief in order to keep the high hydraulic gradients in safe distance to the excavation boundary. However, the seepage forces develop concentrated in the outer shell of the grouting body and may there lead to local over stressing.

In case where for example the water head prevailing renders impossible grouting operation due to maximum feasible injection pressure, drainage in advance of the injection works is an option (e.g. by means of boreholes coaxial to the tunnel axis in Fig. 6.1d). The pore pressure relief resulting from the drainage measure increases the shear resistance of the ground (pre-consolidation) prior to grouting. The seepage forces develop in safe distance from the tunnel and, depending on the location of the axial drainage boreholes, also from the grouting body. However, potentially large inflows may develop due to the drainage measures.

The chapter is organized as follows: Section 6.2 explains the modelling concept for calculation of the characteristic line, i.e. the stress-displacement behaviour of the excavation boundary when assuming ideal drainage. The analytical solutions considering ideal drainage before and after grouting are derived in Section 6.3, on the basis of which characteristic results are presented (Section 6.4). Section 6.5 explains the modelling concept of considering specific drainage borehole arrangements by hydraulic-mechanical coupled FE-modelling. The results (Sections 6.6 and 6.7) quantify the stabilizing effect of several different drainage borehole arrangements drilled after as well as in advance of grouting and place emphasis on the effect of number, length and location of boreholes. Section 6.7 finishes with an example of a fault zone of limited extend and quantifies the pre-consolidation effect of drainage boreholes being drilled in advance of grouting. Finally, Section 6.8 summarizes the findings with focus on the difference between consideration of the analytical solution assuming ideal drainage and consideration of the drainage borehole arrangements.

6.2 Modelling ideal drainage

6.2.1 Static system, initial and boundary conditions

The static system for the stability analysis of a deep, circular tunnel through a grouted fault zone of infinite length is considered as made up by three elements: the tunnel lining support pressure, the grouting body and the untreated, surrounding ground (Fig. 6.2; Anagnostou and Kovári, 2003).

The untreated ground is considered as hollow cylinder with an outer radius at infinity and an inner radius \( b \) coinciding with the boundary to the grouting body. The state of initial stresses is assumed to be homogeneous and isotropic. The external boundary of the ground is subjected to the effective initial stress \( \sigma_0 \)', which magnitude equals to the effective overburden pressure.

The injection body is a thick-wall cylinder with an inner radius \( a \) and an outer radius \( b \). Injection is assumed not to lead to any substantial changes in the stress state and thus the stresses acting after injection are equal to the initial stresses and the stress state is still homogeneous and isotropic (cf. Anagnostou and Kovári, 2003). The stress state immediately prior to the tunnel excavation is taken as reference state for deformations. At the excavation boundary \( r = a \), a uniform lining support pressure \( \sigma_a \) is assumed and at \( r = b \), effective pressure \( \sigma'_b \) is acting.

Ideal drainage up to radius \( r = l \) is considered as pore pressure relief to atmospheric pressure and thus the hydraulic boundary at the drainage radius \( l \) is assumed as a seepage faces (atmospheric pressure at \( r = l \)). The far-field boundary condition assumes an undisturbed head field (pressure is equal to the initial pressure; \( p = p_0 \)) for the area beyond the radius \( r \geq R \), where \( R \) is taken equal to the depth of the tunnel below the water table.
6.2.2 Material properties

The ground (both untreated and grouted) is assumed to be a homogeneous and isotropic porous material with linear-elastic, perfectly-plastic behaviour obeying the principle of effective stress, the Mohr-Coulomb yield criterion and Darcy’s law. The seepage forces are equal to the gradient of the pore water pressure field (assumption of deep tunnel, cf. Anagnostou and Kovári, 2003).

The material parameters of the untreated ground have no subscript, the ones of the injection body are denoted with subscript \( I \) (effective cohesion \( c_I \), angle of internal friction \( \phi_I \), Young’s Modulus \( E_I \), Poisson’s ratio \( \nu_I \), uniaxial compressive strength \( f_{ci} \), angle of dilatancy \( \psi_I \), loosening factor \( \kappa_I \), material constant \( m_I \), and permeability \( k_I \)).

The material constant is defined as

\[
m = \frac{1 + \sin \phi}{1 - \sin \phi} . \tag{6-1}
\]

The uniaxial compressive strength is interrelated by the cohesion and the friction angle:

\[
f' = \frac{2c \cos \phi}{1 - \sin \phi} ; \tag{6-2}
\]

the loosening factor may be expressed depending on the dilatation angle (Anagnostou and Kovári, 2003)

\[
\kappa = \frac{1 + \sin \psi}{1 - \sin \psi} . \tag{6-3}
\]
The permeability of the grouting body is assumed to be several times lower than the one of the untreated ground. The higher the initial ground permeability, the higher is the feasible reduction (e.g. for cement grouts and highly permeable ground upmost to a factor 1000; Greenwood and Thomson, 1984).

Please note that the permeability of the grouting body is assumed to remain constant during plastification (Fig. 6.3a). An increase in permeability, which is possible during plastification due to micro-cracks developing within the plastic zone, would lead to a shift of the hydraulic boundary condition up to radius \( r = \rho \), which increases the hydraulic gradients acting in the outer zone of the grouting body and in turn may lead to additional overstressing, i.e. an even larger plastic zone (Fig. 6.3b). On the other hand and if grouting would not be able to fill all joints, stress redistribution and joint closure around the tunnel could lead to a decrease in permeability, which in turns increases the hydraulic gradients close to the tunnel face (Fig. 6.3c; cf. Mas Ivars, 2006; Fernandez and Moon, 2010a,b,c).

The stiffness of the grouting body is assumed to be considerably higher than the one of the untreated ground. After injection work, no displacement of the outer boundary of the grouting body is assumed \((\bar{u}_b = 0)\).

![Figure 6.3. Schematic sketch of pore pressure distribution \( p \) in the grouting body when considering (a) uniform permeability, (b) an increased permeability due to plastification and, (c) a decreased permeability due to stress redistribution](image)

### 6.2.3 Solution method and dimensioning criterion

Due to the various assumptions been made, the system is rotational symmetric to the tunnel axis. As the length of the fault zone is assumed to be infinite (which is a conservative assumption in case encountering short fault zones; cf. Anagnostou and Kovári, 2003), plane strain conditions apply.

Each independent structural element of the static system is described by means of its characteristic line (also known as “ground response curve”) expressing the interdependence between radial stress \( \sigma \) and radial displacement \( u \). The characteristic line for the total structure is achieved by providing the mechanical requirement of equilibrium and compatibility at the boundaries between adjacent elements; i.e. continuity of both radial stress and displacement. The system can be subdivided any further (e.g. into an elastic and a plastic zone, see Fig. 6.2) and reassembled to the overall system providing continuity at the interfaces.

Each’s element stiffness and bearing capacity influences the stress distribution in the overall static system. Some parameters are pre-determined in a specific tunnelling situation (the mechanical properties of the untreated ground, the initial stress state prior to any measure, the water pressure); some may be chosen by the engineer: the tunnel lining support, the diameter and the material properties of the grouting body and the drainage measures.
In tunnelling practice, tunnel linings provide typically a support of $\sigma_a < 1$ MPa. Grouting bodies with a diameter corresponding to two or at most three times the tunnel diameter proved to be adequate (Kovári, 1992). Injections usually allow uniaxial compressive strength up to $f_{ci} = 4-5$ MPa (strength of lean concrete). When the strength of the grouting body is inadequate or the load too high, the grouting body plastifies up to the plastic radius $\rho$ (Fig. 6.2). An extensive plastification may lead to loosening of the grouting body, which impairs its sealing effect, and/or may lead to inner erosion. Thus limiting plastification constitutes another dimensioning criterion, which is in the following referred to as “degree of plastification”

$$\lambda = \frac{\rho - a}{b - a},$$  \hspace{1cm} (6-4)

i.e., the share of the plastic zone $\rho$ of the extent of the grouting body ($b - a$).

### 6.3 Equations of ideal drainage

#### 6.3.1 Sequence of drainage and grouting

Two cases have to be distinguished concerning the sequence of works of drainage and grouting: drainage prior to injection or drainage after construction of grouting bodies (Fig. 6.4).

#### 6.3.1.1 Previous investigations: drainage after grouting ($l = b$)

Anagnostou and Kovári (2003) focused on the case of drainage after formation of the grouting body. They considered a rigid grouting body ($u_b = 0$), which is then ideally drained up to radius $r = b = l$ (Fig. 6.4a) and derived the closed-form equations for the characteristic line of the excavation boundary ($r = a$) for a partially plastified grouting body ($a \leq \rho \leq b$) as a function of the stresses acting on the grouting body $\sigma_b'$.  

![Figure 6.4. Loading $\sigma$ and pore water pressure distribution $p$ for the case of (a) ideal drainage after grouting and (b) ideal drainage in advance of grouting](image-url)
They showed - for comparison only - that the changes in pressure distribution due to advance drainage cause deformation and increase the stresses prior to the grouting measure. Assuming a plastic radius \( \rho < b \), the drainage-induced increase of the radial stress at radius \( r = b \) is

\[
\Delta \sigma'_{bDR} = \frac{P_0}{2(1-\nu)},
\]

and the drainage-induced radial displacements (due to the low stiffness of the untreated ground)

\[
u_{s,DR} = b \frac{P_0 (1+\nu)(1-2\nu)}{E 2(1-\nu)}.
\]

After drainage, the stress state is isotropic within the drained area (radius \( r < b \)).

6.3.1.2 New investigations: drainage in advance of grouting \( (l \geq b) \)

Here, drainage up to radius \( r = l \geq b \) prior to injection is considered (Fig. 6.4b). After drainage, a rigid grouting body is constructed \( (u_b = 0) \). The state immediately prior to tunnel excavation is taken as reference state for the deformations and the characteristic line of the excavation boundary is determined.

The stresses at the future boundary of the grouting body \( (r = b) \) consider the stress-increase according to Eq. (6-5):

\[
\sigma'_{bDR} = \sigma'_0 + \Delta \sigma'_{bDR} = \sigma'_0 + \frac{P_0}{2(1-\nu)},
\]

i.e., the load of the grouting body is independent of the drainage radius \( l \).

The drainage-induced deformations (Eq. (6-6)) occur prior to the injection works, i.e. they do not need to be considered for our concern.

Thus the only modification of the previous investigation of Anagnostou and Kovári (2003) on the case of drainage after formation of the grouting body is the stress state acting on the grouting body (specifically, \( \sigma'_{bDR} \) instead of \( \sigma'_b \) in Fig. 6.4). The modified equations are summarized below and Appendix B provides the matlab-code considered for the investigations in hand.

The grouting body will be partly plastified up to the radius \( r = \rho \) under the following condition:

\[
\sigma'_{bDR} = \left( \frac{b}{a} \right)^{\nu_{c+1}} \left[ 1 - m_i + (m_i + 1) \left( \frac{b}{\rho} \right)^2 \right],
\]

where overscores denote the transformed stress defined as

\[
\sigma_x = \sigma_x + \frac{c}{\tan \varphi}.
\]

According Eq. (6-9) and (6-7) it is

\[
\sigma'_{bDR} = \sigma'_{bDR} + \frac{c_i}{\tan \varphi_i} = \sigma'_0 - P_0 + \frac{P_0}{2(1-\nu)} + \frac{c_i}{\tan \varphi_i}.
\]
The displacement of the excavation boundary \(r = a\) is:

\[
\bar{u}_a = \frac{u E_I}{a \sigma_y'} - A_1 + \frac{\sigma_y'}{\sigma_{b,DR}} \left[ A_2 + A_3 \left( \frac{m + \kappa}{a} \right)^{m + \kappa} \right],
\]

where the constants \(A\) are

\[
A_1 = (1 + \nu)(1 - 2\nu),
\]

\[
A_2 = \frac{(1 + \nu)(1 - 1 + m_i \kappa_i - \nu_i (m_i + \kappa_i))}{m_i + \kappa_i},
\]

\[
A_3 = \frac{(1 - \nu)(m_i^2 - 1)}{m_i + \kappa_i}.
\]

The displacement of the outer boundary of the grouting body is

\[
u_b = \frac{a \sigma_{b,DR}'}{E_I} \frac{2 \left( m_i - 1 \right) \left( 1 - \nu_i \right) \left( \frac{b}{a} \right)}{1 - m_i + \left( m_i + 1 \right) \left( \frac{b}{\rho} \right)^2}.
\]

### 6.3.1.3 Water inflow

Seepage analysis assumes a thick-walled cylinder of inner radius \(l\) and outer radius \(R\) (Fig. 6.4b) with boundary conditions \(p(l) = 0\) and \(p(R) = p_0\). The water inflow \(Q\) per tunnel meter is

\[
Q = \frac{2 \pi k}{\rho_w g} \frac{p_0}{\ln \left( \frac{R}{l} \right)}.
\]

### 6.3.2 Extent of the radius of drainage

Anagnostou and Kovári (2003) derived the closed-form solutions of the characteristic line for a partially plastified grouting body \((a \leq \rho \leq b)\), which is considered either without any drainage, or ideally drained up to radius \(r = l\) (Fig. 6.5a and b, respectively). The grouting body without drainage (Fig. 6.5a) is subjected to large seepage forces (which may overstress the grouting body), but obtains its possible sealing effect. The ideally drained grouting body (Fig. 6.5b) impairs its sealing effect (which may cause large inflows), but is not subjected to seepage forces.

A compromise and the contribution of this section is the grouting body, which is ideally drained in its inner part \((a \leq l \leq b)\) (Fig. 6.5c), while the outer part still provides a sealing effect. The mechanical system is subdivided depending on the extent of the plastic zone \(\rho\) \((a \leq \rho \leq b)\) and the extent of the radius of drainage \(l\) \((l \leq r \leq b)\) (Fig. 6.6).
Figure 6.5. Loading $\sigma$ and pore water pressure distribution $p$ for the case of drainage after grouting: (a) grouting body without drainage, (b) ideally drained grouting body and (c) ideally drained inner part of the grouting body (a and b after Anagnostou and Kovári, 2003)
Figure 6.6. Considered system for ideal drainage of the inner part of the grouting body (a) in Case I, where the plastic radius $\rho$ is smaller than the radius of drainage $l$, (b) in Case II, where the plastic radius $\rho$ is larger than the radius of drainage $l$ and (c) the notation used in case of evolving of two plastic zones ($\rho_{in}, \rho_{out}$).

6.3.2.1 Case I ($a \leq \rho < l$)

"Case I" refers to a plastic radius which is smaller than the radius of drainage ($a \leq \rho < l$; Fig. 6.6a). The system is composed of an outer elastic grouting body ($r \geq l$ in Fig. 6.6a), which takes into account the acting seepage forces and an inner, partially plastified ring ($a \leq r \leq l$ in Fig. 6.6a), which is ideally drained. For the outer system, Anagnostou and Kovári (2003) provide the solution of an elastic grouting body without drainage (Fig. 6.5a), when modifying it slightly and considering the excavation boundary $l$ instead of $a$:

$$\sigma'_i = \frac{E_i}{l(1 + \nu_i)} \left( \frac{C_i}{1 - 2\nu_i} - C_2 \right) - \frac{1}{2} f_2,$$

(6-17)
\[ \sigma_b' = \frac{E_i}{l(1+\nu_i)} \left( C_1 - C_2 \left( \frac{l}{b} \right)^2 \right) - \frac{f_2}{2} \left( 1-\nu_i + \ln \left( \frac{b}{l} \right) \right), \]  
\tag{6-18}

\[ u_i = C_1 + C_2 - \frac{l(1+\nu_i)(1-2\nu_i)}{E_i} \sigma_o', \]  
\tag{6-19}

\[ u_b = C_1 \left( \frac{b}{l} \right) + C_2 \left( \frac{l}{b} \right) - \frac{f_2 b \ln \left( \frac{b}{l} \right) \left( 1+\nu_i \right) \left( 1-2\nu_i \right)}{2E_i} \left( 1-\nu_i \right) - \frac{b \left( 1+\nu_i \right) \left( 1-2\nu_i \right)}{E_i} \sigma_o', \]  
\tag{6-20}

where \( f_2 \) is

\[ f_2 = \frac{p_b}{\ln \left( \frac{b}{l} \right)} \]  
\tag{6-21}

with the water pressure acting at the outer boundary of the grouting body \( r = b \)

\[ p_b = \frac{\ln \left( \frac{b}{l} \right)}{\ln \left( \frac{l}{a} \right) + \frac{k_i}{k_g} \ln \left( \frac{R}{l} \right)} \cdot p_o, \]  
\tag{6-22}

and \( C_i \) denote the integration constants to be determined.

The inner system is solved the same as an ideally drained grouting body (Fig. 6.5b), but with the size of the grouting body \( l \) instead of \( b \):

\[ \sigma_{u_i}' = \frac{\left( \frac{l}{a} \right)^{m_i-1} \left( 1-m_i + \left( m_i + 1 \right) \left( \frac{l}{\rho} \right)^2 \right)}{2 \left( \frac{l}{\rho} \right)^{m_i+1}}, \]  
\tag{6-23}

\[ u_i = a \left( 1+\nu_i \right) \left( 1-2\nu_i \right) \left( \sigma_{u_i}' - \sigma_o' \right) + \frac{a\sigma_i''}{E_i} \left\{ -A_i + \frac{\sigma_o'}{E_l} \left( A_2 + A_3 \left( \frac{\rho}{l} \right)^{m_i+1} \right) \right\}, \]  
\tag{6-24}

\[ u_b = l \left( \frac{l}{a} \right)^{m_i-1} \left( 1-m_i + \left( m_i + 1 \right) \left( \frac{l}{\rho} \right)^2 \right) \]  
\tag{6-25}

with \( A_i, A_2, A_3 \) according to Eq. (6-12), (6-13) and (6-14), respectively.

At the boundary between the inner and outer system \( r = b \), the radial displacement \( u_i \) and the radial stress \( \sigma_i \) must be compatible, which leads to following solutions for the integration constants:

\[ C_i = f \left( \sigma_b', \rho \right) = \frac{P_2}{P_1} C_2 \left( 2-2\nu_i + \frac{P E_i}{l(1+\nu_i)} \right) + \frac{1}{2} f_2 \left( P_1 + P_2 \right) \frac{P_2}{P_1} \frac{f_{al}}{m_i-1} P_1, \]  
\tag{6-26}
\[ C_2 = f\left(\sigma'_i, \rho\right) = \frac{1}{2} f_2 \left[ 1 + \frac{P_1}{P_i} - \frac{1 - \nu I + \ln(b/l)}{1 - \nu I} - \frac{f_{rl}}{m_I - 1} - \sigma'_I \right] - \frac{f_{rl}}{m_I - 1} - \sigma'_I , \] (6-27)

and the uniaxial compressive strength of the grouting body \( f_{cl} \) according to Eq. (6-2). The factors \( P_i \) allow for a compact notation:

\[ P_i = \frac{a}{E_I} \left\{ \frac{2(m_I - 1)(1 - \nu^2)(l/a)}{1 - m_I + (m_I + 1)(l/\rho)^2} \right\} , \] (6-28)

\[ P_2 = \frac{l(1 + \nu_I)(1 - 2\nu_I)}{E_I} . \] (6-29)

6.3.2.2 Case II \((l \leq \rho \leq b)\)

“Case II” refers to a plastic radius which is larger than the radius of drainage \((l \leq \rho \leq b, \text{Fig. 6.6b})\). The system is composed of an outer, partially plastified ring subjected to seepage forces \((l \leq r \leq b \text{ in Fig. 6.6b})\) and an inner, drained and fully plastified ring \((a \leq r \leq l)\). Again, Anagnostou and Kovári (2003) provide a solution for each independent system. The outer system is accordingly considered as a partially plastified grouting body without drainage (Fig. 6.5a), but with excavation boundary \(l\) instead of \(a\). The relation between the stresses at \(r = l\) and \(r = b\) is expressed as

\[ f_{cl,r} = B_c \sigma'_c + B_I p_b - B_I \sigma'_I , \] (6-30)

where \( f_{cl,r} \) is uniaxial compressive strength of the grouting body required for a specific plastic radius \(\rho\); \( p_b \) is the pressure at \(r = b\) according to Eq. (6-22) and \( B_i \) the dimensionless parameters:

\[ B_i = B_2 + \frac{2 \sin \varphi_i}{1 - \sin \varphi_i} , \] (6-31)

\[ B_2 = \frac{2 \sin \varphi_i}{(1 - (\rho / b)^2 \sin \varphi_i)(\rho / a)^{2 \sin \varphi_i/(1 - \sin \varphi_i)} - 1 + \sin \varphi_i} , \] (6-32)

\[ B_I = \frac{2(1 - \nu_I) + B_2 \left( \ln(\rho / b) - 0.5 + 0.5(\rho / b)^2 \right)}{2(1 - \nu_I) \ln(b / a)} . \] (6-33)

The radial displacements at \(r = l\) is

\[ u_I = \frac{l}{E_I} \left\{ -A_1 \frac{f_{cl,r} - f_2}{m_I - 1} + \sigma'_I \left[ A_4 + A_3 \left( \frac{\rho}{l} \right)^{m_I + \kappa_I} \right] \right\} , \] (6-34)
where

\[
\tilde{\sigma}_i' = \tilde{\sigma}_i' - \frac{f_2}{m_j - 1} = \sigma_i' + \frac{c_i}{\tan \varphi_i} - \frac{f_2}{m_j - 1},
\]  

(6-35)

and \(f_2\) according to Eq. (6-21), the factors \(A_1, A_2, A_3\) according to Eq. (6-12)-(6-14); and \(A_6\) is

\[
A_6 = \frac{1}{\kappa_i + 1},
\]  

(6-36)

The radial displacements at \(r = b\) is

\[
u_b = C_1 (b/l) + C_2 (l/b) - \frac{f_2 b \ln(b/l) (1+\nu_l)(1-2\nu_l)}{2E_i} \left(1-\nu_l\right) - \frac{b(1+\nu_l)(1-2\nu_l)}{E_i} \sigma_0'\].

(6-37)

The inner system is solved with reference to a fully plastified, ideally drained grouting body (Fig. 6.5b), but with the size of the grouting body \(l\) instead of \(b\):

\[
\frac{\tilde{\sigma}_i}{\sigma_0} = \left(\frac{l}{a}\right)^{m_i-1},
\]  

(6-38)

\[
u_a = a \left(1+\nu_l\right) \left(\sigma'_i - \sigma_0'\right) + u_{\text{case I}} \mid_{\rho = l} + \left(u_i - u_{\text{case I}} \mid_{\rho = l}\right) \left(l/a\right)^{\nu_l},
\]  

(6-39)

with \(u_{\text{case I}} \mid_{\rho = l}\) and \(u_{\text{case I}} \mid_{\rho = l}\) denoting the displacements at the transition from Case I to Case II, i.e. immediately before all the inner system is fully plastified, and is calculated by inserting \(\rho = l\) in the r.h.s term of Eq. (6-24) and (6-25), respectively.

At the boundary between the inner and outer system \((r = l)\), the radial displacement \(u_l\) and the radial stress \(\sigma_l\) must be compatible, which leads to following solutions for the constants \(C_i\):

\[
C_1 = f(\rho, \sigma'_b) = A_1 \rho \frac{\tilde{\sigma}_b \left(l/a\right)^{m_i-1} - f_2/m_j - 1}{E_i \left(1 - (b/\rho)^2\right)} \left(\rho/l\right)^{m_i-1} + A_1 \rho \left(0.5 f_2 - \frac{f_{cl,e} - f_2}{m_j - 1}\right)
\]  

(6-40)

\[
- A_1 b^2 \frac{\sigma'_b}{E_i \left(1 - (b/\rho)^2\right)} - A_1 b^2 \frac{0.5 f_2}{E_i \left(1 - (b/\rho)^2\right)} 1-\nu_l + \frac{\ln(b/\rho)}{1-\nu_l}
\]

\[
C_2 = f(\rho, \sigma'_i) = \left(\frac{b}{\rho}\right)^2 C_1 - b^2 \frac{1+\nu_l}{E_i} \sigma'_b - \frac{0.5 f_2 b^2 (1+\nu_l)}{\rho E_i} \left(\frac{1 - \nu_l \ln(b/\rho)}{1-\nu_l}\right),
\]  

(6-41)

and to a formulation of the stress \(\sigma_a\) independently of \(\sigma_i\):

\[
\sigma_a = \frac{1}{B_1 \left(l/a\right)^{m_i-1}} \left(\sigma'_i B_2 + p_b B_3\right) - f_{cl,e} \left(\frac{1}{B_1 \left(l/a\right)^{m_i-1}} + \frac{(l/a)^{m_i-1} - 1}{(m_j - 1) \left(l/a\right)^{m_i-1}}\right).
\]  

(6-42)
6.3.2.3 **Outer plastification**

It was observed that for high hydraulic gradients and for large radii of drainage and plastification, the stresses at the outer boundary of the grouting body \((r = b)\) may violate the failure criterion before the injection body fully plastifies (i.e., \(p < b\); Fig. 6.6c). This indicates a second plastic zone emerging, which starts at the outer boundary of the grouting body and propagates inwards.

Therefore, *Case I* and *Case II* have to be extended for an outer plastic zone evolving depending on the stress state at the outer boundary of the grouting body at \(r = b\). If the stress state is elastic, the equations of the previous two sections apply. If the stress state violates the failure criterion, another plastic ring of extent \(\rho_{\text{out}}\) is introduced (Fig. 6.6c). The value of \(\rho_{\text{out}}\) is determined iteratively by checking the failure criterion (starting from \(r = b\) and propagating inwards, i.e. in \((-r)\) direction; ending when the stress state is elastic or when \(\rho_{\text{in}} = b_{\text{red}}\)) and simultaneously reducing the size of the inboard system to \(b_{\text{red}} = b - \rho_{\text{out}}\). The solution of the inboard system is given by the equations of *Case I* or *Case II* with \(b_{\text{red}}\) instead of \(b\) and taking into account the stress \(\sigma'_{b,\text{red}}\) as boundary condition at \(r = b_{\text{red}}\). The latter is determined when considering the outer plastic zone as a fully plastified grouting body subjected to seepage forces (Fig. 6.5a), which results in

\[
\sigma'_{b,\text{red}} = \frac{\overline{\sigma}'_{b}}{(b/b_{\text{red}})^{m_f-1}} \cdot \frac{f_{c_l} - f_{2}}{m_f-1} .
\]

Due to system complexity, Appendix C provides a matlab-code for calculating the characteristic line of grouting bodies for several degree of ideal drainage and for given input parameters comprising the problem layout (in-situ stress and pore water pressure, radius of excavation and grouting, permeability of the untreated ground) and material properties of the grouting body (effective cohesion, Young’s Modulus, angle of internal friction, Poisson’s ratio, and permeability). The matlab-code in Appendix D allows determining the required strength of grouting bodies as a function of the lining support pressure in order to limit the degree of plastification to a prescribed value.

6.3.2.4 **Water inflow**

A thick-walled cylinder of outer grouting radius \(b\) with initial water pressure \(p_0\) acting at \(r = R\) (Fig. 6.5c) is assumed to be ideally drained (\(p = 0\) for \(a \leq r \leq l\)). The water inflow per tunnel meter is

\[
Q = Q_0 \cdot \frac{\ln(R/a)}{\ln(R/b) + \frac{k}{k_f} \ln(b/l)} , \tag{6-44}
\]

where \(Q_0\) denotes the quantity of water inflow in the absence of any grouting measure:

\[
Q_0 = \frac{2\pi kp_0}{\rho_w g \ln(R/a)} , \tag{6-45}
\]

with \(\rho_w\) and \(g\) denoting the unit weight of water and the acceleration due to gravity, respectively.
6.4 Effect of ideal drainage of the grouting body

The effect of ideal drainage is discussed by means of the application example of Figure 6.7. Similar to the degree of plastification (Eq. (6-4)), a “degree of drainage” is introduced, which quantifies the share of the drained part of the grouting body:

\[ \eta = \frac{l - a}{b - a} . \]  

(6-46)

Figure 6.7. Problem layout of the tunnel example

6.4.1 Ideal drainage after grouting

6.4.1.1 Characteristic line

Figure 6.8 shows the characteristic line of our tunnel example considering several degrees of drainage \( \eta \). The case of grouting bodies without drainage represents the upper borderline case (red line for \( \eta = 0 \) in Fig. 6.8). The lining support pressure \( \sigma_a \) steeply decreases for small displacement of the excavation boundary \( u_a \), but then stays nearly constant. In other words, the grouting body is too weak and the excavation boundary tends to close the opening completely for a lining support pressure of less than \( \sigma_a = 1 \) MPa.

Increasing the degree of drainage favours both support and displacement of the excavation boundary (indicated by black lines for \( 0 < \eta < 1 \) in Fig. 6.8). For very low displacement, the characteristic lines coincide (which is due to the assumption of a very stiff grouting body with \( u_b = 0 \): “the total load acting on a stiff grouting body will be equivalent to \( \sigma_b \) irrespectively of any drainage”, Anagnostou and Kovári, 2003). But with increasing displacement, ideal drainage decreases the required support pressure. The case of an ideally drained grouting body represents the lower boundary (green line for \( \eta = 1 \) in Fig. 6.8).

To give an example, assume the profile in our tunnel example to allow for a displacement of \( u_a = 0.2 \) m. An ideal drainage degree of \( \eta = 0.2 \) decreases the lining support pressure by about 30% compared to the upper boundary of no drainage (compare points A to B in Fig. 6.8). Increasing the degree of drainage lowers the required support, but with decreasing effect (compare point A to B to C in Fig. 6.8). A drainage degree of \( \eta > 0.6 \) finally provides no additional benefit and the lines coincide with the characteristic line of and ideally drained grouting body (point D in Fig. 6.8).
Figure 6.8. Characteristic lines for several degrees $\eta$ of drainage after grouting (parameters see Fig. 6.7)

Figure 6.9. Support pressure $\sigma_a$ as a function of the degree $\eta$ of drainage after grouting and for several degrees of plastification $\lambda$ (parameters see Fig. 6.7)

6.4.1.2 Radius of drainage and plastification

The lining support pressure $\sigma_a$ as a function of the degree of drainage $\eta$ is shown in Figure 6.9 when considering several degrees of plastification $\lambda$ of the grouting body. For grouting bodies at purely elastic stress state ($\lambda = 0$ in Fig. 6.9), the support pressure increases with increasing drainage degree. On the other hand for a fully plastified grouting body ($\lambda = 1$ in Fig. 6.9), the support pressure decreases with increasing drainage degree, i.e. drainage is tentatively unfavourable for stability. For a partially plastified grouting body ($0 < \lambda < 1$ in Fig. 6.9), a combination thereof is observed: the required support pressure first decreases, then increases. The lowest support pressure is required at the point of discontinuity $\lambda = \eta$ (points A-D in Fig. 6.9) and thus represents the optimal drainage degree.
A higher drainage degree offers no benefit in terms of support, but on the contrary is tentatively unfavourable for the stability of the grouting body due to the risk of inner erosion caused by high gradients prevailing in the outer ring of grouting body.

### 6.4.1.3 Required strength of grouting body

In the course of pre-dimensioning, the engineer might have determined the admissible degree of plastification $\lambda = 0.6$ and now needs to quantify both strength and lining support pressure for grouting bodies fulfilling that requirement (Fig. 6.10). The necessary uniaxial compressive strength $f_{ci}$ depends linearly on the support pressure $\sigma_a$ (see also Eq. (6-30)). Drainage degrees smaller than the degree of plastification require a higher support or strength, respectively ($\eta < \lambda = 0.6$ in Fig. 6.10).

In case of choosing a drainage degree equal to the degree of plastification ($\eta = \lambda = 0.6$), the grouting body is stable for the combination of strength $f_{ci} = 1.65$ MPa and support pressure $\sigma_a = 0.5$ MPa (point A in Fig. 6.10). If only a support pressure of $\sigma_a = 0.25$ MPa is provided, the strength of the grouting body has to be increase to $f_{ci} = 2.24$ MPa (point B in Fig. 6.10).

![Figure 6.10](image.png)

**Figure 6.10.** Required uniaxial compressive strength $f_{ci}$ as a function of the support pressure $\sigma_a$ for several degrees $\eta$ of drainage after grouting (parameters see Fig. 6.7)

### 6.4.1.4 Parametric study

A parametric study was conducted concerning the lining support pressure $\sigma_a$ as a function of the degree of plastification $\lambda$ when varying the decisive design parameters of the grouting body: the geometry $(b/a)$, the required uniaxial compressive strength $(f_{ci})$ and the degree of drainage $\eta$ (Table 6.1). The results are shown in Figure 6.11. (Note that the marginal deviation in lining support pressure at the onset of plastification (i.e., at $\lambda = 0$) is due to the slow increase in support pressure with increasing degree of drainage $\eta$; cf. Fig. 6.9.)

Drainage of the inner part of the grouting body decreases the required lining support pressure only if a plastification of at least 10% of the grouting body is admissible. For lower degree of plastification, the lining support pressures coincide for all considered parameters (compare plots for $\lambda < 0.1$ in Fig. 6.11).
Figure 6.11. Required lining support pressure $\sigma_a$ as a function of the degree of plastification $\lambda$ for the parametric study of Table 6.1.
In case of aiming at a degree of plastification of $\lambda = 0.3$, the required lining support pressures coincide for all degree of drainage $\eta \geq 0.2$ independently of the size or the stiffness of the grouting body in our parametric study (compare Fig. 6.11 for $\lambda = 0.3$). For higher degree of plastification, the required support clearly depends on the degree of drainage. An increase of degree of drainage from $\eta = 0$ to 0.2 decreases the required support more than an increase from $\eta = 0.6$ to 0.8 (in evidence e.g. for $\lambda = 1$ in Fig. 6.11b). A drainage degree of $\lambda = 0.8$ necessitates a value of lining support pressure comparable to the one required in case of a completely drained grouting body ($\lambda = 1$), but with benefit of (at least some) sealing effect.

The bigger the grouting body, the larger are the differences in support pressure required when considering minimum or maximum drainage degree (compare e.g. $\Delta \sigma$ of Fig. 6.11a to 6.11b).

Grouting bodies of $b/a = 2$ are only expedient when having high compressive strength ($f_{cI} > 3$ MPa for our tunnel example, Fig. 6.11). Otherwise, they do not allow for a practically feasible support pressure of about $\sigma_a < 1$ MPa, even for full plastification (see e.g. Fig. 6.11c). Of course, the support pressure decreases with increasing strength and size of the grouting body (compare e.g. Fig. 6.11i to 6.11j).

### Table 6.1. Problem layout of the parametric study of Figure 6.11

<table>
<thead>
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<th>Problem layout</th>
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<tr>
<td>Total stresses, in-situ</td>
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<tr>
<td>Pore water pressure, in-situ</td>
<td>$p_0$</td>
<td>2.0 MPa</td>
</tr>
<tr>
<td>Radius of tunnel excavation</td>
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</tr>
<tr>
<td>Radius of influence for seepage flow</td>
<td>$R$</td>
<td>200 m</td>
</tr>
<tr>
<td>Ratio of permeability of grouting body to untreated ground</td>
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</tr>
<tr>
<td>Poisson’s ratio of grouting body</td>
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</tr>
<tr>
<td>Angle of internal friction equal to angle of dilatancy</td>
<td>$\phi_I = \psi_I$</td>
<td>25°</td>
</tr>
</tbody>
</table>

<table>
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</tr>
<tr>
<td>Degree of plastification</td>
<td>$\lambda$</td>
<td>0-1</td>
</tr>
<tr>
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</tr>
<tr>
<td>Uniaxial compressive strength</td>
<td>$f_{cI}$</td>
<td>0.5-5 MPa</td>
</tr>
</tbody>
</table>

### 6.4.1.5 Water inflow

One intention of injection measures may be reducing the water inflow into the tunnel. Figure 6.12 shows the normalized inflow (Eq. (6-44) normalized with the inflow in untreated ground according to Eq. (6-45)) as a function of the degree of drainage $\eta$. In case of equal permeability of grouting body and untreated ground ($k_I/k = 1$ in Fig. 6.12), the inflow linearly increases with increasing degree of drainage $\eta$ (i.e. increasing radius of drainage $l$). The lower the permeability ratio and the lower the degree of drainage, the higher is the sealing effect of the injection body.

Consider as an example ideal drainage of 80% of the grouting body ($\eta = 0.8$ in Fig. 6.12). In case grouting reduces the permeability to 1/100 of the initial value, inflow is reduced considerably to $Q/Q_0 = 0.23$ (point A in Fig. 6.12). But in case sealing grouting is less successful and decreases permeability only to 1/10, water inflow is only reduced to 90% of the inflow collected in untreated ground (point B in Fig. 6.12).
6.4.2 Ideal drainage in advance of grouting

6.4.2.1 Characteristic line

Figure 6.13 compares the characteristic line for drainage prior or after grouting. Drainage in advance of grouting is clearly favourable concerning stresses and displacement of the excavation boundary. Assume as an example a feasible tunnel support of $\sigma_a = 0.8$ MPa. The displacements in case of drainage in advance of grouting are 6 cm (point C in Fig. 6.13) and correspond to a degree of plastification of 32% (aiming at a lower degree of plastification would require an increase of the strength or the diameter of the grouting body – or to install a higher tunnel support). Drainage after grouting leads to 8.5 cm displacement (point D in Fig. 6.13), which is about 50% more than with drainage prior to grouting.

Figure 6.13. Comparison of the characteristic lines of drainage in advance of grouting to drainage after grouting (parameters see Fig. 6.7)
6.4.2.2 Dimensioning aids

Anagnostou and Kovári (2003) provide dimensioning aids allowing a simple and fast pre-dimensioning of grouting bodies in “dry” fault zones in the form of normalized nomograms for variable parameters to be chosen by the engineer (e.g. geometry, stiffness, lining support pressure etc.). These dimensioning aids apply for drainage in advance of grouting when using $\sigma_{bDR}$ according to Eq. (6-7) instead of $\sigma_{b}$ in Figures 5.3-1 to 5.3.4 of Anagnostou and Kovári (2003).

6.4.3 Assumption of stiff grouting body

In our analytical model, we assume the grouting body to be very stiff and thus experiencing no displacement at its outer boundary ($u_b = 0$ at $r = b$; cf. Anagnostou and Kovári, 2003). But displacements $u_b$ during plastification depend on the stresses acting at $r = b$ and the stiffness of the grouting body (i.e. size $b/a$, strength $f_{cl}$ and degree of plastification $\lambda$). Lines C and D in Figure 6.14 indicate a displacement of about $u_b = 8$ cm for our tunnel example in the worst-case of full plastification of the grouting body ($\lambda = 1$), virtually independent of drainage in advance or after grouting. This displacement is only 6 % of the radius of the grouting body. By neglecting it, the analytical model marginally overestimates the stresses acting on the grouting body, i.e. the solution is slightly safe-side concerning stresses and displacement of the excavation boundary.

![Figure 6.14](image)

**Figure 6.14.** Displacement of the inner ($u_a$) and outer ($u_b$) boundary of the grouting body as a function of the degree of plastification $\lambda$ for drainage in advance and after grouting (parameters see Fig. 6.7)
6.5 Modelling specific drainage borehole arrangements

6.5.1 Problem and approach

In the previous sections, complete pore pressure relief due to ideal drainage was assumed. We stay with the tunnel example of Figure 6.7, but now, the pore pressure relief resulting from specific borehole arrangements will be considered and the characteristic lines are determined by FE-modelling.

6.5.2 Arrangement of drainage boreholes

Pore pressure relief resulting from several different borehole arrangements are investigated. For drainage in advance of grouting, drainage boreholes coaxial to the tunnel axis are considered. For drainage subsequent to construction of the grouting body, radial drainage borehole arrangements are studied.

The borehole arrangements considered for drainage in advance of grouting can be grouped in three drainage layouts (Fig. 6.15): Layout A considers drainage boreholes of diameter 10 cm evenly spaced around the circumference of $r = l$ ($l \geq b$; Fig. 6.15a). Layout B comprises lateral drainage boreholes of 10 cm diameter each, arranged in horizontal centre distance $l$ to the tunnel axis and of vertical spacing $y_{DR}$ (Fig. 6.15b). For the purpose of comparison, ideal drainage is modelled assuming pore pressure relief within the radius $r = l$ ($l \geq b$; Fig. 6.15c). The geometric values are given in the table within Figure 6.15.

![Drainage layouts](image)

**Figure 6.15.** Drainage layouts considered for drainage in advance of grouting: a) coaxial drainage boreholes evenly spaced along the circle line $r = l$ (“layout A”), (b) lateral arrangement of coaxial drainage boreholes (“layout B”) and (c) ideal drainage of the area $l \geq b$
Figure 6.16 shows the group of borehole arrangements considered for drainage measures after grouting. Ideal drainage (Fig. 6.16a) assumes pore pressure relief within the radius \( r = l \) \((a \leq l \leq b)\) and serves for the purpose of comparison to the analytical solution. The two-dimensional layout simulates drainage slits of 10 cm diameter, which are evenly spaced at sector angle \( \alpha \) around the circumference (Fig. 6.16b; plane strain conditions). Both length \( l \) \((a \leq l \leq b)\) and number \( n \) of boreholes per cross-section are investigated \((n = 1\text{-}60)\). The three-dimensional drainage layout (Fig. 6.16c) simulates 12 drainage boreholes \((a = 30^\circ)\) of 10 cm diameter each. The study considers several length of boreholes \( l \) \((a \leq l \leq b)\) and varies the spacing \( e \) of the boreholes coaxial to the tunnel axis from 1.25 to 10 m.

![Figure 6.16. Drainage layouts considered for drainage after grouting: (a) ideal drainage of area \( l \leq b \), (b) radial drainage slits considered in the two-dimensional model (plane strain) and (c) radial drainage boreholes considered in the three-dimensional model](image)

### 6.5.3 Computational model

Figure 6.17 shows the problem layout for a representative drainage arrangement. As in the previous sections, it considers a deep, circular tunnel of radius \( a \) and the grouting body in form of a thick-walled cylinder of outer radius \( b \) in plane strain condition. The ground (both untreated and injected) is considered as an elasto-plastic porous medium obeying the principle of effective stresses, Coulomb's failure criterion and Darcy's law. Associated flow rule is assumed. The initial stress state of untreated ground is assumed to be homogeneous and isotropic; the magnitude of the effective initial stress \( \sigma_0' \) equals to the effective overburden pressure. The initial water pressure \( p_0 \) acts at a radial distance \( R \) from the tunnel, with \( R \) being equal to the height of the undisturbed water table above the tunnel. The lining support pressure \( \sigma_l \) is assumed uniform along the excavation boundary \( r = a \). The boundaries of excavation and drainage boreholes represent seepage faces of atmospheric pressure \((p = 0)\). At the axis of symmetry, a no-flow boundary-condition is assumed and no normal displacement is allowed.
Figure 6.17. Computational model for the numerical analyses

The FE-model considers the sequential coupling of hydraulic analysis influencing the mechanic analysis by augmenting the effective stresses in the equations of the mechanical equilibrium by the spatial gradients of fluid pressure $\nabla p$ (e.g. Potts, 1999). The used FE-code (Comsol) enables manual coupling of hydraulic and mechanical analysis. The implementation was verified by comparison to results of a built-in routine in the FE-code of Abaqus (steady-state consolidation) and Hydmec (Anagnostou, 1991). The computational steps are described for drainage in advance and after grouting separately (see below).

6.5.3.1 Computational steps: drainage in advance of grouting

Figure 6.18 shows the sequence of computational steps simulating drainage in advance of constructing the grouting body and subsequent excavation of the tunnel for representative drainage layout A. The hydraulic step H1 calculates the hydraulic head field due to the drainage measure. The mechanical step M1 serves for initialisation of the stresses in the untreated ground. Step M2 incorporates the spatial derivative of the pore pressure field resulting from step H1, considers the initial stresses of step M1 and calculates the increased stresses due to consolidation. In step H2, the pore pressure field when considering the changed permeability due to the grouting body is computed. Step M3 considers the pore pressure field resulting from step H2 and the initial stresses of step M2, and computes the stress-field after injection (no displacements allowed at the future excavation boundary). The last hydraulic step H3 calculates the pore pressure field due to excavation. The final mechanical step M4 (considering the initial stresses of step M3 and pore pressure field of step H3) simulates the excavation of the tunnel by step-by-step reduction of $\sigma_a$. For evaluation of the displacements $u_a$ at the excavation boundary, step M4 is considered only.

6.5.3.2 Computational steps: drainage after grouting

Figure 6.19 depicts the sequence of computational steps simulating grouting bodies with subsequent drainage and excavation of the tunnel for a representative drainage layout. The only hydraulic step H calculates the hydraulic head field due to excavation and drainage measure, which are assumed to take place simultaneously. Mechanical step M1 is used for initialisation of the stresses acting in the untreated ground and the grouting body. Step M2 incorporates the spatial derivative of the pore pressure field resulting from step H, considers the initial stresses of step M1 and calculates the increased stresses due to consolidation. At the future excavation boundary $a$ no displacements are allowed. Still incorporating the spatial derivative of the pore pressure field resulting from step H, but
Figure 6.18. Computational steps of the numerical model simulating drainage in advance of grouting and subsequent excavation of the tunnel.
Figure 6.19. Computational steps of the numerical model simulating drainage after grouting and subsequent excavation of the tunnel

considering the initial stresses of step M2, the final step M3 calculates the excavation of the tunnel by stepwise reduction of $\sigma_a$. For evaluation of displacements $u_a$ at the excavation boundary, step M3 is considered only.

6.5.4 Validation

Comparison of the characteristic lines by means of the analytical and the numerical model may serve as validation of the latter (problem layout see Fig. 6.7). The results of the analytical (solid line) and the numerical (markers) model agree very well for both curves of support-displacement (Fig. 6.20a and c) and support-plastification (Fig. 6.20b and d). More specifically, the characteristic lines are congruent
for a wide range of support pressures, but differ at very low lining support pressure (Fig. 6.20a and c). Reason is the assumption of the stiff grouting body in the analytical solution ($u_b = 0$), which is not fulfilled in the numerical model and due to which the stresses acting on the grouting body are slightly higher in the analytical solution (see e.g. Fig. 6.20b at $\lambda \approx 1$).

The development of an outer plastic zone for ideal drainage after grouting at low values of support pressure was confirmed in the numerical model (Fig. 6.21; extent of plastic zones by means of the analytical model indicated with green arrows). For $\sigma_a = 1.2$ MPa, only an inner plastic zone appears (Fig. 6.21b). At a lining support pressure of $\sigma_a = 0.45$ MPa, plastification starts also from the outer boundary $b$ of the grouting body (Fig. 6.21c), where high hydraulic gradients act (Fig. 6.21a). With decreasing lining support pressure ($\sigma_a = 0.44$ MPa in Fig. 6.21c), the outer plastic zone grows towards the excavation boundary. After merging of these two plastic zones, plastification (in the FE-model) may propagate outwards into the untreated ground.
Figure 6.21. (a) Hydraulic head fields and development of plastic zones for decreasing support pressure for ideal drainage after grouting: (b) single plastic zone for $\sigma_a = 1.2$ MPa, (c) second plastic zone for $\sigma_a = 0.45$ MPa and (d) $\sigma_a = 0.44$ MPa (extent of plastic zones according to the analytical model indicated with green arrows; parameters see Fig. 6.7)

Figure 6.22. Hydraulic head fields for several number $n$ of drainage slits installed after grouting ($\eta = 0.8$; parameters see Fig. 6.7)
6.6 Effect of drainage borehole arrangements drilled after grouting

6.6.1 Drainage slits (2D-model)

6.6.1.1 Number of drainage slits

Figure 6.22 shows the hydraulic head field of the two-dimensional drainage layouts (Fig. 6.16b) for variable number \( n \) of drainage slits of length \( l = 11 \) m, as well as for the belonging boundary cases without and with ideal drainage. In grouting bodies without drainage, pore water pressure dissipates mainly within the grouting body and nearly 2 MPa of water pressure acts on the outer boundary of the grouting body (Fig. 6.22a). In case of 4 drainage slits, the pressure is slightly reduced to \( p_b = 1.9 \) MPa, but the number of drainage slits is too small to relief pressure overall within the grouting body (Fig. 6.22b). Increasing the drainage number to \( n = 12 \) (Fig. 6.22d; \( p_b = 1.75 \) MPa) provides a good approximation to the ideally drained case (Fig. 6.22f; \( p_b = 1.6 \) MPa), where very high hydraulic gradients act within the outer ring of the grouting body.

The pore pressure relief within the grouting body is plotted in Figure 6.23 as a function of the number \( n \) (or the sector angle \( \alpha \), respectively) of drainage slits. The pressure decreases steeply for low number of drainage slits and then levels out for larger amounts. Increasing the number of drainage slits above \( n = 20 \) (\( \alpha < 18^\circ \)) offers marginal utility concerning pore pressure relief, but would require an extremely high drilling effort in tunnelling practice. However, even very dense drainage slits (\( n = 60 \) or \( \alpha = 6^\circ \)) do not relief pressure as much as ideal drainage (point A in Fig. 6.23).

![Figure 6.23. Average pore pressure \( p \) in the grouting body normalized with the initial pressure \( p_0 \) as a function of the number \( n \) (or the sector angle \( \alpha \)) of drainage slits installed after grouting (\( \eta = 0.8 \); parameters see Fig. 6.7)](image)

6.6.1.2 Plastic zones

During the stepwise reduction of the lining support pressure \( \sigma_a \), a plastic zone develops starting from the inner boundary of the grouting body (orange area in Fig. 6.24b for \( \sigma_a = 0.8 \) MPa). The very high gradients between the outer boundary of the grouting body and the end of the drainage borehole (Fig. 6.24a) might lead to local overstressing of the grouting body. A second plastic zone develops from the outer boundary of the grouting body for lower support pressures (\( \sigma_a = 0.58 \) MPa in Fig. 6.24c), but in contrast to ideal drainage starting from local spots instead of the full ring (compare
Fig. 6.21 to Fig. 6.24). A further decrease of lining support leads to plastic “bridges” between the inner and the outer plastic zone (Fig. 6.24d; $\sigma_a = 0.57 \text{ MPa}$).

For our study, full plastification (i.e., a degree of plastification $\lambda = 1$) is defined as the state where the first bridge between outer and inner plastic zone develops.

![Figure 6.24](image)

**Figure 6.24.** (a) Hydraulic head fields and development of plastic zones for decreasing support pressure when considering drainage slits after grouting: (b) inner plastic zone for $\sigma_a = 0.8 \text{ MPa}$, (c) additional plastic spots for $\sigma_a = 0.58 \text{ MPa}$ and (d) plastic “bridges” for $\sigma_a = 0.57 \text{ MPa}$ ($\eta = 0.8$; parameters see Fig. 6.7)

### 6.6.1.3 Required lining support pressure

The results of a parametric study into the effects of the numbers of drainage slits ($n = 4, 8, 12, 16$), the degrees of drainage ($\eta = 0.4, 0.8, 1$) and the size of grouting body ($b/a = 2$ and $2.5$) is shown in Figure 6.25. The lower borderline case of the ideally drained grouting body is plotted in red; the upper borderline case of no drainage slits is shown in green.

As an example, we assume a maximum admissible degree of plastification of $\lambda = 0.5$ for a mid-size grouting body of $b/a = 2.5$, which is drained to a degree of $\eta = 0.8$ (Fig. 6.25b). In the best-case of ideal drainage, a lining support pressure of $\sigma_a = 0.7 \text{ MPa}$ would be required (A in Fig. 6.25b). The effect of 12-16 drainage slits nearly coincides with ideal advance drainage ($\sigma_a = 0.7-0.73 \text{ MPa}$). In case of 8 drainage slits, the required support increases by 17% (to $\sigma_a = 0.81 \text{ MPa}$; B in Fig. 6.25b). The case of only four drainage slits requires 50% more lining support pressure than the ideal case (compare C to A in Fig. 6.25b).

The same trends apply overall in Figure 6.25. The higher the number of drainage slits, the better is of course the approximation to the ideal solution. However, 12 or more drainage slits provide a good approximation to the ideal solution (about 10-20% accuracy). For low degree of plastification up to $\lambda = 0.4$ (which is about the admissible degree of plastification in tunnelling practice for reasons of safety and usability), the required support pressures for both drainage lengths virtually coincide (less than 5% deviation for both $\eta = 0.4$ and 0.8).
The smaller the grouting body, the higher has to be the degree of drainage to allow for pressure relief at small drainage slit numbers. Otherwise (and as shown in Figures 6.25d and 6.25e for four drainage slits), the drainages have no effect on the required support. On the other hand and assuming slits piercing the entire grouting body ($\eta = 1$), already 4 drainage slits allow a considerable reduction of pore pressures and the lines for higher drainage numbers nearly coincide, independently of the size of the grouting body (Fig. 6.25c and 6.25f).

Figure 6.25. Required lining support pressure $\sigma_a$ as a function of the degree of plastification $\lambda$ for drainage after grouting considering several numbers $n$ of drainage slits, three degrees of drainage $\eta$ and two sizes of grouting body $b/a$ ($b = 12.5$ and $10$ m; drainage layout see Fig. 6.16b; other parameters see Fig. 6.7)
6.6.1.4 Inflow

Figure 6.26 shows the normalized inflow as a function of the degree of drainage $\eta$. Without any drainage measure ($\eta = 0$ in Fig. 6.26), the grouting body considerably reduces the inflow to about 4% of the inflow in untreated ground. The inflow increases with growing seepage area, i.e. with increasing number $n$ of drainage slits and/or increasing degree of drainage $\eta$. The inflow considering drainage slits is lower than in the borderline case of ideal drainage (compare ‘ideal’ to other lines in Fig. 6.26). Reason is that although the seepage area increases when considering drainage slits compared to ideal drainage, the thereon acting hydraulic gradients are considerably smaller (see also Fig. 6.22).

![Figure 6.26](image)

**Figure 6.26.** Inflow $Q$ collected from all boundaries of excavation and drainage (normalized with the inflow when considering untreated ground $Q_0$) as a function of the degree $\eta$ of drainage after grouting (drainage layout see Fig. 6.16b; parameters see Fig. 6.7)

6.6.2 Drainage boreholes (3D-model)

According to the results of the two-dimensional model, twelve drainage slits approximate the solution of ideal drainage reasonably well (Section 6.6.1.1). Thus we limit our considerations to 12 drainage boreholes in cross-section of the three-dimensional model and focus on the effect of axial borehole distance $e$ and borehole length $l$ (see Fig. 6.16c).

6.6.2.1 Axial borehole distance

Figure 6.27 shows the hydraulic head fields in the cross section (r.h.s.) and in the longitudinal section to the tunnel axis (l.h.s.) for the example of 11 m long boreholes. It is self-evident that the hydraulic head field of the drainage slits in the two-dimensional model (see Fig. 6.22d) is approximated best by a very small axial borehole distance $e$ (e.g., $e = 2.5$ m in Fig. 6.27a). But this is not expedient keeping in mind the required drilling effort for tunnelling practice. An almost homogeneous hydraulic head field in both radial and axial direction develops when keeping the axial distance $e$ about roughly equal to the middle centre-distance of the boreholes $m$ (Fig. 6.27b with $e \approx m = 0.5(l+a) \tan \alpha = 4.6$ m). In case of longer axial borehole distance, the benefit of the number of drainage boreholes will be lost and the hydraulic head distribution in the axial direction is less favourable than in the radial (see $\Delta r$ for $e = 10$ m in Fig. 6.27d).
6.6.2.2 Required lining support pressure

A parametric study into the effects of the axial borehole spacing \( e = 1.25-10 \text{ m} \); Fig. 6.16c) and borehole length \( l = 11 \text{ m} \) and 12.5 m or \( \eta = 0.8 \) and 1, respectively) was conducted. The required lining support pressure \( \sigma_a \) as a function of the degree of plastification \( \lambda \) is shown in Figure 6.28. Again, the lower borderline case of the ideally drained grouting body is plotted as green line; the upper borderline case of no drainage boreholes is shown as red line.

An axial borehole distance of \( e = 1.25 \text{ m} \) shows nearly the same support-plastification line as the two-dimensional case (compare Fig. 6.28a to Fig. 6.25b). In case of full degree of drainage \( \eta = 1 \), this dense drainage layout nearly coincides with the ideal line (compare dotted to green line in Fig. 6.28b).

For larger axial borehole distance, the deviation to the case of ideal drainage increases fast. Assuming a maximum admissible degree of plastification of \( \lambda = 0.5 \), a lining support pressure \( \sigma_a = 0.92 \text{ MPa} \) is required (point A in Fig. 6.28a for \( \eta = 0.8, e = 5 \text{ m} \)), which is 30 \% more than when considering ideal drainage \( \sigma_a = 0.7 \text{ MPa}, \text{ point B in Fig. 6.28a} \). However for the practical relevant ranges of plastification, the deviation to the ideal solution diminishes to only 3-12\% (for \( \lambda = 0.2 \) and 0.3, respectively).

6.6.2.3 Comparison of characteristic lines

Assume a maximum feasible lining support pressure of \( \sigma_a = 0.75 \text{ MPa} \) for the tunnel example of Figure 6.7. The engineer has to choose from the drainage arrangements sketched as insets A to D in Figure 6.29, all of them considering 12 boreholes, and wants to know the belonging displacements of the excavation boundary \( u_a \) including the degree of plastification \( \lambda \).

The lower borderline case of ideal drainage leads to 10 cm displacement of the excavation boundary (point A in Fig. 6.29) and the degree of plastification is \( \lambda = 0.46 \) (see e.g. Fig. 6.20). Nearly the same
values of deformation and plastification can be obtained with borehole arrangement of inset D, where radial boreholes are drilled every second tunnel meter (point D in Fig. 6.29; $u_a = 11 \text{ cm}, \lambda = 0.48$). Increasing the axial borehole distance to 4.6 m (leading to an almost homogeneous head field as discussed in Section 6.6.2.1), a slightly higher displacement results (point B in Fig. 6.29; $u_a = 12 \text{ cm}$). The degree of plastification is $\lambda = 0.53$ (see Fig. 6.28a). Shorter drainage boreholes would increase deformation considerably (inset and point C in Fig. 6.29; $u_a = 18 \text{ cm}, \lambda = 0.79$).
In case the 12 drainage boreholes were arranged coaxial to the tunnel axis (see inset F in Fig. 6.29), 25% more displacement and plastification than in the radial arrangement result (compare point B to point F in Fig. 6.29 with $u_a = 15$ cm, $\lambda = 0.65$).

Although displacements of all considered drainage layouts are relatively small, the engineer might aim for a lower degree of plastification for safety reasons. Note that this would only be possible (for all other parameters remaining unchanged) by an increase in uniaxial compressive strength of the grouting body (inset and point E in Fig. 6.29 for twice the uniaxial compressive strength of Fig. 6.7; $u_a = 4$ cm, $\lambda = 0.15$).

6.6.2.4 Inflow

Figure 6.30 shows the normalized inflow as a function of the degree of drainage $\eta$ for several axial borehole distances ($e = 1.25-10$ m; borehole layout of Fig. 6.16c). The inflow increases with increasing seepage area; i.e. smaller axial borehole distance $e$ and increasing degree of drainage $\eta$. The inflow is reduced to less than 20% of the inflow in untreated ground for all considered axial distances and – due to reduced seepage area of the drainage boreholes – reduced remarkably compared to the ideally drained case (compare line “ideal” to the other lines in Fig. 6.30).

**Figure 6.30.** Inflow $Q$ collected from all boundaries of excavation and drainage (normalized with the inflow when considering untreated ground $Q_0$) as a function of the degree $\eta$ of drainage after grouting for drainage layout of Figure 6.16c with variable axial spacing $e$ (parameters see Fig. 6.7)
6.7 Effect of drainage borehole arrangements drilled in advance of grouting

6.7.1 Layout A: circular borehole arrangements

6.7.1.1 Number of drainage boreholes

The effect of the number of advance drainage boreholes is discussed by means of the borehole layout A with circle line at $l=15$ m (see Fig. 6.15a).

Figure 6.31 shows the hydraulic head field (l.h.s.) and the plastic zones when assuming a lining support pressure $\sigma_a = 0.4$ MPa (r.h.s.) for several borehole numbers $n$. The borderline cases of no and ideal advance drainage are added for comparison. The pore pressure relief is evident when comparing the hydraulic head field considering no boreholes to the case of only two advance drainage boreholes (compare l.h.s. of Fig. 6.31a to b). The plastic zone is considerably smaller, too and shows a slightly oval shape due to the lateral location of drainage (compare orange area indicating the plastic zone in r.h.s. of Fig. 6.31b to a). Increasing the borehole number reduces the plastic radius and leads to an even shape of the plastic zone for $n \geq 8$ (r.h.s. of Fig. 6.31).

Figure 6.31. Hydraulic head fields (l.h.s.) and plastic zones for a support pressure of $\sigma_a = 0.4$ MPa (r.h.s.) when considering several numbers $n$ of drainage boreholes drilled in advance of grouting (extent of plastic zones according to the analytical model indicated with green arrows; drainage layout A of Fig. 6.15a; parameters see Fig. 6.7)
Figure 6.32. (a) Water pressure acting on the grouting body $p$ (normalized with the initial pressure $p_0$) and (b) inflow $Q$ collected from all boundaries of excavation and drainage (normalized with the inflow $Q_{id}$ which considers ideal drainage up to $r = l$) as a function of the number $n$ (or the sector angle $\alpha$) of drainage boreholes drilled in advance of grouting (drainage layout A; parameters see Fig. 6.7)

The average normalized pore pressure acting on the grouting body is plotted as a function of the number $n$ (or the sector angle $\alpha$, respectively; Fig. 6.32a). Already a small number of drainage boreholes decreases the pore pressure clearly, while increasing the borehole number $n > 8$ provides marginal additional benefit. A large borehole number ($n = 20$) nearly relieves the pressure as much as ideal drainage in advance of grouting (point A in Fig. 6.32a).

The normalized inflow increases with increasing seepage area (i.e. increasing borehole number), but does not reach the values as when assuming ideal drainage (point A in Fig. 6.32b). In the case of 8 advance drainage boreholes, the inflow is by 25% lower than when assuming ideal advance drainage in advance of grouting (compare point B to A in Fig. 6.32b).

The characteristic lines for $n = 2$-20 drainage boreholes drilled in advance of grouting are shown in Figure 6.33. The lower borderline case of the ideally advanced drained grouting body is plotted in green; the upper borderline case of no drainage is shown as red line. For borehole number $n \geq 16$, the characteristic lines virtually coincide with the one of ideal advance drainage (compare black to green...
lines in Fig. 6.33). Assuming a feasible lining support pressure of $\sigma_a = 0.4$ MPa, the displacement of the excavation boundary considering ideal advance drainage is $u_a = 14$ m (point A in Fig. 6.33). In case of 8 advance drainage boreholes, a 20% higher displacement results (point B in Fig. 6.33; $u_a = 17$ cm). In case of only 4 drainage boreholes, the deformation increases (point C in Fig. 6.33; $u_a = 25$ cm) and for two drainage boreholes, the system is close to failure (point D in Fig. 6.33; $u_a = 85$ cm).

### 6.7.1.2 Circle line of drainage boreholes

Figure 6.34 shows the characteristic lines when considering 8 drainage boreholes at several distances between grouting body and drainage boreholes, here referred to as “circle line $l$”. At too large circle line, the boreholes are not able to relief the pore pressure close to the grouting body, leading to unfavourable characteristic lines (e.g. line $l = 30$ m in Fig. 6.34a). On the other hand at very small circle line, the drainage boreholes cannot be fully effective due to the close permeability interface to the grouting body and thus do not offer additional benefit in terms of stresses and displacements (e.g. line $l = 12.5$ m in Fig. 6.34a).

A parametric study was conducted to determine the circle line for common sizes of grouting bodies ($b/a = 2-3$) and $n = 4$, 8 and 16 boreholes of borehole layout A (Fig. 6.15a and Table 6.2). It showed low sensitivity to the value of circle line, as long as the boreholes are arranged roughly 1-2 m outside of the grouting body (note that the ratio of permeability of grouting body to untreated ground $k_I/k \leq 0.1$ proved therefore to be negligible). A recommendation of circle line $l$ normalized with the tunnel radius $a$ is given in Figure 6.35.

<table>
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<tr>
<th>Problem layout</th>
<th>Total stresses, in-situ $\sigma_0$</th>
<th>4.4 MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pore water pressure, in-situ $p_0$</td>
<td>2.0 MPa</td>
<td></td>
</tr>
<tr>
<td>Radius of tunnel excavation $a$</td>
<td>5 m</td>
<td></td>
</tr>
<tr>
<td>Radius of influence for seepage flow $R$</td>
<td>200 m</td>
<td></td>
</tr>
</tbody>
</table>

### Ground and grouting body

| Effective cohesion $c$ | 0.05 MPa |
| Angle of eff. internal friction $\phi$ | 25° |
| Angle of dilatancy $\psi$ | 25° |
| Poisson’s ratio $\nu$ | 0.33 |
| Young’s modulus $E$ | 100 MPa |
| Uniaxial compressive strength $f_{c_1}$ | 1.5 MPa |
| Poisson’s ratio of grouting body $\nu_I$ | 0.33 |
| Angle of internal friction equal to angle of dilatancy $\phi_I = \psi_I$ | 25° |

### Variable design parameters

| Number of drainage boreholes $n$ | 4, 8, 12 |
| Circle line $l$ | 10-50 m |
| Ratio of radii of grouting body to excavation $b/a$ | 2, 2.5, 3 |
| Ratio of permeability of grouting body to untreated ground $k_I/k$ | 0.1-0.001 |
Figure 6.34. (a) Characteristic lines for several circle lines \( l \) when considering drainage in advance of grouting (layout A of Fig. 6.15a with \( n = 8 \); other parameters see Fig. 6.7)

Figure 6.35. Recommended circle line \( l \) as a function of the size of the grouting body \( b \) (normalized with tunnel radius \( a \)) for drainage via borehole number \( n \) in advance of grouting (drainage layout A)

6.7.1.3 Required lining support pressure

The results of a parametric study into the effects of the number of drainage boreholes \((n = 4, 8, 16)\) and the size of grouting body \((b = 10, 12.5\) and \(15\)m, \(i.e.\ b/a = 2, 2.5\) and \(3\)) is evaluated in Figure 6.36 showing the required lining support pressure of \(\sigma_a\) as a function of the degree of plastification \(\lambda\). The lower borderline case of the ideal advance drainage of the grouting body is shown as green line; the upper borderline case of no drainage measure is shown as red line.
The effect of every advance drainage borehole arrangement is remarkable with respect to lining support or degree of plastification and similar for all sizes of grouting bodies (compare Fig. 6.36a to b to c). Consider a maximum admissible degree of plastification $\lambda = 0.5$ for a mid-size grouting body ($b/a = 2.5$ in Fig. 6.36b). In the case of ideal advance drainage, a lining support pressure of $\sigma_a = 0.5$ MPa would be required (point A in Fig. 6.36b; note that this value is 30% lower than when considering ideal drainage after grouting; point A in Fig. 6.25b). The model of ideal drainage is adequate with 10% accuracy in the case of 16 boreholes, and with still 20% accuracy for 8 boreholes ($\sigma_a = 0.55$ and 0.6 MPa for $n = 16$ and 8, respectively; Fig. 6.36b). In case of 4 boreholes, accuracy decrease to 32% ($\sigma_a = 0.66$ MPa at point B in Fig. 6.36b; see also later in Section 6.7.3 for an application example).

**Figure 6.36.** Required lining support pressure $\sigma_a$ as a function of the degree of plastification $\lambda$ for drainage in advance of grouting when considering three sizes of grouting bodies $b/a$ ($b = 10$, 12.5 and 15 m; drainage layout A of Fig. 6.15a; other parameters see Fig. 6.7)
6.7.2 Layout B: lateral borehole arrangements

6.7.2.1 Effect of Layout B compared to Layout A

In tunnelling practice, a circular arrangement of boreholes might not be possible due to the limited accessibility for drilling. An option might be a lateral arrangement of drainage boreholes (layout B of Fig. 6.15). Figure 6.37 compares the hydraulic head field (l.h.s.) and the plastic zone when assuming a lining support pressure $\sigma_a = 0.4$ MPa (r.h.s.) of layout A to layout B for the example of $n = 8$ advance drainage boreholes. Layout B lowers the hydraulic head lateral of the grouting body similar to layout A, but is less effective above the tunnel roof (l.h.s. of Fig. 6.37). The plastic zone is therefore marginally more extended in vertical direction and the displacements are distinguished into “roof” and “lateral” (r.h.s. of Fig. 6.37b). The average degree of plastification is $\lambda = 0.8$, which is 20% higher than for layout A ($\lambda = 0.67$; compare r.h.s. of Fig. 6.37).

The characteristic line confirms the advantage of layout A compared to layout B (Fig. 6.38). When assuming a maximum admissible support pressure of $\sigma_a = 0.4$ MPa, the lower borderline case of ideal advance drainage has a displacement of only $u_a = 14$ cm (point C in Fig. 6.38). The displacements increase for layout A by 20% and by additional 50% for layout B ($u_a = 17$ and 24 cm for point A and B in Fig. 6.38). The differences in characteristic lines due to the egg-shaped plastic zone (denoted as “roof” and “lateral” according to Fig. 6.37b) appear only for very low support pressure close to failure of the grouting body, while for lower displacement, both lines coincide.

Hereinafter, the study considers the worst-case of displacements at the tunnel roof.

Figure 6.37. Hydraulic head field (l.h.s.) and plastic zone for a support pressure of $\sigma_a = 0.4$ MPa (r.h.s.) when (a) considering drainage in advance of grouting by means of borehole layout A and (b) layout B ($n = 8$; other parameters see Fig. 6.7)
6.7.2.2 Required lining support pressure

As in Section 6.7.1.3, the required lining support pressure $\sigma_a$ as a function of the degree of plastification $\lambda$ is evaluated in Figure 6.39 for variable borehole number $n$ (drainage layout B of Fig. 6.15b) and for three sizes of the grouting body ($b = 10, 12.5$ and $15$ m, i.e. $b/a = 2, 2.5$ and $3$). The lower borderline case of ideal advance drainage is indicated for comparison as green line; the upper borderline case of no advance drainage measure is shown as red line.

Comparing the results of layout A (Fig. 6.36) to layout B (Fig. 6.39) proves the trend discussed in the previous section: layout B might allow easier accessibility than layout A. But layout A provides benefit in terms of stresses and displacements at the excavation boundary. Section 6.7.3 examines these effects by means of an application example.

6.7.3 Application example

Consider planning the grouting body at a depth and in a ground shown in the tunnel example of Figure 6.7. The uniaxial compressive strength of the grouting body is limited to $1.5$ MPa; its size to maximum $b/a = 2.5$. The lining will provide a support pressure of $\sigma_a = 0.6$ MPa. For safety reason, maximum half of the grouting body is allowed to plastify (degree of plastification $\lambda = 0.5$). Without drainage measures, the grouting body will fail (see e.g. red lines in Fig. 6.36a or b). Which drainage arrangements allow a stable grouting body within these parameters – and what are the displacements to be expected at the excavation boundary?

First, the lateral drainage borehole arrangements are studied (layout B). Figure 6.39 shows that a small grouting body requires higher lining support pressure than $\sigma_a = 0.6$ MPa, but the grouting body of size $b/a = 2.5$ might be an option. The limits of admissible plastification $\lambda = 0.5$ is only fulfilled for $n = 16$ advance drainage boreholes (point B in Fig. 6.39b). The displacement of the excavation boundary is $u_a = 12$ cm (point B in Fig. 6.40e).
Advance drainage considering borehole layout A is supposed to be more favourable. But still, only the grouting body of size $b/a = 2.5$ enables the lining support pressure of $\sigma_a = 0.6$ MPa (see Fig. 6.36). The degree of plastification is $\lambda = 0.46$ and 0.5 for $n = 16$ and 8 drainage boreholes, respectively (about point A in Fig. 6.36b). The displacement of the excavation boundary is $u_a = 9$ and 10 cm (point A in Fig. 6.40e), i.e. 20% less than for drainage layout B.

Summarizing, both drainage layouts A and B allow for construction of a stable grouting body. The option of choice is presumably drainage layout A with 8-16 drainage boreholes. The circle line for the advance drainage boreholes is $l = 13.6 - 14.2$ m (Fig. 6.35), the size of the grouting body $b = 12.5$ m ($b/a = 2.5$). If preferring drainage layout B, 20% larger displacements had to be considered, while the degree of drainage was about the same.

(Please note that drainage boreholes drilled after grouting is no option within the demanded parameters of the application example. According to Figure 6.28, only the theoretical, ideal drainage measure allows for a support of $\sigma_a = 0.6$ MPa – and still for a too high degree of plastification of $\lambda > 0.6$.)
Figure 6.40. Characteristic lines for drainage in advance of grouting considering three sizes of grouting bodies $b/a$ ($b = 10, 12.5$ and $15$ m; drainage layouts of Fig. 6.15; other parameters see Fig. 6.7)
6.7.4 Effect of fault zone of limited extent

Anagnostou and Kovári (2003) described that in fault zone of limited extent, a “stabilizing wall-effect” might be observed. Triggered by the deformation occurring within the fault zone, shear stresses develop at the interface to the stiffer host rock. Other than in the previous sections considering a fault of unlimited extent, where the effective stresses increased by about the amount of pressure relief induced by drainage, the increase in effective stresses is therefore diminished by the wall-effect. The stresses acting on the grouting body are lower, which is favourable in terms of stability.

For detailed explanation including parametric studies, reference is made to Anagnostou and Kovári (2003). Here, we limit ourselves to one single example of fault zone without intending an in-depth study, but still extend the previous investigations in three aspects: (i) by considering a fault zone running axis-parallel to the tunnel; (ii) by taking account of the possibly different permeability of fault zone and host rock; and (iii) by considering specific borehole arrangements drilled in advance of grouting.

The exemplary fault zone of 60 m width runs axis-parallel to the tunnel and has a vertical interface to the solid host rock (Fig. 6.41). The width of the fault zone is chosen such as it is supposed to trigger a visible wall effect (in fault zones normal to the tunnel axis, this is the case for zones up to an extent of about 8 times the radius of the grouting body, cf. Anagnostou and Kovári, 2003). The problem layout, the approach and the properties of both fault zone and grouting body are the ones of our previous tunnel example; the only difference being that the ground model additionally considers the properties of the solid host rock (denoted with subscript $H$ in Fig. 6.41).

![Figure 6.41. Problem layout considering a fault zone of limited extent](image)

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**Figure 6.41.** Problem layout considering a fault zone of limited extent
6.7.4.1 Host rock and fault zone of uniform permeability

First we consider ground and host rock of uniform permeability (i.e., $k_H/k = 1$) and quantify the wall-effect due to the different rock properties of fault zone and solid host rock for $n = 4$, $8$ and $16$ drainage boreholes drilled in advance of grouting (borehole layout A of Fig. 6.15a). Comparison is drawn to the previously discussed results of a fault of unlimited extent (Section 6.7.1).

Figures 6.42b and 6.42c illustrate the wall-effect by means of the displacement vectors which develop when considering 8 advance drainage boreholes. In a fault of unlimited extent, deformations are high and occur also in some horizontal distance to the tunnel axis (Fig. 6.42c). In a fault of limited extent, the deformation diminishes with decreasing distance to the interface of fault zone to host rock (Fig. 6.42b), where shear stresses, triggered by the deformation in the fault zone, act against the displacement. The reduction in effective stresses acting on the grouting body sums up to about 10% (after drainage and subsequent grouting: $\sigma_h' = 3.8$ MPa in a fault of unlimited extent, $\sigma_h' = 3.4$ MPa in the fault of limited extent).

Figure 6.42a shows the characteristic lines for both a fault of limited and unlimited extent (the latter denoted as “hom”). The displacement of the excavation boundary is clearly favourable when considering the limited fault zone (compare black to grey lines in Fig. 6.42). For the example of 8 drainage boreholes and considering no lining support pressure, the displacement is 50% smaller than in fault of unlimited extent (compare point A to B in Fig. 6.42a). The stabilizing wall-effect increases with larger number of drainage boreholes, i.e. increased seepage area (compare dashed grey to black line in Fig. 6.42a). Of course, the characteristic lines coincide for high support pressure, i.e. allowing virtually no displacement.

![Figure 6.42](image-url)

**Figure 6.42.** (a) Comparison of characteristic lines for drainage in advance of grouting considering a fault of limited and a fault of unlimited extent. Displacement vectors (b) in a fault of limited extent and (c) in a fault of unlimited extent (drainage layout A of Fig. 6.15a; $k_H/k = 1$, other parameters see Fig. 6.41)
6.7.4.2  Host rock and fault zone of different permeability

The effect of host rock and fault permeability is discussed for 8 drainage boreholes drilled in advance of grouting (Fig. 6.43). Figure 6.44 compares three permeability ratios of host rock to fault zone: the left column shows the case of a low-permeability fault \( k_{H}/k = 10 \); the middle column the case of uniform permeability \( k_{H}/k = 1 \) and the right column the case of a high-permeability fault \( k_{H}/k = 0.1 \). The top-line shows the overall hydraulic head field, the second line the enlargement in vicinity of the tunnel, the third line the plastic zone for a lining support pressure of \( \sigma_a = 0.4 \) MPa when considering drainage borehole layout B. The bottom line shows the characteristic lines for both layout A and B. It includes for comparison the lower borderline case of ideal drainage in advance of grouting as green line \( (l = 15 \) m in Fig. 6.15c); the upper borderline case of no drainage measure is shown as red line.

In uniformly permeable ground \( (k_{H}/k = 1) \), the hydraulic head dissipates uniformly in both rock and fault zone (Fig. 6.44b and e). The characteristic lines when considering drainage measures are clearly favourable (compare e.g. ideal drainage of Fig. 6.44k to 6.40b). Previously discussed lower efficiency of layout B (Section 6.7.2.2) is no longer of relevance: borehole layouts A and B are equivalent (Fig. 6.44k). However, when assuming no lining support \( (\sigma_a = 0) \), the displacement considering borehole layouts A and B is by factor 2.5 larger than when assuming ideal drainage in advance of grouting (compare black to green line in Fig. 6.44k).

In case of a low-permeability fault \( k_{H}/k = 10 \), pore pressure relief takes place within the fault zone (Fig. 6.44a). The hydraulic gradients acting on the grouting body are higher (compare Fig. 6.44d to e), thus the plastic zone is larger than in ground of uniform permeability (compare Fig. 6.44g to h). Therefore, the characteristic lines are less favourable (Fig. 6.44j). The stress-displacement behaviour of both the considered drainage layouts A and B are very similar with small advantage for layout A (compare dotted-dashed to dashed line in Fig. 6.44j). Note that the deviation to the case of ideal drainage is substantial (compare black to green lines in Fig. 6.44j).
The most favourable situation is the case of a high-permeability fault \(k_H/k = 0.1\). Pore pressure relief takes place within the (less permeable) host rock and very low gradient act on and within the grouting body (Fig. 6.44c and f). The plastic zone is smaller and has an egg-shape in case considering layout B (compare Fig. 6.44i to h). However, drainage layouts A and B are equivalent in terms of stability and
both the characteristic lines are very close to the case of ideal drainage for lining support pressures larger than $\sigma_a = 0.1$ MPa (compare black to green lines in Fig. 6.44l).

Note that the characteristic line of the upper borderline case (i.e. grouting bodies without drainage boreholes; red lines in Fig. 6.44jkl) is virtually unaffected of the ratio $k_H/k$, as pore pressure relief takes place mainly within the grouting body and thus does not lead to significant consolidation of the untreated ground, which in turn would trigger the favourable wall-effect.

Summing up, the wall-effect developing in case of advance drainage measures in the exemplary fault of 60 m width is clearly favourable for the stability of grouting bodies. 8 boreholes drilled in advance of grouting may decrease displacements by more than 50% compared to a fault of unlimited extent. The effects of drainage layout B is almost equal to layout A. The permeability ratio of solid host rock to fault zone is a key factor for the hydraulic head field and needs to be considered when determining the characteristic line considering drainage in advance of grouting in a fault of limited extent. The model of ideal drainage in advance of grouting proved to be adequate only for a fault zone of higher permeability than the host rock ($k_H/k \leq 0.1$).

### 6.8 Conclusions

The chapter in hand showed that drainage measures considerably increase the stability of grouting bodies in water-bearing fault zones. Considerations were limited to an exemplary deep tunnel without claiming general validity. However, there is no reason to doubt that the analytical solutions of ideal drainage represent lower limits for pre-dimensioning grouting bodies and that the accuracy will be in a comparable range when considering specific borehole arrangements similar to the ones discussed above.

#### 6.8.1 Analytical solution of ideal drainage

##### 6.8.1.1 Drainage after grouting

Equations have been derived for the displacement at the excavation boundary $u_a$, which is a function of the geometry of the grouting body $b/a$, the degree of drainage $\eta$, the degree of plastification $\lambda$, the material properties of the grouting body (effective cohesion $c_I$, Young’s Modulus $E_I$, angle of internal friction $\varphi_I$, angle of dilatancy $\psi_I$, Poisson’s ratio $\nu_I$), the ratio of permeability of grouting body to untreated ground $k_I/k$, the in-situ pore pressure $p_0$ and the lining support pressure $\sigma_a$.

Ideal drainage does increase the stability of the grouting body only if at least some moderate plastification is allowed ($\lambda > 0.1$). In a fully elastic grouting body, drainage is tentatively unfavourable for stability. Other measures to increase the stability of grouting bodies (for a given degree of plastification) is to increase the size $b/a$, the uniaxial compressive strength $f_{cI}$ or the lining support pressure $\sigma_a$.

The optimal drainage degree is equal to the degree of plastification ($\eta = \lambda$). A larger drainage area does not affect stability, but might involve the risk of inner erosion due to the high gradients prevailing in the outer ring of the grouting body.

Limiting the degree of drainage to $\eta < 0.8$ ensures the sealing effect of grouting bodies, which is 100 times less permeable than the untreated ground, and inflow is reduced to 20% of the value without considering any grouting body.
6.8.1.2 Drainage in advance of grouting

Drainage in advance of grouting decreases both displacements and stresses at the excavation boundary in comparison to drainage after grouting. The displacement at the excavation boundary \( u_\alpha \) are a function of the geometry of the grouting body \( b/a \), the degree of plastification \( \lambda \), the material properties of the grouting body (effective cohesion \( c_I \), Young’s Modulus \( E_I \), angle of internal friction \( \varphi_I \), angle of dilatancy \( \psi_I \), Poisson’s ratio \( \nu_I \)) and the lining pressure \( \sigma_a \), but not of the radius of advance drainage \( l \). Dimensioning aids developed in previous studies apply for drainage in advance of grouting when using \( \sigma'_b \) according to Eq. (6-7) instead of \( \sigma' \) in Figures 5.3-1 to 5.3-4 of Anagnostou and Kovári (2003).

The model-assumption of a very stiff grouting body \( (u_b = 0) \) may marginally overestimate the displacements at the excavation boundary.

6.8.2 Specific drainage borehole arrangements

6.8.2.1 Drainage borehole arrangements drilled after grouting

Considering ideal drainage after grouting by means of the analytical solution represents the lower limit for the stress-displacement behaviour at the excavation boundary. 12 drainage slits in the cross-section (two-dimensional consideration) approximate ideal drainage conditions with 5% accuracy for a degree of plastification of \( \lambda = 0.5 \) and with 10% accuracy for higher plastification. A small grouting body requires longer drainage slits than a large grouting body to account for the same pore pressure relief.

When arranging 12 boreholes such as they lead to a spatial uniform hydraulic head field (three-dimensional consideration with with about an equal spacing between the boreholes in radial and axial direction; \( e \approx m \) in Fig. 6.27b), up to 12% higher displacements result than in the borderline case of ideal drainage for a degree of plastification of \( \lambda \leq 0.3 \) (30% accuracy for \( \lambda = 0.5 \)). For better accordance, the axial spacing has to decrease. Virtually unaffected of the axial distance, the sealing effect of the grouting body leads to less than 20% of the water inflow in untreated ground.

6.8.2.2 Drainage borehole arrangements drilled in advance of grouting

Considering ideal drainage in advance of grouting by means of the analytical solution determines the lower limit of the characteristic line when considering specific advance drainage borehole arrangements. A minimum of 8 coaxial drainage boreholes, arranged evenly distributed around a circle line of a radius which is 1-2 m larger than the grouting body (layout A), leads to a uniform shape of the plastic zone. An additional increase in borehole number provides marginal additional benefit in terms of pore pressure relief. The model of ideal drainage is adequate for consideration of the stress-displacement behaviour at the excavation boundary up to \( \lambda \leq 0.5 \) with 20% accuracy in case of 8 drainage boreholes and with 10% for 16 boreholes. A lateral borehole arrangement (layout B) may be preferred due to accessibility and drilling operation, but it increases the displacements by about 20-30% compared to the circumferential arrangement of layout A.

The smaller seepage area of individual advance drainage boreholes decreases the water inflow considerably compared to ideal drainage (e.g. by 25% for 8 boreholes).

For a fault of limited extent, favourable shear stresses develop at the interface to the solid rock due to consolidation induced by advance drainage. Displacements may decrease by more than 50% when considering 8 drainage boreholes drilled in advance of grouting. The permeability ratio of solid host rock to fault zone needs to be considered when determining the characteristic line. Good accordance to the case of ideal drainage was found only for fault zones of higher permeability than the host rock.
7. Conclusions and outlook

In this thesis, the effects of drainage measures for two particularly important problems of tunnelling through water-bearing ground have been investigated systematically and comprehensively: the stability of the tunnel face and the stability of grouting bodies. Drainage measures improve the stability and deformation behaviour of underground openings decisively by relieving the pore pressure which in turn, (i), reduces the magnitude of the seepage forces acting on the ground, and (ii), increases the effective stress and thus the resistance of the ground to shearing.

The first part of the thesis focused on face stability which was analysed through limit equilibrium computations taking account of the numerically determined seepage flow conditions prevailing in the ground after the implementation of a drainage measure. The findings were summarized within the respective chapters. The main contributions may be summarized as follows:

- A design equation was developed (Chapter 2), which allows a quick assessment to be made of tunnel face stability considering the effects of various advance drainage schemes and thus representing a very valuable design aid for tunnel engineers. The coefficients that appear in the equation can be depicted using dimensionless nomograms worked out by analysing the computational results of a comprehensive parametric study incorporating a wide parameter range in a homogeneous ground.

- In ground of non-uniform permeability, the drainage boreholes should be arranged so that they lower the pressure in and around critical barrier layers; they will then be as effective as in homogeneous ground. Various critical situations in terms of the orientation, elevation and thickness of ground layers of different permeability to the surrounding rock were set out in Chapter 3. Also, the suitability of using an equivalent homogeneous anisotropic model for calculating the seepage flow condition was shown in the case of thin layered ground.

- The application range of drainage measures is restricted due to feasibility aspects related to the drainage boreholes themselves (associated with their hydraulic capacity and casings; Chapter 4), the ground permeability in combination with the construction process (the lead-time required for pore pressure relief) or environmental constraints (admissible groundwater drawdown, settlement or inflow; Chapter 5). An equivalent conductivity model taking account of pipe-flow hydraulics within the borehole allowed a determination to be made of the potential loss of pore pressure relief in the surrounding ground with regard to the hydraulic capacity of the borehole. Also, Chapters 4-5 discussed the potential loss of pore pressure relief in the surrounding ground due to the aforementioned factors and showed the applicability limits of the design nomograms of Chapter 2.

The second part of the thesis focused on the stability of grouting bodies in geological fault zones under high hydrostatic pressure. The relationship between support pressure and displacement of the excavation boundary was investigated by considering the ground as a porous, elasto-plastic medium obeying the principle of effective stress, Coulomb’s failure criterion and by taking into account the seepage forces developing in the presence of a series of different drainage measures. The findings were summarized within Chapter 6. The main contributions may be summarized as follows:

- For the virtual case of ideal drainage (either in advance of grouting or after grouting), existing analytical solutions were extended to consider an arbitrary radius of the drained zone.

- The effectiveness of realistic drainage schemes was studied by means of hydraulic-mechanical coupled FE-modelling considering the pore pressure relief resulting from the individual
boreholes. The computational results provide valuable information about number, length, spacing and location of drainage boreholes.

- Drainage (either in advance of grouting or after grouting) increases the stability of the grouting body considerably, both for ideal drainage and for specific arrangements of drainage boreholes. The analytical solutions for the case of ideal drainage provide a lower limit for lining support pressure and displacement at the excavation boundary. Chapter 6 shows under which conditions these solutions can be used in the case of realistic drainage schemes for pre-dimensioning of grouting bodies in engineering practice.

In the author’s opinion, there are still a number of open questions concerning drainage measures in tunnelling, which merit further investigation. With regard to face stability, the following aspects are of interest:

- **Failure mechanism**: The study in hand analysed face stability through the limit equilibrium of a wedge and prism. But when encountering a fault zone that acts as an aquitard, punching failure may also need to be considered. In layered ground, the failure of individual layers is possible. Also, deformations occurring in weak ground could be decisive (e.g. extrusion of the tunnel face, but also lining convergence in squeezing rock conditions). The interaction of these aspects with drainage measures has not yet been investigated.

- **Casings**: The selection and handling of casings allowing for pore pressure relief is a demanding task on site and solutions are often found only by trial and error. A more detailed investigation into the design of casings (with materials and shapes providing high stiffness and torque, but still offering adequate spacing for pore pressure relief; the hydraulic behaviour of inflow into casings when taking account of local losses at the openings etc.) and execution aspects of drilling and insertion would be of interest for drainage applications in tunnelling practice.

- **Groundwater drawdown**: The study in hand calculated the maximum drawdown in groundwater according to the residual flow method (a sharp drop in permeability between overlying, unsaturated ground and a lower, saturated region) in homogeneously permeable ground. A broader study into the maximum admissible groundwater drawdown is recommended with a focus on three aspects: (i) consideration of ground of non-uniform permeability; (ii) comparisons with the maximum drawdown according to more sophisticated FE-modelling (e.g. by considering unsaturated flow by means of Richard’s equation; FEM-procedures for moving mesh etc.); and (iii) consideration of the possible decrease in permeability during consolidation (by coupling the hydraulic and mechanical behaviour for the calculation of settlements).

With regard to the stability of grouting bodies, there is need for research concerning:

- **Mutual interference of permeability and plastification**: The study in hand assumes a constant permeability in the grouting body. During plastification, however, micro-cracks may develop within the plastic zone, which would increase permeability. Since the seepage field proved to be a crucial aspect in stability analysis, any change in permeability might have a major impact. Further investigations are necessary, employing fully coupled numerical studies that can reproduce the mutual interference of permeability and plastification. In addition, field or laboratory tests to provide input-data on the changes in permeability during plastification are recommended.

- **Pore pressure relief resulting from drainage boreholes**: The study in hand considered the drainage boreholes by assuming atmospheric pressure within a borehole of fixed size in
seepage analysis. This might overestimate the pore pressure relief resulting from the drainage boreholes with regard to two scenarios: (i) highly permeable ground under a high water table, where the capacity of the drainage boreholes drilled outside the grouting body might hinder full pressure relief; and (ii) highly plastified ground, where deformation of the drainage boreholes drilled inside the grouting body might hinder full pressure relief. Both scenarios should be taken into consideration in further research by applying appropriate boundary conditions; (i) considering flow conditions, and (ii) considering the physical interaction of drainage boreholes (with and without casing) with the surrounding ground.

- **Fault of limited extent:** The study in hand focused on grouting bodies in faults of practically unlimited extent and discussed the wall-effect occurring in narrow faults by means of only one single example. A systematic investigation of faults of limited extent would need to consider geometrical factors (fault width, orientation to the tunnel axis and spatial extent, size of grouting body), hydraulic aspects (permeability of the fault zone, grouting body and untreated ground as well as the interaction with several different drainage borehole arrangements) and variable soil parameters (stiffness and strength of the fault zone and the untreated ground).
Appendix A. Publications from the present thesis

Major parts of Chapter 2 have been published in:

and preliminary results have been published in:

and presented in:

Preliminary results of Chapter 3 have been presented in:

Preliminary results of Chapter 5 have been presented in:
Within the framework of the present PhD thesis, several Master’s theses evolved under lead and with the close support and collaboration of the author.

Preliminary numerical studies for Chapter 2 were carried out in:

Preliminary numerical studies for Chapter 3 were carried out in:

Numerical studies for Chapter 6 were carried out in:
Appendix B. Matlab-code drainage in advance of grouting

Matlab-code for the characteristic line considering ideal drainage in advance of grouting

% calculates the characteristic line (displacements and stresses at the excavation boundary) of the grouting body which is drained in advance (l≥b).

% definition of problem
sigma_0 = 4.4*10^3; % in-situ total stresses [kPa]
p0 = 2*10^3; % in-situ pore water pressure [kPa]
u = 1/3; % Poisson ratio [-]
R = 200; % radius of influence for pore pressure decrease [m]
E = 0.1*10^6; % Youngs Modulus of the ground [kPa]

% injection body
E_I = 1*10^6; % Youngs Modulus [kPa]
c = 500; % cohesion [kPa]
u_I = 1/3; % Poisson ratio [-]
phi = 25; % angle of friction [degree]
psi = 25; % angle of dilatancy [degree]
a = 5; % radius of tunnel excavation [m]
b = 12.5; % radius of grouting [m]

% assembling governing equations
kappa = (1+sind(psi))/(1-sind(psi));
m = (1+sind(phi))/(1-sind(phi));
f_c = 2*c*cosd(phi)/(1-sind(phi));
sigma_b = sigma_0-p0+p0/(2*(1-u));
sigma_b_quer = sigma_b+f_c/(m-1);

% substitutions
A1 = (nu*I+1)*(1-2*nu_I);
A2 = (nu*I+1)*(nu*I+1)+nu*I+1
A3 = (nu*I+1)^2

% displacements as function of the degree of plastification (rho= radius of plastification [m])
for i=1:100
    B(i) = (rho/a)^2*(1-m+m*(b/rho(i))^2)/(2*(b/rho(i))^2);
end

% displacements and plastification due to excavation
for j=1:100
    ua(j) = E/I*sigma_b_quer*(-A1+1/B(j)*(A2+A3*(rho/a)^2));
end

% plot of characteristic line
figure(2);
plot(sigma_a/1000,ua,'k','LineWidth',1);

% end
Appendix C. Matlab-code drainage after grouting I

Matlab-code for the characteristic line considering ideal drainage of the inner part of the grouting body

% Definition of problem
E = 1000*10^6; % Youngs Modulus of grouting body [Pa]
mu = 0.33; % Poissons ratio of grouting body [-]
phi = 25/180*pi; % angle of internal friction of grouting body [rad]
c = 0.5*10^6; % effective cohesion of grouting body [Pa]
kappa = 1/(1-sin(phi)); % loosening factor of grouting body [-]
a = 5; % excavation radius [m]
b = 12.5; % grouting radius [m]
s_o_tot = 4.4*10^6; % in-situ total stress [Pa]
pO = 2*10^6; % in-situ pore water pressure [Pa]
R = 200; % radius of influence for pore pressure decrease [m]
kl = 10^(-8); % permeability of the injection body [m/s]
kg = 10^(-6); % permeability of the untreated ground [m/s]
s_o = s_o_tot-pO; % in-situ effective stress [Pa]

% Assembling the governing equations
s_o_dash = s_o+c/tan(phi);
fc = 2*c*cos(phi)/(1-sin(phi));
m = (1+sin(phi))/(1-sin(phi));
kappa = m;
p_undrained = porewaterpressure(a,b,R,kl,kg,pO);
s_b_undrained = s_o + (pO-p_undrained);
s_b_drained = s_o+pO;
A1 = (1+nu)*(1-2*nu);
A2 = (1+nu)^*(1-m*kappa)-nu*(m+kappa)/(m+kappa);
A3 = (1-m^2)*((m/2-1)/(m+kappa));
A6 = (1+m^2)*kappa*(m+kappa)/(m+kappa);

jmax = 4; % number of drainage factors considered (see below definition of drain_deg)
for j = 1:jmax
    counter = 1;
drain_deg = [0.2,0.4,0.6,0.8];
l(j) = a+(b-a)*drain_deg(j);
pb = porewaterpressure(l(j),b,R,kl,kg,pO);
s_b(j) = s_o +(pO-pb); % no stress redistribution at the outer boundary of the grouting body
f = pb/log(b/l(j));

    i = 0;
    while i < 100 % loop for the plastic radius
        i = i +1;
rho_out = 0; % initial value of outer plastic radius
        plast_deg(i) = 1/100*i;
        rho(i) = a+(b-a)*plast_deg(i);
        K = 2*(b/rho(i))^(m+1)/((b/a)^(m-1)*(1-m+(m+1)*(b/rho(i))^2));
        s_a_undrained(i) = B2/B1*s_b_undrained+B7/B1*p_undrained-fc/B1;
        s_a_tilde = s_a_undrained(i) + (fc-f_undrained)/(m-1);
        s_o_tilde = s_o+(fc-f_undrained)/(m-1);
        s_a_tilde = s_a_tilde+S*(s_a_undrained(i)+s_o_tilde*A1+s_o_tilde*A2)/(m+kappa)+f_undrained*(A6+(1+nu)/(kappa+1)*(rho(i)/a)^(kappa+1));
    end

% Ideal drained grouting body
K = 2*(b/rho(i))^(m+1)/((b/a)^(m-1)*(1-m+(m+1)*(b/rho(i))^2));
s_a_drained(i) = (s_b_drained-fc/m)^K-fc/m;
C2 = (0.5*f*(1+P2/P1-(1-nu+log(b/l(j)))/(1-nu))-fc/(m-1)-s_b(j))/(E/(l(j)*(1+nu))*(l(j)^2/b^2-1)-2*(1-nu)/P1);
C1 = P2/P1*C2*(2+nu)*log(b/l(j))/(1+nu)^

% grouting body without drainage ("undrained")
if drain_deg % This corresponds to case I
    P1 = a/E*(2*(m-1)^2*(1-m^2)*l(j)^2)/(l(j)^2+i)+l(j)+l(j)^2); % initial value of outer plastic radius
    B2 = 2*sin(phi)/(1-sin(phi));
    B1 = 2*sin(phi)/(1-sin(phi))+B2;
    B7 = (2*(1+nu)-B2)*log(rho(i)/(1+nu))-B2*(1+nu)*log(b/l(j));
    s_a_undrained(i) = B2/B1*(s_a_drained(i)-B7/B1*p_undrained(i)+C1);
    s_o_tilde = s_o_tilde*A1+s_o_tilde*A2/(m+kappa)+f_undrained*(A6+(1+nu)/(kappa+1)*(rho(i)/a)^(kappa+1));
end

% Deformation and stresses of the cylinders do not account for the initial hydrostatic loading of sigma_o before excavation. The correction of the % elastic deformation due to the difference of the hydrostatic initial stresses and the actual loading of the grouting body is done in the last % equation when the displacement of the excavation boundary is calculated
if rho(i)==l(j) & & rho(i)==a
    P1 = a/E*(2*(m-1)^2*(1-m^2)*l(j)^2)/(l(j)^2+i)+l(j)+l(j)^2); % initial value of outer plastic radius
    P2 = l(j)^2/(1+nu)-l(j)^2); % initial value of outer plastic radius
    C2 = (0.5*f*(1-P2/P1(l(j)^2))/(1+nu)-log(b/l(j)))/((1+nu)-f(1+nu)^*log(b/l(j)))+l(j)/(1+nu)))+l(j); % initial value of outer plastic radius
    C1 = P2/P1*C2*(2+nu)*log(b/l(j))/(1+nu)^

% Calculation of the displacements
s_l = E/(l(j)*(1+nu))*(C1/(1-2*nu)-C2)-0.5*f;
s_b_tang(i) = E/(l(j)^2)/(1+nu)*(C1/(1-2*nu)+C2*(1+nu)/b^2)-0.5*f*(nu+log(b/l(j)))/(1-nu);
s_l_vector(i) = s_l;
while frac(i,j) < -4
rhou = rho_out + 0.01;
bnew = b - rho_out;
s_b_tilde = s_b(j) + (fc - s_b_tang(i)) / (m - 1);

\frac{i,j} = \frac{m\cdot s_b(j) + fc - s_b\text{\_tang}(i)}{m - 1};
\frac{P_1} = \frac{E\cdot (2\cdot l(j)\cdot (1 -\nu^2)\cdot (l(j)/a)/(1 - m + (m + 1)\cdot l(j)/rho(i))\cdot (l(j)/rho(i))^2)}{1 - m + (m + 1)(l(j)/rho(i))^2};
\frac{P_2} = \frac{E\cdot (l(j)/rho(i))^2}{1 - m + (m + 1)(l(j)/rho(i))^2};
\frac{C_1} = \frac{P_2\cdot \nu \cdot C_2}{1 - m + (m + 1)(l(j)/rho(i))^2};
\frac{C_2} = \frac{P_1\cdot F_2}{1 - m + (m + 1)(l(j)/rho(i))^2};

s_b\text{\_tang}(i) = \frac{E\cdot (l(j)/rho(i))^2}{1 - m + (m + 1)(l(j)/rho(i))^2}\cdot C_1\cdot (C_2\cdot l(j)/rho(i)^2);
% calculation of the displacement at the excavation boundary
if rho(i) > l(j) & & rho_out == 0
  u_l = l(j)/E*(-s_o_tilde*A1+s_l_tilde*(A2+A3*(rho(i)/l(j))^(m+kappa))+f*(A6+(1+nu)/(kappa+1)*(rho(i)/l(j))^(kappa+1)));
  u_l_el = -l(j)*(1+nu)*(1-2*nu)/E*(0-s_l);
  u_l3(i) = u_l - u_l_el;
  if counter ==1
    u_a3_fully_plast = (s_l + fc/(m-1))*a/E*(-A1+A2*(l(j)/a)^(1-m)+A3*(l(j)/a)^(kappa+1));
    u_l3_fully_plast = (s_l + fc/(m-1))*a/E*((m-1)*(1-nu^2)*l(j)/a);
    end
  u_a3(i) = u_a3_fully_plast + (u_l3(i)-u_l3_fully_plast)*(l(j)/a)^kappa;
  counter = counter +1;
else rho.in == l(j)
  u_a3_fully_plast = (s_l + fc/(m-1))*a/E*(-A1+A2*(l(j)/a)^(1-m)+A3*(l(j)/a)^(kappa+1));
  u_l3_fully_plast = (s_l + fc/(m-1))*a/E*((m-1)*(1-nu^2)*l(j)/a);
  end
end
u_a3(i) = u_a3(i) + u_a_el(i);
end
figure(1);
plot(plast_deg, s_a/10^6 , 'b','LineWidth',1);
hold on
%extension for the fully plastic and elastic part
m_elastic = (u_a(2)-u_a(1))/(s_a(2)-s_a(1));
s_elastic = s_a(1)*1.25;
u_elastic = u_a(1)+(s_elastic-s_a(1))*m_elastic;
s_a_full = [s_elastic, s_a, s_a(i)];
u_a_full = [u_elastic, u_a, u_a(i)+0.5];

figure(2);
plot(u_a_full, s_a_full/10^6,'g','LineWidth',1);
plot(u_a(2), s_a(2),'r','LineWidth',1);
hx = xlabel('$\lambda \ [\%]$  ','interpreter','latex','FontSize',18);
yh = ylabel('$\sigma_{a,req} \ [MPa] $ ','interpreter','latex','FontSize',18);
grid on
hold off
% extension for the fully plastic and elastic part
m_drained = (u_a_drained(2)-u_a_drained(1))/(s_a_drained(2)-s_a_drained(1));
s_drained = s_a_drained(1)*1.25;
u_drained = u_a_drained(1)+(s_drained-s_a_drained(1))*m_drained;
s_drained_full = [s_drained, s_a_drained, s_a_drained(i)];
u_drained_full = [u_drained, u_a_drained, u_a_drained(i)+0.5];

figure(2);
plot(u_drained_full, s_drained_full/10^6,'g','LineWidth',1);
plot(u_a(2), s_a(2),'r','LineWidth',1);
hx = xlabel('$u_{a} \ [m] $ ','interpreter','latex','FontSize',18);
yh = ylabel('$\sigma_{a} \ [MPa] $ ','interpreter','latex','FontSize',18);
grid on
hold off
Appendix D. Matlab-code drainage after grouting II

Matlab-code for the required strength of the grouting body considering ideal drainage of the inner part thereof

a) Required strength

% Calculates the required strength of the grouting body in function of the support sigma_a in order to limit the plastic radius to a certain extent
% given by the degree of plastification lambda

% definition of problem
E = 1000*10^6; % Youngs Modulus of grouting body [Pa]
mu = 0.33; % Poisson ratio of grouting body [-]
phi = 25/180*pi; % angle of internal friction of grouting body; assumed to be equal to angle of dilatancy [rad]
kappa = 1/(1-sin(phi)); % loosening factor of grouting body [-]
a = 5; % excavation radius [m]
b = 12.5; % grouting radius [m]
s_o_tot = 4.4*10^6; % in-situ total stress [Pa]
pO = 2*10^6; % in-situ pore water pressure [Pa]
R = 200; % radius of influence for pore pressure decrease [m]
kI = 10^(-8); % permeability of the injection body [m/s]
kG = 10^(-6); % permeability of the untreated ground [m/s]
s_o = s_o_tot-pO; % in-situ effective stress [Pa]
lambda = 0.6; % degree of plastification [-]
m = (1+sin(phi))/(1-sin(phi));

% assembling the governing equations
% in-situ stresses and pressures ("undrained") at the outer boundary of the injection body
p_undrained = porewaterpressure(a,b,R,kI,kG,pO);
s_b_undrained = s_o + (pO-p_undrained);
s_b_drained = s_o+pO;

% number of lines in the final plot. Each line represents a drainage length l
jmax = 6;
for j = 1:jmax
l(j) = a+(b-a)/(jmax-1)*(j-1);

% pore water pressure and effective strength at the outer boundary of the grouting body for drainage length l
pb = porewaterpressure(l(j),b,R,kI,kG,pO);
s_b = s_o +(pO-pb);
f = pb / log(b/l(j));

% the excavation support is evaluated in function of the compressive strength
for i = 1:150
rho = lambda*(b-a)+a;
rho_out = 0;
c = (i-1)*10^(-4);
f(i) = 2*(c*cos(phi))/(1-sin(phi));

if rho < l(j)
% This corresponds to case I
P1 = a/E*(2*(m-1)^2*(l(j)/a))/(1-m*(1-l(j)/rho));
P2 = l(j)^*(1-m*a*(1-2*nu)/E);
P3 = (0.5*P1+P2)/(P1+P2);
P4 = P1+P2;
P5 = P1+P2;
P6 = P1+P2;
P7 = P1+P2;
P8 = P1+P2;
P9 = P1+P2;
P10 = P1+P2;
P11 = P1+P2;
P12 = P1+P2;
P13 = P1+P2;
P14 = P1+P2;
P15 = P1+P2;
P16 = P1+P2;
P17 = P1+P2;
P18 = P1+P2;
P19 = P1+P2;
P20 = P1+P2;
P21 = P1+P2;
P22 = P1+P2;
P23 = P1+P2;
P24 = P1+P2;
P25 = P1+P2;
P26 = P1+P2;
P27 = P1+P2;
P28 = P1+P2;
P29 = P1+P2;
P30 = P1+P2;
P31 = P1+P2;
P32 = P1+P2;
P33 = P1+P2;
P34 = P1+P2;
P35 = P1+P2;
P36 = P1+P2;
P37 = P1+P2;
P38 = P1+P2;
P39 = P1+P2;
P40 = P1+P2;
P41 = P1+P2;
P42 = P1+P2;
P43 = P1+P2;
P44 = P1+P2;
P45 = P1+P2;
P46 = P1+P2;
P47 = P1+P2;
P48 = P1+P2;
P49 = P1+P2;
P50 = P1+P2;
P51 = P1+P2;
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P79 = P1+P2;
P80 = P1+P2;
P81 = P1+P2;
P82 = P1+P2;
P83 = P1+P2;
P84 = P1+P2;
P85 = P1+P2;
P86 = P1+P2;
P87 = P1+P2;
P88 = P1+P2;
P89 = P1+P2;
P90 = P1+P2;
P91 = P1+P2;
P92 = P1+P2;
P93 = P1+P2;
P94 = P1+P2;
P95 = P1+P2;
P96 = P1+P2;
P97 = P1+P2;
P98 = P1+P2;
P99 = P1+P2;
P100 = P1+P2;
end

while frac(i,j) > 0.4
frac(i,j) = m^*(s_b+fc(i)-s_b_tang(i))/s_b_tang(i);
end
\[ P3 = (k(j)a)^{(m-1)(1-m-1)^{2}}(\frac{2\pi \rho}{r})^{2}\frac{2\pi \rho}{r}^{(m+1)}; \]
\[ s_a(i) = s_l^{(i)/P3 + f_c(i)/(m+1)}; \]
elseif \( \rho > l(j) \)
\[ B2_{out} = (2\sin(\phi)/(\rho/l(j))^{(2\sin(\phi)/(1-\sin(\phi)))\rho/(1-\sin(\phi))^{\rho/\rho}}; \]
\[ B7_{out} = (2\sin(\phi)/(\rho/l(j))^{(2\sin(\phi)/(1-\sin(\phi))^{\rho/\rho}}; \]
\[ s_a(i) = s_l^{B2_{out}(B1_{out}^{(i)/a})^{(m+1)}}; \]
\[ s_\rho = (\rho/l(j))^{(m-1)}(s_\rho^{(i)-f(i)}(m+1)}; \]
\% check whether elasticity condition is fulfilled at the outer grouting body boundary
\[ prho = \log(\rho/l(j))\log(\rho/l(j))^{\log(b/rho)}; \]
\[ s_b_tang(i) = s_b^{((b/rho)^{2}+1)/((b/rho)^{2}-1)-s_\rho^{2/((b/rho)^{2}-1)}; \]
\% Figures
\[ plot(s_a/10^6,fc/10^6,'b','LineWidth',1); \]
\[ hold on \]
\[ end \]
\end{verbatim}

\textbf{b) Pore water pressure}

\% calculates the pore water pressure at the interface of two formations of different permeability \( kI \) (injection body) and \( kG \) (untreated ground)
\[ f_c = \log(b/l)/(\log(b/l)+kI/kG*\log(b/l)); \]
\[ s_b = \log(b/l)/(\log(b/l)+kI/kG*\log(b/l)); \]
\[ s_b = \log(b/l)/(\log(b/l)+kI/kG*\log(b/l)); \]
\[ pb = \log(b/l)/(\log(b/l)+kI/kG*\log(b/l)); \]
\[ pO = \log(b/l)/(\log(b/l)+kI/kG*\log(b/l)); \]
\[ \]
Notation

**Latin symbols**

- $a$: radius of the tunnel
- $A_1, A_2, A_3, A_6$: factors
- $a_{dr}$: distance of drainage curtains
- $A_i$: area of cross-section at location $i$
- $a_i$: fraction of layer $i$
- $B$: average hydraulic head of a specific drainage scheme at time $t$
- $b$: radius of the grouting body
- $B_1, B_2, B_7$: factors
- $b_{red}$: reduced radius of the grouting body
- $c$: effective cohesion of the (untreated) ground
- $C_1, C_2$: coefficients
- $c_H$: effective cohesion of the solid host rock
- $c_i$: effective cohesion of the injection body
- $c_{lim,d}$: limiting cohesion
- $c_{lim,s}$: limiting cohesion
- $c_w$: compressibility of water
- $D$: diameter of the tunnel
- $D'$: side length of equivalent square tunnel cross-section
- $d_b$: diameter of grouted borehole
- $d_{dr}$: diameter of drainage borehole
- $d_{hy}$: hydraulic diameter
- $d_i$: thickness of a layer
- $d_{lin}$: thickness of the lining
- $d_p$: diameter of pilot tunnel
- $E$: Young’s modulus of the (untreated) ground
- $e$: axial distance of drainage boreholes
- $e_d$: coefficient
- $E_H$: Young’s modulus of the solid host rock
- $E_i$: Young’s modulus of the injection body
- $e_s$: coefficient
- $E_S$: constrained modulus of the ground
- $F$: filling ratio
- $f(...)$: function of ...
- $F_0 - F_3$: coefficients provided by nomograms
- $f_2$: factor of pore water pressure
- $f_{ci}$: uniaxial compressive strength of the injection body
- $f_{ci,r}$: required uniaxial compressive strength of the injection body
- $f_s$: seepage force
- $g$: acceleration due to gravity
\( G' \) submerged weight of wedge
\( H \) depth of cover
\( h \) hydraulic head
\( \bar{h} \) normalized hydraulic head
\( h_0 \) initial hydraulic head (depth of the tunnel axis underneath the water table)
\( \bar{h}_0 \) normalized initial hydraulic head
\( h_{av} \) average hydraulic head over prism cross-section
\( \bar{h}_{av} \) normalized average hydraulic head
\( h_{e,i} \) energy head at point \( i \)
\( \bar{h}_{max} \) normalized maximum hydraulic head in borehole
\( h_V \) head loss in drainage borehole
\( H_w \) elevation of water table (with respect to the tunnel crown)
\( I_1, I_3, I_s \) coefficients for the contribution of seepage flow
\( i_{adm} \) admissible hydraulic head gradient
\( I_s \) hydraulic head gradient of the drainage borehole
\( k \) permeability
\( k_1 \) permeability of layer 1
\( k_2 \) permeability of layer 2
\( K_g \) permeability of the ground
\( k_H \) permeability of the solid host rock
\( k_I \) permeability of the injection body
\( k_L \) permeability of a layer
\( k_{lower} \) permeability of the lower formation
\( k_n \) equivalent permeability normal to stratified layers
\( k_p \) equivalent permeability parallel to stratified layers
\( k_{s,eq} \) equivalent sand roughness of drainage borehole wall
\( k_{upper} \) permeability of the upper formation
\( K_x \) equivalent permeability in x-direction
\( K_{x,open} \) equivalent permeability in drainage borehole considering open-channel flow
\( K_{x,pipe} \) equivalent permeability in drainage borehole considering pipe flow
\( k_z \) permeability of a zone
\( L \) centre distance
\( l \) circle line of the drainage measure
\( l_{dr} \) length of drainage boreholes
\( l_{dr, char} \) characteristic length of drainage boreholes
\( L_h \) horizontal centre distance
\( L_v \) vertical centre distance
\( M_{eff} \) degree of pore pressure relief of a drainage scheme at time \( t \)
\( m \) average radial distance of drainage boreholes
\( m_I \) material constant of the injection body
\( n \) number of drainage boreholes (or drainage slits)
\( N' \) effective normal force on the slip pane of the wedge
$n_b$  
bolt density

$N_c$  
value of $N_{co}$ at $\omega_{cr}$

$N_{co}$  
cohesion influence coefficient for wedge angle $\omega$

$n_g$  
porosity of saturated ground

$N_h$  
value of $N_{ho}$ at $\omega_{cr}$

$N_{ho}$  
seepage flow influence coefficient for wedge angle $\omega$

$n_k$  
unit normal vector

$N_f$  
value of $N_{fo}$ at $\omega_{cr}$

$N_{fo}$  
weight influence coefficient for wedge angle $\omega$

$p$  
pore water pressure

$p_0$  
initial pore water pressure

$P_1, P_2$  
coefficients

$p_a$  
pore water pressure at radius $r = a$

$p_{atm}$  
atmospheric pressure (seepage face)

$p_b$  
pore water pressure at radius $r = b$

$p_T$  
transition pressure in drainage borehole

$Q$  
discharge of water, water inflow

$Q_0$  
water inflow in untreated ground

$Q_{id}$  
water inflow when considering ideal drainage

$q_r$  
radial inflow velocity

$q_x$  
axial flow velocity in the simplified borehole model

$q_{\bar{x}}$  
normalized axial flow velocity

$r$  
coordinate in radial direction

$R$  
radius of the hydraulic head field

$r_c$  
ratio of volume to circumferential area of the prism

$r_{dr}$  
distance of boreholes from tunnel axis

$Re$  
Reynolds number

$r_{hy}$  
hydraulic radius

$R_s$  
percentage of seepage face area on the drainage shell surface

$s$  
effective face support pressure

$S$  
integration area

$s_0$  
effective face support pressure for cohesionless ground

$s_{mono}$  
effective face support pressure according to the nomograms

$s_{max}$  
maximum effective face support pressure

$S_s$  
specific storage

$S_{sa}$  
support force for wedge angle $\omega$

$s_{sa}$  
support pressure for wedge angle $\omega$

$T$  
distance of the upper boundary of the seepage flow domain to the tunnel crown

$t$  
time

$t_{dr}$  
fill-level of fluid in drainage borehole

$T_l$  
shear resistance of the inclined slip plane of the wedge

$T_y$  
shear resistance of the two lateral slip planes of the wedge
$u$ radial displacement

$u_a$ radial displacement of the excavation boundary ($r = a$)

$\bar{u}_a$ normalized displacement of the excavation boundary (at $r = a$)

$u_b$ radial displacement of the outer boundary of the grouting body ($r = b$)

$u_{bDR}$ drainage-induced radial displacement (at $r = b$)

$u_l$ radial displacement at the boundary of ideal drainage ($r = l$)

$u_S$ settlement of ground surface

$U_{wet}$ wetted perimeter of drainage borehole

$V$ volume of the wedge

$V'$ effective vertical force of prism

$v_r$ flow velocity horizontal to the drainage borehole

$v_z$ flow velocity vertical to the drainage borehole

$W_k$ seepage forces acting on the wedge

$x$ (local) coordinate parallel to the drainage borehole

$x_1$ (global) horizontal coordinate parallel to the tunnel axis

$x_2$ (global) horizontal coordinate perpendicular to the tunnel axis

$x_3$ (global) vertical coordinate

$x_b$ coordinate along to the drainage borehole casing

$x_f$ position of tunnel face

$x_g$ distance of interface of a layer to tunnel face

$x_s$ spacing between slots or perforations in borehole screens

$y$ (local) coordinate perpendicular to the drainage borehole

$z$ (local) vertical coordinate of the drainage borehole; geodetic height

$z_b$ position of bottom edge of a layer

$z_l$ distance of tunnel axis to interface of a layer

**Greek symbols**

$\alpha$ sector angle between drainage boreholes/slits

$\alpha_1, \alpha_2, \alpha_3$ auxiliary variables of the simplified borehole problem

$\beta_1, \beta_2$ coefficients

$\gamma'$ submerged unit weight of the ground

$\gamma_d$ dry unit weight of the ground

$\gamma_w$ unit weight of water

$\delta_l$ coefficient

$\delta_b$ coefficient

$\Delta \sigma_{bDR}$ drainage-induced change of effective radial stress (at $r = b$)

$\varepsilon$ strain

$\varepsilon_r$ relative error

$\zeta$ auxiliary variable
\( \eta \) degree of drainage; \( \eta = (l-a)/(b-a) \)
\( \kappa_I \) loosening or dilatation factor of the injection body
\( \lambda \) degree of plastification; \( \lambda = (p-a)/(b-a) \)
\( \lambda_h \) hydraulic friction coefficient
\( \lambda_p \) coefficient of lateral stress in prism
\( \lambda_w \) coefficient of lateral stress in wedge
\( \nu \) Poisson’s ratio of the (untreated) ground
\( \nu_H \) Poisson’s ratio of the solid host rock
\( \nu_I \) Poisson’s ratio of the injection body
\( \xi \) normalized borehole length
\( \xi_k \) normalized coordinate
\( \rho \) radius of the plastic zone
\( \rho_{in} \) radius of the inner plastic zone
\( \rho_{out} \) extent of the outer plastic zone
\( \rho_w \) unit density of water
\( \sigma \) stress
\( \sigma'_{b} \) effective radial stress at the outer boundary of the grouting body (at \( r = b \))
\( \sigma'_{b,red} \) effective radial stress (at \( r = b_{red} \))
\( \sigma'_{b,DR} \) effective radial stress after drainage (at \( r = b \))
\( \overline{\sigma}_{b,DR} \) transformed drainage-induced effective radial stress (at \( r = b \))
\( \overline{\sigma}'_i \) transformed effective radial stress (at \( r = i \))
\( \sigma'_i \) transformed radial effective stress (at \( r = i \))
\( \sigma'_v \) effective radial stress at the boundary of ideal drainage (at \( r = l \))
\( \sigma'_{v} \) silo pressure acting upon the wedge
\( \sigma'_{vd} \) contribution of the ground above the water table to the silo pressure
\( \sigma'_{ve} \) contribution of the ground underneath the water table to the silo pressure
\( \sigma'_{vs} \) contribution of the seepage flow to the silo pressure
\( \sigma'_{vT} \) silo pressure at the elevation of the water table
\( \sigma_a \) radial stress at the excavation boundary (at \( r = a \)); lining support pressure
\( \sigma_b \) radial stress at the outer boundary of the grouting body (at \( r = b \))
\( \overline{\sigma}_i \) transformed radial stress (at \( r = i \))
\( \tau_m \) bond strength
\( \nu \) kinematic viscosity of water
\( \varphi \) angle of effective internal friction of the (untreated) ground
\( \varphi_H \) angle of effective internal friction of the solid host rock
\( \varphi_I \) angle of effective internal friction of the injection body
\( \chi \) ratio of idealized square tunnel cross-section side length to tunnel diameter
\( \psi \) angle of effective dilatancy of the untreated ground
\( \psi_H \) angle of effective dilatancy of the solid host rock
\( \psi_I \) angle of effective dilatancy of the injection body
\( \omega \) angle between face and inclined slip plane
\( \omega_{cr} \) critical angle \( \omega \)
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Figure 3.21. Results of a parametric study into the effect of ground cohesion $c$ in the problem of Figure 3.20

Figure 3.22. (a) Distribution of the hydraulic head $h$ along two lines above and ahead of the tunnel face for two vertical zones of different permeability (cases A and B of Fig. 3.20). Belonging surface plots of the hydraulic head field as well as required support pressure $s$ (b) without advance drainage measures and (c) with six axial drainage boreholes ($c = 50$ kPa, other parameters according to Fig. 3.4 and Table 3.1)

Figure 3.23. (a) Distribution of the hydraulic head $h$ along two lines above and ahead of the tunnel face when encountering a highly/poorly permeable fault zone. Belonging surface plots of the hydraulic head field as well as required support pressure $s$ (b) without advance drainage measures and (c) with six axial drainage boreholes ($c = 50$ kPa, other parameters according to Fig. 3.4 and Table 3.1)

Figure 3.24. (a) Distribution of the hydraulic head $h$ along two lines above and ahead of the tunnel face when entering in a fault zone of different permeability. Belonging surface plots of the hydraulic head field as well as required support pressure $s$ (b) without advance drainage measures; with six axial drainage boreholes of length (c) $l_d = 30$ m; (d) $l_d = 10$ m and (e) $l_d = 5$ m ($c = 50$ kPa, other parameters according to Fig. 3.4 and Table 3.1)

Figure 3.25. Required support pressure $s$ as a function of the vertical layer thickness $d_i$ in ground without any cohesion $c$ (homogeneous permeability; no drainage measure; other parameters according to Fig. 3.4 and Table 3.1)
Figure 3.26. (a) Parametric study of a single vertical layer of variable thickness. (b) Required support pressure \( p_0 \) as a function of the layer thickness \( d \) in ground without any cohesion \( c \) (permeability contrast \( k_{f}/k = 0.01-100 \); blue: no drainage measure; red: six axial boreholes according to Fig. 3.4; other parameters according to Table 3.1)

Figure 3.27. Results of a parametric study into the effect of ground cohesion \( c \) in the problem of Figure 3.26

Figure 3.28. (a) Longitudinal section of a tunnel crossing 5 m thick vertical layers of permeability contrast \( k_{f}/k = 100 \). (b) Required support pressure \( s \) as a function of the position of the tunnel face \( x_f \) in ground of cohesion \( c = 50 \) kPa (blue: no drainage measure; red: six axial boreholes according to Fig. 3.4; other parameters according to Table 3.1)

Figure 3.29. (a) Multiple vertical layers and (b) required support pressure \( s \) as a function of the layer thickness \( d \) in ground without any cohesion \( c \) (black: discretely modelled layers of permeability contrast \( k_{f}/k = 0.01 \) and 100; red: equivalent anisotropic model; other parameters according to Table 3.1)

Figure 3.30. (a) Permeability anisotropy of the equivalent homogeneous model representing very thin vertical layers. (b) Required support pressure \( s \) as a function of the permeability ratio \( k_{f}/k \) and the degree of anisotropy \( k_{f}/k_{a} \) at logarithmic scale in ground without any cohesion \( c \) and (c) belonging distribution of the hydraulic head \( h \) along two lines above and ahead of the tunnel face (blue: no drainage measure; red: six axial boreholes according to Fig. 3.4; other parameters according to Table 3.1)

Figure 3.31. Results of a parametric study into the effect of ground cohesion \( c \) in the problem of Figure 3.30

Figure 4.1. (a) Pressurized flow from a borehole during construction of Olafsfjödur street tunnel, Island (courtesy of Karl Gunnar Holter) and outflow of drainage borehole (Lake Mead Intake No. 3) of (b) discharge of about 0.25 m\(^3\)/h and (c) dripping only

Figure 4.2. Pipe flow

Figure 4.3. Open-channel flow

Figure 4.4. (a) Average water pressure \( p \) in a borehole cross-section of diameter \( d_0 \) at fill-level \( t_0 \); transition pressure \( p_{tr} \) at \( t_{f}/d_{f} = 1 \). (b) Open-channel discharge \( Q_{p,0} \) from a single borehole normalized by maximum discharge \( Q_{p,0} \) as a function of the relative fill-level \( t_{f}/d_{f} \). (c) Equivalent permeability \( K_e \) on logarithmic scale as a function of the hydraulic gradient \( I \) for several average pressures \( p \) (see Eqs. (4-9), (4-10) with \( d_0 = 0.1 \) m, \( p_{tr} = 500 \) Pa, \( k_{e,eq} = 5 \) mm, \( v = 1.307 \cdot 10\text{ }^6 \) m\(^2\)/s)

Figure 4.5. (a) Spatial discretization of borehole and surrounding ground. Seeage flow velocity vectors inside the borehole for transverse permeability (b) equal to \( K_b \) and (c) equal to 100 \( K_b \). Hydraulic head field for transverse permeability (d) equal to \( K_x \) and (c) equal to 100 \( K_x \)

Figure 4.6. Problem setup for the comparative analysis of a single drainage borehole

Figure 4.7. Distribution of, (a) pressure \( p_b \), (b) axial velocity \( v_x \) and, (c), transverse velocities \( v_y \), \( v_z \) along the borehole axis \( x \) \( (k_{e,eq} = 5 \text{ mm}, h_0 = 100 \text{ m}, K_x = 2 \cdot 10^{-7} \text{ m/s}) \) other parameters according to Table 4.1

Figure 4.8. Top row: Sketches of flow condition. (a) to (c): Pressure distribution \( p \) along the borehole axis \( x \); (d) to (f): Normalized pressure distribution in \( y \)-direction for several ground permeabilities \( K_x \) \( (k_{e,eq} = 5 \text{ mm}, p_0 = 1000 \text{ kPa}, other \text{ parameters according to Table 4.1}) \)

Figure 4.9. (a) Pressure \( p \) normalized by the initial pressure \( p_0 \) along the borehole axis \( x \) and (b) hydraulic head field for variable ground conductivity \( K_y \) as well as the belonging hydraulic head fields \( (k_{e,eq} = 5 \text{ mm}, h_0 = 100 \text{ m}, other \text{ parameters according to Table 4.1}) \)

Figure 4.10. Pressure \( p \) normalized by the initial pressure \( p_0 \) along the borehole axis \( x \) for variable initial hydraulic head \( h_0 \) in (a) highly permeable and (b) medium permeable ground with \( p/p_0 = 1 \) indicating the transition point from open-channel to pipe flow \( (k_{e,eq} = 5 \text{ mm}, other \text{ parameters according to Table 4.1}) \)

Figure 4.11. Pressure \( p \) normalized by the initial pressure \( p_0 \) along the borehole axis \( x \) for variable surface roughness \( k_{s,eq} \) of the borehole in (a) highly permeable and (b) medium permeable ground \( (h_0 = 100 \text{ m}, other \text{ parameters according to Table 4.1}) \)

Figure 4.12. Discharge \( Q \) from a single borehole as a function of the permeability of the ground \( K_x \) for variable borehole surface roughness \( k_{s,eq} \) \( (h_0 = 100 \text{ m}, other \text{ parameters according to Table 4.1}) \)
Figure 4.13. Simplified borehole problem: (a) radial inflow into a borehole of radius $r$ from ground with permeability $k_g$ (b) conservation of mass at a circular borehole with diameter $d_b$, and infinitesimal length $dx$ when considering radial inflow $q_r$ at its shell surface

Figure 4.14. Simplified borehole problem: Normalized hydraulic head $\hat{h}$ as a function of the normalized borehole length $\xi$ for several values $\alpha_i$

Figure 4.15. Simplified borehole problem: Comparison of the numerical and the simplified solution in terms of (a,b) the normalized hydraulic head $\hat{h}$ and (c,d) the normalized flow velocity $\hat{q}_r$ as a function of the normalized borehole length $\xi$

Figure 4.16. Simplified borehole problem: Comparison of the FEM and the numerical solution in terms of the normalized hydraulic head $\hat{h}$ as a function of the normalized borehole length $\xi$ for several ground permeability $K_g$ (one single borehole with $h_0 = 100$ m, $l_b = 30$ m, $d_b = 0.1$ m, $k_{s,eq} = 5$ mm)

Figure 4.17. Problem setup for the comparative analysis of the tunnel example

Figure 4.18. Face stability and water discharge of the tunnel example ($k_{s,eq} = 5$ mm, other parameters according to Table 4.2): (a) required support pressure $s$ as a function of failure angle $\omega$ and (b) as a function of the permeability of the ground $K_g$, (c) water discharge $Q$ as a function of the permeability of the ground $K_g$ on a logarithmic scale

Figure 4.19. Required support pressure $s$ normalized by the support pressure $s_{norm}$ (according to the nomograms when assuming sufficient capacity, i.e. boresoles under atmospheric pressure) as a function of the permeability of the ground $K_g$ for (a) variable surface roughness $k_{s,eq}$ of the borehole, (b) ground cohesion $c$, (c) elevation in water table $H_w$ and (d) borehole length $l_b$ (other parameters according to Table 4.2)

Figure 4.20. Simplified borehole problem: (a) ideal drainage up to a length $l_b$ ahead of the tunnel of radius $a$ and assumption of the same discharge $Q$ being collected via (b) $n$ drainage boreholes

Figure 4.21. Admissible permeability of the ground $K_g$ as a function of the admissible hydraulic gradient $i_{adm}$ providing sufficient capacity in the drainage boreholes (drainage cases see Table 4.3)

Figure 4.22. Exemplary screens of drainage casings: (a) perforated, (b) slotted (courtesy of Baosu Pipe), (c) oblong slotted (courtesy of Pancera Tubi e Filtri S.r.l.), (d) Failure of the screw thread at the joint of the segmental casings

Figure 4.23. (a) Screens of the considered drainage casings. (b) Pressure $p$ normalized by the initial pressure $p_0$ along the borehole axis $x$ for the single cased borehole. (c) Required support pressure $s$ as a function of the ground cohesion $c$ for the tunnel example (parameters according to Table 4.4)

Figure 5.1. Sketches of the construction process of tunnel excavation (No. 1) and subsequent drilling works of drainage boreholes (No. 2) when approaching a weak rock requiring drainage measures for face stability

Figure 5.2. (a) Problem setup with drainage schemes considered of (a) no advance drainage, (b) ideal advance drainage by means of complete pore pressure relief in the ground ahead of the tunnel face and (c) advance drainage by means of axial boreholes from the face

Figure 5.3. Time-dependent pore pressure relief when considering ideal advance drainage: (a) Normalized hydraulic head distribution $h/h_0$ above the tunnel face starting at initial condition ($t = 0$) at several time steps up to stationary conditions ($t = \infty$; $K_g/S_s = 0.1$) and (b) degree of pore pressure relief $M_{eff}$ as a function of the time $t$ on a logarithmic scale for several ratios of permeability to specific storage $K_g/S_s$

Figure 5.4. Degree of pore pressure relief $M_{eff}$ as a function of the dimensionless time-factor $tK_g(S_sD^2)$ on a logarithmic scale (drainage schemes see Fig. 5.2c)

Figure 5.5. (a) Tunnel example, (b-d) hydraulic head field and (e) required face support pressure $s$ as a function of the ground cohesion $c$ for selected degree of pore pressure relief $M_{eff}$ (other parameter according to Table 5.1)

Figure 5.6. Time-factor $tK_g(S_sD^2)$ as a function of normalized distance $T/D$ when considering $n = 2$, 6 and 12 axial drainage boreholes (Fig. 5.2c) for a degree of pore pressure relief $M_{eff} = 95$ (a) and 90% (b)

Figure 5.7. Problem setup for the comparative analysis of the tunnel example

Figure 5.8. Relative groundwater drawdown $\Delta H_i/H_i$ as a function of the initial water table $H_i/D$ when considering ideal drainage areas of variable length $l_b$ (parameters according to Table 5.4)
Figure 5.9. Relative groundwater drawdown $\Delta H_w/H_w$ as a function of the initial water table $H_w/D$ when considering several axial advance drainage borehole schemes ($n = 2, 6$ and 12 bores at location of Fig. 5.2c; other parameters according to Table 5.4)

Figure 5.10. Pore pressure distribution $p$ along a vertical line at the location of maximum groundwater drawdown $\Delta H_w$ showing the pore pressure relief $\Delta p$ (highlighted in grey) between initial conditions (dashed line) and the pressure prevailing after drainage (solid line)

Figure 5.11. Ground settlement $u_S$ multiplied by the constrained modulus $E_S$ as a function of the normalized initial water table $H_w/D$ when considering several axial advance drainage borehole arrangements ($n = 2, 6$ and 12 bores at location of Fig. 5.2c; other parameters according to Table 5.4)

Figure 5.12. Water discharge $Q$ divided by the permeability of the ground $K_y$ (on a logarithmic scale) as a function of normalized initial water table $H_w/D$ when considering several axial advance drainage borehole arrangements ($n = 2, 6$ and 12 bores at location of Fig. 5.2c; other parameters according to Table 5.4)

Figure 6.1. Hydraulic head field when considering (a) the grouting body without drainage, (b) the ideally drained grouting body (c), radial drainage of the inner part of a grouting body and (d) coaxial drainage of the grouting body

Figure 6.2. Computational model of tunnel excavation in grouting bodies when considering ideal drainage up to radius $r = l$ in an infinite long fault zone: (a) cross section, (b) longitudinal section

Figure 6.3. Schematic sketch of pore pressure distribution $p$ in grouting bodies when considering (a) uniform permeability, (b) an increased permeability due to plastification and, (c) a decreased permeability due to stress redistribution

Figure 6.4. Loading $\sigma$ and pore water pressure distribution $p$ for the case of (a) ideal drainage after grouting and (b) ideal drainage in advance of grouting

Figure 6.5. Loading $\sigma$ and pore water pressure distribution $p$ for the case of drainage after grouting: (a) grouting body without drainage, (b) ideally drained grouting body and (c) ideally drained inner part of the grouting body (a and b after Anagnostou and Kovári, 2003)

Figure 6.6. Considered system for ideal drainage of the inner part of the grouting body (a and b after Anagnostou and Kovári, 2003)

Figure 6.7. Problem layout of the tunnel example

Figure 6.8. Characteristic lines for several degrees $\eta$ of drainage after grouting (parameters see Fig. 6.7)

Figure 6.9. Support pressure $\sigma_a$ as a function of the degree $\eta$ of drainage after grouting and for several degrees of plastification $\lambda$ (parameters see Fig. 6.7)

Figure 6.10. Required uniaxial compressive strength $f_{cI}$ as a function of the support pressure $\sigma_a$ for several degrees $\eta$ of drainage after grouting (parameters see Fig. 6.7)

Figure 6.11. Required lining support pressure $\sigma_a$ as a function of the degree of plastification $\lambda$ for the parametric study of Table 6.1

Figure 6.12. Inflow $Q$ (normalized with the inflow when considering untreated ground $Q_{0u}$) as a function of the degree $\eta$ of drainage after grouting when considering several permeability ratios $k_I/k$ (other parameters see Fig. 6.7)

Figure 6.13. Comparison of the characteristic lines of drainage in advance of grouting to drainage after grouting (parameters see Fig. 6.7)

Figure 6.14. Displacement of the inner ($u_i$) and outer ($u_o$) boundary of the grouting body as a function of the degree of plastification $\lambda$ for drainage in advance and after grouting (parameters see Fig. 6.7)

Figure 6.15. Drainage layouts considered for drainage in advance of grouting: (a) coaxial drainage boreholes evenly spaced along the circle line $r = l$ (“layout A”), (b) lateral arrangement of coaxial drainage boreholes (“layout B”) and (c) ideal drainage of the area $l \geq b$

Figure 6.16. Drainage layouts considered for drainage after grouting: (a) ideal drainage of area $l \leq b$, (b) radial drainage slits considered in the two-dimensional model (plane strain) and (c) radial drainage boreholes considered in the three-dimensional model

Figure 6.17. Computational model for the numerical analyses

Figure 6.18. Computational steps of the numerical model simulating drainage in advance of grouting and subsequent excavation of the tunnel
Figure 6.19. Computational steps of the numerical model simulating drainage after grouting and subsequent excavation of the tunnel

Figure 6.20. Comparison of the results of the numerical model (markers) to the analytical solution (solid lines) by means of (a,c) the characteristic line and (b,d) the support pressure $\sigma_l$ as a function of the degree of plastification $\lambda$ (ideal drainage layouts of Figs. 6.15c and 6.16a; parameters see Fig. 6.7)

Figure 6.21. (a) Hydraulic head fields and development of plastic zones for decreasing support pressure for ideal drainage after grouting: (b) single plastic zone for $\sigma_a = 1.2$ MPa, (c) second plastic zone for $\sigma_a = 0.45$ MPa and (d) $\sigma_a = 0.44$ MPa (extent of plastic zones according to the analytical model indicated with green arrows; parameters see Fig. 6.7)

Figure 6.22. Hydraulic head fields for several number $n$ of drainage slits installed after grouting ($\eta = 0.8$; parameters see Fig. 6.7)

Figure 6.23. Average pore pressure $p$ in the grouting body normalized with the initial pressure $p_0$ as a function of the number $n$ (or the sector angle $\alpha$) of drainage slits installed after grouting ($\eta = 0.8$; parameters see Fig. 6.7)

Figure 6.24. (a) Hydraulic head fields and development of plastic zones for decreasing support pressure when considering drainage slits after grouting: (b) inner plastic zone for $\sigma_a = 0.8$ MPa, (c) additional plastic spots for $\sigma_a = 0.58$ MPa and (d) plastic “bridges” for $\sigma_a = 0.57$ MPa ($\eta = 0.8$; parameters see Fig. 6.7)

Figure 6.25. Required lining support pressure $\sigma_l$ as a function of the degree of plastification $\lambda$ for drainage after grouting considering several numbers $n$ of drainage slits, three degrees of drainage $\eta$ and two sizes of grouting body $b/a$ ($b = 12.5$ and 10 m; drainage layout see Fig. 6.16b; other parameters see Fig. 6.7)

Figure 6.26. Inflow $Q$ collected from all boundaries of excavation and drainage (normalized with the inflow when considering untreated ground $Q_0$) as a function of the degree $\eta$ of drainage after grouting (drainage layout see Fig. 6.16b; parameters see Fig. 6.7)

Figure 6.27. Hydraulic head field for 12 drainage boreholes drilled after grouting for several axial spacings $e$ (drainage layout of Fig. 6.16c; other parameters see Fig. 6.7)

Figure 6.28. Required lining support pressure $\sigma_l$ as a function of the degree of plastification $\lambda$ for drainage after grouting. Drainage borehole arrangement of Figure 6.16c with variable axial spacing $e$ for two degree of drainage $\eta$ (parameters see Fig. 6.7)

Figure 6.29. Characteristic lines when considering several different drainage borehole arrangements after grouting (see insets; parameters see Fig. 6.7)

Figure 6.30. Inflow $Q$ collected from all boundaries of excavation and drainage (normalized with the inflow when considering untreated ground $Q_0$) as a function of the degree $\eta$ of drainage after grouting for drainage layout of Figure 6.16c with variable axial spacing $e$ (parameters see Fig. 6.7)

Figure 6.31. Hydraulic head fields (l.h.s.) and plastic zones for a support pressure of $\sigma_a = 0.4$ MPa (r.h.s.) when considering several numbers $n$ of drainage boreholes drilled in advance of grouting (extent of plastic zones according to the analytical model indicated with green arrows; drainage layout A of Fig. 6.15a; parameters see Fig. 6.7)

Figure 6.32. (a) Water pressure acting on the grouting body $p$ (normalized with the initial pressure $p_0$) and (b) inflow $Q$ collected from all boundaries of excavation and drainage (normalized with the inflow $Q_0$ which considers ideal drainage up to $r = l$) as a function of the number $n$ (or the sector angle $\alpha$) of drainage boreholes drilled in advance of grouting (drainage layout A; parameters see Fig. 6.7)

Figure 6.33. Characteristic lines for several drainage borehole numbers $n$ drilled in advance of grouting (drainage layout A; parameters see Fig. 6.7)

Figure 6.34. (a) Characteristic lines for several circle lines $l$ when considering drainage in advance of grouting (layout A of Fig. 6.15a with $n = 8$; other parameters see Fig. 6.7)

Figure 6.35. Recommended circle line $l$ as a function of the size of the grouting body $b$ (normalized with tunnel radius $a$) for drainage via borehole number $n$ in advance of grouting (drainage layout A)

Figure 6.36. Required lining support pressure $\sigma_l$ as a function of the degree of plastification $\lambda$ for drainage in advance of grouting when considering three sizes of grouting bodies $b/a$ ($b = 10, 12.5$ and 15 m; drainage layout A of Fig. 6.15a; other parameters see Fig. 6.7)
Figure 6.37. Hydraulic head field (l.h.s.) and plastic zone for a support pressure of $\sigma_a = 0.4$ MPa (r.h.s.) when (a) considering drainage in advance of grouting by means of borehole layout A and (b) layout B ($n = 8$; other parameters see Fig. 6.7)

Figure 6.38. Characteristic lines for drainage in advance of grouting by means of borehole layout A and B ($n = 8$; other parameters see insets and Fig. 6.7)

Figure 6.39. Required lining support pressure $\sigma_a$ as a function of the degree of plastification $\lambda$ for drainage in advance of grouting considering three sizes of grouting bodies $b/a$ ($b = 10$, 12.5 and 15 m; drainage layout B of Fig. 6.15b; other parameters see Fig. 6.7)

Figure 6.40. Characteristic lines for drainage in advance of grouting considering three sizes of grouting bodies $b/a$ ($b = 10$, 12.5 and 15 m; drainage layouts of Fig. 6.15; other parameters see Fig. 6.7)

Figure 6.41. Problem layout considering a fault zone of limited extent

Figure 6.42. (a) Comparison of characteristic lines for drainage in advance of grouting considering a fault of limited and a fault of unlimited extent. Displacement vectors (b) in a fault of limited extent and (c) in a fault of unlimited extent (drainage layout A of Fig. 6.15a; $k_H/k = 1$, other parameters see Fig. 6.41)

Figure 6.43. Problem layout when considering a fault zone of limited extent for drainage borehole layout A (l.h.s.) and layout B (r.h.s.)

Figure 6.44. Three permeability ratios of host rock to fault zone $k_H/k$ presented in three columns comprising: (a-c) overall hydraulic head field, (d-f) detail of hydraulic head, (g-i) plastic zones for a support pressure of $\sigma_a = 0.4$ MPa, and (j-l) characteristic lines when considering drainage in advance of grouting in a fault zone of limited extent (Fig. 6.43)
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