CHALMERS

# Soil Rock Interfaces: Problem Identification and Conceptualisation for Sealing Strategies

*Master of Science Thesis in the Master's Programme Infrastructure and Environmental Engineering*

LINN ÖDLUND ERIKSSON

Department of Civil and Environmental Engineering
*Division of GeoEngineering*
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CHALMERS UNIVERSITY OF TECHNOLOGY
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Department of Civil and Environmental Engineering
Division of GeoEngineering
Engineering Geology Research Group
Chalmers University of Technology
SE-412 96 Göteborg
Sweden
Telephone: + 46 (0)31-7721000

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# ABSTRACT

Hydraulic conductivity, susceptibility to internal erosion and groutability are properties of frictional material that are considered in this thesis to be of major importance to tunnelling at a soil rock interface. These material properties can be indicated at early project phases from the grain size distribution curve of the frictional material. Together with evaluation of hydraulic and mechanical behaviour of the hydrogeological system, such as aquifer diffusivity, consolidation properties of clay and presence of bedrock deformation zones, potential risks of a soil rock interface can be indicated. The overall objectives of this thesis are to make a problem identification of soil rock interface tunnelling and to create a conceptual model for sealing purpose. More specifically the objective is also to investigate and exemplify how the grain size distribution curve of frictional material can be used to indicate material properties of crucial importance to successful soil rock interface tunnelling. A literature review provides the theoretical background and basis for fulfilment of the objectives. A case study provides practical example and an opportunity for application of the suggested conceptual model. The case study shows that erosion and out-wash of fine particles has occurred at the studied soil rock interface excavations and has given large vertical deformation. Large inflows to excavation occurred at conditions of a more transmissive bedrock than expected in combination with insufficient excavation sealing. The case study and the literature review has not provided detailed insight into excavation sealing design and its application to a specific site, the literature review indicates that further research on soil grouting is valuable. Large inflow to soil rock interfaces can lead to problems with fulfilment of the regulatory inflow requirement and large gradients to the excavation can lead to erosion of frictional material and additionally make needed post-grouting measures difficult. Grain size distribution curves of the frictional material provide opportunity for a qualitative assessment of erosion susceptibility and groutability. Implementing a conceptual model in early can provide a structured way of handling information and can aid transparency and communication between stakeholders and project phases.

KEY WORDS: soil rock interface, sealing, conceptual model, initial suggestion, hydraulic conductivity, erosion susceptibility, groutability, sensitivity assessment

# Content

# Preface

I want to thank my supervisor and examiner Åsa Fransson for the possibility of performing this thesis and for her profound and committed guidance. Olle Olofsson and Bengt Åhlén at the Swedish Transport Administration have been involved in this thesis since its initiation and I am grateful for their contribution. I also want to thank Minna Karstunen, Jelka Dijkstra and Mats Olsson for the advice given within the field of geotechnics. I am greatly thankful for the opportunity of meeting professionals and interview them within the case study; this was an important contribution to the thesis.

Gothenburg, June 2014

Linn Ödlund Eriksson

## Notations

Roman letters

| | | |
|---|---|---|
| $A$ | [m$^2$] | Cross-sectional area |
| $b$ | [m] | Hydraulic fracture aperture |
| $b$ | [m] | Aquifer thickness, saturated thickness |
| $b_0$ | [m] | Fracture aperture or saturated thickness at distance |
| $b_1$ | [m] | Fracture aperture or saturated thickness locally |
| $b_{ekv}$ | [m] | Equivalent fracture aperture of frictional soil |
| $b_{min}$ | [m] | Designing hydraulic fracture aperture |
| $C$ | [-] | Dimensionless constant depending on pore shape |
| $C_H$ | [-] | Constant in Hazen´s equation |
| $C_u$ | [-] | Coefficient of uniformity |
| $c_v$ | [m$^2$/s] | Coefficient of consolidation, vertical direction |
| $D$ | [m$^2$/s] | Hydraulic diffusivity |
| $D_r$ | [-] | Relative density of soil |
| $D_{10}$ | [m] | Soil particle size at ten per cent passing rate |
| $D'_{15}$ | [m] | Particle size at fifteen per cent passing rate for coarse soil |
| $D_{15}$ | [m] | Particle size at fifteen per cent passing rate |
| d | [m] | Mean pore diameter and well depth |
| $d_{85}$ | [m] | Grout particle size at 85 per cent passing |
| $d'_{85}$ | [m] | Particle size at 85 per cent passing for fine soil |
| $d_{90}$ | [m] | Soil particle size at 90 per cent passing rate |
| $d_{95}$ | [m] | Grout particle size at 95 per cent passing rate |
| $e_v$ | [m] | Void aperture of fracture |
| $g$ | [m/s$^2$] | Gravitational acceleration constant |
| $H$ | [m] | Groundwater head above tunnel or underground space |
| $h$ | [m] | Groundwater head |
| $I_{dim}$ | [m] | Design penetration length of grout in fracture |
| $I_{max}$ | [m] | Theoretical maximum penetration length |
| $i$ | [m/m] | Hydraulic gradient |
| $K$ | [m/s] | Hydraulic conductivity |
| $k_i$ | [m$^2$] | Intrinsic permeability |
| $k_n$ | [Pa/m] | Fracture normal stiffness |
| $k_1$ | - | Empirical constant |
| $k_2$ | - | Empirical constant |
| $L$ | [m] | Length of specimen |
| $L_e$ | [m] | Average length of capillaries |
| $M$ | [N/m$^2$] | Compressibility modulus |
| $M_0$ | [N/m$^2$] | Compressibility modulus below pre-consolidation pressure |
| $M_L$ | [N/m$^2$] | Compressibility modulus above pre-consolidation pressure |
| $m_v$ | [m$^2$/N] | Coefficient of volume compressibility |

| | | |
|---|---|---|
| $n$ | [-] | Porosity |
| $p$ | [Pa] | Grouting pressure |
| $Q$ | [$m^3/s$] | Flow |
| $q$ | [$m^2/s$] | Flow per length unit |
| $R$ | [m] | Radius of influence at time t |
| $R_b$ | [m] | Equivalent radius of underground space |
| $R_0$ | [m] | Radius of influence |
| $r$ | [m] | Radial distance |
| $r_w$ | [m] | Radius of well |
| $r_1$ | [m] | Radius of local transmissivity and distance between pumping well and mirror well |
| $S$ | [-] | Storativity |
| $S_S$ | [$m^{-1}$] | Specific storage |
| $S_0$ | [-] | Specific grain surface |
| $s$ | [m] | Groundwater drawdown in analytical evaluation and and consolidation settlement in clay |
| $T$ | [$m^2/s$] | Transmissivity |
| $T_{eff}$ | [$m^2/s$] | Specific transmissivity |
| $T_G$ | [min] | Gel time for silica sol |
| $T_{local}$ | [$m^2/s$] | Local transmissivity |
| $t_0$ | [s] | Time where data plot line intercepts *log t*-axis |
| $t_G$ | [s] | Gel induction time for silica sol |
| $u$ | [Pa] | Pore water pressure |
| $V$ | [$m^3$] | Volume of water |

Greek letters

| | | |
|---|---|---|
| $\beta_s$ | [$m^2/N$] | Compressibility of soil skeleton |
| $\beta_w$ | [$m^2/N$] | Compressibility of water |
| $\gamma_w$ | [$N/m^3$] | Unit weight of water |
| $\sigma$ | [Pa] | Total stress |
| $\sigma'$ | [Pa] | Effective stress |
| $\sigma'_c$ | [Pa] | Pre-consolidation pressure |
| $\tau_0$ | [Pa] | Yield strength |
| $\rho$ | [$kg/m^3$] | Density |
| $\mu$ | [Pa×s] | Viscosity of water |
| $\mu_0$ | [Pa×s] | Initial viscosity of silica sol |

Abbreviations

| | |
|---|---|
| FC | Content of fines in soil |
| gw | Groundwater |
| NC | Normally consolidated |
| OC | Over consolidated |
| OCR | Overconsolidatio ratio |
| w/c | Water to cement ratio in concrete |

# 1 Introduction

There is at present an increasing need for underground infrastructure in Sweden and several tunnels are planned and built. These tunnels are commonly situated in densely built urban areas sensitive to groundwater lowering and the regulatory requirements on inleakage to tunnels are getting stricter. Fractured bedrock and fault zones are potential parts of a tunnel that will need large sealing efforts to fulfil the requirements, but also soil rock interfaces are potentially problematic concerning groundwater inflow. A soil rock interface is the part where a tunnel excavation gradually passes from soil to bedrock and where the final tunnel construction changes from a concrete construction into a bedrock tunnel. In the Gothenburg tunnel project Västlänken, soil rock interfaces are identified as very problematic from a hydrogeological point of view (Banverket, 2006a; Banverket 2006b). If ground conditions deviate from expected and the inflow requirement is exceeded, substantial delays and additional costs can follow.

The common division of work in Swedish tunnelling projects is that rock professionals construct the bedrock tunnel and geotechnical professionals construct the soil tunnel. This is the pattern also in research where a lot of knowledge is gained in rock engineering and geotechnical engineering but the complex interface between the soft soil and the stiff bedrock is not explicitly dealt with. The hydraulic properties differ significantly between typical Swedish rock and soil, but also between different soil types and rock qualities. There is a need for deeper understanding of soil rock interfaces and their characteristics in order to choose efficient treatment strategies.

Groundwater issues are crucial in underground projects and soil rock interfaces are potentially particularly problematic (Banverket, 2006a). Large groundwater inflows have arisen in Swedish underground projects where the construction cuts through both soil and rock (Werner *et al*, 2012). In the perspective of future stricter inflow requirements and complex underground projects in urban areas, there is a need for deeper understanding of sealing issues related to soil rock interfaces. Due to the heterogeneity of an interface the approach must be trans-disciplinary, covering conventionally separated fields of expertise such as geotechnics, hydrogeology and geology.

By assessing possible and likely ranges of geological conditions at a site in early project phases a basis for cost estimates and contractual risk allocation is obtained (Kadefors and Bröchner, 2008). It is also a basis for more adequately planned further investigation and following decision about technical design, which moreover is a proposed procedure in the observational method (Peck, 1969; Kvartsberg, 2013a). Such early evaluation of expected geological conditions, relevant to the technical application in question, of a site demands definition of essential processes and parameters and simplification of the complex reality to a suitable degree.

## 1.1 Purpose and objective

As the title indicates the purpose of this thesis is to identify problems in soil rock interface tunnelling as well as conceptualize soil rock interfaces for sealing application. There are thus two overall objectives of this thesis:

- To make a *problem identification* of what difficulties can arise, what technical solutions work well and less well in soil rock interface tunnelling and what other aspects than geology and engineering that might add to the difficulties.

- To create a *conceptual model* for groundwater impacts and sealing purpose, based on hydraulic and mechanical behaviour of materials in soil rock interfaces. A conceptual model is a set of assumptions used to describe a system for a given purpose and a framework developed by Olsson *et al* (1994). Creating a conceptual model includes identifying important processes, geometries and material properties.

This involves compiling theory and knowledge of relevance to understand and conceptualize the hydrogeological system of a soil rock interface. Theory of basic physical processes and how they are described, relevant material properties, conceptualisation of geology and groundwater flow forms a basis of understanding and a basis for the creation of a conceptual model. From an engineering point of view also typical subsurface investigation and evaluation, tunnelling and sealing of soil and bedrock is important. These engineering aspects are important since the conceptual model is derived for a given technical application describing relevant processes, parameters and assumption (Kvartsberg, 2013a; Gustafson, 2012; Olsson *et al*, 1994).

The conceptual model shall be possible to use as a basis for choosing investigation strategies in order to upgrade the model to site-specific conditions, as a basis for evaluation of difficulty or severity of tunnelling at the interface, as a basis for analytical evaluation and numerical modelling, and as a basis for choosing technical treatment and sealing strategy of the soil rock interface. This conceptual model and its applications can be further studied, refined and applied in projects. In this thesis some extra effort will be put on using the conceptual model for assessment of soil rock interface sensitivity in early project phases, as described in the introduction above.

A case study provides example of real tunnelling at soil rock interfaces and the conceptualisation made in this thesis can be compared to and applied to real cases. The case study involves interviews with different stakeholders who worked in the projects. Together with the literature review and the case study, a list of identified possible problems or problematic aspects can be presented. This can be valuable in further studies on soil rock interface sealing.

A third specific objective of this thesis is:

- To present an initial suggestion on how material properties of the lower aquifer can be assessed from grain size distribution curves. Such local material properties at the excavation location of focus in this thesis are erosion susceptibility, groutability and hydraulic conductivity. The hypothesis is that these properties can be assessed at early projects phases from the grain size distribution curve of the frictional material.

The theoretical background for this hypothesis is described throughout the literature review with special attention paid to erosion susceptibility and groutability of frictional material. Theory of evaluation of erosion susceptibility is based on dam construction, in which internal erosion of frictional material is a major concern. Within the case study a suggested approach to assessing these local material properties is given. However, grain size distribution curves from another location than the case projects are used since grain size distribution curves from the studied soil rock interfaces have not been obtained.

# 2 Methodology and report disposition

There are two methodological parts in this thesis: a literature review and a case study including interviews. The main part is the literature review and following conceptualisation whereas the case study provides a real word example and opportunity of exemplifying findings and suggestions based on the literature review. Groundwater issues, hydrogeology and sealing are believed to be of major importance and the literature review thus focuses on these areas. Of the same reason hydraulic, hydromechanical and mechanical properties of materials are considered to be the most relevant. Throughout the thesis there is a need for a trans-disciplinary approach covering the fields of hydrogeology, geotechnics and geology.

The report disposition and methodology parts are schematically presented in Figure 2-1, which also shows how the different parts contribute to fulfilment of the overall thesis objectives described in Chapter 1.1.

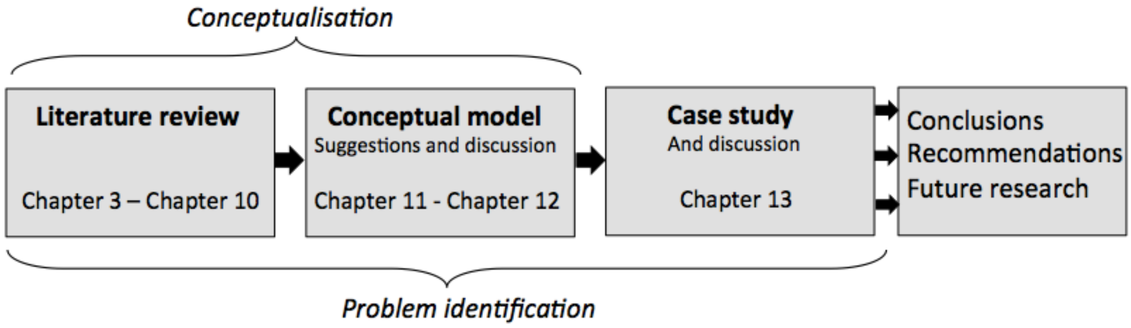


*Figure 2-1 Report disposition and how different parts contribute to the thesis objectives.*

The literature review is carried out in order to obtain basic as well as in-depth knowledge in subjects concerning soil rock interfaces. Such subjects are geological settings, material properties, characterisation, investigations, tunnelling and sealing technique. Also regulatory inflow requirement, forms of organisation and contracts, and the planning process and design of an underground project are included in the literature review. The regulatory inflow requirement to an underground governs the demand of sealing, and also contracting and organisation have great influence on underground projects and might be important in the problem identification. Links are made to the observational method according to Eurocode 7 since this design method is applicable and suitable to complex large-scale projects.

A main outcome of the literature review is the creation of a conceptual model as described above. This also forms basis for the case study in the sense that the literature review shall indicate what to be further studied in the case projects. Research work carried out at the division of GeoEngineering at Chalmers is to a relatively large extent the basis in the literature review. The conceptual model is in this thesis defined by a framework introduced by Olsson *et al* (1994), which is used to present and visualise the created conceptual model.

The case study focuses on two large Swedish tunnel projects: Götatunneln in Gothenburg and Citybanan in Stockholm. Götatunneln was finalised 2006 and Citybanan is presently being constructed.  Both tunnels are situated in urban areas sensitive to groundwater changes. The case study provides true examples of soil rock interfaces as well as comparison and exemplification of the created conceptual model.

The case study shall give deeper understanding and practical examples of how the soil rock interfaces were investigated, conceptualized and treated in these two projects. A comparison can be made to the conceptual model created from the literature review. The case study shall also try to answer how forms of contracts, organization and communication affected the final result of the soil rock interfaces. A main outcome of the case study is a problem identification concerning sealing of soil rock interfaces. Based on the problem identification, recommendations can also be given. Interviews with professionals within the case study can provide information to the problem identification and can also act as a basis for discussion about the conceptual model.

## 2.1 Delimitations

The literature review in this thesis is seen as a first, basic compilation of theory aspects considered relevant to soil rock interface tunnelling and groundwater impacts. The problem identification and conceptualization of soil rock interfaces is mainly dealing with the early phases of a project since soil rock interfaces are believed to be of greatest concern during construction; groundwater issues and sealing need to be captured within the pre-investigations and the early engineering geological prognoses of a project.

The compilation of experience from Swedish tunnelling projects is limited to a relatively small amount of projects and will instead focus on the two tunnels. Götatunneln is chosen since it is finalised and the results are known, it is also situated in Gothenburg where the tunnel Västlänken will be constructed in the coming years including ten soil rock interfaces (Banverket, 2006a). However, data collection from Götatunneln may be difficult due to the elapsed time. Citybanan is chosen since it is a large underground project, including several soil rock interfaces, and information and data might be easier to obtain since this tunnel is more recent. Figure 2-2 shows the location of the case projects.

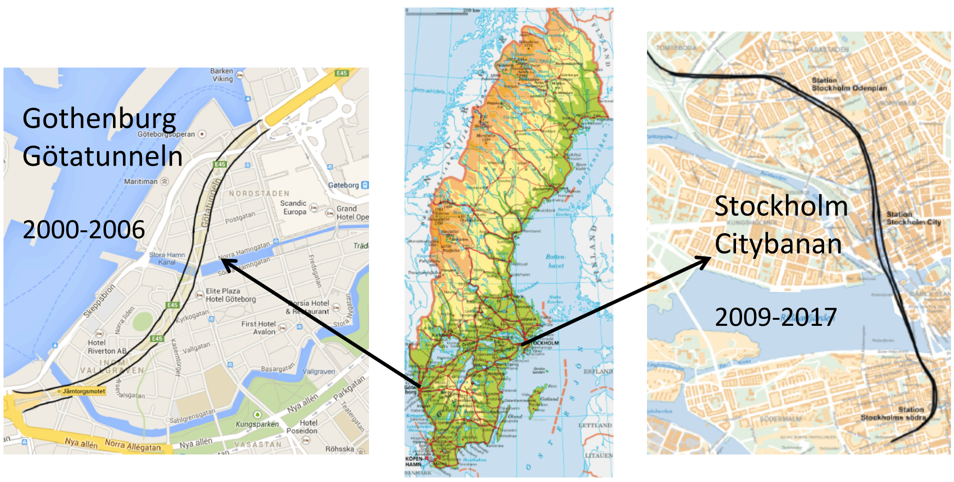


*Figure 2-2 Locations of the two tunnels studied in this thesis. Maps from Google Maps, NE.se and Swedish Transport Administration.*

# 3 Regulatory requirement for tunnel inflow

An underground construction situated below the water table will affect the surrounding hydrology since groundwater will leak into the underground space, changing flow patterns and draining the ground (Werner *et al*, 2012). In cases when the ground contains layers of clay, there is a particular risk for settlement damages on buildings and infrastructure due to ground drainage. If the groundwater lowering gives a stress increment and effective stress exceeding the pre-consolidation pressure of the clay consolidation will start, possibly giving large deformations. Clay creep rate is also increased and might give significant settlement in the long-term perspective (Persson, 2007). In the environmental perspective, ecosystems can be negatively affected if the groundwater flow pattern and access of a site are changed.

In order to avoid negative consequences, all waterworks with notable impact need permission from a Swedish Land and Environment Court. In a permit trial, the regulatory inflow requirement for every unique project is decided. The permit applicant suggests a tunnel inflow requirement based on a prognosis of the effects of the waterworks (Werner *et al*, 2012; Hansson *et al*, 2010). The applicant shall also provide the court with investigatory material and an Environmental Impact Assessment document. The final regulatory requirement is commonly stated as volume per time unit but can be formulated somewhat differently, either as inflow requirement for the whole tunnel or as different requirements in different parts of the tunnel depending on the sensitivity of the surrounding area (Hansson *et al*, 2010). A combination can also be used. The requirement could also be stated as a maximal allowed groundwater drawdown, or in combination with an inflow (Nilsson, personal communication). Since every underground project is unique, the formulation of requirement depends on the characteristics of the project (Johansson, personal communication). Inflow requirement are typically assigned only to bedrock tunnels since a final concrete tunnel in soil is considered water-tight (Hansson *et al*, 2010). Until present, the requirements have commonly been formulated as benchmarks but Hansson *et al* (2010) identify a trend towards limiting requirements in the coming years. Violation of the regulatory requirement is punishable.

The prognoses that suggested inflow requirement are founded upon are, according to Werner *et al* (2012), based on very different methods and varying investigatory material from project to project. Werner *et al* (2012) show that prognoses of effects from groundwater diversion from rock facilities are seldom based on conceptual models and that it is often unclear what part of the hydrogeological system is being modelled. The true inflow often turns out to be significantly different from what the prognoses indicate. If ground conditions turn out to deviate negatively form what is expected, the costs for sealing can comprise a large share of the total project cost. Issues regarding inflow and sealing are common causes of delay and unforeseen costs in tunnelling projects (Engström *et al*, 2009), the costs for sealing can sometimes be as high as half of the total production cost (Kadefors and Bröchner, 2008).

# 4 Phases in underground construction projects

Underground projects are by their nature often large-scale and complex. The Swedish Transport Administration (Trafikverket) is a major client (Kvartsberg, 2013b). Consultants perform the design of the project and contractors execute the actual construction. How tasks and responsibility is divided between these main stakeholders depends on the form of organisation and contracts in the project, which is further dealt with in Chapter 5. The main phases to distinguish in a project are the feasibility phase, the design and production planning phase, the construction phase and the operational phase. These phases and their content is schematically summarized and presented by Kvartsberg (2013b) in Figure 4-1.

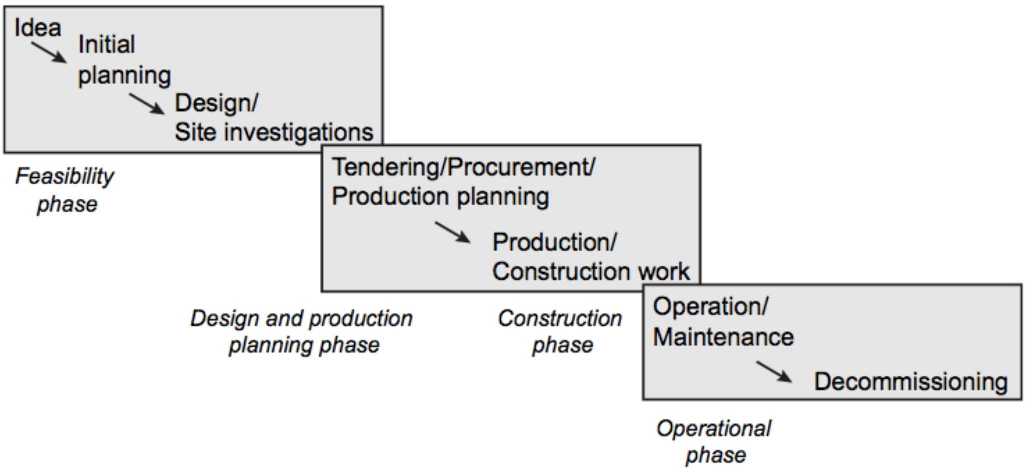


*Figure 4-1 Phases in an underground construction project, from Kvartsberg (2013b).*

In the first part of the 'Design and production planning phase' in Figure 4-1 there is an integrated cooperation between the client and the consultant and all documentation needed for tendering and procurement is prepared (Hansson *et al*, 2010). This work is also the basis for the suggestion of regulatory inflow requirement to the Land and Environment Court. Issues concerning technical aspects of measurement of water inflow needs to be dealt with at this stage so that such aspects are clear to contractors in the following procurement process. In order to increase the quality of the inflow measurements Hansson *et al* (2010) proposes that such technical aspects should be communicated early in the construction phase between the client and the contractor. The regulatory inflow requirement is followed up with a control programme during construction and the contractor and the client shall update the control programme continuously throughout the construction phase (Hansson *et al*, 2010).

In case the Swedish Transport Administration is the client, the feasibility phase in Figure 4-2 will follow the planning process at the Swedish Transport Administration. All projects are initiated by a need or demand and a decision of a new construction project will only be made if no other optimization or reconstruction options are suitable (Trafikverket). Then pre-study, investigation and planning follows to create all needed documentation for approval, tendering and procurement, schematically presented in Figure 4-2.

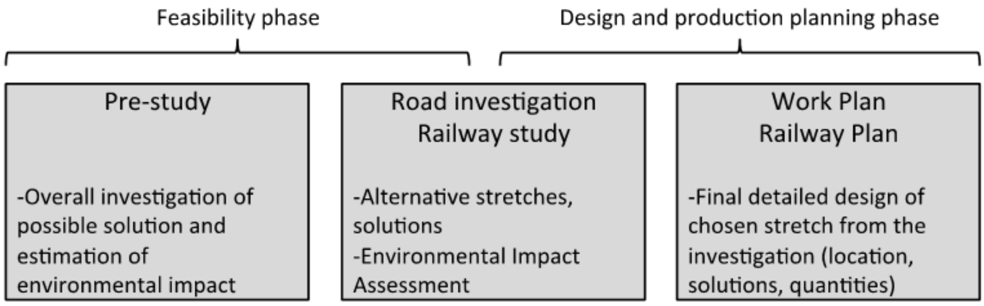


*Figure 4-2 The Swedish Transport Administration's planning process for large infrastructure road and railway projects (modified from Trafikverket).*

Since January 2013 there is no distinction of stages in the Swedish Transport Administration's planning process, the aim of this is to increase the efficiency (Trafikverket; Lindfors *et al*, 2014).

## 4.1 Design by the observational method

Eurocode 7 for geotechnical construction is the European standard applying to underground project. The design of large-scale underground projects, performed in the design and production planning phase described above, can thus be performed in accordance with this standard. Design methods proposed by Eurocode 7 are (EN-1997-1):

1. Design by calculations
2. Design by prescriptive measures
3. Load tests and tests on experimental models
4. The observational method

Traditionally in complex Swedish tunnelling projects a design approach called 'Active Design' has been used (Holmberg and Stille, 2007). This design approach has similarities to the observational method but the latter one provides a more formal basis and a stricter framework (Holmberg and Stille, 2007; Kvartsberg, 2013b). The observational method was originally proposed by Peck (1969) and the purpose of it is to build after actual demand in order to avoid insufficient design and particularly overly conservative design (Kvartsberg, 2013b; Zetterlund, 2009; Kadefors and Bröchner, 2008). There is at present an effort on implementing the observational method in Swedish tunnelling projects (Kadefors and Bröchner, 2008; Brantberger, 2009).

The observational method in Eurocode 7 can be used as design method in projects where ground conditions are difficult to predict (Holmberg and Stille, 2007). The method is suitable in large-scale underground projects in urban areas and it is particularly suitable in projects where there are large uncertainties in bedrock quality and sealing effort (Kadefors and Bröchner, 2008). Figure 4-3 from Kvartsberg (2013b) shows the different parts of the observational method, and also where in the project process the parts belong.

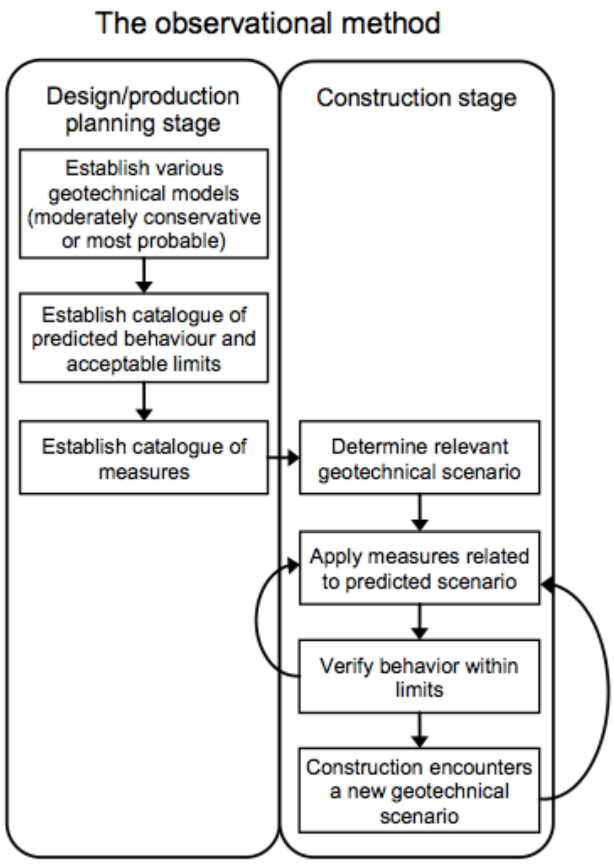


*Figure 4-3 The content of the Observation method, from Kvartsberg (2013b) modified from Schubert (2010).*

A successful application of the observational method needs a thorough and trans-disciplinary understanding of the geological system. This is due to that measures, or design-options, for different predictions of system behaviour are developed at an early stage in the project (Zetterlund, 2009). The most probable site conditions shall be the basis for the design, which demands high qualitatively planned, performed and interpreted pre-investigations. For less, but still relatively, probable situations design-options are prepared and it is also important to estimate the most unfavourable conditions (Zetterlund, 2009). Key-parameters are measured and monitored throughout construction and depending on these observations a suitable pre-determined design-option is applied. Such key-parameters to monitor needs to be determined in a delicate manner since it can be difficult to identify and chose relevant parameters (Gustafsson *et al*, 2010; Holmberg and Stille, 2007; Kvartsberg, 2013b).

# 5 Organization and forms of contracts in tunnelling

Underground construction projects inherit a large degree of uncertainty, simply due to the fact that the ground conditions are not completely known until actual excavation. The uncertainties are firstly to be found in geological and hydrogeological conditions at the site. There is also a large share of uncertainty connected to the needed amount and actual efficiency of performed sealing efforts; if the sealing achieved with for example pre-grouting is insufficient, post-grouting needs to be applied. It could also be that the proposed pre-grouting design is observed not to be sufficient and additional sealing is needed. Such additional work can give significant delays in the production cycle, particularly since the tunnel sealing is a critical part in the tunnel production cycle (Brantberger, 2009; Hansson, 2010). Furthermore, a lion's share of the costs in tunnelling projects is time dependent, which leads to substantially increased costs if the project is delayed (Hansson, 2010).

The contractual ways of handling uncertainties in Swedish underground projects are deficient (Hansson, 2010; Kadefors and Bröchner, 2008), especially when it comes to strict sealing requirements for tunnels (Brantberger, 2009). There is a need for greater flexibility and adjustment to actual conditions than the forms of contracts and organisation of today allow for. Concerning sealing of tunnels, the economic compensation also needs to be changed in order to fit the contractor's actual costs more adequately (Brantberger, 2009; Hansson, 2010). Flexibility and adjustment to actual conditions is fundamental in the observational method and using this design method is one way to handle uncertainties concerning tunnel sealing (Brantberger, 2009). If the observational method shall be used successfully as design method in tunnelling, the forms of contracts and organisation need to be adopted to suit the characteristics of this method (Kadefors and Bröchner, 2008). For example the allocation of responsibility and risk needs to be clearly stated as the design might be changed throughout the construction process (Brantberger, 2009).

Dividing the tunnel stretch into classes of different geological or hydrogeological conditions with following design is considered a suitable method for facilitating risk allocation and economic compensation (Holmberg and Stille, 2007; Kadefors and Bröchner, 2008; Brantberger, 2009). The classification, or systematic characterisation, shall be made on a site- and project specific basis founded upon geological settings of relevance for the technical application in question (Kvartsberg, 2013a). Empirical rock mass classification systems could be used as an aid at early phases but it shall be remembered that they were developed for tunnel stability and thus not applicable to for example sealing (Kvartsberg, 2013a; Palmström and Broch, 2006).

There is a direct link between the quality of pre-investigations and the correctness in prognoses (Hansson *et al*, 2010), and pre-investigations may need to be supplemented throughout the phases of planning and construction (e.g. Kvartsberg, 2013a). Still, there will be uncertainties in actual ground conditions, particularly in the early phases of the project. Consequently uncertainties need to be handled explicitly in contractual manner. In the case the observational method is used, Kadefors and Bröchner (2008) propose that the client shall have a budget span that embraces reasonable geological variations. In the case of tunnel sealing, i.e. grouting, Kvartsberg (2013a) suggests that the use of hydraulic domains (see Kvartsberg, 2013a and Rhén *et al*, 2003) as a basis for class division aids covering the range of geological conditions and thereby technical design.

Design-bid-build contract is the most common organizational form for underground construction projects (Brantberger, 2009). This means that the client is responsible for the design and the contractor is responsible for the construction and execution of the design. In a design and construct contract the contractor is responsible for both design and construction. Design and construct contracts have been used relatively often in projects where the Swedish Transport Administration is client (Riksrevisionen, 2012). Tender specifications are prepared by a consultant, and sometimes also the client, and commonly include a description of bedrock classes or rock prognoses (Brantberger, 2009; Hansson, 2010). This description is the basis for tendering and procurement. However, the actual conditions most often deviate from these initial prognoses due to limited amount of information on actual ground conditions (Hansson *et al*, 2010). The tender specifications can, particularly in more large-scale projects, include a description of risks or hazards identified in the early phases of the project (Brantberger, 2009). Knowledge from the phases of investigation and design is however often difficult to transmit to the construction of the projects since new parties, i.e. companies and personnel, take over (Bröchner *et al*, 2006).

Throughout the process of pre-investigation, design, tendering and construction, communication between different stakeholders is important. The understanding and handling of uncertainties in geological and hydrogeological conditions, both contractually and technically, needs to be communicated between stakeholders and between phases in the project. Bröchner *et al* (2006) states that workshops performed during early phases of a project provide an opportunity for increased communication, and thereby understanding.

Aspects concerning water inflow during the construction of a tunnel could be treated earlier in the project in order to increase the control of inflow measurements and thereby knowing more accurately whether the inflow requirement is fulfilled (Hansson *et al*, 2010). Figure 5-1 shows how different stakeholders are involved in the fulfilment of the regulatory inflow requirement.

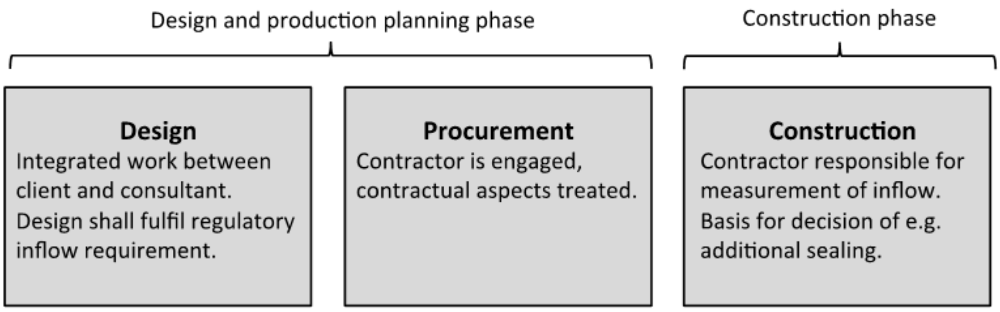


*Figure 5-1 Stakeholders' involvement in the fulfilment of the regulatory inflow requirement, based on Hansson* et al *(2010).*

Hansson *et al* (2010) suggest that technical questions concerning measurement shall be clearly stated in the documentation for tendering and procurement and that such technical aspects shall be treated by the client, consultant and contractor at the start of the construction. It is also suggested that updating of the measurement plan shall be made continuously throughout the construction.

# 6 Conceptualisation of geological and hydrogeological conditions

The geological and hydrogeological conditions are of crucial importance in an underground project and it is important to know soil stratigraphy, aquifers, water-conducting features of the bedrock and similar. The geology and hydrogeology of a site can however not be specifically deeply known without investigations and following interpretation. The amount of available information is particularly limited in the early phases of a project. Suitable investigations are typically costly and are for this reason often performed to a limited extent (Zetterlund, 2009). One valuable approach could then be to use the concept of hydrogeological type settings studied by Stejmar Eklund (2002) amongst others. By comprising already existing information into hydrogeological type settings, further investigations can be planned in order to verify or reject features of the initially assumed settings. This concept provides a systematic way of using available information and is further presented and described in Chapter 6.1 below; it is considered a valuable tool for estimation of soil stratigraphy and groundwater aquifers at a site. Groundwater in bedrock is strongly linked to rock fractures and deformation zones since the material else has a very low porosity. There may thus be a large hydraulic difference between intact rock and fractured rock and a conceptualisation scheme originally introduced by Caine *et al* (1996) is presented in Chapter 6.2.

This chapter thus provides some general features and concepts of the hydrogeological system since the focus of this thesis is on groundwater issues. The chapter also aims at showing how relatively much can be assumed about the ground conditions at a site already in early projects phases when little site-specific data is known. It can also help identify what features are particularly important to identify and verify for a certain purpose in the following site-investigation.

## 6.1 Soil profile and aquifers

Stejmar Eklund (2002) has developed hydrogeological type settings, defined by characteristic geological and hydrogeological conditions, which can be assigned to a site. By using the concept of type settings, there is an increased understanding of the site condition in the early phases of a project and information obtained subsequently can be used more efficiently. Type settings can also be used for identifying hydrogeological anomalies and to extrapolate information from a well-known area to an area with less information. The type settings comprises information mainly from geological and topographical maps but can also include earlier investigations, experiences and similar. Two generally applicable geological stratigraphies to Swedish conditions are identified and used as a basis in the development of the hydrogeological type settings in Stejmar Eklund (2002) and are shown in Figure 6-1 below.

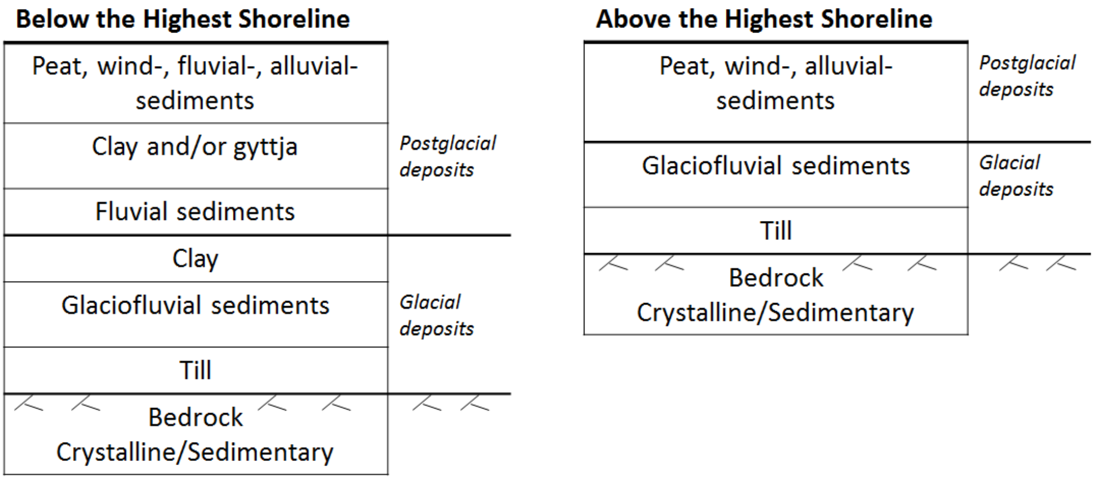


*Figure 6-1 Geological stratigraphies below and above the Highest Shoreline, modified from Stejmar Eklund (2002) based on Bengtsson and Gustafson (1996).*

The geological history at the site determines what layers in Figure 6-1 are represented and their relative thicknesses (Stejmar Eklund, 2002). As can be seen, the distinction is made between areas located above or below the Highest Shoreline; above the Highest Shoreline, glacial clay, postglacial clay and fluvial sediments are not present. All material layers can contain groundwater though their water conductive properties vary. Characteristics of the materials in Figure 6-1 are described in Chapter 8. Permeable unconsolidated material layers in the soil profile, typically fluvial or glaciofluvial sediments, constitute groundwater aquifers since the material contains significant amounts of water. Any clay layer above an aquifer, as in the soil profile below the Highest Shoreline, is significantly less permeable and constitutes a groundwater aquitard since water can seep through the clay but only very slowly an in small amounts (Carlsson and Gustafson, 1997).

A characteristic hydrogeological setting or feature below the Highest Shoreline is clay filled valleys or depressions between good quality bedrock plinths (Gustafson, 2012; Fredén *et al*, 2009; Persson, 2007), the bedrock often has water conductive fracture zones in the valleys (Gustafson, 2012). This geometry typically constitutes the location for a tunnel soil rock interface when a tunnel stretches through the soil in the valley into the rock. Between the bedrock and the clay there is commonly a layer of frictional material such as till or sand, providing a groundwater aquifer together with the uppermost part of the bedrock (Gustafsson, 2012). There is also an upper aquifer in soils above the clay, for example fill material in urban areas, or in the uppermost dry crust of the clay. This upper aquifer is thus separated from the aquifer below by the clay aquitard in the middle. A profile of the stratigraphy in a valley like this is shown in Figure 6-2.

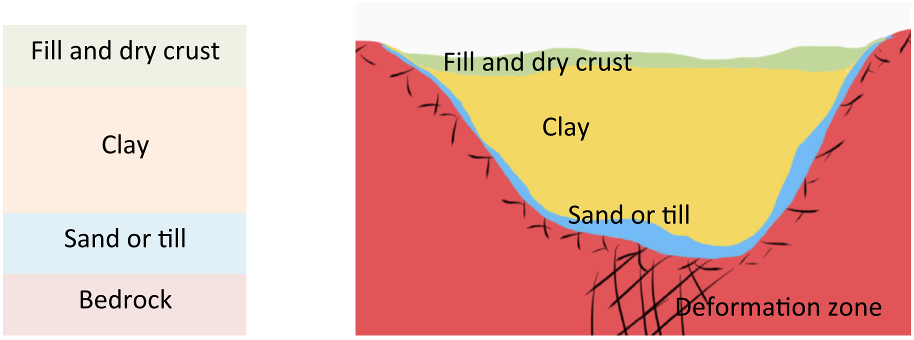


*Figure 6-2 Stratigraphy typical in clay-filled valleys in Gothenburg and Stockholm, modified and visualized from Gustafson (2012).*

The upper aquifer in the dry crust and fill material in Figure 6-2 is an unconfined aquifer with hydrostatic pressure distribution. The lower aquifer is a confined aquifer since it is covered by clay. An unconfined aquifer is recharged by infiltration of precipitation whereas a confined aquifer is recharged only from its sides where is outcrops (Carlsson and Gustafson, 1997).

Variation in presence and extent of the coarse material between the bedrock and the clay can for example affect the hydrogeological system significantly (Persson, 2007). Material layers, their thicknesses, groundwater recharge and topography are factors that are likely to differ between different sites. For example a valley with the stratigraphy in Figure 6-2 where the sand/till layer extends beyond the clay and up along the bedrock hill, experiences a large amount of groundwater recharge (Gustafson, 2012). In such cases, artesian pressure in the lower aquifer is common. Other characteristic hydrogeological settings of features in Swedish conditions are glacial eskers and deltas. The glaciofluvial material in the esker or the delta provides a groundwater aquifer and they can contain significant amounts of water (Fredén *et al*, 2009). The presence of highly water-conducting features affects the overall hydrological system at the site and can have major impact. The same applies to water-conducting fracture zones in bedrock as described in the following (Gustafson, 2012).

## 6.2 Bedrock and its permeability

Swedish bedrock is either crystalline or sedimentary, the crystalline being predominant since most sedimentary rocks have eroded away. The crystalline bedrock is pre-Cambrian, having been subjected to significant tectonic activity and large-scale deformation patterns are due to plate tectonics (Larsson and Tullborg, 1993; Söderbäck, 2008; Thörn, 2013). Due to tectonic events bedrock fractures at regional and local scale can be divided into fracture sets in which fractures have the same origin and direction (van der Pluijm and Marshak, 2004). It might be valuable to study the geological and geomorphological history of a site in order to understand and predict geometrical features of importance to an underground project (Lindblom, 2010; Kvartsberg, 2013a).

The latest glaciation has affected the top layer bedrock; loading, ice movement, melt water and low temperature, has exposed bedrock in valleys to fracturing, abrasive erosion and frost weathering (Lindblom, 2010; Gustafson, 2012). The bedrock is in general more fractured closer to the ground surface (Engström *et al*, 2009); this is due to unloading of former overburden and environmental impacts. Fractures close to the bedrock surface can be filled with permeable sediments from nearby soil layers, having entered the fractures by flowing groundwater (Carlsson and Christiansson, 2007). At depth, fractures tend to be more closed due to the compression from overburden rock and soil (Stephansson *et al*, 1991; Jiang *et al*, 2010).

The intact rock can be considered impermeable due to its low porosity and water is present in open, water-conducting fractures. A fault or deformation zone is a part of the bedrock with higher fracture intensity than the surrounding rock. The fault has a core, a central part that has been subjected to large deformation and around the core there is a damage zone of fractured rock (Gustafson, 2012). Caine *et al* (1996) have developed a conceptual scheme of fluid flow in a fault zone; the model is based on the relative extent of core and damaged zone. This conceptual model links structural regimes of a fault zone to corresponding hydrogeological regimes. The conceptualisation scheme is seen in the right half of Figure 6-3.

Depending on the size of the core it acts as a conduit or a barrier to flow. The damage zone around the core can be highly fractured and then act as a groundwater feeder to the less fractured host rock (Kvartsberg, 2013a; Hernqvist, 2009). The permeability contrast between host rock, damage zone and fault core can be significant and a water-conducting damage zone can have large impact on the local groundwater flow pattern. The conceptualisation of the permeability structure of a fault zone by Caine et al (1996) is extended by Fransson and Hernqvist (2010) with a description of the flow dimension and fracture pattern as seen to the left in Figure 6-3 below.

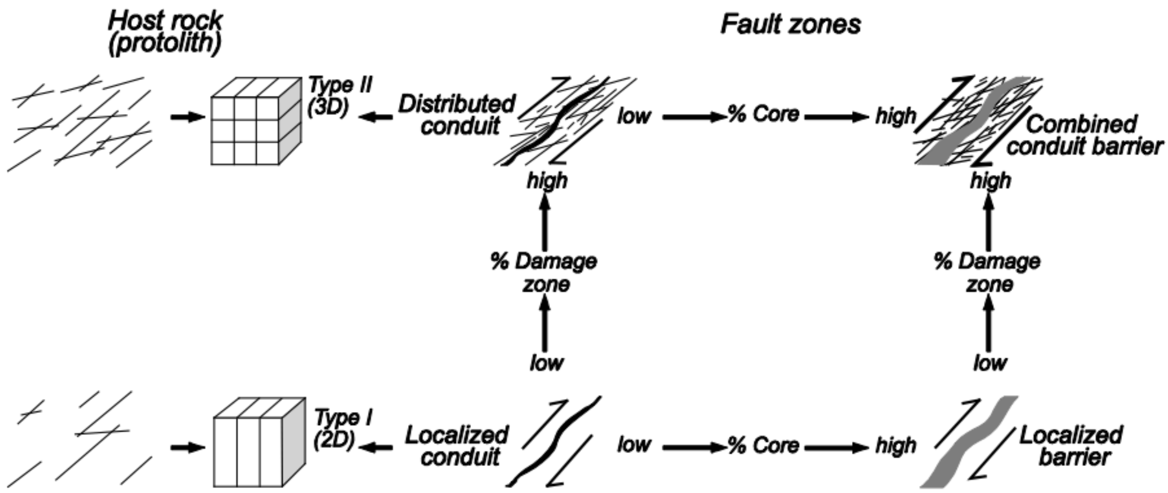


*Figure 6-3 To the right conceptualisation of fluid flow in fault zone by Caine et al (1996), extended with flow dimension by Fransson and Hernqvist (2010) to the left.*

In bedrock with few fractures and few fracture interconnections groundwater will flow in a one dimensional channel manner; in the fracture interconnections the water can flow radially in a two dimensional manner (Fransson and Hernqvist, 2010; Kvartsberg, 2013a). This is visualized in the leftmost bottom of Figure 6-3. A densely fractured rock, such as a fracture zone or bedrock close to ground surface, may instead experience a three-dimensional, spherical water flow due to the large interconnectedness in the fracture system (leftmost top of Figure 6-3). The conditions in the upper part of Figure 6-3, with a densely fractured rock and three-dimensional groundwater flow are the most likely to give problems with water inflow in Nordic underground projects (Zetterlund, 2009). Healed or filled fractures might not contribute to any water flow (Thörn, 2013; Kvartsberg, 2013a), and the permeability generally decreases with depth (Jiang *et al*, 2010; Rhén *et al*, 2008).

# 7 Theory of groundwater flow and definitions

The general features of soil profile, groundwater aquifers and bedrock permeability were presented in Chapter 6. This chapter presents theories and equations that are needed to quantitatively describe groundwater flow and hydraulic behaviour of materials. Basic concepts such as Darcy's law, hydraulic conductivity and hydraulic fracture aperture govern groundwater flow and are described in the following. Transmissivity and storativity are aquifer properties which together govern the diffusivity of the aquifer; the diffusivity is considered important in tunnelling at a soil rock interface since it determines the rate of pressure changes in the aquifer, for example due to inleakage to excavations or tunnels.

Since consolidation settlement in clay is one of the major risks of groundwater lowering, also deformation properties of clay are considered. The coefficient of consolidation determines the rate of consolidation settlement in clay. Quantifying the time dependency of pressure changes in the aquifer and consolidation could thus potentially provide a quantification of overall system sensitivity to groundwater changes or when countermeasures need to be undertaken (Persson, 2007).

In Persson (2007) settlement due to tunnelling was studied and internal erosion of frictional material was identified as a cause of significant vertical deformation. Such erosion is also considered a risk in the Gothenburg tunnel Västlänken (Banverket, 2006a). The phenomenon of internal erosion of frictional material is treated in this thesis based on literature from the area of dam construction and stability in which internal erosion is a major concern.

Persson (2007) illuminates in her work the direct link between geotechnical and hydrogeological terms. In this thesis also hydraulic and hydromechanical behaviour of bedrock fractures is considered due to the nature of a soil rock interface, containing both soil and bedrock. This chapter focuses particularly on the phenomenon and description of internal erosion since this is traditionally not explicitly dealt with within large-scale road and railway infrastructure projects.

## 7.1 Darcy's law

Groundwater flow obeys the natural laws of conservation of mass and conservation of energy. Groundwater flow is described by the theory of linear flow, i.e. Darcy's law. Darcy's law (Darcy, 1856) describes fluid flow through a porous medium as seen in for example Fetter (2001) and de Marsily (1986):

$$Q = -K \times A \times \frac{dh}{dl} \quad [\mathrm{m^3/s}] \qquad \text{Equation 1}$$

Where $K$ is the hydraulic conductivity of the medium, $A$ is the cross-sectional area subjected to flow and *dh/dl* is the hydraulic gradient, i.e. the change in groundwater head per unit distance. The hydraulic gradient represents a difference in pressure, i.e. energy. Darcy's law is valid for laminar flow and constitutes a linear relationship between flow velocity and hydraulic gradient. Turbulent flow is only likely to arise in coarse-grained soils (Persson, 2007; Alabi, 2011). For soils finer than gravel, Darcy's law is considered to be valid for gradients below 0.07 (Persson, 2007). There are several suggestions from researches that there is a non-linear relationship for very low gradients and there is a threshold value of hydraulic gradient below that no flow occurs (Alabi, 2011). Especially soils containing clay show this non-linear behaviour.

## 7.3 Hydraulic conductivity

The hydraulic conductivity $K$ quantifies the ability of a soil to transmit water. Hydraulic conductivity has the unit m/s and is given as, see for example Fetter (2001):

$$K = \frac{k_i \times \rho \times g}{\mu} \quad [\mathrm{m/s}] \qquad \text{Equation 2}$$

$\rho$ and $\mu$ are the density and the viscosity of the fluid, $g$ the gravitational constant and $k_i$ the intrinsic permeability of the porous medium. Both density and viscosity are temperature dependent. The intrinsic permeability represents the properties of the medium itself and is described as:

$$k_i = C \times d^2 \quad [\mathrm{m}^2] \qquad \text{Equation 3}$$

Where d is the mean pore diameter and $C$ is a dimensionless constant depending on the shape of the pore space (Fetter, 2001). The permeability in a material and thereby the hydraulic conductivity is usually lognormally distributed throughout the material (de Marsily, 1986; Persson, 2007). For radial, two-dimensional flow the average value of a lognormally distributed permeability is given by the geometric mean (de Marsily, 1986). For uniform flow the average is between the harmonic mean and the arithmetic mean. In the case of one-dimensional flow trough layers of different hydraulic conductivity, the harmonic mean applies to flow perpendicular to the layering and the arithmetic flow applies to flow parallel to it (Fetter, 2001; de Marsily, 1986).

To obtain the mean pore diameter in Equation 3 or other measures of particle sizes the material is sampled and sieved in laboratory. A grain size distribution curve is obtained and presented in a diagram showing percentage of mass passing a sieve to the logarithm of particle diameter corresponding to openings of the sieve. Three examples of grain size distribution curves are given in Figure 7-1.

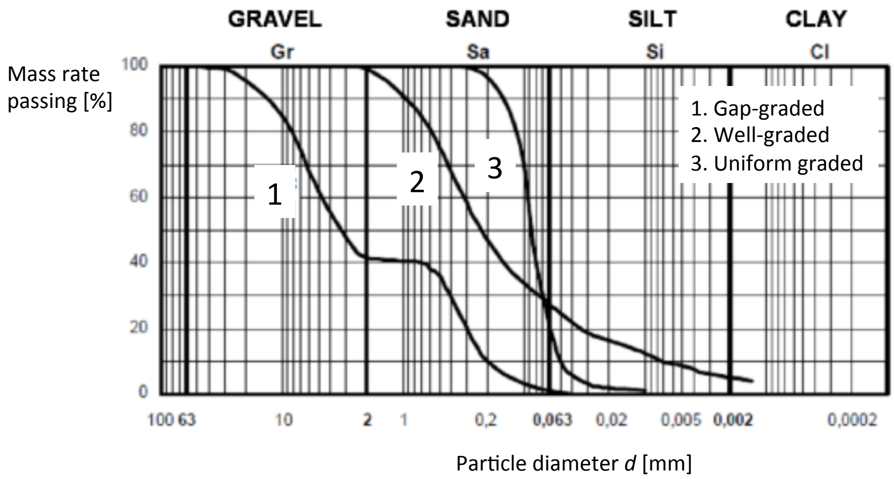


*Figure 7-1 Examples of the grain size distributions of three different soils. Picture modified from tankonyvtar.hu.*

Measuring the hydraulic conductivity of a soil material can be difficult (Svensson, 2014). Commonly, empirical relationships to grain size such as Hazen's and Gustafson's formulas are used. Hazen's formula (Carrier, 2003; Fetter, 2001):

$$K = C_H \times d_{10}^2 \quad [\mathrm{m/s}] \qquad \text{Equation 4}$$

Where $C_H$ is an empirical constant and $d_{10}$ is the particles size of which ten percent of the total material content is finer, obtained from a grain size analysis curve as in Figure 7-1. Gustafson's formula is also based on the grain size distribution curve of the material as follows (Svensson, 2014; Andersson *et al*, 1984):

$$K = E(C_u) \times \left(\frac{d_{10}}{1000}\right)^2 \quad [\mathrm{m/s}] \qquad \text{Equation 5}$$

Where $E(C_u)$ is a function dependent on $C_u$, which is the coefficient of uniformity, i.e. the ratio between $d_{60}$ and $d_{10}$. The Kozeny-Carman equation (Carman, 1956) is suggested as more appropriate, for example in Chapuis and Aubertin (2003) and Carrier (2003). This equation includes significantly more parameters than the two above commonly used:

$$K = \frac{\frac{\gamma}{\mu} \times n^3}{k_0 \times \left(\frac{L_e}{L}\right)^2 \times (1-n)^2 \times {S_0}^2} \quad [\mathrm{m/s}] \qquad \text{Equation 6}$$

Where $n$ is the porosity, $k_0$ is a constant, $L_e$ is the average length of capillaries, $L$ is the length of the specimen and $S_0$ is the specific surface area per unit volume of particles. The term $L_e/L$ can also be called the tortuosity (Svensson, 2014) and is usually handled as an empirical constant (Carrier, 2003). The porosity is measured by comparing for example the weight of a saturated material to the weight of a dry material, i.e. the volume of water corresponds to the volume of voids (Axelsson and Gustafson, 2007). The specific surface can be estimated from a grain size distribution or from a hydraulic measurement in situ; in the latter way the degree of compaction of the material is taken into account.

Carrier (2003) lists limitations of the Kozeny-Carman equation as following:

- The equation is not suitable for clayey soils since its does not account for electrochemical reactions between soil particles and water. Also, the extreme shape of the clay particles makes it difficult to describe the specific surface accurately.
- The equation is based on Darcy's law; in coarse soils where turbulent flow can occur, a more advanced formula, taking account for permeability changes due to different flow gradients needs to be used. For example Åberg (1992) has developed such a formula.
- Soils with a multi-graded grain size distribution are not suitable to evaluate with the Kozeny-Carman equation if they have a large range of fines. The smallest particle size in the soil needs to be known.

## 7.4 Transmissivity

The water conducting features of bedrock is mainly limited to the fracture system and hydraulic conductivity is then not a relevant description (Engström *et al*, 2009).

Hydraulic transmissivity is used instead. Transmissivity and conductivity is interrelated as:

$T = K \times b$ [m$^2$/s] Equation 7

The transmissivity is thus the integral of the conductivity of a saturated thickness $b$ (Carlsson and Gustafsson, 1997). Due to the linear relationship between transmissivity and hydraulic conductivity, also transmissivity has a lognormal probability distribution.

With the idealisation of the fracture to consist of two, smooth plane parallel plates, the transmissivity of a fracture is given by the cubic law, seen in Gustafson (2012) amongst others:

$T = \frac{\rho \times g \times b^3}{12 \times \mu}$ [m$^2$/s] Equation 8

$b$ is the hydraulic aperture, the width of the space between the fracture walls which conducts water (Thörn, 2013). Real fractures are however not smooth and there are expressions accounting for this variability, for example Zimmerman and Bodvarsson (1996).

Darcy's law applies also to flow in bedrock fractures. The flow velocity depends on the governing gradient and the hydraulic aperture of the fracture (Stille *et al*, 2012). Turbulent flow can arise in fractures with large aperture, more easily in fractures with rough surfaces than in smooth fractures (Gustafson, 2012; Thörn, 2013). Hydraulic transmissivity needs to be measured in situ and it can be estimated from short duration hydraulic test by its linear relationship to the specific capacity (Fransson, 2001):

$T \approx \frac{Q}{dh}$ [m$^2$/s] Equation 9

## 7.5 Storativity

Storativity is a measure of the volume of water that is stored or expelled per unit surface area of an aquifer due to a change in groundwater head and is expressed as (see for example Carlsson and Gustafsson, 1997; Persson, 2007):

$S = \frac{\Delta V}{\Delta h} \times \frac{1}{A}$ [-] Equation 10

Specific storage, $S_S$, is the same term but water expelled or stored per volume of aquifer. Specific storage and storage relates to each other as (Fetter, 2001):

$S = S_S \times b$ [-] Equation 11

Where $b$ in this case is the thickness of the aquifer. In a confined aquifer, the storativity depends on the porosity, the compressibility of the soil skeleton and the compressibility of water (Persson, 2007):

$S_S = \gamma_w \times (\beta_w \times n + \beta_S)$ [m$^{-1}$] Equation 12

In geotechnical terms, the compressibility of water is neglected and the storativity becomes (Persson, 2007; Carlsson and Gustafson, 1997):

$S_S = \gamma_w \times m_v$ [m$^{-1}$] Equation 13

Where $m_v$ is the coefficient of volume compressibility, which is the reciprocal of constrained modulus $M$.

For a bedrock fracture, the storativity can according to Doe and Geier (1990) be expressed as:

$S = \gamma_w \times (\beta_w \times e_v + \frac{1}{k_n})$ [-] Equation 14

$k_n$ is the normal stiffness of the fracture and $e_v$ is the void aperture of the fracture. The compressibility of water, $\beta_w$, can often be neglected. For fractures subjected to only small previous deformation, there is a correlation between fracture normal stiffness and hydraulic aperture, the stiffness decreasing with increasing aperture (Fransson, 2009).

## 7.6 Hydraulic diffusivity

Diffusivity is in hydrogeological terms the rate of change in hydraulic head along the horizontal direction of the aquifer (Persson, 2007). The hydraulic diffusivity of aquifers, such as frictional material or fractured rock, controls how fast the system responses to a pressure change (Gustafson, 2012; Singhal and Gupta, 2010). If pressure is lowered in one spot water will flow there due to the induced hydraulic gradient, as water flows there the pressure reduction is spread through the aquifer. The rate at which this spread of pressure changes occurs is governed by the hydraulic conductivity and the compressibility of the material or the soil skeleton and is expressed as (Persson, 2007):

$D = \frac{K}{\gamma_w \times \beta_s} = \frac{K}{S_S} = \frac{T}{S}$ [m$^2$/s] Equation 15

The diffusivity thus depends both on the ability of the material to transmit water (hydraulic conductivity) and the specific storage of the material, i.e. how large volume of water is expelled or stored due to a pressure change. In geotechnical terms the diffusivity for fine-grained, normally consolidated soils equals the coefficient of consolidation for one-dimensional vertical consolidation (Persson, 2007; Carlsson and Gustafson, 1997):

$c_v = \frac{K}{\gamma_w \times m_v} = \frac{K}{S_S}$ [m$^2$/s] Equation 16

The coefficient of consolidation gives a measure of the rate of consolidation, i.e. how fast excess pore water pressure is dissipated. Persson (2007) illuminates that the terms diffusivity, $D$, and coefficient of consolidation, $c_v$, equals if the compressibility of water is neglected. Still, the diffusivity is for flow in horizontal direction and the coefficient of consolidation is for flow in vertical direction. The order of magnitude also differs significantly between them, which depends on the different response in time of the processes (Persson, 2007).

## 7.7 Consolidation

Deformation properties of cohesive soil have already been touched upon by the coefficient of volume compressibility, $m_v$, and the coefficient of consolidation, $c_v$, where $m_v$ controls the vertical deformation and, $c_v$, controls the rate of the process. The denotation $m_v$ used here is the reciprocal of the compressibility modulus, $M_L$, for one-dimensional consolidation settlement in clay (Knappett and Craig, 2012; Persson, 2007). Elastic deformation taking place for stress levels below the pre consolidation pressure, $\sigma_c'$, of the clay is neglected in this thesis since it is small relative to potential consolidation settlement. The vertical deformation, $s$, due to consolidation of clay layer of thickness $h$ is given by (Knappett and Craig, 2012):

$s = h \times m_v \times \Delta\sigma$   [m]   Equation 17

$\Delta\sigma$ is the surcharge pressure governing the consolidation. The surcharge pressure can be due to applied overburden or decreased pore water pressure since these factors both increases the effective stress according to:

$\sigma = u + \sigma'$   [kPa]   Equation 18

$\sigma'$is the effective pressure, $\sigma$ is the total pressure and $u$ is the pore water pressure. The principle of Equation 18 also applies to pressure in a bedrock fracture. An increase in applied total pressure will initially result in an equally large increase in pore pressure since it takes time for the excess pore water pressure to dissipate and the soil skeleton to compact and carry more pressure, i.e. increased $\sigma'$. In addition to elastic deformation and consolidation deformation also creep contributes to the total vertical deformation of clay (Persson, 2007). Creep is significant at stress levels exceeding the pre-consolidation pressure.

A clay is termed normally consolidated if it is presently being subjected to the highest stress level it has ever been subjected to. An over consolidated clay has earlier consolidated to a stress level, $\sigma_c'$, higher than the present stress. The over consolidation ratio, *OCR*, is the relation between the pre-consolidation pressure and the current pressure (Knappett and Craig, 2012). The consolidation process starting at stress levels above $\sigma_c'$ and the consolidation rate is quantified by the coefficient of consolidation as presented in Equation 16 above.

In case clay is subjected to unloading, for example from infiltration of water to the ground, the coefficient of consolidation corresponding to unloading depends on the unloading modulus instead of the modulus for vertical compression, $m_v$ (Persson, 2007). For clarity, consolidation is a phenomenon present only in cohesive soil of low permeability.

## 7.8 Internal erosion

Hjulström's chart was developed for erosion from rivers and subsequent transport and deposition of soil grains. The chart in Figure 7-2 indicates what flow velocities are needed for erosion, transport and disposition of different grain sizes.

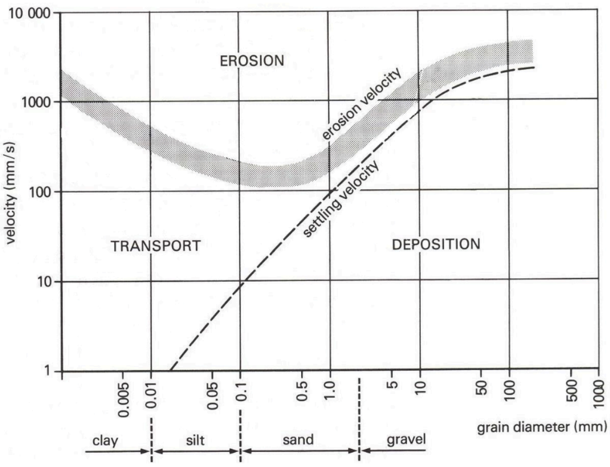


*Figure 7-2 Hjulströms chart. Flow velocities needed to erosion, transport and depostition of different grain sizes in a river bed (Geography is easy).*

A division can be made in Hjulström's chart between erosion of cohesive material and non-cohesive material. In such a case it is seen that a flow velocity of 2 cm/s will erode particles of a size up to 1 mm from a non-cohesive bed (Axelsson, 2009; Stille *et al*, 2012). Axelsson (2009) uses this velocity as an indication of at what velocity non-cohesive fracture infilling material will be eroded in a rock fracture. The flow velocity in a rock fracture depends on the hydraulic aperture of the fracture (described in Chapter 5.4) and the hydraulic pressure gradient (Gustafson, 2012). This is in accordance with Darcy's law, Equation 1, since the hydraulic aperture governs the transmissivity of the fracture.

The phenomenon of internal erosion of frictional material is an essential concern in dam formations. Internal erosion means that the finest particles within the soil skeleton are moved by seeping water, giving a following change in the soil structure. According to Li (2008), internal erosion is governed by a combination of the geometric characteristics of the soil, i.e. shape of the grain size distribution curve, porosity and amount of fine particles, and hydromechanical conditions, i.e. hydraulic gradient and effective stress. A hydraulic gradient acting over a soil creates a seepage drag force on the soil particles as water flows through the material. Fine particles are moved if this drag force exceeds the gravity on the soil particle as well as the friction against other particles in the soil skeleton (Jantzer and Knutsson, 2010). If the pore spaces of the soil skeleton are not small enough to prevent the moving fines from travel further with the water, internal erosion will occur. Depending on the geometric characteristics of the soil it is susceptible or not to internal erosion, and the onset of internal erosion is determined by the hydraulic gradient and effective stress, i.e. the hydromechanical conditions, acting on the soil (Li, 2008).

A soil with a gap-graded grain size distribution is unstable to internal erosion compared to a material with a continuous, linear curve; a gap-graded soil can experience erosion and transport of the material's fines whereas a linear material creates a stable, inter-locked soil skeleton (Rönnqvist, 2002). Figure 7-3 shows an example of (A) gap-graded and thereby unstable material, (B) a material with a fine content ranging over several fine sizes making the material unstable to internal erosion, and (C) a well-graded, stable material (Hammerstedt, 2010).

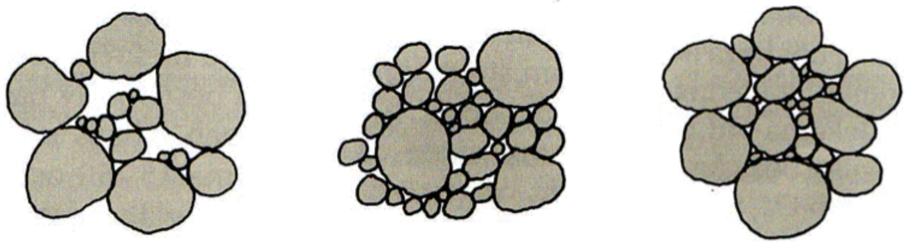


*Figure 7-3 Examples of stable and unstable frictional materials, modified from Hammerstedt (2010).*

The geometric character of the soil structure is visualized in a particle size distribution curve. Figure 7-1 shows an example of (1) a gap-graded material, (2) a well-graded or multi-graded material, and (3) a uniform graded material. Material (1) is clearly gap-graded and corresponds to (A) in Figure 7-3, thus susceptible to internal erosion. Material (2) is well-graded like (C) in Figure 7-3 but has a large range of fines like (B) which could make it unstable. Material (3) is uniform graded and thereby less stable than a well-graded material with a low amount of fines.

Only a small pathway for seepage flow of water can with time lead to a backward erosion, i.e. piping. The effect is escalating since the material becomes less and less stable as fines are washed out and the permeability increases. It can be seen from Darcy's law, Equation 1, that the flow and flow velocity will increase as the permeability increases. Soils with a higher porosity might start to erode at lower gradient than lower porosity soils (Li, 2008). Erosion can be initiated at borders between materials with different permeability (Rönnqvist, 2002).

There are several empirical or semi-empirical relationships developed for describing susceptibility and onset of internal erosion and Li (2008) has scrutinized a majority of them. Of the ones most often used to describing susceptibility, Li (2008) found experimentally that the Kezdi criterion and the Kenney and Lau criterion are the most appropriate. In Kezdi's criterion (Kezdi, 1979), the grain size distribution curve is divided into one part for fine content and part for coarse content. The soil is considered stable if following applies for both parts of the grain size distribution curve:

$$\frac{D'_{15}}{d'_{85}} \leq 4 \qquad \text{Equation 19}$$

$D'_{15}$ is the soil grain size of which 15 per cent of the particles are smaller and $d'_{85}$is the grain size of which 85 per cent of the particles are smaller. The Kenney and Lau criterion can also be said to divide the grain size distribution curve into a fine part and a coarse part and relate these to each other (Li, 2008). The criterion is purely based on the shape of the grain size distribution curve and the finest part of the curve is most important, as it is these particles that are most easily eroded (Li, 2008; Ahlinhan *et al*; 2012). See Kenney and Lau (1985) for a more detailed description.

Other criteria for soil susceptibility to internal erosion deal with the slope of the grain size distribution curve, coefficient of uniformity, amount of fines, porosity and effective diameter of the particles (Li, 2008). When it comes to the onset of internal erosion a critical hydraulic gradient is estimated based on the geometric properties on the gradation curve. Some expressions also include Reynolds number, i.e. flow velocity (Rönnqvist, 2002), and porosity (Li, 2008). Li (2008) also proposes, in accordance with Moffat (2005), that effective stress needs to be included when determining the onset of stability. However, this hydromechanical approach is yet not established and commonly only critical gradient, or critical flow velocity, is used. Some researches (e.g. den Adel *et al* (1988) and Richards and Reddy (2009)) point out the critical velocity is a more correct measure for onset of internal erosion, however velocity is difficult to measure correctly and the gradient is representative enough (Jantzer and Knutsson, 2010).

The critical gradient for internal erosion can be significantly smaller than the gradient for buoyancy or heave failure (Li, 2008; Jantzer and Knutsson, 2010). For a stable material the critical gradient is 1.0, just like the criterion for buoyancy. This is however for a vertical upward flow direction; the critical gradient for a stable material is lower for a horizontal or downward flow (Jantzer and Knutsson, 2010). Skempton and Brogan (1994) found the critical gradient for unstable material to be as low as one fifth of the critical gradient for buoyancy. In Perzlmaier *et al* (2007) it is also seen that the critical gradient decreases with decreasing particle size of the material, e.g. fine sand has a lower critical gradient than gravel. Also for unstable materials he direction of the flow is important since a horizontal or downward flow direction gives lower critical gradient than upward flow (Jantzer and Knutsson, 2010).

# 8 Material properties
The following subchapters gives a description of the different materials included in the stratigraphies presented in Figure 6-1. The descriptions are given in general terms and it is emphasised that specific properties can vary significantly within the same material. The subchapters focuses on properties of interests from a hydrogeological point of view, since groundwater issues is considered to be of major importance when it comes to soil rock interfaces. Hydraulic and mechanical behaviour are of importance in practical application of sealing strategies, which is the focus of this thesis. Geological history, origin and deposition are factors that affect the relevant properties of materials.

## 8.1 Bedrock properties
The geological settings for bedrock and its permeability are described earlier in Chapter 6.2 and only briefly summarized here. Most of the groundwater volume is located in open fractures in the bedrock since the porosity of the rock matrix else is significantly low. The frequency of open fractures and their interconnections determines the water conductive characteristics of the rock mass. Fracture zones or deformation zones in the bedrock give a heterogeneous and anisotropic hydraulic behaviour and there are large contrasts in hydraulic conductivity within the rock mass. A lion's share of the groundwater flow in bedrock takes place in the zones and they play important roles in the total hydrological system (Gustafson, 2012). It is therefore important to identify them early in any large-scale underground project.

## 8.2 Soil properties
Permeability of a soil is according to Knappett and Craig (2012) primarily governed by the average pore size of the material, which in turn is governed by the distribution of particle sizes, the particle shapes and the soil structure. Also the degree of compaction of the natural soil affects the hydraulic conductivity significantly (Svensson, 2014). A naturally deposited soil commonly shows anisotropic behaviour, being more permeable in the direction of the stratification than perpendicularly (Knappett and Craig, 2012). The more fines and the denser material, the smaller pore voids and the lower permeability. Well-sorted materials have higher permeability than unsorted or fine-grained materials since fine particles fill up the voids in the particle skeleton (Fetter, 2001). Clay is for example low permeable but sensitive to pore water dissipation since it can give rise to deformation. Pore pressure changes in a high permeable soil will develop quicker.

Values of hydraulic conductivity of different soil types vary a lot depending on macro fabrics, deposit environment, origin and similar. This inherits a large variation in the hydraulic conductivity, also within the same deposit where the hydraulic conductivity can vary with several orders of magnitude (Persson, 2007). Below follows a description of origin, deposition and characteristics of the different soil types seen in the type stratigraphies in Chapter 6.1. This chapter focuses on the water-conducting properties of the soils. It shall be mentioned that in the following, the term frictional material will be used to cover all cohesionless soils, i.e. till, gravel, sand and silt.

### 8.2.1 Till
Till is a glacial deposit consisting of material eroded from the rock below the ice sheet (Knappett and Craig, 2012). The material was transported with the moving ice and water and deposited during the melting of the ice sheet; as moraines at the ice front or

just below the ice as lodgement till (van der Meer *et al*, 2003). The material is crushed and unsorted or poorly sorted, though there can be layers of more sorted material such as gravel, sand or silt (Fredén *et al*, 2009). The range of particle sizes in till is large, from clay particles up to boulders, giving a multi-graded particle size distribution. The share of the different sizes differs between deposits due to the conditions during deposition. The Swedish till content is commonly dominated by of sand and silt fractions (Fredén *et al*, 2009, SGU[2]). The permeability of till is highly dependent on the amount of fines in it; a lot of fines filling the pores will significantly reduce the permeability (Knappett and Craig, 2012). If the till has layers of more sorted material, it is likely to be more permeable in this direction. Due to the large variation in particle size distribution and degree of compaction, depending on variations in deposition environment, the hydraulic conductivity of a till can be very difficult to decide from the grain size distribution curve of the material (Svensson, 2014). Due to the heterogeneity of a till, its hydraulic conductivity can also potentially vary a lot over an area.

### 8.2.2 Glaciofluvial sediments

Glaciofluvial sediments are rounded particles, sorted to well-sorted, of boulders, stones, gravel, sand or silt (Fredén *et al*, 2009). It is transported, sorted and deposited by ice melt water. The sortation is dependent on the velocity of the flowing water, larger particles sizes settling at the highest velocities and vice versa (Lundqvist *et al*, 2011). Glaciofluvial sediments are commonly layered since the settling conditions at one site have changed during the disposition. The layering inherits anisotropic permeability features of the sediment, being more permeable in the direction parallel to the layering. Since the material is sorted, is has an inter-connected pore space system and thereby a high permeability.

### 8.2.3 Glacial clay

Glacial clay is deposited in water far from the ice margin since very slow flow velocity is needed for these small particles to settle and is often settled at great depths (Fredén *et al*, 2009; SGU[1]). If the clay particles are deposited in freshwater, the clay is varved with light layers and dark layers depending on the season of disposition. If deposited in marine environment, the clay is not varved depending on chemical reactions with the salt water (Fredén *et al*, 2009; Lundqvist *et al*, 2011). Varved clays or clays with layer of silt are more permeable in the horizontal direction than the vertical (Persson, 2007). The pore space in clay is small and sometimes sparsely interconnected, resulting in a low permeability (Fetter, 2001). On the other hand, clay has a high capillarity. The water content is important in a clay, determining whether it is liquid, solid or in between (Knappett and Craig, 2012). The macro fabric can be crucial for the hydraulic behaviour of clay, as an example the existence of fissures in clay increases the permeability significantly.

### 8.2.4 Postglacial deposits

Postglacial deposits can be post-glacial clay, beach deposits, wind sediments, fluvial sediments and organic deposits (Fredén *et al*, 2009). They are deposited after the glaciation but commonly consist of glacial material that has been rearranged by wind and water. Wind and fluvial sediments are relatively sorted, wind sediments usually deposited as dunes and fluvial sediments deposited as bank or deltas. Beach deposits may consist of stones, gravel, sand or even finer particles. Layers of postglacial clay and silt are commonly overlying glacial clay. Peat and gyttja are organic soils, consisting of incompletely degraded organic material. Such soils have large water content and are very unstable.

### 8.2.5 Dry crust and fill

The upper part of the soil layer in urbanized areas typically consists of fill material, commonly frictional materials (Gustafson, 2012). When the uppermost natural soil layer is clay, it has been subjected to environmental forces, which changes the clay into dry crust (Persson, 2007). The dry crust is usually a couple of meters thick but can extend some meters deeper, approximately two to four meters below the ground water table. The dry crust and potential filling material above it creates an upper aquifer above the underlying clay layer. The ground water pressure can be assumed to be hydrostatic within this upper aquifer, mostly affected by precipitation and leaking water pipes, and groundwater changes develop immediately (Persson, 2007).

## 8.3 Systematic characterization of material properties

When material types and their spatial location have been identified (which is done in the site investigation more described in Chapter 9) the information needs to be arranged in a way in which it is possible to handle it. The information used to characterize materials is preferably related to the specific engineering application it shall be used for and materials with similar characteristics can be grouped together (Kvartsberg, 2013a). Rhén *et al* (2003) introduced a concept of hydraulic domains as a basis for hydrogeological modeling of the Simpevarp area in Sweden (a research facility for future storage of spent nuclear fuel (SKB)). In this concept rock is divided into hydraulic rock domains and hydraulic conductor domains respectively. There are also hydraulic soil domains consisting of soil units of similar hydraulic characteristics grouped together. The hydraulic domains are seen in Figure 8.1. Hydraulic conductor domain relates to deformation zones and hydraulic rock domain relates to relatively sparsely fractured rock.

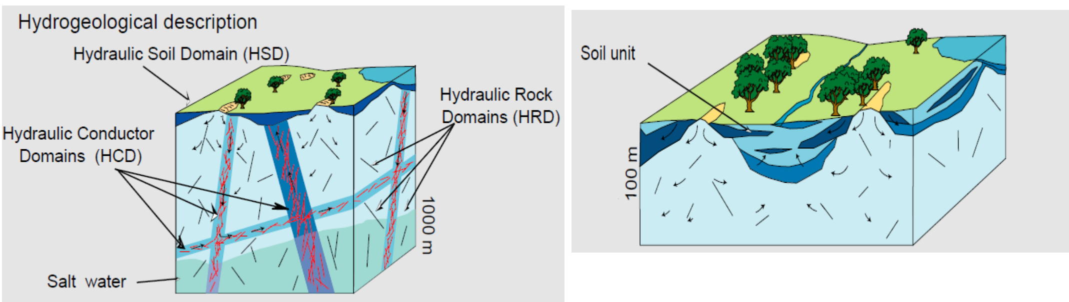


*Figure 8-1 Hydraulic domains to the left and soil units to the right, by Rhén* et al *(2003).*

Kvartsberg *et al* (2013) and Kvartsberg (2013a) used a concept presented by Rhén *et al* (2003) to classify a site for tunnel sealing purpose. The groundwater flow dimension differs between hydraulic rock domains and hydraulic conductor domains; the hydraulic conductor domains have a distinct anisotropic flow while the flow in the hydraulic rock domains is more distributed seen in a larger scale (it is still single fracture that transmits the lion's share of groundwater) (Kvartsberg, 2013a). The hydraulic conductor domains were by Kvartsberg *et al* (2013) further distinguished, or classified, using the permeability structure of a fault zone by Caine *et al* (1996) in Figure 8-1. Hydraulic domains constitute a relevant classification of the rock for tunnel sealing purpose since it is the hydraulic behavior of the bedrock which is of interest in sealing application (Kvartsberg, 2013a).

# 9 Investigation of underground conditions

Investigation of geological and hydrogeological conditions is crucial in any underground project. Measurements and investigations are initiated during the pre-investigation phase of a project since it is an important base for the Environmental Impact Assessment document as well as for the design of the construction (Werner *et al*, 2012). If the observational method is used as design method, the pre-investigation phase is particularly important since the design shall be based on the most probable conditions and contingency plans for deviations shall be decided on beforehand (Zetterlund, 2009). Throughout the construction process updating shall be made continuously as there more information becomes available (Kvartsberg, 2013a).

The feasibility phase is as seen in Figure 4-1 the first stage in an underground project process and the desk study, i.e. collection of relevant existing data concerning the site, constitutes an essential part of the feasibility phase (Kvartsberg, 2013b). The main part of the investigations is carried out in the following design and production planning phase of the project. Ideally, site investigations shall be based on geological models of the area in order to improve the efficiency of the investigations; such models are however usually performed after the site investigation (Kvartsberg, 2013a). Zetterlund (2009) states that disagreement that often arises in connection to interpretations and prognoses of expected geology could be avoided if investigations focus on project specific factors.

The following subchapters deal briefly with investigations that typically are performed in early phases of an underground project, such as the pre-investigation which is followed by establishment of engineering geological prognoses for the project. The aim of the chapter is to act as a basis for understanding how the underground conditions commonly are investigated. This is important in order to identify potential fields where co-operation and co-interpretation of investigations can improve the overall knowledge and reduce uncertainties concerning soil rock interfaces.

## 9.1 Geological investigation

Fracture mapping and desk studies, including studying large-scale lineaments and tectonics, are important initial stages in the geological investigation (Lindblom, 2010). The bedrock itself is investigated by more or less vertically drilled holes (Gustafson, 2012). Bedrock cores from the drilling are mapped and bedrock type, mineral content, mineral grains, fracture surfaces, fracture filling materials, fracture frequency and similar is determined. Commonly the rock quality is decided by some standard classification system (Kvartsberg, 2013a). Zones of brittle deformations can be interpreted locally from loss of material in the drill core. Larger brittle deformation zones can be identified by seismic investigations and are important to know both from a stability and groundwater point of view (Gustafson, 2012). The water conductive feature of the rock is estimated by performing hydraulic tests in probe holes. Measurements of hydraulic features are dominated by the most transmissive features of the rock mass and is thereby not representative for individual fractures. However, water conductive features of bedrock are very difficult to evaluate accurately in the early phases of a project and there is an essential need for probe hole tests and measurements during the construction (Hernqvist, 2011; Gustafson, 2012). For example, the verticality of investigation boreholes in the pre-investigation possibly makes it difficult to capture anisotropic features of the bedrock (Gustafson, 2012).

## 9.2 Geotechnical investigation

The stratification, i.e. the soil profile, at the site is investigated in situ by vertical penetration of the ground. The relative stiffness of the ground, the depth to firm ground (bedrock) and the approximate thickness of material layers are interpreted at an early stage. This is done with penetrative, intrusive methods such as standard penetration test of cone penetration test, in which the relative resistance through the ground is monitored and interpreted (Knappett and Craig, 2012). If the soil includes stones or boulders, drilling might be needed. Geophysics is another method to investigate the soil profile and depth to bedrock. Clay samples are taken for laboratory testing of permeability and deformation properties such as deformation moduli, pre-consolidation pressure and coefficient of consolidation. Taken samples need to as little disturbed as possible to obtain representative results. Equipment is also established at site to measure the pore pressure distribution within the soil profile. Shear strength of soil and soil sensitivity (e.g. silt content, water content) are important properties, which also are interpreted from investigations and tests.

## 9.3 Geophysical investigation

Geophysical investigation methods are used in both soil and rock. The advantage is that the method is non-intrusive and can give large-scale pictures of the underground conditions, however it is mainly suitable as a complementary method for co-interpretation with borehole data (Knappett and Craig, 2012). Geophysics can be used to get an idea of the conditions between boreholes or points of investigation and it can indicate where additional information might be valuable. The method is also suitable for determining approximate soil depth, soil types and ground water table (Knappett and Craig, 2012; Lindblom, 2010). Electrical resistivity measures the ground's ability to transmit current and can be used to make estimates of soil types and layer thicknesses, as well as lateral variation of these (Knappett and Craig, 2012). In bedrock investigations, seismic refraction is primarily used and gives information about how fractured the rock is since fractured rock has lower shear wave velocity (Gustafson, 2012). Geophysical methods can also be used inside drilled bedrock boreholes (Kvartsberg, 2013b).

## 9.4 Hydrogeological investigation and evaluation

Hydrogeological investigations are used for evaluating the hydrogeological system and the large-scale water conducting features of a site. The stratigraphy at the site and the thickness of the different soil layers are important to know when planning and performing a hydrogeological investigation (Fetter, 2001) and the evaluation of performed tests is facilitated by an adequately detailed geological model (Persson, 2007). Desk studies of geological and topographical maps as well as a site visit provides useful information of the hydrology, possible hydraulic boundaries and groundwater dividers and similar.

Investigation of aquifer hydraulic properties is typically performed as pumping test in which water is injected or extracted to a bored well (Fetter, 2001). The effects of the pumping are observed in observation wells in the surroundings; groundwater head is measured at certain time intervals during up to several days of pumping. Results from a pumping test are commonly evaluated analytically and often also with numerical modelling (Persson, 2007). This aids identification of the type of aquifer and its behaviour; parameters such as the effective transmissivity, storativity, boundary conditions as well as hydraulic properties of a confining layer can be obtained. Pumping test can also be used in bedrock to identify the extent, interconnectivity and behaviour of fracture zones (Gustafson, 2012). When pumping tests are too expensive or not practically feasible to carry out, slug tests can be performed instead (Fetter, 2001). Slug tests gives the local hydraulic properties of the aquifer material in the vicinity of the well.

### 9.4.1 Local hydraulic properties

Slug tests demands no observation wells, instead it is the conductivity of the soil closest to the well that is measured. Water is extracted or injected to a well and the transient recovery behaviour is recorded in the same well (de Marsily, 1986). Different methods, depending of the kind of recovery behaviour of the well, are then used to calculate hydraulic parameters (Fetter, 2001). The recovery can be either smooth, then called overdamped, or oscillating, then called underdamped. Different evaluation methods for wells fully or partly penetrating a confined aquifer are given in Table 9-1. Obtained parameters are aquifer transmissivity and storage coefficient, the latter one with less precise than the first one (de Marsily, 1986).

*Table 9-1 Evaluation methods for slug-tests, based on Fetter (2001) in which also more about the methods can be seen.*

| | **Overdamped** | **Underdamped** |
|---|---|---|
| **Complete penetration of confined aquifer** | -Cooper-Bredehoeft-Papadopulos method<br>-Bouver and Rice method | -Van der Kamp method<br>-Kipp method |
| **Partly penetration of confined aquifer** | -Hvorslev method<br>-Bouver and Rice method | |

The hydraulic conductivity of specific material layers can also be obtained by in situ sampling and laboratory measurement as described in Chapter 7. The sampling method needs to be performed with suitable equipment and technique in order to obtain representative samples (Svensson, 2014). Sampling and laboratory measurements might however not be totally representative since material fractions can be lost and the degree of compaction is not captured.

### 9.4.2 Aquifer hydraulic properties

There are several analytical solutions describing radial and one-dimensional flow to wells (Persson, 2007). They are all founded upon Darcy's law, extended to flow in different directions and based on idealized situations (Svensson, 2014). The analytical solutions are derived for different assumed idealized conditions, such as confined aquifers, unconfined aquifers, leaky aquifers, as well as a leaky aquifer overlain by a compressible aquitard (Persson, 2007). Leaky aquifer means that there is a leakage of water from an overlying aquitard down to an underlying aquifer. If the aquitard layer is compressible, i.e. consists of normally consolidated clay, compression of this layer gives additional contribution to the leakage to the aquifer. The significant difference, apart from what was described in Chapter 6, between a confined and an unconfined aquifer is that the water-conducting thickness of an unconfined aquifer decreases as the drawdown increases and that an unconfined aquifer has a significantly lower specific storage but instead a larger specific yield (Carlsson and Gustafsson, 1997; Fetter, 2001). The upper and lower aquifer described in Chapter 6 are then unconfined and confined respectively, the confined one probably also leaky with a compressible aquitard overlying it.

The different analytical solutions apply either to transient conditions or steady state conditions, which is obtained when the cone of depression has a constant shape. A summary of different conditions and applicable solutions for confined aquifers with radial, two dimensional, flow is given in Table 9-2 below.

*Table 9-2 Analytical solutions for idealized conditions. Confined aquifers with radial flow. The table is based on, and equations given in Persson (2007), Carlsson and Gustafson (1997) and Fetter (2001).*

| | **Confined aquifer Radial flow** | **Leaky aquifer Radial flow** |
|---|---|---|
| **Steady state conditions** | Thiem well equation $\Delta h = \frac{Q}{2\pi T} \times ln \frac{R_0}{r}$ | Hantush – Jacob method |
| **Transient conditions** | Cooper-Jacob approximation Theis type curve | Theis – Walton method |

Data logged from the test pumping is plotted logarithmically or semi-logarithmically as time to drawdown or radial distance from pumping well to drawdown. With such plots the analytical solutions can be used to evaluate hydraulic properties of the aquifer tested. Table 9-2 above only includes confined aquifers and radial flow conditions but there are also solutions for unconfined aquifers and channelled, i.e. one-dimensional, flow (Carlsson and Gustafson, 1997; Fetter, 2001). The use of image wells and superposition enables taking hydraulic boundaries into account in the analytical evaluation. Hydraulic boundaries are for example a boundary with a constant ground water head or an impermeable boundary with insignificantly small groundwater flow.

Spatial extent of the aquifer, heterogeneity of the aquifer and possible inter-connections between different aquifers will affect the results of the interpretation of a test pumping (Persson, 2007). It is important that the observation wells are located at suitable spots and distances from the pumping well. Hydraulic conductivity or transmissivity evaluated from a pumping test represent an average of the different soil layers or a heterogeneous soil layer; this can be particularly important to consider the case of very heterogeneous soils (Svensson, 2014). Generally, anomalies with different hydraulic conductivity in an aquifer will only have local effect and the effective, or average, transmissivity of the large-scale aquifer will dominate the hydraulic behaviour (Persson, 2007; Fransson, 1999).

### 9.4.3 Bedrock hydraulic properties

Hydrogeological investigations are used to identify the water-conducting features of bedrock. It is particularly important to identify any large water-conducting deformation zones since they may have a major impact on the overall hydrogeological system and large investigation effort is preferably put on identification and characterization of such zones (Gustafson, 2012). Smaller zones of less significance can be represented stochastically by grouping zones of similar size, orientation and behavior. Bedrock wells can be used else testing is performed in drilled boreholes. Packer tests are common in which water is injected during a short time into isolated sequences of the borehole, called a water pressure test. With the injected flow of water known, together with the used injection pressure and assumed steady state conditions (Kvartsberg, 2013a), the transmissivity of the tested section is known from Equation 9 (Fransson, 1999). The same theory applies also for inflow tests, where the natural flow from into the borehole due to the hydraulic gradient is measured (Fransson, 1999; Hernqvist, 2011). In this manner the transmissivity of each section along the borehole can be obtained. The obtained transmissivity originates from all water-conducting, open fractures crossed by the borehole along the tested section; the main share of the flow comes from only a few fractures along the section (Gustafson, 2012).

The injection of water gives a pressure increase around the borehole and the analytical concepts described for transient conditions in aquifers above can be used for evaluation by viewing the pressure increase as negative drawdown and the borehole as a well (Gustafson, 2012). With injection in one hole and pressure measurement in other holes the connectivity of the fracture system can be evaluated, known as an interference test. The hydraulic diffusivity of the fracture system determined how fast the pressure change reaches the observation wells or boreholes. With pressure build up test recovery data is logged and evaluated instead. This test also allows for evaluation of flow dimension and thereby the interconnectedness of the fracture system, see the left part of Figure 6-3 (Hernqvist, 2011).

### 9.4.4 Analytical evaluation versus numerical modelling

There are several assumptions and simplifications in the conceptual models used for analytical evaluation presented above. Still, analytical methods are of valuable importance in the early phases of a large-scale project for identifying aquifer conditions and properties, key parameters and to cross-validate other models (Persson, 2007). In numerical models, more complex geometries can be considered as well as variations in soil parameters, flow, boundary conditions and initial conditions (Persson, 2007). Overall, the numerical methods are more flexible and advanced, however the quality of the model result is it significantly dependent on the quality of the conceptual model it is based upon. A problem with numerical models is commonly that there is a lacking availability of accurately high quality data (Gustafson, 2012).

### 9.4.5 Early estimation of inflow

In might be valuable to estimate expected inflow to a tunnel or an underground space for example for comparison to the regulatory inflow requirement, for example as in Kvartsberg *et al* (2013). This can be done by viewing the tunnel as a well and use the analytical solutions presented above. By using Thiem's well equation in Table 9-2 and data from the Swedish well archive on well capacities (provided by the Geological Survey of Sweden) the inflow to a tunnel can be estimated as (e.g. Gustafson, 2012):

$$q \approx \frac{H}{2 \times R_0} \times \left(\frac{Q}{d}\right)_{50} \times ln\left(\frac{R_0}{r_w}\right) \qquad \text{Equation 20}$$

The transmissivity is here assumed to be the geometric mean of the specific capacity of representative wells, $(Q/d)_{50}$, where Q is the well capacity and d is the well depth. $r_w$ is the well radius, $R_0$ is the radius of influence which roughly can be estimated as five times the groundwater head above the tunnel (Gustafson, 2012). In the case of an underground space instead of a tunnel the inflow can be estimated as (Gustafson, 2012):

$$Q \approx H \times \left(\frac{Q}{d}\right)_{50} \times \frac{ln(R_0/r_w)}{ln(R_0/R_b)} \quad \text{Equation 21}$$

$R_b$ is here the equivalent radius of the underground space. These expressions are based on assumptions that make the obtained values rough but might still give valuable indications at early project phases (Kvartsberg *et al*, 2013). As the project proceeds the estimated inflows can be updated with a more appropriate transmissivity than the median specific capacity.

# 10 Tunnelling and sealing

Constructing a deep excavation or a tunnel creates a pressure gradient since the pressure in the cavity is atmospheric and there is a higher pressure outside the cavity, governed by the groundwater head. The gradient will lead to changed groundwater flow patterns since water starts to flow towards the cavity. The effects of this are dependent on the hydraulic properties and extent of soil layers, the bedrock and the contact between soil and bedrock as well as the groundwater head and recharge. Relatively often, the sealing efforts are not efficient enough to fulfil regulatory requirements on tunnel inflow (Werner *et al*, 2012; Banverket, 2006a). This chapter presents tunnelling techniques in Sweden very briefly, focusing on groundwater issues and sealing strategies. Grouting is considered to be an important part of the sealing concerning soil rock interfaces and the penetrability of grout in different materials is crucial for a successful design and performance. Subsurface ground is a complex material and according to Stadler (2004), grouting design demands proper pre-investigation, a well-defined geological model and knowledge of the materials, observations during execution, experience and competence as well as an engineer's open mind.

## 10.1 Rock tunnelling and grouting

The common rock tunnelling technique in Sweden is drill and blast, though there are some examples of tunnelling with tunnel boring machine. Issues typically arise when the tunnel passes fracture zones, concerning both stability and inflow (Gustafsson, 2012) and great uncertainties arise when the sealing demands are high (Kadefors and Bröchner, 2008). The regulatory requirements on tunnel inflow are strict in areas sensitive to ground water changes and pre-grouting is a standard solution in fulfilling the regulatory requirements keeping a safe and functional working environment. Post-grouting is used after construction if the inflow is still too high, post-grouting is however more difficult to perform due to high groundwater gradients (Hernqvist, 2011). Knowing the actual conditions on beforehand is very difficult and the pre-grouting performance needs to be adapted to observations and measurements throughout the work (Kvartsberg, 2013a).

Sealing design shall according to Trafikverket (2011) be presented in sealing classes or design classes (Kvartsberg, 2013a). Sealing classes are based on varying outer requirements along the tunnel stretch, such as regulatory inflow requirement or sensitivity of the surroundings, whereas design classes apply to varying geological conditions (Kvartsberg, 2013a). A combination of the two types of classes is often used and design shall be adjusted to actual geological conditions. However, Kvartsberg (2013a) recognises that it is often unclearly defined exactly how this adjustment shall be made and in what quantities deviating conditions are likely to occur. The geological conditions can be presented as rock classes (Gustafson, 2012). For grouting purpose the hydraulic domains described in Chapter 8.3 can act as different rock classes when grouting is the engineering application, the term hydraulic domains is though preferably kept since it emphasizes what processes and characteristics that are in focus (Kvartsberg, 2013a). In general terms the geological settings at a site and the requirements of the project together determine what pre-defined design shall be applied as seen in Figure 10-1 (Gustafson, 2012; Kvartsberg, 2013a).

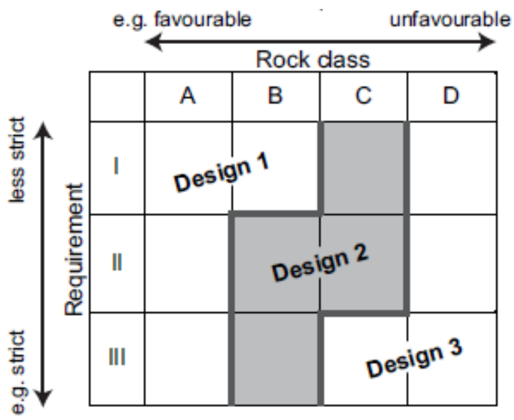


*Figure 10-1 Matrix showing how to choose pre-defined design based on geological settings and project requirements. Picture from Kvartsberg and Fransson (2013) from Gustafson (2012).*

The design is made in the 'design and production planning phase of the project' and the decision of what design to use is made in the 'construction phase'. Therefore suitable observation parameters are needed to base the choice of pre-defined design (Kvartsberg, 2013a).

### 10.1.1 Permeation grouting and pre-grouting design

A pre-grouting design consists of a number of grout holes typically drilled out from the tunnel periphery with a certain length, inclination and spacing. The grouting pressure and duration of grouting are also specified in the design. If successful, the penetration of grout from the grout holes into intersected fractures creates a sealed zone around the tunnel to be excavated. The fan geometry design can also be adapted to the fracture orientations (strike and dip) in relation to the tunnel direction (Funehag, 2012; Hernqvist, 2011). Grouting design involves three major factors: the rock, the grout and the technique. These are described in the following text. The final result of a grouting needs to be checked and control measurements to perform can be stated in the design (Brantberger, 2009).

The design is ideally based on individual fracture apertures and transmissivities (Fransson *et al*, 2012). From water pressure tests performed in section in a probe hole, together with mapping (fracture frequency) of the core from the probe hole, the transmissivity of each fracture is estimated. It has been found that the distribution of fracture transmissivities can be represented by a Pareto distribution (Funehag and Gustafson, 2008; Fransson and Gustafson, 2000; Gustafson, 2012). The Pareto distribution reflects the fact that a majority of the flow in bedrock fractures is located to very few large fractures, while there can be a large amount of small fractures together contributing only to a small amount of the total flow in the bedrock. The notation large or small fractures here relates to the hydraulic aperture, $b$, of the fracture. Commonly, measurements of water flow in bedrock are given in Lugeon unit (Stille *et al*, 2012). This measure is however an average of the water flow, and not directly linked to conducting feature of individual fractures (Kvartsberg, 2013a ;Fransson *et al,* 2012).

The hydraulic aperture, $b$, is obtained from the cubic law (Equation 8) if the transmissivity is known, assuming the fracture walls to be parallel plates. Having a probability distribution of the hydraulic apertures means that the minimum aperture that needs to be sealed, $b_{min}$, in order to fulfil the inflow requirement can be determined.

This minimum aperture governs the choice of grout material (Hernqvist, 2011; Funehag and Gustafson, 2008) apertures larger than 50 to 100 µm can be sealed with a cement grout and sealing of smaller apertures needs to be done with s chemical solution grout (Kvartsberg, 2013a ; Fransson *et al,* 2012). Combinations of cement grouts and chemical grouts can be used within a grout fan (Butron, 2009). Figure 10-2 below shows an analysis process for a preliminary design based on individual fracture apertures, given by Gustafson *et al* (2004). Knowledge of the grout's rheology and penetrability is of major importance in adequate development of a grouting design (Hernqvist, 2011) and the Chapters 10.1.2 and 10.1.3 present this briefly.

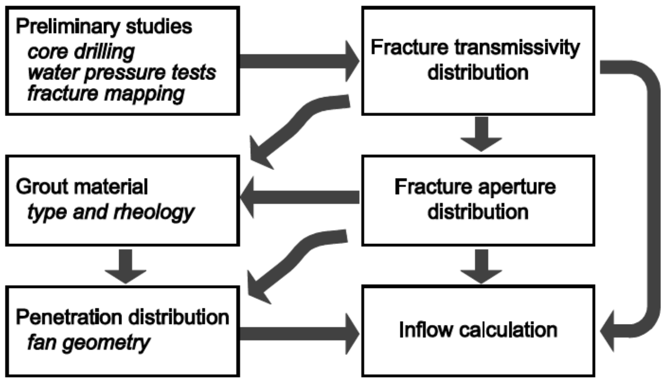


*Figure 10-2 Analysis process for a preliminary grouting design (Gustafson et al, 2004*

As the grout injection hole is filled, grout will start to penetrate all fractures, which are intersected by the grout hole, simultaneously (Thörn and Fransson, 2013). However, the grout penetration velocity will be higher in the larger fractures and vice versa, resulting in longer penetration lengths in larger fractures and shorter penetration is smaller fractures. Figure  shows a grouting fan and grout penetration from one of the grout holes (grouted with an overpressure *Δp*). Figure 10-3 shows the minimum aperture to be sealed, $b_{min}$, which determines the design penetration length, $I_{dim}$, for the spacing of grout holes in this case.

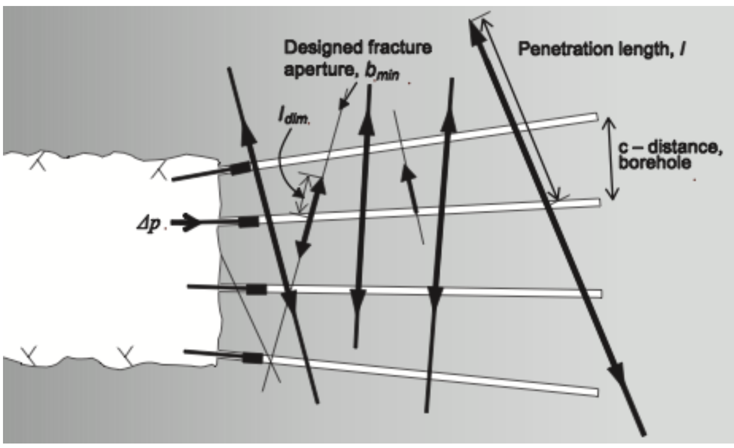


*Figure 10-3 Relationshipe between penetration length, fracture aperture and dimensioning penetration length for a given spacing between grout holes. From Funehag (2012).*

Grouting time and grouting pressure can be altered to achieve needed sealing efficiency. Since time often is limited in projects, increased pressure might be the solution to increase the penetration length. There is however a risk of hydraulic jacking, i.e. opening of fractures due to the applied pressure, in general the risk increases with decreased tunnel depth due to decreasing overburden (Thörn and Fransson, 2013). Large fracture apertures, with a two-dimensional flow and few small contact points, generally have low fracture normal stiffness and thereby more easily jack (Thörn and Fransson, 2013; Fransson, 2010). This affects the nearby fracture system and can for example lead to closure of parallel fractures, leading to even poorer grout penetration due to the reduced fracture aperture. The opening and closing of fractures can lead to changed paths for fluid flow, especially where the in situ stress is low (Kvartsberg, 2013a). In the case that fractures have any infilling material, this can be moved by the grout penetration, also leading to opening or closing for fluid flow.

Engineering geological information and data needed for a grouting design and performance described here is obtained during the investigation phase of a project but also during the actual construction (see e.g. Hernqvist, 2011; Kvartsberg, 2013a). Needed hydrogeological investigations described in Chapter 9.4 are pressure build up tests, water pressure tests and inflow test (Hernqvist, 2011). These together with core mapping provide the basis for the pre-grouting design. During performance additional inflow and water pressure tests in the tunnel front, together with logging of grouting data constitutes the basis for decision of grouting design to use. Grouting data of pressure, volume of grout and time provides a measure of the flow dimension in the fracture system.

### 10.1.2 Cement grout material

A cement grout behaves as a Bingham fluid, meaning that is has a yield strength, $\tau_0$, which needs to be exceeded (by the grouting pressure) in order to make the grout flow and the flow stops when equilibrium is reached (Axelsson *et al*, 2009; Funehag and Gustafson, 2008). The penetration into fractures, i.e. the flow of the grout, depends on the grouting overpressure, the aperture of individual fractures, the yield strength, $\tau_0$, and the viscosity, $\mu$, of the grout. The time of applied grouting pressure also affects the obtained penetration length of the grout since the theoretical penetration length would take long time to reach. Gustafson *et al* (2013) have developed an analytical solution for grouting design taking time and relative penetration length into account. The theoretically maximum penetration length, time of applied grouting pressure not accounted for, is calculated as:

$$I_{max} = \frac{\Delta p}{2 \times \tau_0} \times b \quad [\mathrm{m}] \qquad \text{Equation 22}$$

Cement grouts consist of water and cement, and the size of the particle grains need to be small enough to penetrate bedrock fractures. A rule of thumb is that 95 per cent of the particles in the cement (obtained as $d_{95}$ from a grain size curve) needs to be smaller than at least one third of the hydraulic aperture (Fransson *et al*, 2012; Hernqvist, 2011). This can be expressed as penetration is possible if (Axelsson *et al*, 2009):

$$\frac{b_{fracture}}{D_{95}^{grout}} > 3 \qquad \text{Equation 23}$$

Apart from the grain size of the cement grout, also the water cement ratio can be changed in order to change the penetrability of the grout: the higher water content, the larger penetrability (Axelsson *et al*, 2009). Too high water cement ratio tough increases the risk of grout filtration. Microcement can also be used; this cement has very small particles and could then penetrate smaller voids than conventional cement grout. However, the microcement has a larger specific surface due to the small sizes of the particles and this leads to a charge between the particles, which might flocculate (Axelsson *et al*, 2009; Powers *et al*, 2007). In order for microcement to penetrate a fracture the fracture aperture needs to be about ten times larger than the largest grain size in the microcement (Stille *et al*, 2012), compared to the rule of thumb for conventional cement in Equation 23. Higher water cement ratio can reduce the risk for flocculation and clogging (Axelsson and Gustafson, 2007).

### 10.1.3 Chemical grout material

Chemical solution grouts, like Silica sol or polyurethane foams. Amongst others, Andersson (1998) studied rock grouting with polyurethane foams and Funehag (2007) has studied silica sol. Chemical grouts behave as Newtonian fluids, this means that a chemical grout has no yield stress and it flows as long as there is a gradient acting on it (Funehag and Gustafson, 2008). The chemical grout will eventually gel and this is what makes is stop and stiffen in the bedrock fractures. The gelling features of the grout are determined by how the grout is mixed with a saline solution, and the gel induction time, $t_G$, is the time it takes for the grout viscosity to double (Funehag, 2012). The time it takes until the grout has gelled is denoted $T_G$ and denoted gel time. The achieved penetration length of a chemical grout into a fracture depends on the fracture aperture, the grouting pressure, the viscosity and the gel induction time of the grout but also the duration of applied grouting pressure. As a rule of thumb, maximum penetration length is achieved after half the gel time $T_G$ (Funehag, 2012). Analytical solutions for the penetration of Silica sol are seen in for example Funehag and Gustafson (2008) and the penetration length for a radial (two-dimensional) grout spread is calculated as:

$$I_{max-2D} = 0.45 \times b \times \sqrt{\frac{\Delta p \times t_G}{6 \times \mu_0}} \quad [\mathrm{m}] \qquad \text{Equation 24}$$

## 10.2 Soil tunnelling and grouting

Tunnel stretches in soft soil are constructed as concrete tunnels in open excavations, which might need to be large and deep. Tunnel boring machines can be used in soft soils which is often done internationally but there are no Scandinavian examples of using tunnel boring machine in soft, marine clay (Banverket, 2006b). Using a boring machine still needs large excavations at start and end points of the tunnel stretch. Sheet pile walls are commonly used to retain excavation walls, though there are also examples of diaphragm walls and secant pile walls. Diaphragm walls are constructed by excavating a rectangular pit and then fill it with a cage of reinforcement and then concrete, this is repeated continuously along the large excavation area creating a line of reinforced concrete panels. Secant pile walls consist of smaller concretes piles inserted with an overlap. First piles without reinforcement are installed, and then reinforced pillars are installed between the first ones with an overlap. Excavations are kept dry by pumping inflowing water away and sealing efforts might be needed to avoid too large groundwater drawdown outside the excavation. Sealing strategies in soil are either permeation grouting, meaning that the voids in the soil are filled with grout, or jet grouting in which the soil is mixed with grout material. Both techniques are described below.

### 10.2.1 Permeation grouting in soil

Permeation grouting in rock was previously described in Chapter 10.1 and can also be performed in soil with either cement, microcement or chemical grout. If groundwater control is the purpose of the permeation grouting, chemical grout is likely to be needed since this grout material can penetrate the fine pores of a soil skeleton (Powers *et al*, 2007). Permeation grouting with cement grout is only applicable to clean coarse sands or gravel. Permeation grouting can be applied to most soil types but a large amount of fines reduces the grout penetrability, especially clayey fines. The moving grout can pick up fine particles, which reduces the grout's ability to penetrate small voids (Warner, 2004). Cohesive soils might not accept grout at all (Karol, 2003). Figure 10-4 shows an example of the penetrability of different grout materials for different grain sizes (Powers *et al*, 2007).

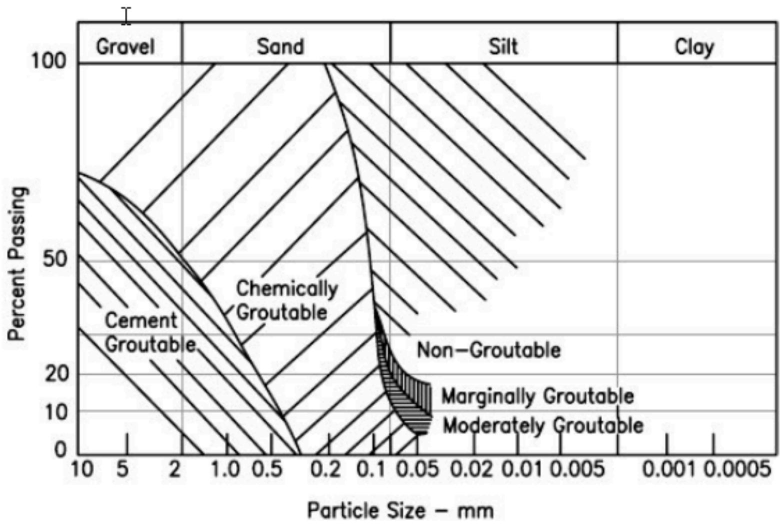


*Figure 10-4 Grout material penetrability in soil shown in a grain size distribution diagram, from Powers et al (2007).*

Heterogeneous soil conditions, such as layers of more permeable features, can be difficult to treat since the grout then penetrates the most permeable layers more easily and less permeable material risk to be left ungrouted or partly ungrouted (Powers *et al*, 2007). If there is a high groundwater gradient fine material like silt or silty sand in such ungrouted spots can be eroded the material can then be moved of washed out. In heterogeneous conditions tighter grout hole spacing is needed to obtain a proper sealing efficiency (Warner, 2004). The results of the permeation grouting can be verified by hydraulic tests in situ (Powers *et al*, 2007).

The groutability of a soil with a chemical grout depends on the viscosity of the grout, the grouting pressure and grouting time, and the permeability of the soil (Akbulut and Saglamer, 2002; Powers *et al*, 2007). For cement grout, the groutability depends on the size of the opening available for grout to penetrate as well as the grain size of the grout. The groutability ratio of a cement grout is defined by a rule of thumb as $D_{15}^{soil}/d_{85}^{grout}$ (Powers *et al*, 2007, Axelsson *et al*, 2009). $D_{15}^{soil}$ is the soil grain size of which 15 per cent of the particles are smaller and $d_{85}^{grout}$ is the cement grain size of which 85 per cent of the cement particles are smaller. Also the ratio between $D_{10}^{soil}$ and $d_{95}^{grout}$ is sometimes used. The intervals of groutability for the two rules of thumb are given in table 10-1.

*Table 10-1 Interval of groutability for rules of thumb criteria, modified from Axelsson et al (2009) based on Mitchell (1982).*

| Criterion | Groutable | Not groutable |
|---|---|---|
| $\frac{D_{15}^{soil}}{d_{85}^{grout}}$ | > 24 | < 11 |
| $\frac{D_{10}^{soil}}{d_{95}^{grout}}$ | > 11 | < 6 |

Akbulut and Saglamer (2002) add water cement ratio, grouting pressure, amount of fines and relative density of the soil as important factors in predicting the groutability of a soil with a cement grout. This means that their groutability ratio takes additional soil properties into account but also the grouting technique. The groutability ratio modified by Akbulut and Saglamer (2002) needs to be higher than 28 for the soil to be groutable:

$$\frac{D_{10}}{d_{90}} + k_1 \times \frac{w/c}{FC} + k_2 \times \frac{P}{D_r} > 28 \qquad \text{Equation 25}$$

*w/c* is the water to cement ratio of the grout, *FC* is the fines content of the soil, *P* is the applied grouting pressure, $D_r$ is the relative density of the soil and $k_1$ and $k_2$ are constants. The empirical expression in Equation 25 shows in what way different soil parameter, grout parameters and grouting pressure affects the penetrability of the grout into the soil. It can be seen for example that the higher the soil fine content is, the higher the water cement ratio needs to be; the higher the relative density of the soil is (i.e. the more compact the soil is), the higher the grouting pressure needs to be. More detailed description of the impact of the different parameters is given in Akbulut and Saglamer (2002).

Sometimes also the hydraulic conductivity of the soil is used as an indication of the penetrability of grout into it (Lees and Chuaqui, 2003). The hydraulic conductivity and the empirical groutability ratios together provide a first, rough estimate of the groutability of the soil and what grout material that might be needed, i.e. cement, microcement or chemical grout. Also one-dimensional grout injection tests have been used for assessing the groutability of sands with microcmement (Mittag and Savidis, 2003). According to Mittag and Savidis (2003) such tests do not give representative penetration lengths for true grout spread (which commonly is two- or three-dimensional).

Axelsson *et al* (2009) have developed an equation giving the maximum penetration length of a Bingham fluid, i.e. cement grout, into a granular medium, the derivation can be seen in Axelsson and Gustafsson (2007). The penetration length is governed by the grouting pressure, the shear strength of the grout and the porosity and specific surface of the soil as follows:

$$I_{max} = \frac{\Delta p}{2 \times \tau_0} \times \frac{8}{\pi \times (1-n) \times S_0} \quad [\mathrm{m}] \qquad \text{Equation 26}$$

In Axelsson *et al* (2009), Bingham grout penetration into frictional material, i.e. sand, was studied. The size of the pore voids available for grout penetration was expressed as a theoretical aperture of the porous medium. Theoretical aperture was chosen instead of for example porosity or hydraulic conductivity since an aperture can be directly compared to the size of the grout particles. The theoretical aperture can be expressed based on the Kozeny-Carman equation for hydraulic conductivity or by the fictitious aperture developed by Bergman (1970). The Bergman expression for aperture only takes the median grain size of the material into account, as Kozeny-Carman includes several properties (described in Chapter 7.3). Both the Bergman and Kozeny-Carman are developed for a Newtonian fluid, i.e. water. Axelsson *et al* (2009) also used Equation 26 to describe the theoretical aperture by comparing Equation 26 to the expression of grout penetration into a fracture in Equation 22. The latter term in Equation 26 thus gives the available aperture, $b_{eqv}$, for a Bingham fluid as:

$$I_{max} = \frac{\Delta p}{2 \times \tau_0} \times \frac{8}{\pi \times (1-n) \times S_0} = \frac{\Delta p}{2 \times \tau_0} \times b_{eqv} \quad [\mathrm{m}] \qquad \text{Equation 27}$$

### 10.2.2 Jet grouting

Jet grouting is performed with high velocity jets from a jet pipe, which is rotated and lifted slowly (Powers *et al*, 2007). Soil is eroded by the jets and mixed with the cement grout material with a radial spread, creating a column. There are different kinds of system for jet grouting performance: single fluid system with only cement being injected, double fluid system with cement and compressed air, and triple fluid system with water additionally (Nikbakhtan and Osanloo, 2009). The triple one offers the largest possibility of controlling the result. The general procedure is shown in Figure 10-5 below: A jet grout hole is first drilled, the equipment is tested, a column is created and then several columns created in a pattern.

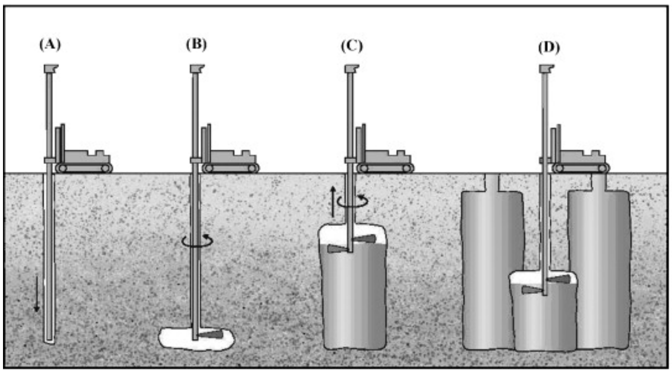


*Figure 10-5 Creation of jet columns, picture from Nikbakhtan and Osanloo (2009).*

Columns can be created in any direction, vertically or inclined and columns placed in parallel rows can in theory create a complete groundwater cut-off, depending on the design, the mixed soil and concrete material and the grouting performance. Jet grouting is less sensitive to soil properties than permeation grouting due to the high velocity injection (Powers *et al*, 2007). All soils to which a jet pipe can be drilled are suitable for jet grouting. However, the penetration is shorter in cohesive soils (Nikbakhtan and Osanloo, 2009) and there is a relationship between the column diameter and the recorded resistance from the geotechnical Standard Penetration Test. The performance can be optimized by changing grout pressure, lifting speed, rotation speed, water pressure, water flow rate, and water-cement ratio. These factors in turn need to be adapted to the soil at the site (Wang *et al*, 2012). Difficulties arise if the soil contains a lot of cobbles or boulders since the soil behind these obstacles might be left untreated (Karol, 2003; Powers *et al*, 2007). It is important to identify heterogeneous soil conditions on beforehand of execution in order to adopt the design, it is especially important to identify any soils that can erode since even a small point of leakage through the jet grouting wall can give erosion (Essler and Yoshida, 2004). High groundwater flow velocities can be problematic since the cement can be washed away before it has stiffened (Powers *et al*, 2007).

In the case that the columns are created at a shallow soil depth it is possible to excavate some soil and investigate the result; at larger depths this is not possible and the resulting sealing efficiency must be verified by test pumping (Essler and Yoshida, 2004). Due to the soil conditions mentioned above, the resulting jet columns will have varying hydraulic properties since the columns consist of a material mixed from the soil at site and the concrete. There can also be deficiencies for example due to insufficient overlap of columns or not properly drilled holes for injection. The conditions might be highly variable and the result of the jet grouting design and field trials are sometimes the only way to assess achieved column diameter and quality (Wang *et al*, 2012). Field trials as control for execution efficiency of jet grouting is also proposed by in European standards (Arroyo *et al*, 2012; EN 12716, 2001). According to Tekin and Akbas (2010) and Arroyo *et al* (2012) there is a lot of handicraft experience from jet grouting but yet no universal, science based methodology. Arroyo *et al* (2012) applied jet grouting design to a tunnelling project based on a probabilistic approach. A cumulative frequency distribution of the minimum thickness of the jet column structure was created. It was based on column diameter and column alignment, which together constitutes the thickness of the structure.

A jet grouting design needs to include detailed geometrical aspects of placement and alignment of columns, as well as execution parameters and grout properties (Essler and Yoshida, 2004). Real time data such as pressure, material flow and drilling and jetting rotation shall be logged during execution.

## 10.3 Soil rock interface tunnelling and sealing

Excavations are needed at locations where tunnels or their access ramps pass between soil and rock and the rock tunnel construction and the soil tunnel construction needs to meet. An example of a typical tunnel construction at a soil rock interface is seen in Figure 10-6 below, consisting of a trough, a concrete tunnel, a concrete tunnel with bedrock bottom and finally a bedrock tunnel. The final tunnel design through the interface can also, instead of a concrete tunnel with bedrock bottom, consist of a concrete lining that stretches between the two different constructions. Using a concrete lining allows for both making the construction at the interface water-tight and stable; the length of the lining is made sufficient to reach bedrock quality with enough stability. However, large excavations in which the construction needs to take place are likely to be challenging concerning groundwater inflow, especially if the excavation cuts through highly permeable soils or water-conducting fracture zones in rock (Banverket, 2006b;

Banverket, 2007a). Parts one, two and three in Figure 10-6 all needs to be constructed in an excavation, part three needing the greatest depth in this example.

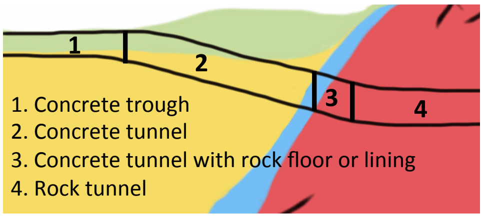


*Figure 10-6 Tunnel construction at a soil rock interface, picture after Banverket (2006a).*

Permeable material layers below the excavation bottom can inherit a risk of bottom heave and the bottom then needs to be stabilized (Banverket, 2006b). Lowering the groundwater pressure in the permeable layer reduces the risk but the pressure decrease cannot be allowed to spread over a large area and needs to be limited to the excavation area. This demands watertight excavation walls and bottom, else pumping water from inside the excavation can give a widespread groundwater drawdown outside the excavation. Sealing is preferably performed in advance since any post-sealing measures in the already excavated shaft can be difficult due to large groundwater gradients (Persson, 2007).

Even though the excavation is sealed there is likely to be some pathways for water to seep or flow as will be described later. The flow is described by Darcy´s law (Equation 1) and depends on the groundwater gradient acting over the wall. If the seepage forces from the flowing groundwater are high, unconsolidated granular material can be internally eroded and if flow velocities are high enough the eroded material can be washed out (Persson, 2007). Such out-wash of material can lead to significant ground settlements outside the excavation. Infiltration is often used as a countermeasure to raise the groundwater table after it has been lowered due to underground construction and is relatively often needed also as a permanent action throughout the operational phase of the construction (Banverket 2006a; Banverket, 2007a; Persson, 2007). Such infiltration, if not handled correctly, can however trigger erosion and out-wash of material (Persson, 2007). Also installation of retaining wall anchors can erode any present frictional material since high-pressure air or water is used for the installation. Persson (2007) emphasizes the use of observation wells and a monitoring program measuring groundwater levels as necessary in order to avoid negative consequences, both due to excavation inflow and infiltration.

The location of the bedrock surface is important to know, especially in the surrounding of soil rock interfaces since it is important to make retaining walls and their deficiencies water-tight (Banverket, 2006b; MÖD, 2010). It can however be difficult to distinct actual bedrock from a compact layer of coarse material or randomly distributed boulders (MÖD, 2010). Finding the actual depth of the bedrock surface and driving of sheet pile walls to a sufficient depth is identified as a large uncertainty for example in the Gothenburg project Västlänken (Banverket, 2006b). Where frictional material is present above the bedrock this soil layer needs to be sealed since it is likely that the sheet pile wall, diaphragm wall or secant pile wall will not provide sufficient water cut off. The upper part of the bedrock can also be more densely fractured than the intact rock as described in Chapter 6.2, and might also need to be sealed. Thus both soil grouting and rock grouting is needed. It is particularly important to identify and investigate the hydraulic features of any water-conducting deformation zone in the proximity of the tunnel soil rock interface (Banverket, 2006a). The above described issues are visualised in Figure 10-7 where water can flow through the frictional material or through the bedrock fractures.

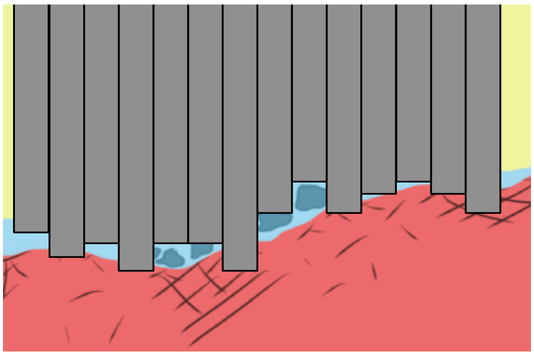

*Figure 10-7 Example of a sheet pile wall driven to bedrock. Coarse frictional material (blue colour) and boulders might lead to that the wall does not reach the bedrock.*

Jet grouting of the frictional soil creating a wall of jet columns together with permeation grouting of the bedrock is one solution (Banverket, 2006b; Banverket, 2007a; MÖD, 2010). The geological conditions at the site decide what technical solutions and sealing effort might be needed. In order to adopt an appropriate design, the site needs to be thoroughly investigated prior to the construction work begins (Banverket, 2006b). The efficiency of the sealing measures needs to be verified by for example test pumping (Banverket, 2006b; Persson, 2007). However, even if sealing is performed there can still be leakage into the excavation. Persson (2007) identifies possible paths for inflow to an excavation illustrated in Figure 10-8. Additionally, any water-conducting fracture zone present nearby or within the excavation would give an increased risk of groundwater inleakage (Banverket, 2007a).

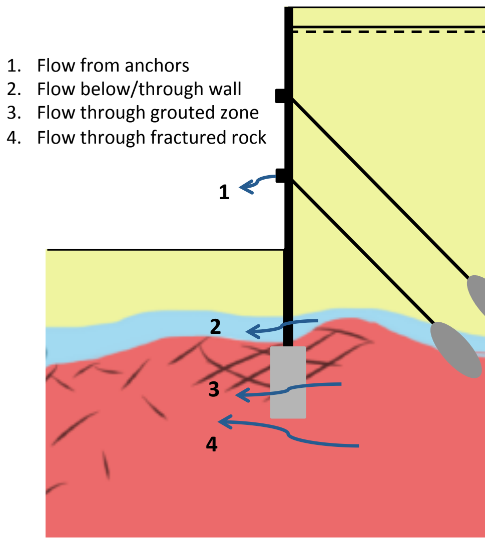


*Figure 10-8 Paths of water inflow to an excavation, modified from Persson (2007).*

The bedrock in Figure 10-8 has an almost horizontal surface. In the case of a soil rock interface the bedrock surface has a more or less significant inclination; this could add to the difficulties with getting a tight interface between the retaining wall and the bedrock. There could potentially also be higher in situ groundwater gradients at the soil rock interface depending on the ground surface topography (Banverket, 2006a).

### 10.3.1 Key material parameters

Persson (2007) identified key parameters for groundwater drawdown due to inleakage into an excavation or to a tunnel. These key parameters applies also to soil rock interfaces in this thesis and are summarized from Persson (2007) as:

- The hydraulic conductivity of a deficiency in the jet grouting wall or the retaining wall governs the amount of inleakage to the excavation. The transmissivity of the aquifer governs the groundwater drawdown. Aquifer transmissivity, aquifer storage, leakage through aquitard and groundwater recharge at boundaries govern the size of the area influenced area.
- The pre-consolidation pressure is crucial to the severity of the groundwater impact and needs to be determined high-qualitatively as well as the in situ stress and pore pressure distribution.
- The hydraulic conductivity of the clay is important in case the aquifer has a limited recharge and leakage from the clay provides a large contribution of water to the aquifer.

### 10.3.2 The operational phase

The text above described soil rock interface tunnelling during the construction phase of the project and to some extent also investigation and design phase. The operational phase of a project is not within the scope of this thesis but some aspects are pointed out in Banverket (2006a) as potentially problematic in the operational phase. One technical concern is the water-tightness of the joint between the final concrete tunnel and rock tunnel. A managerial concern is to maintain an efficient control programme with measurements, evaluation and quick implementation of countermeasures if needed. Additionally, processes initiated during the construction phase might be slow and give problem later on, such ground settlement due to consolidation (Banverket, 2006a; Banverket, 2007a; Persson, 2007).

# 11 Conceptual model of a soil rock interface

A conceptual model can be said to constitute a relevant set of assumptions to describe a system for a given purpose (Olsson *et al*, 1994). Zetterlund (2009) states that it is important to have a conceptual model in the early phases of a project design and that pre-investigation need to be based on project specific questions and problems. It is beneficial if investigations are based on a site geology model, which is updated with results from investigation, instead of creating a model after the investigations are made (Kvartsberg, 2013a). The use of a problem-specific conceptual model can facilitate adequate investigation, interpretation and design.

The conceptual model developed in this thesis aims at providing a basis for decision of needed pre-investigations and subsequent choice of sealing strategy and sealing design for tunnel excavation at soil rock interface. The conceptual model in Table 11-1 is presented in a format developed by Olsson *et al* (1994) for the Äspö Hard Rock Laboratory project and the original format is seen in Appendix 1. A conceptual model includes processes and parameters of importance as well as the geometrical framework for a specific purpose. No specific data is given in the conceptual model but used when the conceptual model is realised (Gustafson, 2012). The relationship between the conceptual model and its realisation is seen in Figure 11-1.

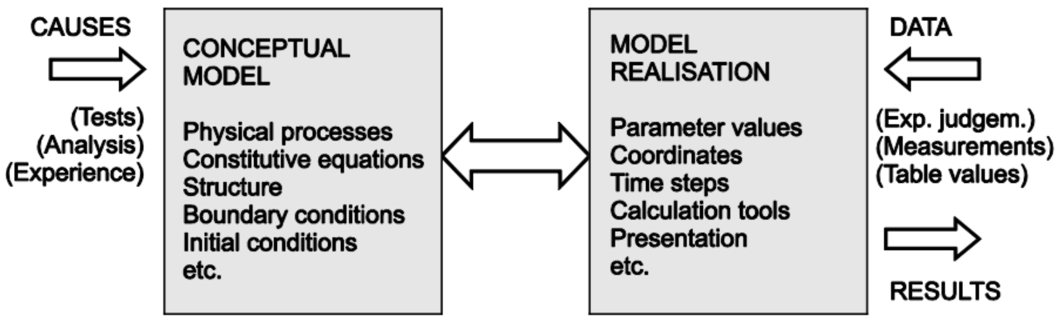


*Figure 11-1 Relationship between the conceptual model and its realisation, from Gustafson (2012).*

A conceptual model is thus based on identified processes governing the phenomena of interest for a specific purpose or a given application. In this thesis, the conceptual model in Table 11-1 is based on findings and conceptualisation from the literature review, compared to 'Tests, Analysis, Experience' on the left hand side of Figure 11-1. The parts dealing with settlement are to a large extent inspired by Persson (2007) who studied settlements caused by inleakage to excavations and tunnels from a hydrogeological standpoint. The purpose of the conceptual model developed in this thesis is to provide a tool for understanding, investigating and modelling:

- Settlement from consolidation of clay or erosion of frictional material due to inflow to the excavation at the soil rock interface, and:
- Sealing of the excavation at the soil rock interface.

The conceptual model thus aims at (i) providing a basis for estimation and analytical or numerical analysis of the sensitivity of the system to pore pressure changes (i.e. consolidation or erosion), and (ii) to evaluate the groutability of the soil and bedrock surrounding an excavation and perform an adequate sealing design. The bearing hypothesis is that the hydrogeological system response to pressure changes is crucial since pressure lowering leads to induced groundwater flow, which in turn can give erosion and outwash of frictional material, and consolidation settlement in clay depending on the magnitude of the pressure reduction.

The first rows describe the purpose or application of the conceptual model as well as the processes governing the phenomena of interest. Such processes are in this case considered to be groundwater flow fundamentally described by Darcy's law and mass-balance, consolidation theory for settlement in clay, geometric and hydraulic criteria for erosion susceptibility and onset of erosion with subsequent settlement, and coefficient of consolidation and aquifer diffusivity for time dependency of pressure changes in the hydraulic system. The columns are then divided into description of concepts and source of data, for sealing design and settlement respectively. Concepts and data are described for the geometry of the problem, the material properties, assignment of material properties throughout the geometry, and boundary conditions. The final rows suggest numerical tool for modeling and what the output parameters of interest for the actual application of the model are. The columns belonging to 'settlement' applies to consolidation and erosion and are suitable for modeling.  The columns belonging to 'sealing' within this thesis more at a stage of being a checklist for suitable investigations and consideration of geometries, and not a basis for modeling. Within future work and studies it could potentially be further developed into modeling of grout penetration for example.

*Table 11-1 Conceptual model of a soil rock interface. The framework for the model is developed by Olsson et al (1994). See list of notations for explanations of used denotations.*

| **SOIL ROCK INTERFACE MODEL** | | | |
|---|---|---|---|
| Settlement due to consolidation of clay or erosion and out-wash of frictional material<br>Excavation sealing design<br>Hydraulic and mechanical processes governing consolidation, internal erosion and groutability | | | |
| Flow: Darcy's law, mass-balance Consolidation settlement: 1D consolidation theory<br>Pore pressure response: $c_v$ (clay), $D$ (frictional material) $D = \frac{K}{\gamma_w \times \beta_s} = \frac{T}{S}$, clay $c_v = \frac{K}{\gamma_w \times m_v}$<br>Erosion of frictional material: erosion susceptibility, flow velocity, hydraulic gradient | | | |
| **CONCEPTS** | | **DATA** | |
| *Settlement* | *Sealing* | *Settlement* | *Sealing* |
| **Geometric framework and parameters** | | | |
| Soil profile, soil depth/location of bedrock surface, thickness of soil layers, deformation zones, fracture frequency upper bedrock layer.<br>3D extent of above features. | | Desk study (topography, rock types, tectonics, deformation zones, soil types, use of type stratigraphies).<br>CPT/SPT in soil, drilling, geophysics, hydrogeological investigations. | |
| Thickness of clay.<br>3D extent of lower aquifer (along z-axis and in xy-plane). Size of seepage/flow pathways for inleakage to excavation | Material layers: thicknesses (z-axis), heterogeneity (z-axis and xy-plane).<br>Extent and direction of any deformation zone. | Size: Extent of lower aquifer, from pumping tests and interpretation.<br>Resolution: in excavation high, pathways of flow seepage. Else couples of metres (soil layers, def. zones) | Size: Excavation size and its proximity.<br>Resolution: representatively capture e.g. frictional soil boulders and thin layers of contrasting permeability soils. |
| **Material properties** | | | |
| $K_{bedrock}$, $K_{deformation\ zone}$, $K_{frictional\ soil}$, $K_{clay}$ | | K from lab tests and/or hydraulic tests. | |
| Clay: $\sigma'_c$, *OCR*, $c_v$, compressibility $m_v$.<br>Frictional soil: storage coefficient *S* and *K*, stiffness *M*<br>Deformation zone: *T*, stiffness *M*.<br>Out-wash of material: shape of grain size distribution. | Frictional soil: *K*, *n*, $S_0$, groutability ratio, fines content.<br>Rock: $T_{fractures}$, *b*, fracture frequency/density, fracture infilling. | Lab oedometer test for clay properties.<br>Pumping test for aquifer effective properties. Suitable hydraulic test or combination of tests for deformation zone *T*. | $S_0$ from grain size distribution, *n* from measurements.<br>Groutability ratio from grain size distribution.<br>*K* from laboratory test on samples or in situ slug test.<br>*T* from water pressure test or similar in rock → *b* |
| **Spatial assignment method** | | | |
| Each material layer or hydraulic domain is considered homogeneous.<br>*K* (soils, bedrock): e.g. lognormal distribution.<br>Large water-conducting deformation zones: deterministic (Gustafson, 2012)<br>Smaller water-conducting deformation zones: stochastically (Gustafson, 2012)<br>Deficiencies in grouting or gap below retaining wall: suitable average for the model cell (Persson, 2007) | | Interpolation between points of investigation.<br>Deformation zones: desk study, field visit + mapping, intrusive investigations, geophysics.<br><br>Grout or wall deficiency: homogeneous grouted zone → harmonic mean grouted zone and frictional soil, gap → arithmetic mean grouted zone and frictional material (Persson, 2007). | |
| $m_v$: normal<br>$\sigma'_c$: normal<br>*OCR*: normal or lognormal | *K*: lognormal<br>$T_{\mathrm{fractures}}$: Pareto<br>*b*: Pareto | Field sampling and laboratory measurement. | K from sampling and laboratory measurements, or in situ slug test and |

<table>
<tr><td>Suggested distributions of clay properties from Persson (2007).</td><td>Suggested distributions from Gustafson (2012) amongst others.</td><td></td><td>pumping test. T and b from in situ test.</td></tr>
<tr><td colspan="4">Boundary conditions</td></tr>
<tr><td colspan="2">Aquifer connectivity at location of excavation.<br>Distance to hydraulic boundaries.<br><br>Pore pressure distribution with depth.<br>Depth of excavation/amount of pressure relief → gradient.<br>Boundaries (Persson, 2007):<br>– No flow (impermeable)<br>– Constant head (water level)<br>– Infiltration (artificial)/groundwater recharge<br>– Leaky aquifer (OC clay)<br>– Leaky compressible aquifer (NC clay)<br><br>Large water-conducting deformation zone and its recharge possibly also relevant to consider.</td><td colspan="2">Aquifer connectivity interpreted from several slug tests. Distance to boundaries from test pumping and suitable analytical evaluation.<br>Interpretation of boundaries from desk study (physical borders) and pumping test and evaluation.<br>Recharge from hydrological data.<br>Excavation depth: changes during construction.<br>Piezometer measurement or similar of pore pressures and initial distribution with depth.</td></tr>
<tr><td colspan="4">Numerical tool</td></tr>
<tr><td colspan="4">Steady state analysis to assess what inflow corresponds to what drawdown and what conductivity of the grouted zone in soil is needed (Persson, 2007). Sensitivity analysis with the steady state model to investigate different parameters' influence. Analytical evaluation for estimation of influenced area (depends on T and S of aquifers, leakage and recharge at boundaries) and time dependency of pressure changes the aquifer (Persson, 2007).<br>Steady sate and transient analysis to evaluate development, magnitude, direction and distribution of hydraulic gradients (and flow velocities) due to inflow to excavation and also due to infiltration in wells.</td></tr>
<tr><td colspan="4">Output parameters</td></tr>
<tr><td colspan="4">Pore pressure (response with time, size of $\Delta u$ and how fast and far it develops), radius of influence to inleakage to excavation. Groundwater drawdown, settlement.<br>Hydraulic gradients and flow velocity, for different excavation depths (interpretation gives indications of risk of soil erosion and potential difficulties in sealing execution).<br>Effects of different potential sealing results/efficiency.<br>Importance of different parameters (sensitivity analysis). Allowable infiltration rate and pressure.</td></tr>
</table>

## 11.1 Discussion and concluding remarks

The literature review indicates that it is difficult to transmit knowledge between stakeholders and project phases in a large-scale, complex underground project such as tunnelling in urban environment. A lot of knowledge about the hydrogeological environment is for example gained during the investigations for the Environmental Impact Assessment document and suggestions of regulatory inflow requirement, but perhaps not transmitted throughout the project and communicated to stakeholders to whom potential risks are important to be aware of. Introducing a conceptual model in the start of the project could aid planning investigations in order to obtain needed data. As more information becomes available the model is updated and can follow throughout the project phases, thus transmitting important knowledge and information. This is in line with what for example Zetterlund (2009) and Kvartsberg (2013a) suggest. Continuous meetings between stakeholders throughout the design and construction planning phase and particularly the construction phase can be suitable in order to update the conceptual model. Meetings and cooperative updating of the conceptual model could also provide a platform for a collective knowledge and understanding of new information concerning the soil rock interface.

Using a conceptual model presented in this format provides a structured way of handling information and the model can be the basis for communication of important processes, parameters and technical aspects between stakeholders. Hansson *et al* (2010) proposes that technical aspects concerning measurement of inflow shall be discussed as early as possible when a contractor is engaged. Important processes, geometries and material properties suggested in the conceptual model in Table 11-1 could in a likewise manner also be raised and discussion early in order to increase involved stakeholder's understanding of potential problems and risks. This is in accordance with the early workshops proposed by Bröchner *et al* (2006).

The conceptual model is a basis for modelling and could be used within the work with the Environmental Impact Assessment document to assess impacts to the surroundings, for example making water-balance calculations and assess groundwater drawdown and inleakage. The conceptual model and numerical modelling could perhaps also be used within the work for suggestion an inflow requirement to improve the transparency of that work; Werner *et al* (2012) saw in their study that inflow prognoses seldom are based on conceptual models and that it often it is unclear what parts of the hydrogeological system is being modelled. The conceptual model in Table 11-1 can be used to give a structured and transparent basis for the modelling. Relatively simple modelling could also be performed to assess needed demand of sealing of the soil rock interface excavation. It can also indicate what processes are of potential concern and need to be communicated clearly in procurement documents and contracts.

The most significant feature of a soil rock interface from a sealing point of view is the heterogeneity of the hydraulic system, i.e. relatively stiff bedrock, any deformations zone, frictional soil of different type and degree of permeability, and low-permeable clay. With the bedrock types found in Scandinavia there is a distinct difference in the permeability structure in bedrock compared to soil. The conceptual model in Table 11-1 could be applicable to other situations where the same characteristics govern potential hydrogeological issues. One example of such is a bedrock tunnel passing a deformation zone below a soil-filled valley as seen in Figure 11-2. In that case settlement can be evaluated using the conceptual model above. For any deep excavation through soil down to bedrock, which according to Werner *et al* (2012) has turned out to often impact groundwater conditions, the concepts of sealing in the conceptual model can be used. In fact the conceptual model in this thesis could be applied to any excavation in contact with water-conducting frictional material, whether it is a tunnelling project or not.

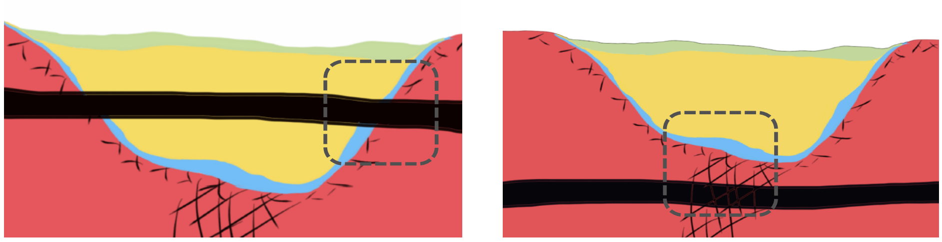

*Figure 11-2 The black band illustrates a tunnel stretching through a clay valley between bedrock outcrops. To the left a typical soil rock interface marked by dashed box. To the right a situation with similar characteristics, the conceptual model in Table 11-1 with settlement application can be used.*

Due to the significant hydrogeological features of a soil rock interface and the potential risks for negative groundwater impact and subsequent costly risk of exceeding the regulatory inflow requirement, it might be efficient to treat soil rock interfaces explicitly within a project. Treating soil rock interfaces explicitly within a project can be beneficial both during site investigations, environmental impact assessment, assessment of economic costs and sealing design, and during preparation of procurement documents, contractual risk allocation and economic compensation. The reason for treating soil rock interfaces explicitly is that the hydraulic characteristics during construction deviate from other parts of a tunnel (i.e. blasted bedrock tunnel and concrete tunnel constructed in open soft soil cuts) and the sealing design thus must be treated differently. There are also other potential hazards than typically encountered in other tunnel parts, such as direct contact with the lower aquifer and risk of erosion of frictional material. If there are several soil rock interfaces in a project, they have the potential to together contribute to large projects uncertainties concerning fulfilment of inflow requirement, needed sealing effort and subsequent project costs.

One way to handle soil rock interfaces explicitly as described above in practice could be to handle them as a design class. In similarity to bedrock, which preferably is divided into classes for tunnel design as described in Chapter 10.1, soil rock interfaces could constitute a design class with its characteristic hydraulic features differing significantly from rock classes. The term ‘design class’ here applies to the application of sealing design. Kvartsberg (2013a) suggests in her work of bedrock pre-grouting design that the term ‘hydraulic domain’ preferably is used since it accentuates that hydraulic features are the once of interest for sealing application. In accordance with the terminology of hydraulic domains used in Figure 8-1, a suitable denotation of a soil rock interface class could be hydraulic soil-rock unit.

Furthermore, dealing with soil rock interfaces as a separate class is also in line with what Kadefors and Bröchner (2008) amongst others identifies as beneficial for contractual risk allocation, which in turn is needed for successful implementation of the observational method in Swedish tunnelling projects. Handling soil rock interfaces explicitly in a tunnelling project, and structure the work by using the conceptual model in Table 11-1 (or parts of it), can also be useful when establishing predicted behaviour and acceptable limits within the observational method, see Figure 4-3 by Kvartsberg (2013b) modified from Schubert (2010).

The observational method as design method is probably most suitable for pre-grouting design of rock tunnel since an optimized pre-grouting design demands measurements and observations during construction. The application of the observational method to soil rock interfaces is most clear during the investigatory phase of a project (establishment of system behaviour, most likely and most unfavourable conditions). Parameters needed for a suitable excavation sealing design need to be obtained during the site investigations. Parameters to observe during construction could be groundwater levels at several radial distances from the excavation, measurement of vertical deformation and infiltration rates and pressure if infiltration is used. Other parameters that need to be measured are grouting parameters during grouting performance. This is needed both as a quality control (as identified in the literature review) but possibly also as a way to change and optimize the soil grouting design depending on the actual response, i.e. site conditions. Exactly what parameters to measure during grouting performance needs to be further studied but could for example be grouting pressure, grout flow, grout volume with time.

# 12 Approach to early soil rock interface sensitivity assessment

Based on the conceptual model in Chapter 11 and the literature review, a suggestion of an approach to early soil rock interface sensitivity assessment is presented in this chapter. The approach is thus based on theory, the aim is however to provide a concrete suggestion of how essential features of a soil rock interface could be assessed. The conceptual model in Table 11-1 includes relevant concepts and sources of data for analytical or numerical analysis and can be seen as a checklist for information to obtain, verify or reject throughout the project. With a lot of data and information gathered it is also a basis for numerical modelling. However, sensitivity of a soil rock interface might need to be assessed early in the project process in order to plan appropriate investigation and treatment of the interface. It might also be useful within contractual risk allocation and economic estimates of project costs (in accordance with Kvartsberg (2013a), Kvartsberg *et al* (2013) and Kadefors and Bröchner (2008)). This is particularly the case if the observational method is used as design method since an interval of probable system behaviour, including the most unfavourable and most likely behaviour, needs to be assessed early as described in Chapter 4.1. At such early phases a lot of data in the conceptual model in Chapter 11 is by nature lacking. Instead a condensed version of the conceptual model, specifically focusing at early assessment of soil rock interface sensitivity is provided in this chapter.

The early approach to soil rock interface sensitivity assessment suggested in this thesis includes following main aspects:

- Geometries and initial conditions
- Boundary conditions
- Connectivity of the frictional material
- Hydraulic conductivity
- Erosion susceptibility of frictional material
- Groutability of frictional material
- System-response in time

The hypothesis is that hydraulic conductivity, aquifer storage coefficient, aquifer transmissivity, boundary conditions and connectivity of the aquifer together states the system behaviour to pressure changes, i.e. the extent and amount of consequences of excavation inleakage and also the system response in time. Local properties of the frictional material that are considered to be of great importance are hydraulic conductivity, susceptibility to erosion and groutability. The term local here indicates the direct vicinity of the excavation. This hypothesis of division into local properties and more overall system properties is seen in Figure 12-1.

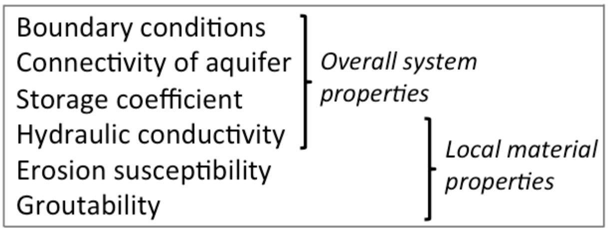


*Figure 12-1 Essential properties included in the early approach to soil rock interface sensitivity assessment, divided into overall system properties and local properties at excavation size.*

The local properties will be the main focus in the following case study according to the third objective of this thesis, presented in Chapter 1.1. A hypothesis is that all three properties can be evaluated or at least indicated from a grain size distribution curve.

Subchapters 12.1 to 12.7 describe possible ways to approach and estimate each of the factors in the list above. Finally, in subchapter 12.8 this is summarized into the condensed conceptual model for early assessment of soil rock interface sensitivity. Depending on the combination of factors included in this suggested sensitivity assessment, tunnelling at a soil rock interface becomes more or less difficult. Subchapter 12.9 provides a synthesized comment on possible interpretation of the outcome of the condensed conceptual model and also some remarking comments on the suggested approach to early soil rock interface sensitivity assessment.

## 12.1 Geometries and initial conditions

Geometries here apply to the soil stratigraphy and relative thicknesses of soil layers, the depth to bedrock, the ground topography and bedrock topography, initial groundwater head and hydraulic gradients. At least an idea of what geometries and materials to expect at the side is needed and can be obtained from geological maps, earlier investigations and similar. The type stratigraphy in Chapter 6.1, Figure 6-1 can also be used. In addition to these geological geometries also the depth of the excavation is important to consider since this will create new hydraulic gradients, which governs induced groundwater flow to the excavation. The hydraulic gradient also governs internal erosion and impacts any performed sealing, e.g. needed grouting pressure and the risk of erosion of fresh grout (see for example Axelsson (2009) on erosion of grout).

## 12.2 Boundary conditions

The boundary conditions for the lower aquifer could be evaluated from pumping test data. In this case the regional hydraulic properties of the aquifer are of interest. Two methods from Chapter 9.4.2 for evaluation of boundaries are suggested here:

- Semi-logarithmical plot of logarithm of time versus drawdown, according to Cooper-Jacob approximation.
- Logarithmical plot of logarithm of time versus logarithm of drawdown according to Theis type curve.

By performing a large-scale pumping test and plot the obtained data in a semi-logarithmic diagram as *log t – s* (where t is the time of pumping and s is the groundwater head drawdown), some things can be said about the boundaries. Evaluation of pumping test data also allows for determination of aquifer hydraulic properties such as storage coefficient and transmissivity (which with known material thickness also reveals the hydraulic conductivity of the frictional material, further described in Chapter 12.4).

According to the principle of superposition, a negative boundary is seen in the semi-logarithmic plot as the rate of drawdown increases to the double (Carlsson and Gustafson, 1997; Persson 2007). A positive boundary would give a constant drawdown (rate of drawdown equal to zero). A positive boundary could be a lake, a river, a highly transmissive fracture zone, or similar hydraulic feature. Both these situations, with a negative boundary and a positive boundary influencing the drawdown, are seen in Figure 12-2. Significant leakage from a compressible overlying aquitard would cause the plot to deviate from its ideal, linear shape.

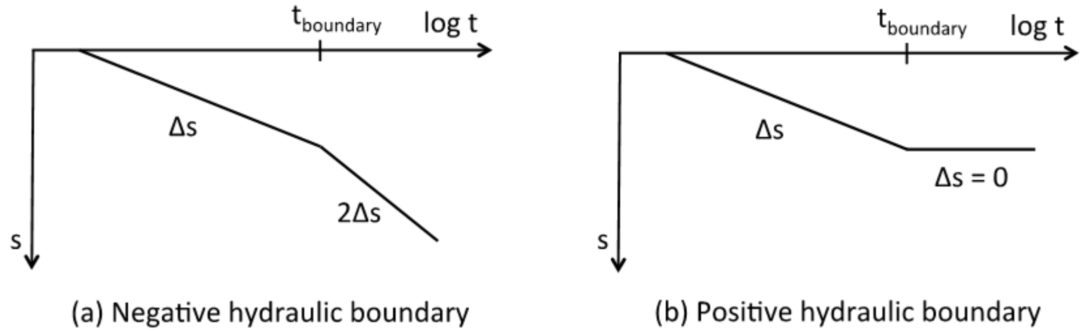


*Figure 12-2 Semi-logarithmical* log t - s *plots of drawdown data from a well. At time t-boundary a hydraulic boundary starts to influence the drawdown, in (a) a negative boundary and in (b) a positive boundary. Figure modified from Carlsson and Gustafsson (1997).*

From these plots the effective transmissivity of the aquifer, *T*, as well as its storage, *S*, can be evaluated according to Cooper-Jacob's approximation (Carlsson and Gustafsson, 1997):

$$T = 0.183 \times \frac{Q}{\Delta s} \quad [\mathrm{m^2/s}] \qquad \text{Equation 28}$$

$\Delta s$ is the drawdown over one decade of time. The storage is evaluated as:

$$S = \frac{135 \times T \times t_0}{r^2} \quad [-] \qquad \text{Equation 29}$$

$t_0$ is the time where the plot line intercepts the *log t*-axis and $r$ is the distance from the well, i.e. the distance between the well and the observation well the data is recorded from. With *S* and *T* evaluated, the aquifer diffusivity is obtained with Equation 15. The radius of influence, i.e. the radial extent of the drawdown from the well, at the time t (in minutes) can then be evaluated as:

$$R = \sqrt{\frac{135 \times T \times t}{S}} \quad [\mathrm{m}] \qquad \text{Equation 30}$$

This measure gives an indication of how large area will is affected by a groundwater drawdown. As seen, the radius of influence is determined by the duration of the pumping (of inleakage to underground construction) and the aquifer diffusivity. When the parameters *T* and *R* are obtained, Thiem's well equation (in Table 9-2) can be used to calculate for example the drawdown at different distances from the disturbance.

The Cooper-Jacob's approximation is used for late time data since the early data do not fit to the linear relationship between drawdown and logarithm of time due to well storage (Persson, 2007; Carlsson and Gustafson, 1997). To evaluate earlier time data for the aquifer transmissivity and storage, a *log t – log s* plot and Theis type curve can be used. The earliest data can however still be affected by well storage and well skin, which in that case is indicated by a slope of 1:1 for the earliest data in a *log t – log s* plot. Late time data can deviate from the type curve depending on that boundaries are reached, as seen in Figure 12-3. An increasing rate of drawdown with a slope 1:2 indicated two parallel negative boundaries and thereby channelled flow. A confined aquifer totally surrounded by negative boundaries would get an even steeper slope of 1:1. If there is considerable leakage from the overlying aquitard, the drawdown might be balanced by this leakage and the rate of drawdown would then be zero, indicated by no slope of the plot.

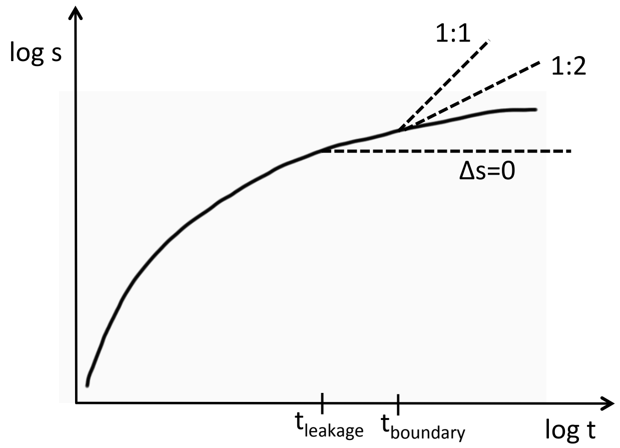


*Figure 12-3 Plot* log t - log s*, Theis type curve. Dashed flat line indicates leakage to aquifer, sloping dashed lines indicate negative boundaries (slope 1:1) and channelled flow (slope 1:2). Modified from Persson (2007).*

Hydraulic boundaries close to the pumping well can influence the evaluated hydraulic properties, i.e. $T_{eff}$ and $S$, of the aquifer. The same applies to observation wells that are located close to hydraulic boundaries. The main focus of this chapter is however the evaluation of boundaries of the lower aquifer and some tools are provided, based on Persson (2007) and Carlsson and Gustafsson (1997). As Persson (2007) states, true conditions might be significantly more complex than described here, but performing a proper pumping test (with observation wells both in the close vicinity of the pumping well and more far away, and frequent logging in the early phase of the test) is still essential for evaluating boundaries and the effective transmissivity of the aquifer.

If the boundaries are located relatively far away from the excavation site, the concept in Chapter 12.3 below applies. The radius of influence then needs to be compared to the distance to the hydraulic boundary; if the radius of influence extends beyond the hydraulic boundary, the hydraulic boundary will determine the shape of the drawdown curve. The theory of superposition and mirror wells can then be used to estimate groundwater drawdown. Evaluated aquifer parameters are in such a case, when the boundary has been reached, influenced by the boundary (Persson, 2007). The distance to the boundary can be evaluated by (Carlsson and Gustafson, 1997):

$$r_0 = r_1 \times \sqrt{\frac{t_1}{t_0}}$$ Equation 31

$r_0$ is the distance between the observation well and the pumping well, and $r_1$ is the distance between the observation well and the mirror well. $t_0$ is obtained from a *log t – s* plot and is the time at which drawdown line cuts the *log t* – axis. $t_1$ is the time at which the effect from the boundary starts to affect the drawdown ($t_{boundary}$ in Figure 12-3). The evaluated distance to the boundary is however relatively rough (Carlsson and Gustafson, 1997).

## 12.3 Connectivity of the frictional material

The spatial extent and continuity, i.e. the connectivity, of the frictional material constituting the lower aquifer is essential to the groundwater issues concerning soil rock interface since it governs the local transmissivity; if the frictional material has a certain hydraulic conductivity at a site the transmissivity can still vary significantly from spot to spot depending on the thickness of the frictional material layer. A relatively simple way to investigate the connectivity around a soil rock interface excavation would be valuable. The following suggestion is originally developed for bedrock fractures with varying fracture aperture and applies if the hydraulic boundaries are located far away from the well.

Fransson (1999) investigated analytically, by using Thiem's well equation, how a locally deviating transmissivity around a well affects the effective transmissivity within the radius of influence of the well. The well in this case is borehole used for a water pressure test of short duration and it was transmissivity of a bedrock fracture that was studied. The effective transmissivity, $T_{eff}$, is the transmissivity over the fracture within the radius of influence; since the fracture aperture is not constant but varies also transmissivity is different at different parts of the fracture. Figure 12-4 shows schematically the situation where a well or a borehole penetrates a fracture with a varying thickness or aperture:

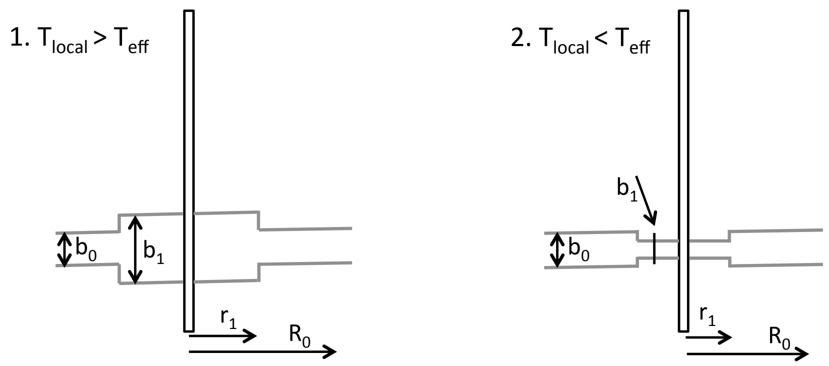


*Figure 12-4 Schematical view of wells cutting through locally varying thicknesses of frictional material. Modified from Fransson (1999). The leftmost sketch shows a locally thicker layer and the rightmost sketch shows a locally thinner layer.*

The aperture is $b_1$ along a radius $r_1$ closest to the well and $b_0$ at the distance between $r_1$ and $R_0$ from the well. $R_0$ is the radius of influence from the injection to the well and is likely to be rather small due to the short duration of the test. The conclusions in Fransson (1999) concerning local deviations in transmissivity are:

- When the local aperture $b_1$ around the well is larger than $b_0$, the transmissivity is somewhat larger than the effective transmissivity within the radius of influence and approaches the effective transmissivity as $b_1$ increases.
- When the local aperture $b_1$ around the well is smaller than $b_0$, the transmissivity is determined by the local aperture $b_1$.

For soil rock interface application, the well can be viewed upon as the excavation at the soil rock interface. The fracture aperture is instead the thickness of the frictional material. If the excavation is totally within a local deviation where the frictional material locally is thicker than outside, the first conclusion above will apply. This means that the effective transmissivity of the aquifer, $T_{eff}$, will dominate the hydraulic behaviour of the aquifer. If the excavation is totally within a local deviation where the frictional material is missing or significantly thinner than outside, the second conclusion above will apply. The different scenarios are shown in Figure 12-5 where the excavation location A is situated in a locally thicker layer of frictional material and excavation location B is situated within a spot of locally thinner frictional material.

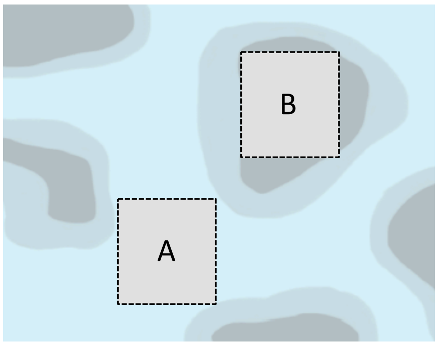


*Figure 12-5 Locations of excavation. Transmissivity for A will be the effective aquifer transmissivity and for B the local transmissivity will apply.*

A suggestion is that the effective transmissivity, $T_{eff}$, of the aquifer is obtained from a pumping test as described in Chapter 12.2. Since a large area is affected by the pumping test, the evaluated transmissivity is a measure of the effective transmissivity of the aquifer at regional scale (hydraulic boundaries can affect the result depending on the radius of influence of the test and the distance to the boundaries). With a slug test the local hydraulic conductivity could be obtained, representing the soil properties in the proximity of the test. With a number of slug tests around the area of the excavation to be, it can be seen if the excavation area belongs to a local lower transmissivity spot or if the effective transmissivity is predominant. If obtained transmissivities from the slug tests are lognormally distributed, the median value is representative for the effective aquifer transmissivity, $T_{eff}$ (de Marsily, 1986; Fransson, 1999).

Figure 12-6 shows an example of how obtained transmissivities from slug tests correspond the thickness of the frictional material (if the frictional material is assumed to have the same hydraulic conductivity over a larger area). The left part of Figure 12-6 shows the soil stratigraphy, bedrock surface and wells in profile, the right part shows the varying thickness of the frictional material indicated by colour; the darker the thinner.

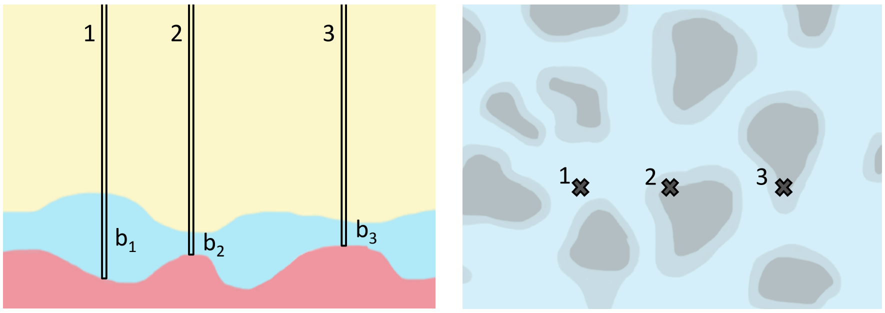


*Figure 12-6 Locations of slug tests and corresponding aquifer thickness governing the local transmissivity evaluated from the slug test data. Modified from Fransson (1999).*

As described, the hydraulic behaviour of the excavation site will thus vary depending on if the local transmissivity or the effective transmissivity is predominant. In Figure 12-5, the local, lower transmissivity will govern the hydraulic behaviour in case B whereas in case A the effective transmissivity of the whole aquifer will determine the hydraulic behaviour. The reason for knowing the transmissivity and whether $T_{eff}$ or $T_{local}$ applies to the excavation area is that something about the cone of depression, or the ground water pressure drawdown, can be said. In theory, with a relatively predominating $T_{local}$ the drawdown will be large in vicinity of the well or the excavation, but the radial extent of influence will be limited. If the aquifer $T_{eff}$ is predominant, the drawdown will be smaller but have a larger radius of influence.

## 12.4 Hydraulic conductivity

Hydraulic conductivity of frictional material has already been dealt with in Chapter 12.2 and 12.3 in terms of evaluation of aquifer transmissivity and connectivity. The hydraulic conductivity of the frictional material is essentially important to a soil rock interface excavation since it determines the groundwater flow (Darcy's law, Equation 1). The hydraulic conductivity in combination with the material layer thickness governs the aquifer diffusivity and the radius if influence of a groundwater pressure disturbance. Indirectly the hydraulic conductivity also affects the onset of internal erosion (flow velocity) in a given material. Groutability of a soil might be roughly indicated by the soil's hydraulic conductivity.

Due to the large variability of hydraulic conductivity of soils, site-specific investigation is needed. Sampling, sieving and evaluation from the grains size distribution curve or slug tests can be used. Gustafson's formula (Equation 5) can be preferred before Hazen's (Equation 4) since Gustafson's takes the shape of the grain size distribution curve into account in addition of the small grain size at $D_{10}$ (Andersson *et al*, 1984). Kozeny-Carman equation (Equation 6) might be even more representative (Svensson, 2014) but demands more work since several additional parameters are needed. It is anyway important that suitable equipment and sampling method is used so that both fines and large particle seizes are captured. The sedimentological history at the site can be important to bear in mind when planning sampling an evaluation results (Svensson, 2014). In situ slug tests gives results where also the degree of compaction and its effect on hydraulic conductivity is represented. A number of slug tests around the excavation to be can show whether the excavation will be situated in a local anomaly of lower transmissivity or if the effective aquifer transmissivity applies.

Hydraulic conductivity of clay is also important since it governs the rate of consolidation, i.e. the coefficient of consolidation (Equation 16). Sampling and laboratory measurement is needed to determine the coefficient of consolidation, which is needed in order to know for how long time a certain pressure lowering in the lower aquifer can be allowed (described in Chapter 12.7). However, in early project phases clay parameters from earlier experience and investigations in the area can be used.

## 12.5 Erosion of frictional material

Based on the literature review of internal erosion, Chapter 7.8, it is here suggested that theory from dam construction could be used to get an estimate of the risk for internal erosion of frictional material. However, it as has to be further studied to what extent the procedure in dam construction can be applied to this kind of infrastructure application (erosion due to gradient at excavation) and natural, in situ soils typically encountered. As described in Chapter 7.8, the susceptibility of a frictional material to internal erosion depends on the geometry of the grain size distribution curve. Thus, representative sampling needs to be done and the samples shall be sieved in lab in order to obtain the distribution curve. From the grain size distribution curve it can be evaluated whether a material is susceptible to erosion if:

- The material is gap-graded (see Figure 7-1).
- The material has a large share of fine ranging over several grain sizes, indicated by a flat tail of the distribution curve (see Figure 7-1 and 7-3).
- The material is susceptible to erosion according to Kezdi's criterion (Equation 19) and/or Kenney and Lau criterion.

Kezdi's criterion and Kenney and Lau's criterion are considered to be the most appropriate by Li (2008) and they are based purely on the grain size distribution curve, thus easily used once the grain size distribution curve is obtained. It shall though be remembered that these empirical criteria have been studied and verified based on materials typically used in dam constructions and perhaps not natural materials of glacial origin and depostision.

An erosion susceptible soil will only erode if the velocity or gradient is high enough, however this critical gradient can be very low for susceptible soils, as low as 0.2 (Jantzer and Knutsson, 2010). The geometry of the grain size distribution curve provides a first indication of whether the material is clearly susceptible or not. In order to further investigate the risk of internal erosion, the flow velocity or the flow gradient acting on the frictional material can also be evaluated. For example the hydraulic gradients towards an excavation to be can be estimated from expected or allowed drawdown. In a later phase, for example when the table in chapter 12.8 is filled in, some modeling of induced gradients and flow velocities due to the excavation could be performed. The results can then be compared to critical gradients of the material in question, either from literature values or from using criteria of erosion onset, e.g. presented and seen in Li (2008). This kind of modeling is not done in the case study in Chapter 13 but an estimation of hydraulic gradients is made.

Another important aspect is infiltration of groundwater as a countermeasure to drawdown (if the performed sealing is not tight enough) since this artificially infiltrated flow can erode material (Persson, 2007; Banverket, 2006a). The used infiltration pressure creates a gradient, which needs to be compared to critical gradient of erosion onset and also other gradients in the system. The infiltration pressure and flow rate can also be compared to the specific capacity, Equation 9, of the lower aquifer to indicate a suitable infiltration rate (Persson, 2007).

The initial pore pressure distribution and gradients are important to evaluate in a hydrogeological manner, as described in Chapter 12.1. Initially high gradients and potentially high gradients are of course less favorable than the opposite. It might be valuable to consider natural topography and the direction of groundwater seepage since downward seep direction decreases the critical gradient for onset of internal erosion (see Chapter 7.8). It might also be valuable to look at the hydraulic boundaries when estimating erosion; open hydraulic boundaries may contribute to sustaining gradient during the construction if the excavation is not totally sealed.

## 12.6 Groutability of frictional material

Sealing of the frictional material and uppermost part of the bedrock is indeed likely to be needed. For all kinds of soil grouting, it is important that the soil profile or soil stratigraphy is known relatively detailed, this is for example due to that any thin layer of higher permeability will inherit a significantly larger grout take than the actual soil to be grouted. Different permeability will also lead to different penetration lengths of the grout. Thus, an idea of the ratio of vertical hydraulic conductivity to horizontal vertical permeability is important to have. It can be obtained from every intrusive investigation of the subsurface conditions, from all geotechnical investigations as well as well installations of wells and similar. A measure of the relative resistance throughout the soil profile is thus valuable, indicating soil type, layering ad degree of compaction. It is thus important that every intrusive kind of investigation that is performed, both geotechnical and hydrogeological investigations, is logged and recorded. It would be very valuable if all intrusive investigations and installations are planned in a way that as much information as possible can be obtained. For example observations during installation of retaining walls can be used to verify or reject for example the expected bedrock surface or presence of boulders.

Independently of grouting method of the soil, the fines content of the soil is important to know (by proper sampling and lab evaluation to obtained the grain size distribution); the more fines, the lower penetrability grouts since the grout picks up fines as it moves through the soil (Warner, 2004). Presence of cobbles and/or boulders, as well as a highly variable bedrock surface, can give effects of shadowing and reduce the penetration length. If a grain size distribution curve of the frictional material is obtained, the groutability ratio for a cement grout into the soil can be obtained (see table 10-1). This can give a first indication of whether permeation grouting is possible with a cement grout or not. This is important to consider in order to choose a suitable grouting method (permeation or jet grouting) and grout material (cement, micro cement, chemical grout). The empirical groutability ratios only consider the finest shares of the soil but also the rest of the material and the whole pore space might be important to the penetration length of the grout. Thus, also the hydraulic conductivity might be useful indicating the groutability, as proposed by Lee and Chuaqui (2003). To further indicate and verify groutability and penetration length of a given grout in a specific soil, test grouting in situ and in lab is probably needed.

In summary, the groutability of the frictional soil is reduced or more difficult if (based on Chapter 10.2):

- The soil has a large share of fines, especially clayey fines.
- The soil is unsorted, containing significantly varying sizes of particles and/or stones and boulders. This might be difficult to capture representatively in sampling.
- The soil is heterogeneous and/or anisotropic since the grout spread will follow the most permeable features.

The first two are obtained from a grain size distribution curve, the latter one can be evaluated from logging and observation during intrusive investigations and sampling.

## 12.7 System-response in time

There are several aspects of time dependency to consider:

- The diffusivity of the aquifer, i.e. rate at which pressure changes throughout the aquifer (how fast a pressure change is spread).
- The consolidation rate in the clay.
- Duration of inleakage to excavation and duration of infiltration.
- Stages in construction (in what order actions like excavation and sealing are performed).

The rate of pressure response in the aquifer is governed by the diffusivity, $D=T/S$, which is obtained from pumping test and evaluation. The diffusivity affects how fast hydraulic boundaries are reached and also how fast any pressure lowering is spread in the aquifer below the clay layer. The larger spread, the larger area of clay is affected. The pumping test shall be carefully planned and performed so that as much information as possible can be obtained from the evaluation of it.

If the pressure lowering in the lower aquifer is large enough to exceed the pre-consolidation pressure of the clay consolidation will start. The amount of potential consolidation settlement depends on the amount of groundwater pressure reduction in relation to the pre-consolidation pressure, and the rate of consolidation is governed by $c_v$. $c_v$ and the amount of pressure reduction, i.e. amount of increase in effective stress, then determines how fast countermeasures need to be undertaken with respect to sensitive objects in the surroundings. It shall be remembered that the pre-consolidation pressure is an interval and not a distinct value (Persson, 2007). Additionally, for loads beyond the pre-consolidation pressure creep needs to be considered in the evaluation of deformations and when to undertake countermeasures (with respect to sensitive building and constructions).

The duration of inleakage to the excavation is important to consider. The longer duration, the more drainage of the lower aquifer and maintained pressure lowering in the lower aquifer, increasing the probability of consolidation settlement. Also the risk of erosion and following mass transport of fine material increases with increasing duration of inleakge; as described in Chapter 7.8 internal erosion can have an escalating development and a small leakage can get large effects with time, especially if large hydraulic gradients are maintained.

When it comes to construction, the last aspect in the list above, the general rule is that any sealing is more difficult to perform the larger pressure gradients there are (since higher applied grouting pressures are needed; there is a larger risk that the fresh grout is eroded (Axelsson, 2009) and using high injection pressure lead to jacking of fracturing, changing the flow patterns.

## 12.8 Conceptual model for early approach to soil rock interface sensitivity assessment

Based on the approach to soil rock interface sensitivity assessment above the conceptual model in Chapter 11 is condensed into Table 12-1, specifically developed for the purpose of assessing a soil rock interface in early project phases.

*Table 12-1 Conceptual model for early sensitivity assessment of soil rock interface.*

| **SOIL ROCK INTERFACE MODEL – EARLY SENSITIVITY ASSESSMENT** | |
|---|---|
| Assessment of boundary conditions and connectivity of aquifer.<br>Assessment of groutability and risk of erosion of frictional material.<br>Assessment of system-response in time of pressure changes.<br>Estimation of drawdown, radius of influence, gradient. | |
| Darcy's law, mass-balance, analytical solutions (Thiem, Cooper-Jacob, Theis), 1D consolidation theory<br>System-response in time: frictional soil $D = \frac{K}{\gamma_w \times \beta_s} = \frac{T}{S}$, clay $c_v = \frac{K}{\gamma_w \times m_v}$<br>Erosion of frictional material: geometric criterion of stability, shape of grain size distribution curve<br>Groutability: geometric criterion | |
| **CONCEPTS** | **DATA** |
| **Geometric framework and parameters** | |
| Soil depth, soil types and stratigraphy.<br>Topography. Bedrock type and topography.<br>Deformation zones. Hydraulic boundaries (lakes, rivers). | Desk study: geological maps e.g. from the Geological Survey of Sweden. Other maps. Site visit. Previous investigations. |
| **Material properties** | |
| Hydraulic conductivity of aquifer.<br>Stability to internal erosion.<br>Groutability of frictional material.<br>Diffusivity and connectivity of aquifer.<br>Hydraulic conductivity and coefficient of consolidation of clay. Pre-consolidation pressure. | Hydraulic conductivity from slug test, pumping test. Hydraulic conductivity, erosion stability and groutability from *grain size distribution curve*.<br>Aquifer diffusivity and connectivity from pumping test and slug test. Clay properties from sampling and laboratory measurement.<br>Previous knowledge/information about material properties at the site valuable, especially for clay. |
| **Spatial assignment method** | |
| Evaluated/measured conductivities lognormally distributed → median K = average aquifer K.<br>Local conductivity for excavation location and effective aquifer conductivity. | Slug tests (short duration test), evaluated K from frictional soil samples.<br>Slug tests around excavation location. Distance to hydraulic boundaries. |
| **Boundary conditions** | |
| Main hydrogeological features; distinct positive or negative boundaries. Possibility of large water-conducting fracture zone. Isolated, small aquifer or large, connectivity of aquifer.<br>Depth of excavation + topography → gradient. | Main features from desk study and from *pumping test*. Initial head follows ground surface.<br>Aquifer connectivity from pumping test and slug tests. |
| **Calculation tool** | |
| Pumping test: semi-logarithmical and logarithmical plots + analytical solution. Thiem's well equation for estimation of drawdown. Steady state spread-sheet model including boundaries, material conductivities and conductivity of sealed zone around excavation (Persson, 2007). | |
| **Output** | |
| Aquifer parameters ($T_{eff}$, S, D). Connectivity of aquifer around excavation ($T_{local}$ versus $T_{eff}$ → amount and extent of drawdown). Estimation of drawdown, radius of influence, gradient, flow velocity.<br>Internally unstable or stable frictional material. High or low groutability if frictional material.<br>Pressure changes fast or slow ($D$, $c_v$).<br>*Summation of above factors gives indication of overall sensitivity of the soil rock interface.* | |

An evaluation of each factor in the list above can together provide a sensitivity assessment of the soil rock interface in whole. This can be compared to the surrounding environment and depending on the sensitivity of for example surrounding buildings and constructions, the uncertainty in the factors above needs to be reduced to a convenient level. The more unfavourable the combination of the seven above factors is for tunnelling at a soil rock interface, the more thorough investigations might be needed to ensure fulfilment of regulatory requirements and avoid negative consequences. Obtaining the data proposed in Table 12-1 also allows for creating a relatively simple spread-sheet model in which it possible consequences and needed sealing efficiency can be evaluated, as was made by Persson (2007). This allows for an estimate of required sealing efficiency to fulfil for example a maximum groundwater drawdown and what hydraulic gradients arise.

## 12.9 Discussion and concluding remarks

Generally, it is beneficial if all investigations in the vicinity of a soil rock interface are planned so that as much relevant information as possible can be derived from them. This probably needs cooperation between several fields of expertise, especially between hydrogeology and geotechnics as emphasized by Persson (2007) but also between project phases and between stakeholders. Since this approach applies to early project phases it belongs to the client (and the consultants) to implement and update the conceptual model. All data in Table 12-1 might not be available or possible to obtain at the earliest project phases but the condensed conceptual model can help prioritize investigations. It is important that it is clear throughout the updating of the conceptual model what assumptions and simplifications are made so that the limitations of the model are known.

If the early soil rock interface sensitivity assessment indicates that the interface is potentially problematic more effort might be put on investigations in order to reduce uncertainties in the geometrical framework and material properties. This is particularly the case if the potential area of influence (of inleakage to an excavation at the soil rock interface) contains sensitive objects. Overall, a pumping test is needed in order to evaluate aquifer boundaries, transmissivity and diffusivity. These factors state the overall hydraulic system sensitivity (the consolidation properties of clay are of course crucial to include in the evaluation of the sensitivity of the surroundings). More locally around the location of the excavation it is believed that a number of slug tests or similar are valuable in order to say something about the aquifer connectivity, and hydraulic conductivity, at the local site. Sieving analysis of the frictional material can provide useful information about the potential erosion susceptibility, the hydraulic conductivity and also the groutability of the material, the latter one indicating suitable sealing method and needed grout material. It is in this thesis suggested that the grain size distribution curve of a frictional material allows for qualitative assessment of the local material properties. Suggested investigations are summarized in Figure 12-7.

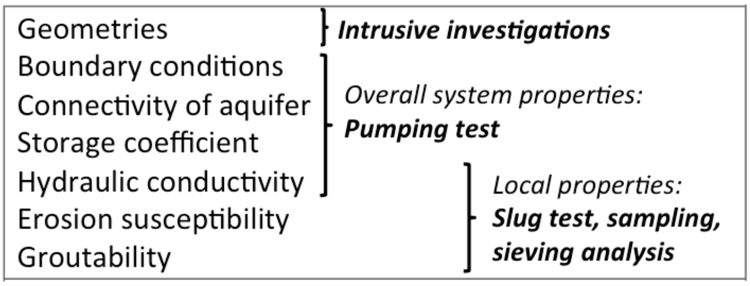


*Figure 12-7 Suggested investigations for obtaining essential overall aquifer properties and local properties at excavation location.*

To be able to say more about erosion susceptibility and groutability of the frictional soil at a site further studies are suggested. With the suggested approach to assess erosion susceptibility and groutability in this thesis it is not possible to say anything certain about these local material properties, however a qualitative assessment can still be made for indications. In the case study in Chapter 13.1.4 an example of assessing erosion susceptibility and groutability (based on Chapter 12.5 and 12.6) is given. This is an example for one site and further studies are needed to verify or reject the made assessment. A larger case study would be valuable to test the assessment of erosion susceptibility and groutability further, and to verify or reject it against field and laboratory tests. A larger case study could also indicate how soil grouting actually is designed and how efficient it is, and also to see whether erosion has given large deformations in similar conditions. Essler and Yoshida (2004) emphasize that erosion susceptible soils need to be identified if jet grouting should be used for groundwater cut-off, and the same most probably applies to permeation grouting. This risk of course depends on if the groundwater cut-off construction is permanent or temporal since internal erosion has an escalating effect; the risk of out-wash of eroded material probably increases with time (as long as there are large hydraulic gradients), for example if the construction work is delayed.

The literature review (Mittag and Savidis, 2003; Arroyo *et al*, 2012) indicates that a scientific-based, analytical design procedure would be valuable for efficient and less uncertain results of soil grouting. The actual soil grouting design belongs to a later project phase than this early sensitivity assessment but the grouting design most certainly has to be based on the soil at the site and suitable characteristics. The grain size distribution curve of the material is probably valuable, at least as an early indication of grout penetrability and suitable grout material. Lab test and in situ test is likely to be needed to further establish a suitable soil grouting design. Leaking points in a grout wall are difficult to localise and additional sealing might need to be applied along a longer wall length than actually needed; if the sealing design is successful from beginning such unnecessary post-grouting measures could be avoided. Test pumping before excavating seems to be the most suitable way to verify the achieved sealing efficiency.

The local properties of the frictional material, i.e. hydraulic conductivity, erosion susceptibility and groutability, can all be indicated from the grain size distribution curve. The hydraulic conductivity is however preferably primarily measured by short duration hydraulic tests in situ since these capture the effects of in situ degree of compaction on the hydraulic conductivity. The degree of compaction affects the material porosity, which in turns affects hydraulic conductivity, stability to erosion and groutability. The effects of compaction on hydraulic conductivity can be evaluated in lab measurements (permeameter test) where the sample has been compacted manually (Svensson, 2014); this could perhaps also be a suitable way to study how degree of compaction and porosity affects groutability.

It is noteworthy that erosion susceptibility and groutability are described with empirical relationships of almost identical form (see Equation 19 and Table 10-1 respectively) and both phenomena have much in common. Erosion is about whether the fine particles in the material can be transported through voids in the larger soil skeleton, whereas groutability is about whether fine cement particles can travel through voids in the soil skeleton. Both these phenomena have clear analogy to hydraulic conductivity, which describes the voids in the pore skeleton available for water to pass through. Based on the literature review it would be interesting to study these phenomena deeper.

Inflow to the excavation without sealing could possibly be estimated with Equation 21 for underground rock caverns in Chapter 9.4.5. This calculated inflow would be rough but could be compared to the suggested regulatory inflow requirement to the tunnel project. Needed sealing efficiency of the excavation can be assessed with some basic modelling as proposed in Table 12-2 (Persson, 2007). An indication of soil rock interface sensitivity and impacts of excavation inleakage can be assessed at relatively early project phase with suggested analytical evaluation of hydraulic boundaries, transmissivity, storage coefficient, radius of influence and using Thiem's well equation for calculation of drawdown. This can also be used to see how much different parameters affect the results.

Worth illuminating is that the inflow requirement and drawdown restrictions traditionally deal with avoiding consolidation settlement and damages to building due to lowering of the upper groundwater level; the regulatory inflow requirement does not take settlement due to erosion and out-wash of material into account. The phenomena of consolidation and erosion are governed by different processes and need to be handled separately, especially in terms of assessing the most unfavourable conditions within the observational method.

The deformation zone in Figure 6-2 and Figure 11-2 is located in the bottom of the bedrock valley but could of course also be located for example along one of the valley sides. If a deformation zone and the location of a tunnel soil rock interface would happen to coincide, this is believed to have the potential to affect the soil rock interface a lot. If the deformation zone is a hydraulic conduit (see Caine *et al* (1996) conceptualisation scheme in Figure 6-3) it can potentially provide large volumes of groundwater and possibly also increase the risk for excavation bottom instability. The deformation zone needs certain sealing measures designed in an adequate way. If the excavation is not properly sealed, groundwater pressure lowering may occur at large distances from the actual excavation depending on the direction and connectivity of the deformation zone. Passing through a deformation zone at the soil rock interface also demands certain rock stability solutions, as well as certain rock tunnel sealing solutions. It is thus believed that it is very important to identify any water-conducting deformation zones at the, or close to the, soil rock interface. Geophysics and hydraulic inference tests in bedrock can be suitable investigation methods (Gustafson, 2012).

# 13 Case study: soil rock interface sensitivity assessment

In this case study the approach to early soil rock interface sensitivity assessment described in Chapter 12 will be applied to real projects. The aim is to exemplify of how to use the suggested approach and the conceptual model provided in Chapter 12. The aim is also to get insight into how the studied soil rock interfaces were/are handled in the case projects, what investigations were made and what processes and material properties were seen as important. This is done by a limited number of interviews with professionals who were involved in the projects. Since the number of interviews is not large and the case study performed during a limited amount of time, this case study is only based on what data has been found and what the interviewed professionals remember. The findings for each studied soil rock interface are presented and discussed within this chapter.

The findings of this case study are thus mostly seen as examples of real soil rock interfaces and indications of what seem to be valuable to study further. The case study is based on the conceptual model for early sensitivity assessment of a soil rock interface suggested in Chapter 12 and focuses on processes and material properties that were identified as important within Chapter 12. Processes are consolidation settlement in clay and internal erosion of frictional material, material properties are hydraulic conductivity, aquifer diffusivity, erosion susceptibility and groutability. The focus in the case study is thus the hypotheses of important aspects presented in Chapter 11, Chapter 12 and Table 12.1, and a comparison of the hypotheses in this thesis to the real project.

The case study projects are two Swedish tunnels: the road tunnel Götatunneln in Gothenburg constructed 2000-2006 and the railway tunnel Citybanan in Stockholm presently being constructed 2009-2017. Both tunnels are situated in urban areas sensitive to groundwater lowering. Both locations are below the Highest Shoreline and have relatively similar general geology consisting of clay filled valleys between good quality bedrock plinths (Gustafson, 2012; Fredén *et al*, 2009).

## 13.1 Götatunneln

Götatunneln is a 1.6 kilometre long road tunnel in the centre of Gothenburg, stretching between Järntorget in west to Lillabommen in east. It was constructed the years 2000 to 2006, mainly in bedrock but also one section of clay tunnel, built with cut-and cover method, at each side of the bedrock construction; it thus has two soil rock interfaces. Both interfaces are studied in this case study. The excavation at the western interface was evaluated by Persson (2007) with respect to ground settlements and exacavation sealing, and her work provides a platform for the study in this thesis.

An early sensitivity assessment of the soil rock interfaces (suggested in Chapter 12) is made in this chapter, ‘early’ indicating that it is early in the project process and little project-specific data is obtained. The outcome of the sensitivity assessment can then be compared to the actual outcome of the soil rock interface tunnelling, and forms a basis for discussion of the suggested approach to sensitivity assessment. The location of the soil rock interfaces at Järntorget and Lillabommen are seen in Figure 13-1, showing the central part of Gothenburg.


*Figure 13-1 Götatunneln in Gothenburg marked with black. The two soil rock itnerfaces are located at Järntorget and Lillabommen, respectively. The orange cross indicates location of performed test pumping, mentioned in the following text.*

### *General geological description of the Götatunnel site*

The following geological description is based on geological maps provided by the Geological Survey of Sweden and can be seen in Appendix 2. From the geological maps it is seen that post-glacial clay is the predominating top soil layer in the area and that the tunnel passes through a bedrock plinth. The soil depth at the sides of the bedrock height is deep, deeper than 50 meters at the eastern side and between 30 and 50 meters at the western side (see the soil depth map in Appendix 2). This indicates steeply sloping bedrock surfaces. The location is below the Highest Shoreline and it might be likely that there is till or glaciofluvial sediments on top of the bedrock (see the type stratigraphies in Figure 6-1), which from the geological maps probably is the case at other locations in the city since bedrock plinths are accompanied by outcrops of till (see the soil type map in Appendix 2). The groundwater map shows that there is a soil aquifer covered by a low permeability material layer over a large area from the eastern end of the tunnel and eastwards. As a first assumption, based on the typical stratigraphy below the Highest Shoreline and the geological maps, a soil profile as in Figure 13-2 can be assumed. Boreholes from geological maps confirm this stratigraphy, the frictional soil is denoted ‘sand’ in these borehole observations. It is difficult to say anything about the extent or connectivity of the frictional material between clay and bedrock based on the information above. The groundwater map however indicates an extent lower aquifer but the map is not particularly detailed.

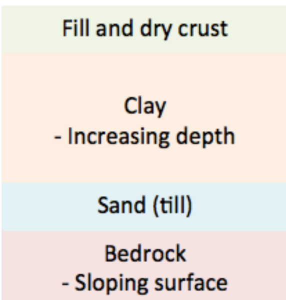


*Figure 13-2 Soil stratigraphy at Götatunneln soil rock interfaces based on geological maps from the Geological Survey of Sweden.*

The bedrock is gneissic granite/granodiorite type, possibly schisted, and some plastic and brittle/plastic deformation zones are marked in the eastern end of the tunnel, mostly in parallel direction but also one more perpendicular. The Göta River indicates a large-scale lineament and runs parallel with the tunnel direction at the eastern tunnel part. No zones are marked at the western end of the tunnel.

*Investigations and regulatory requirement*
The following paragraph is based on an interview with a hydrogeologist who worked with the hydrogeological investigations in early project phase (as a basis for tendering process and environmental impact assessment). The focus of the hydrogeological investigation was on the hydraulic properties of the bedrock since the tunnel is constructed in bedrock. No special attention was paid to the locations of the excavations at Järntorget and Lillabommen respectively. The regulatory requirement was formulated as a maximum allowed drawdown of one meter along the tunnel stretch, this would create an even drawdown curve along the tunnel and the maximum drawdown should be kept with help on infiltration during the construction phase. The drawdown limit was conservative and set with respect to the overconsolidation ratio of the clay, revealed by geotechnical investigations. Infiltration was considered to be needed at the tunnel locations coinciding with water-conducting deformation zones. Needed infiltration rates would depend on the actual drawdown during construction, which in turn depends on the achieved sealing efficiency in the tunnel. No deformation zones at the soil rock interfaces were identified during the early geological investigations. The hydraulic conductivity of the frictional material was assessed from grain size distribution curves on four samples from three different locations, the material varied from fine sand to gravel. No test pumping was performed to investigate the aquifer in the frictional material at this stage of the project since the bedrock was in focus. The lower aquifer and the bedrock were however considered sensitive to pressure lowering since the bedrock aquifer was limited in volume and extent. The frictional material might work as a positive hydraulic boundary to the bedrock.

### 13.1.1 Soil rock interface at Järntorget

The conceptual model for early sensitivity analysis in Table 12-1 is here applied to the Järntorget soil rock interface. Data that was available before construction is used, partly from the agreement between contractor and client (presented in Persson, 2007) and partly from interviews with a hydrogeologist and a contractor. Also information from the geological maps is used. The outcome of the model shall thus be an estimate of the soil rock interface sensitivity. This can then be compared to the chosen technical solutions and outcome at the construction site. It is also interesting to see what data has been found within this case study to have been available at early project phase and how the information has been used. This will be discussed in Chapter 13.1.3 and Chapter 13.3.

*Table 13-1 Conceptual model for early sensitivity assessment of the soil rock interface at Järntorget.*

| EARLY SENSITIVITY ASSESSMENT - WESTERN SOIL ROCK INTERFACE 'Järntorget' | |
|---|---|
| Conceptual model for the soil rock interface at Järntorget in Götatunneln.<br>Soil excavation at soil rock interface at western side of the tunnel. | |
| **CONCEPTS** | **DATA** |
| **Geometric framework and parameters** | |
| Sloping topography and bedrock surface. Fill couples of metres, homogeneous clay, 1-4 metres of frictional soil (sandy silt or silty sand) on top of bedrock surface. No deformation zones.<br>Hydrostatic pressure in clay, upper and lower aquifer pressure levels following fluctuations in Göta River (typically +10.2 metres, between 10-11.5). | *Desk study*: geological maps from the Geological Survey of Sweden. Soil type map, soil depth bedrock and tectonic map, groundwater map.<br>Soil profile and type of material from CPT. Also boreholes from Geological survey of Sweden.<br>Bedrock deformation zones investigated by seismic. gw pressure from piezometers |
| **Material properties** | |
| *Hydraulic conductivity of aquifer* – evaluated from 4 grain size distribution curves in 3 locations.<br>*Stability to internal erosion* – not assessed.<br>*Groutability of frictional material* – not assessed.<br>*Diffusivity of aquifer* – not assessed.<br>*Hydraulic conductivity and coefficient of consolidation of clay* – not found within this case study<br>OCR 1.35 from former loading and unloading.<br>Hydrostatic pressure distribution through clay layer. OCR evaluate from lab measurement of $\sigma_c'$. | $\sigma_c'$ from sampling in one point north of excavation and CRS testing, samples form 10 different depths. Same OCR assumed for the whole area. gw pressure distribution from piezometers.<br>No test pumping before construction →<br>no measures of hydraulic properties of the aquifer. Hydraulic conductivity of silt/sand-fractions can vary several orders of magnitude (literature value $10^{-6}$-$10^{-5}$). Sieving analyses along the tunnel stretch showed $3\times10^{-5}$-$1\times10^{-3}$, fine sand to gravel. Pumping test and evaluation of lower aquifer T at other location showed T ≈ $2\times10^{-5}$(location for text orange cross in Figure 13.1) |
| **Boundary conditions** | |
| No deformation zones indicated by seismics or tectonic map, or early geological investigations.<br>gw levels in lower and upper aquifer following water level in Göta River → positive hydraulic boundary, constant head (Persson, 2007).<br>Limited recharge from bedrock outcrop to lower aquifer (indicated by small natural hydraulic gradient).<br>Excavation depth 16-18 metres. Bedrock at around -15 meters closest to the interface. Bedrock sloping from east to west as well as north towards Göta River valley. | No hydrogeological testing before construction. |
| **Output** | |
| *Geometric framework and parameters*<br>There is a lower aquifer → Sensitive to groundwater lowering in lower aquifer → Demands adequate sealing to avoid spread of pressure lowering and consolidation settlement. Sloping bedrock surface → needs attention when installing retaining wall and performing the sealing. Silt/sand in lower aquifer → possibly large amount of fines → possibly erosion susceptible, low groutability, low hydraulic conductivity depending on the fine content. Soil sampling and grain size analysis could possibly provide opportunity for assessment of theses local material properties.<br><br>*Material properties*<br>No pumping test evaluation of aquifer hydraulic properties → large uncertainties in hydraulic behaviour of lower aquifer and following sealing demand.<br>Continues→ | |

*Boundary conditions*
Constant head governed by Göta River distinct hydraulic boundary in a larger system but very uncertain whether the river affects local system at excavation location. Since no pumping test was carried out it is difficult to say anything about how boundaries will influence. Constant head governed by Göta River perhaps influencing groundwater drawdown at excavation location but this cannot be said without hydraulic investigation.

The following text about technical solutions (retaining walls and sealing) and fulfilment of regulatory requirement at Järntorget is based on Persson (2007) and an interview with a contractor who worked with it.

*Retaining wall and excavation sealing:* The excavation at the Järntorget soil rock interface was retained by a sheet pile wall. The sheet pile wall was anchored at five levels and the excavation depth was about 18 metres at the location of the soil rock interface. The wall was cut to fit the sloping surface and jet grouting was performed with spacing 1.2 meters to seal the gap between wall and bedrock. The top 10 metres of the bedrock below the excavation wall was grouted, spacing 3.6 meters. It has not been found within this study how theses spacing measures were determined since this design was made separately from the overall design of the excavation (retaining walls, stability and similar). Grouting was performed before excavation around the excavation area subjected to risk of bottom instability. Pressure relief wells were used inside the excavation to further reduce the risk of bottom heave.

*Fulfilment of regulatory requirement:* A maximum drawdown had been set to 10 kPa in the lower aquifer. An extensive number of piezometers were used to measure the groundwater level. Pumping was performed within the excavation area to investigate achieved sealing efficiency but no relevant results were obtained from this test pumping. Large groundwater drawdown occurred early, particularly south of the excavation. The maximum allowed drawdown was reached close to the excavation site already when the excavation started and infiltration was undertaken. As excavation continued so did also the groundwater lowering and further infiltration wells were taken into use. Besides infiltration also additional sealing was performed, i.e. jet grouting inside excavation and along wall as well as in bedrock (blanket grouting of bedrock surface inside the excavation). When pressure relief wells were taken into use even further infiltration was needed to slow down the rate of deformation and groundwater drawdown outside the excavation.

Infiltration was started when the groundwater levels approached the level corresponding to the pre-consolidation pressure of the clay, the vertical deformation was at this time already up to 0.2 meters at a distance of 18 meters away from the excavation. This deformation is too large to be due to only elastic deformation of clay and consolidation settlement, besides consolidation has a time dependency which makes it unrealistic that large consolidation settlement could have occurred this fast. The horizontal deformation of the retaining walls was small, around 0.1 meters, and cannot have been the cause of the relatively large vertical deformation. The measured deformation is according to Persson (2007) and the interviewed contractor likely to be due to erosion and out-wash of material during construction work, such as installation of retaining wall anchors; significant settlement occurred at the same occasions as anchors were installed. Infiltration was used to counteract the settlements and the groundwater lowering, large infiltration rates were used, at some occasions as high as 30 – 40 l/min. Infiltration wells are located in both bedrock and frictional material, several wells located very close to the excavation pit. There are indications of that the infiltration caused erosion and outwash of material or creation of new flow paths in the lower aquifer; Persson (2007) compares the used infiltration rates to the specific capacity of a silt/sand material (transmissivity $10^{-6}$-$10^{-5}$ $m^2/s$) and an infiltration rate of 30 l/min is then between 10 and 100 times larger than the specific capacity of the frictional material. Overall, erosion is likely to have been the primary cause of the large deformation according to Persson (2007) and the contractor. When the excavation had reached its final depth the largest measured settlement was almost 0.7 meters 18 meters away from the excavation and 0.5 meters 30 meters away. The smallest deformation was 0.2 meters 26 meters away.

The large need for infiltration at Järntorget excavation was due to the demand of fulfilling the maximum allowed drawdown of 1 meter. A probable reason for the large inflow to the excavation and subsequent drawdown was an area of unexpectedly lower bedrock quality in which the performed sealing before excavation (curtain grouting in frictional soil bedrock below retaining walls) had not been sufficient. The lower quality of the bedrock was noticed during the installation of the sheet pile wall but problems with inflow did not occur clearly until the excavation had reached some depth. At some occasions out-wash of material was observed as ‘boiling ground’ where water and fine particles seeped upwards through the excavation bottom. Additional sealing was difficult to perform due to the large hydraulic gradient and inflow. Finally sealing measures were sufficient to stop this large inflow and groundwater levels could be kept outside the excavation. Throughout construction large boulders in the frictional material had been observed, possibly in combination with a very uneven bedrock surface, which made it difficult to insert the sheet piles to adequate depth.

*Estimation of hydraulic gradients*

A diagram provided in Persson (2007) showing groundwater levels in measurement pipes at different distances from the excavation pit provides opportunity for estimation of what average gradient there were. As described in Chapter 7.8 in the literature review, the hydraulic gradient (i.e. the flow velocity) governs the onset of erosion of an internally unstable material and it can thus be interesting to see what gradients there were at the Järntorget excavation. From the diagram in Persson (2007) the gradients can be calculated as height difference between the groundwater level in an observation well and the bottom of the excavation (effect from used pressure relief wells are then neglected) divided by the distance between the observation well and the excavation. Table 13-2 summarizes hydraulic gradients estimated in this manner.

*Table 13-2 Estimated hydraulic gradients at the Järntorget excavation, calculated from diagram presented in Persson (2007).*

| **Distance from excavation** | **Gradient between observation well and excavation*** | **Comment** |
|---|---|---|
| 10 m | 0.6-0.9 | 1.5 after complete excavation (and efficient additional sealing) |
| 20 m | 0.2-0.5 | - |
| 25 m | 0.3-0.6 | - |
| 60 m | 0.1-0.2 | at 60 m distance from excavation never dropped below maximum allowed drawdown |

** The given intervals of gradients are due to that several measurement pipes/well are located at the same radial distance from the excavation.*

These gradients could have the potential to give internal erosion of erosion susceptible materials. If compared to values of critical gradient to different materials presented in e.g. Jantzer and Knutsson (2010) and Perzlmaier *et al* (2007) the gradients in Table 13-2 could give erosion. For a stable material instability occurs at gradient 1.0 for upward seepage flow, at somewhat lower gradient for horizontal flow. The estimated gradient for one measurement pipe or well 10 meters from the excavation is very close 1.0.

### 13.1.2 Soil rock interface at Lillabommen

Following text follows the same aim and disposition as the one for the soil rock interface at Järntorget above. Firstly, the created conceptual model is presented.

*Table 13-3 Realisation of the conceptual model for early sensitivity assessment of the soil rock interface at Lillabommen.*

<table>
<tr><th colspan="2">EARLY SENSITIVITY ASSESSMENT - EASTERN SRI 'Lillabommen'</th></tr>
<tr><td colspan="2">Conceptual model for Götatunneln<br>Soil rock interfaces at eastern side (Lillabommen) of the tunnel.</td></tr>
<tr><th>CONCEPTS</th><th>DATA</th></tr>
<tr><th colspan="2">Geometric framework and parameters</th></tr>
<tr><td>Local rock plinth, clay filled depressions at side, eastern side very deep soil layer and thick clay layer. Steeply sloping bedrock surface. Some deformation zones indicated at tectonic map around Lillabommen. Soil profile: fill, clay, frictional material (sand and/or silt fractions), gneissic granite/granodiorite rock.</td><td>Desk study: geological maps from the Geological Survey of Sweden. Soil type map, soil depth bedrock and tectonic map, groundwater map.<br>Soil profile: maps and one recorded drilling in vicinity of Lillabommen (Geological Survey of Sweden), interview with contractor</td></tr>
<tr><th colspan="2">Material properties</th></tr>
<tr><td>Hydraulic conductivity of aquifer – evaluated from 4 grain size distribution curves in 3 locations, however not specifically at excavation site.<br>Stability to internal erosion – not assessed.<br>Groutability of frictional material – not assessed.<br>Diffusivity of aquifer – not assessed.<br>Hydraulic conductivity and coefficient of consolidation of clay – not found within this case study<br>Pre-consolidation pressure – OCR 1.25-1.3</td><td>Unstable to erosion by contractor judgement (always a risk of erosion if non-cohesive material).<br>Hydraulic conductivity of silt/sand-fractions can vary several orders of magnitude (literature value $10^{-6}$-$10^{-5}$). Sieving analyses along the tunnel stretch showed $3\times10^{-5}$-$1\times10^{-3}$.<br>Pumping test and evaluation of lower aquifer T at other location showed $T \approx 2\times10^{-5}$(location for text orange cross in Figure 13.1)<br>CPT, sampling and CRS testing in at several locations for clay consolidation properties.</td></tr>
<tr><th colspan="2">Boundary conditions</th></tr>
<tr><td>Göta River very close along northern side of excavation, possibly also southern and eastern side (close to river and canal). Bedrock along western side, impermeable or gradient (groundwater recharge from bedrock outcrop).<br>Excavation depth at deepest 30 meters (with clay bottom), bottom heave risk from bedrock to a clay layer of approximately 8 meters thickness.</td><td>Maps and geological maps.</td></tr>
<tr><th colspan="2">Output</th></tr>
<tr><td colspan="2">Geometric framework and parameters<br>There is a lower aquifer → Sensitive to groundwater lowering in lower aquifer → Demands adequate sealing to avoid consolidation settlement. Significantly sloping bedrock surface → needs attention when installing retaining wall and performing the sealing. Silt/sand in lower aquifer → possibly large amount of fines → possibly erosion susceptible, low groutability, low hydraulic conductivity depending on the silt content. Soil sampling and grain size analysis needed to further evaluate. Possibility of presence of deformation zones → needs to be further investigated, can have major impact of the hydrogeological system especially if in direct vicinity of the excavation<br>Material properties<br>No pumping test evaluation of aquifer hydraulic properties at excavation site→ large uncertainties in hydraulic behaviour of lower aquifer and following sealing demand.<br>Boundary conditions<br>Constant head governed by Göta River perhaps influencing groundwater drawdown at excavation location but this cannot be said without hydraulic investigation.</td></tr>
</table>

The following text about technical solutions (retaining walls and sealing) and fulfilment of regulatory requirement at Lillabommen is based on an interview with a contractor who worked with it.

*Retaining walls and sealing:* The excavation at the soil rock interface at the Lillabommen soil rock interface was retained mainly by diaphragm walls. Sheet pile wall was used where the bedrock outcropped and the soil thickness was small. At the soil rock interface the diaphragm walls were installed down to the rock surface, this was not possible elsewhere in the excavation due to the great soil depth. The walls were also anchored backwards. Grouting was performed to seal both frictional soil and bedrock, below the diaphragm walls around the excavation part that was subjected to risk of bottom heave. No detailed information about the sealing design has been found within this case study.

*Fulfilment of regulatory requirement:* Groundwater levels were measured in both the lower and the upper aquifer and the maximum allowed drawdown was 10 kPa just like for Järntorget and the whole tunnel. The sealing measures were preformed, pumping was made inside the excavation and groundwater effects were observed in wells outside the excavation. A small rate of infiltration was used to compensate the ground water drawdown. When excavation started a pressure drop in the lower aquifer was observed and the infiltration rate was increased. The infiltration rate was however soon reduced again since some outwash of material was observed in the excavation and the contractor was afraid of creating more erosion. Overall the lower aquifer was judged by the contractor to be very sensitive to pressure lowering and erosion.

Infiltration was needed during construction and the maximum allowed drawdown was exceeded short times at some occasion. The used infiltration rate was small and a relatively few number of infiltration wells were used, some located very close to the excavation pit. Some of the occasions of large drawdown occurred at the same time as high pressure grouting was performed. Settlements occurred but only small.

### 13.1.3 Concluding remarks on Götatunneln

Erosion has been identified as a likely cause of vertical large deformations at Järntorget, this occurred during installation of retaining wall anchors and during high infiltration rates (Persson, 2007). At Lillabommen out-wash of material is remembered by the contractor to have been observed in the excavation pit and was judged to be due to large infiltration rate which then was lowered. No problematic deformation occurred at Lillabommen. It would be interesting to study similar data from Lillabommen as Persson (2007) did for Järntorget. Also evaluation of what hydraulic gradient there were and comparison to literature values of critical gradients for erosion (of material of similar kind) would be interesting.

Based on the overconsolidation ratio of the clay, a minimum acceptable groundwater level was put, which included a safety margin. This limit was exceeded by far at Järntorget and occasionally exceeded at Lillabommen. Some of the occasions with large drawdowns at Lillabommen occurred in coincidence with usage of high pressure grouting. It would be interesting to get deeper insight into the sealing designs of the two different excavations in combination with a more detailed geological description and soil stratigraphy to see what might be the reasons for the large excavation inflows to the Järntorget excavation. One possible reason is the varying bedrock surface, low bedrock quality and presence of boulders, which led to uneven depth of sheet piles and possibly deficiencies in the wall where water could flow. The boulders might also have lead to insufficient penetration of grout and shadowing effects (behind boulders) during the soil grouting. One difference between the two soil rock interfaces is that the sealing efficiency at Lillabommen was verified by a test pumping, which was also done at Järntorget but not in way that gave any useful results. However, additional sealing was needed at both excavations since the drawdown turned out to be too large as the excavation reached deeper. Since the maximum allowed drawdown was 1 meter infiltration was used to keep this level, thus also keeping the large hydraulic gradient to the excavation; the deeper the excavation, the higher the gradient. That could have been a potential problem for post-grouting measures since there could have been difficult to make the grout stay long enough in the soil to harden (see e.g. Axelsson, 2009).

Overall, the focus within the hydrogeological investigations for Götatunneln was on the hydraulic properties of the bedrock and attention was not specifically paid to the locations of soil rock interface excavations. Erosion as a potential problem due to high gradients was not a focus within the hydrogeological investigations or the geotechnical investigations. From the contractor's point of view the focus was to create a sufficiently sealed excavation to avoid influencing both upper and lower aquifers.

In Chapter 12 it is suggested that erosion susceptibility, groutability and hydraulic conductivity are local material properties that can be relevant to soil rock interface excavations and that these properties can be assessed from grain size analyses. Within the Götatunneln project sieving analysis was made on four samples of the frictional material, however these curves have not been found within the case study so the suggested assessment in Chapter 12 has not been possible to make. Instead, examples of assessment of erosion susceptibility and groutability from grain size distribution curves of material from another location are given in Chapter 13.1.4. No pumping tests were made at the locations of the soil rock interfaces, which makes it very difficult to, within the suggested early conceptual model, evaluate overall aquifer hydraulic properties and estimate the overall system sensitivity to pressure changes (aquifer effective transmissivity, diffusivity, connectivity). Since a conservative maximal allowed drawdown was set there was no need for investigating the sensitivity of the hydrogeological system since larger drawdowns would never been accepted (all drawdown due to insufficient sealing would be compensated with infiltration). Thus, since no pumping tests were done and grain size distribution curves were not obtained, the suggested conceptual model for early soil rock interface sensitivity assessment cannot be completely filled in for Järntorget and Lillabommen and the output of the conceptual models is not very detailed. The conceptual models however indicates that pumping test and sieving analysis could be suitable complementary investigations if the surrounding area is judged to be sensitive and further sensitivity assessment considered to be needed. Evaluating these suggested local properties from grain size distribution curves is however not custom and needs to be further developed.

### 13.1.4 Evaluation of erosion susceptibility and groutability

In order to exemplify the suggested assessment of erosion susceptibility and groutability in Chapter 12 grain size distribution curves from another project are used. A few grain size analyses were made within early project phase for Götatunneln but these have not been found within this case study. Sampling and sieving analyses of frictional material in the lower aquifer was recently made for the ongoing tunnel project Västlänken in Gothenburg. Samples have been taken from Skansen Lejonet, which is located 1.5 km away from Lillabommen and results and interpretations of the hydraulic conductivity from this investigation are presented in SWECO (2013). Grain size distribution curves from this investigation are used here to exemplify assessment of erosion susceptibility and groutability of the frictional material. The assessment is based on the literature review (Chapter 7.8 'Internal erosion' and Chapter 10.2.1 'Soil permeation grouting'). The assessment of erosion susceptibility is originally introduced and developed within the field of dam construction in which different types of frictional material shall create a filter through which eroded fine particles shall not pass. It is thus not traditionally applied to soil excavations as suggested here.

Between the bedrock outcrops at Lillabommen and Skansen Lejonet there is a continuous, thick clay layer probably with frictional material below it (see geological maps, Appendix 2). Is can not be said whether the frictional material at Skansen Lejonet is representative for Lillabommen or Järntorget but the materials probably have the similar origin and deposition environment. The objective of using these samples is primarily to show the suggested procedure of how to assess erosion susceptibility and groutability. The locations of the sampling are seen in Appendix 4. The grain size distribution curves are seen in Appendix 5, four samples (here denoted number 1-4) are taken from borehole 4002 and two samples (number 5-6) are from borehole 4003.

Firstly the curves are studied optically to identify gap-grading or large shares of fines. Then Kezdi's criterion in Equation 19 is checked by dividing the curve into one part of coarse material and one part of fine material. The soil is unstable to internal erosion if $D'_{15}/d'_{85} \geq 4$ where $D'_{15}$is the coarse part of the soil and $d'_{85}$is the fine part of the soil. This division between coarse and fine has to be done. If there is a tendency of gap-grading the division is made at the gap, if there is no gap tendency the division is made where the tail with fine particles starts. Where the division of the soil into a coarse part and a fine part has been made is seen in Appendix 5. Since no sedimentary analysis has been made in the sieving analyses no silt fractions are presented and all samples have the potential to have a long flat tail of fine. Also the Kenney and Lau criterion is checked to provide a comparison to the Kezdi criterion, the evaluation of the Kenney and Lau criterion is seen in Appendix 6. In order to assess the groutability only the finest shares of the material is considered, $D_{10}$ and $D_{15}$. A conventional cement grout with small particles is here assumed to have $d_{95}$=20 $\mu m$ and $d_{85}$=16 $\mu m$. The rules of thumb for groutability in Table 10-1 are used. A summary of the assessed stability to erosion and groutability is given in Table 13-3.

*Table 13-3 Results from assessment of erosion sucseptiblity and groutability of samples from lower aquifer, Skansen Lejonet.*

| | HOLE 4002 | | | | HOLE 4003 | |
|---|---|---|---|---|---|---|
| | **1** 14-15 m | **2** 16-17 m | **3** 17-18 m | **4** 18-19 m | **5** 12-13 m | **6** 13-14 m |
| **Material** | Sand | Gravelly sand | Sand | Sand | Silty sand | Sand |
| **Ocular assessment** | Relatively uniform | Uneven, gap-tendency | One coarse part and one fine part | Relatively linear | Relatively uniform, larger share of fines | Relatively uniform, small amount of fines |
| **$D_{10}$** | 0.063 | 0.064 | 0.11 | 0.063 | 0.05 | 0.08 |
| **$D_{15}$** | 0.075 | 0.082 | 0.2 | 0.075 | 0.063 | 0.1 |
| **Kezdi** | 1.5 | 4, 10 | 2.7 | 1.8 | 1.6 | 1.5 |
| **Kenney and Lau** | Stable | Stable? | Stable | Stable | Stable | Stable |
| $\frac{D_{10}^{soil}}{d_{95}^{grout}=20\mu m}$ | 3 | 3.2 | 6 | 3.2 | 2.5 | 4 |
| $\frac{D_{15}^{soil}}{d_{85}^{grout}=16\mu m}$ | 4.7 | 5.1 | 12.5 | 4.7 | 3.9 | 6.3 |

The most gap-graded gain size distribution curve is susceptible to erosion according to Kezdi's criterion. However, a notation from the sampling says that there was a layer of gravel above fine sand at this depth, thus perhaps the sample is a mixture of two different (thin) soil layers. Since fractions smaller than 0.063 mm are not evaluated is cannot be seen whether the samples have a long flat tail of fines or not. If they have a long flat tail, another division between coarse and fine part in the Kezdi criterion could be made, perhaps giving a higher value.

The Kezdi criterion in Table 13-3 indicate that the material is stable to internal erosion if number 2 is considered unrepresentative since it is a mixture of two different material layers. However erosion can be initiated along material borders (Rönnqvist, 2002) and the clay layer above the frictional material contains silt fractions. Perhaps it should be considered a risk that the silt in the clay layers can be eroded, perhaps it could also be possible that material is eroded along the borders of the thin layers in material number 2. The Kenney and Lau criterion shows that all materials are stable, however material number two cannot be clearly evaluated and depending on the fine content (smaller than 0.063 mm) it could be internally unstable, see Appendix 6.

From Chapter 7.8 in the literature review it was described that the hydraulic gradient (i.e. the flow velocity) governs the onset of internal erosion, this gradient is called the critical gradient and is different for different materials. For internally stable soils the critical gradient for upward flow is 1.0 but lesser if the flow direction is horizontal or downward. For an unstable material the critical gradient can be assessed for the unique material (see e.g. Li, 2008) and is always smaller than the gradient for a stable material, as small as 0.01 for some materials (see e.g. Jantzer and Knutsson, 2010). The critical gradient is not evaluated in this example, however it decreases with decreasing particle size. The material at Skansen Lejonet is mainly sand whereas the frictional material at Götatunneln might be sand/silt (based on Persson (2007) and interview with contractors), thus having smaller fractions and possibly lower critical gradient. As a comparison some estimated average hydraulic gradients at the Järntorget excavation are seen in Table 13-2.

The groutability ratios indicate that a cement grout is not suitable since the soil voids might be too small for cement grout to penetrate sufficiently. It needs to be evaluated whether a microcement or chemical grout can be used, microcement however needs larger voids in relation to its particles sizes since microcement more easily filtrates and flocculates (Axelsson *et al*, 2009). The material at Götatunneln contains a larger share of fines, the groutability ratios can be expected to be even lower than the ones in Table 13-3.

It would have been interesting to evaluate grain size distribution curves from Lillabommen and Järntorget in a similar manner as was exemplified in Chapter 13.1.4 and compare to the erosion that occurred and also the chosen sealing strategy of the lower aquifer.

## 13.2 Citybanan

Citybanan is a six kilometres long railway tunnel presently being built in the centre of Stockholm, from Tomteboda in North to Södermalm in south. The tunnel will mainly be constructed in bedrock with some parts of concrete tunnel. Two parts of the tunnel are focused at in this case study; one soil rock interface in Tomteboda where the tunnel changes from concrete through to bedrock tunnel and a station entrance in Odenplan where the excavation cuts through soil down to bedrock. Figure 13-3 shows the location for the studied parts.


*Figure 13-3 Citybanan indicated by blue line. Tomteboda and Odenplan are studied in the case study.*

No conceptual models (Table 12-1) are made for Citybanan as was made for Götatunneln, this is due to that no interviews with contractors have been made. Instead the two soil rock interfaces are described and discussed based on an interview with a hydrogeologist who worked with the project representing the client. Geotechnical aspects are based on communication with a consultant. Also the geohydrological memorandum for the Environmental Impacts Assessment document is used (in Banverket, 2007a). First a general geological description based on geological maps is given.

*General geological description of the Citybanan site*

The geological description here is based on maps from the Geological Survey of Sweden if noting else is noted and the maps can be seen in Appendix 3. The bedrock in which the tunnel is constructed is granite and gneissic granite and deformation zones are present in a dominating east-west direction (Banverket, 2007a). The tunnel is constructed as deepest at 45 meters depth (Banverket, 2007b). The greatest soil depths along the tunnel stretch are about 25 meter. Thus, even though the tunnel is constructed in bedrock there will be excavations cutting through soil where stations are constructed. The dominating surface soil type along the tunnel stretch is postglacial sand. At the area around Stockholm central there is postglacial clay and also substantial areas with fill material along the shoreline of Riddarfjärden. There are several bedrock outcrops, accompanied by sandy till (soil type map, see Appendix 3). This leads to the assumption that the typical soil profile looks accordingly to the type stratigraphy in Figure 6-1. The soil stratigraphy can be seen schematically in Figure 13-4 and is very similar to the one for Götatunneln.


*Figure 13-4 General soil stratigraphy in the area of Citybanan.*

A significant geological and hydrogeological feature of the Stockholm city centre is the extent esker stretching through the city in a north south direction. The esker consists of glaciofluvial material and represents a distinct groundwater aquifer. At one eastern part the tunnel and its excavations are located within the esker periphery.

*Investigations and inflow requirement*
The following description is based on an interview with a client's hydrogeologist and Banverket (2007a). The frictional material is glacial till and its transmissivity was evaluated with pumping tests. Sampling and sieving analyses have not been made within the Citybanan project for evaluation of hydraulic conductivity since the pumping tests give the most representative results of the aquifer transmissivity. The hydrogeology at the site has been thoroughly investigated within the work with the Environmental Impact Assessment and suggestion of regulatory inflow requirement. The inflow requirement was divided into different sections long the tunnel stretch based on the natural location of different groundwater aquifers and groundwater flow paths. Infiltration was considered to be needed to ensure no negative impacts, particularly on sensitive buildings in areas with limited aquifers with limited supply of water. The infiltration wells were placed so that groundwater levels were kept around sensitive objects and a large-scale infiltration test showed whether additional well or location of wells was needed. The inflow and subsequent drawdown was controlled during construction by measuring the groundwater levels.

### 13.2.1 Soil rock interface at Odenplan

This chapter about Odenplan is also based on an interview with a client's hydrogeologist, Banverket (2007a) and communication with a consultant who worked with it. At the location of the new station in Odenplan the surrounding buildings were particularly sensitive to groundwater lowering in the upper aquifer, but also to settlement. Secant pile walls were used as retaining walls in the soil excavations and large effort was put on sealing of soil and bedrock. The soil stratigraphy and location of bedrock surface were well known from several investigations. The secant piles were drilled approximately 0.5 meters down in the bedrock. Grouting was performed between bedrock and soil and at two depth levels in the bedrock to create a water cut off around the excavation. Where the excavation had a bedrock bottom this bedrock was also grouted. The curtain grouting (groundwater cut off wall around the excavation pit) was performed before excavation and the blanket grouting (bedrock bottom) was made after excavation. The sealing work at Odenplan worked well and both the regulatory inflow requirement and the maximal allowed groundwater drawdown in the control programme was fulfilled.

The lower aquifer at this location was thoroughly investigated due to the sensitivity of the surrounding buildings. It was found that the aquifer had a very limited extent with negative boundaries. To the east it has contact with the Stockholm esker and between the aquifer and the esker there is a large hydraulic gradient from the aquifer to the water-conducting esker material, i.e. the esker is supplied from the aquifer. The sealing demand of the excavation was thus high since the buildings in the area were very sensitive and the lower aquifer had a limited extent recharge.

### 13.2.2 Soil rock interface at Tomteboda

This chapter is based on the same reference as Chapter 13.2.2. In Tomteboda the tunnel passes gradually from a concrete trough to a concrete tunnel and finally a bedrock tunnel. The soil excavation at the location of the soil rock interface was retained by sheet pile walls down to bedrock and sealing of soil and rock was performed prior to excavation. However the inflow to the excavation was so high that the regulatory inflow requirement (specified as a maximum inflow per month) was exceeded and the work was hindered by changes that had to be made due to the large inflow. Additional sealing was performed and also complementary investigations. It was important to find and to seal the large inflow since the higher the inflow to the excavation was, the lower the inflow to the bedrock tunnel had to be (the regulatory inflow requirement applied to a tunnel stretch containing both the bedrock tunnel and the soil excavation). Finally a more transmissive part of the bedrock in vicinity of the excavation was sealed and the inflow reduced to a convenient level. The bedrock had been more transmissive than expected but there had also been an originally designed sealing that never was performed since it turned out to be practically difficult to reach the area that should have been grouted.

### 13.2.3 Concluding remarks on Citybanan

The soil rock interface excavation at Odenplan was successful and it would be interesting to look further into the soil grouting design at this excavation in order to identify the reasons for this success. The bedrock surface and soil stratigraphy was well known due to the large number of investigations performed (sensitive surroundings and sensitive lower aquifer). The secant piles in the retaining wall were drilled down into bedrock, this in combination with a well known bedrock surface reduces the risk of deficiencies for water seepage. The sealing demand of the Odenplan excavation was high and the excecution was successful, it would thus be interesting to get deeper insight into how the sealing of the soil and bedrock was designed and performed to suit the site conditions.

The problem at Tomteboda seems to have been a more transmissive bedrock than expected but also that the original sealing design was not totally followed. However this thesis has not provided enough insight to the problem to state the reasons for the large inflows. What the large inflow at Tomteboda however shows is that it can be problematic for a whole bedrock tunnel stretch if the excavation at the soil rock interface has a large inflow; if the large inflow to the excavation had not been possible to reduce sufficiently the sealing design of the bedrock tunnel would have needed to be changed in order to achieve a higher sealing efficiency, compensating for the large inflow to the excavation. There are also costly delays when construction works has to be stopped since the allowed volume of water for e.g. a month has been reached earlier than expected.

During site investigations, work with Environmental Impact Assessment and similar, erosion due to large hydraulic gradients was not considered as a potential problem, possibly only in the vicinity of deep excavations. Erosion is not mentioned in any of the railway study or railway plan documents (Banverket (2003) and Banverket (2007b)).

## 13.3 Discussion and concluding remarks

The conceptual model for early sensitivity assessment in Table 12-1 has been filled in for Götatunneln soil rock interfaces. The outcome of the models for Järntorget and Lillabommen is that more site-specific investigations are needed in order to draw any further conclusions about the soil rock interface sensitivity. There is a lower aquifer, probably consisting of sand fractions, and both aquifer properties, and local properties such as erosion susceptibility and groutability could be useful to investigate in order to test the suggested sensitivity assessment further. A steeply sloping bedrock surface indicates that it can be important to localize the bedrock surface in order to assess needed depth of retaining walls.

It was not possible to assess local aquifer properties such as erosion susceptibility, groutability and hydraulic conductivity within the case study since no site-specific grain size distributions curves were obtained. As an example instead grain size distributions from another location was made to exemplify the suggested assessment of erosion susceptibility and groutability. It is difficult to draw further conclusions from this example since the assessment cannot be compared to any actual observations or outcomes. Erosion susceptibility and groutability was not evaluated within the site investigations, neither for Götatunneln nor Citybanan. Since Götatunneln was organized a design and construct project and the contractor makes the design and groutability might have been evaluated within each contractor's sealing design. The interviewed contractors did not have insight into the sealing design details. Citybanan was organized a design-bid-build project and the consultant who made the design might have evaluated groutability. Whether the contractors or the consultants have evaluated groutability of the frictional material in the lower aquifer has not been found in this case study. The case study has provided some information concerning for example spacing of grout holes for the sealing design, but not more detailed background to such figures. It would be interesting in a further study to see how groutability is assessed, how grout material is chosen, what factors determine when to use permeation grouting and when to use jet grouting, how to assess penetration length of grout and needed spacing, how the resulting sealing efficiency is verified, and similar.

Of the studied soil rock interface excavations erosion was observed at Lillabommen and created large deformations at Järntorget. No interviews were made with contractors at Odenplan or Tomteboda so it cannot be said in this case study whether erosion was observed there or not. Lillabommen and Odenplan fulfilled regulatory requirements successfully. Tomteboda exceeded the requirement and had large inflows, probably mainly due to a transmissive bedrock and insufficicent sealing performance, Järntorget exceeded the maximum allowed drawdown and large vertical deformations occurred, the reason is at least partly a zone of higher tranmissivity in the bedrock. Both soil rock interfaces with problematic inflow thus seem to have gotten large inflows due to unexpected transmissive zones in the bedrock. However also the performed sealing and the design of both soil and rock grouting would need to be studied in order to find any other additional reasons. In the ongoing tunnel project Västlänken in Gothenburg it might be important to pay attention to and further investigate the deformation zone at Järntorget since Västlänken has a soil rock interface in the same bedrock plinth nearby.

An example was provided in Chapter 13.1.4 on how to assess erosion suscpetiblity and groutability of a frictional material. Both these material properties can be assessed, at least roughly, from a grain size distribution curve. This was made but based on samples from another location than the case studies, however realtively close to Lillabommen in Götatunneln. The assessment of erosion susceptibility and groutability does not give a quantitative, certain number but offers the opportunity to qualitatively assess these parameters. Studying a grain size distribution curve of the material, based on representative sampling, can give important indications of groutability (focusing on the amount of fines) and the erosion susceptibility (the clearer gap-tendency, the larger risk of instability to erosion).

# 14 Conclusions and recommendations

This Chapter presents the most significant conclusions of relevance for answering the thesis objectives in Chapter 1.1, which are to make a problem identification, to create a conceptual model and to present initial suggestions on how local material properties can be assessed from grain size distribution curves of a frictional material. Topics concerning the thesis objectives have been discussed in Chapter 11.1, 12.9 and 13.3. Chapter 11.1 discussed possible advantages and opportunities of using a conceptual model of a soil rock interface throughout the project phases. Chapter 12.9 discussed possibilities with the suggested early sensitivity assessment of a soil rock interface and focused on local material properties such as erosion susceptibility and groutability. Chapter 13.3 discussed the findings from the case study and the application of the suggested early sensitive assessment.

One objective was to make a *problem identification*, this is presented as a bullet list in below and is based on the literature review and the case study. The second objective was to create a *conceptual model of a soil rock interface*, the purpose of the model was to assess, evaluate or model impacts from groundwater inflow to excavation and to assess needed sealing strategy. The conceptual model is presented in Table 11-1 and discussed in Chapter 11.1, and a condensed model focusing on early sensitivity assessment is presented in Table 12-1 and discussed in Chapter 12.9. The conceptual model for early sensitivity assessment of a soil rock interface also includes the third thesis objective, which was to present an initial suggestion on how *grain size distribution curves could be used to assess important frictional material properties*. Conclusions concerning the conceptual models and the assessment of local material properties are given in a bullet list in Chapter 14.2. Finally, Chapter 14.3 provides a list of suggested future studies based on findings and experiences from this thesis.

## 14.1 Problem identification

A list with identified problems concerning excavations at soil rock interfaces is given below. The identified problems concern technical solutions for sealing, geological aspects of identified problems and organizational factors of identified difficulties.

- Soil rock interfaces are identified in the literature review and in documents for Environmental Impact Assessment for large Swedish tunnel projects as problematic from a groundwater point of view. Werner *et al* (2012) states that large inflows often have occurred in Swedish underground projects at locations of soil rock interfaces.

- The literature study and the case study shows that excavations at soil rock interfaces are not water-tight and complete sealing is difficult to obtain. No clear insight in practical design of soil grouting and rock curtain grouting is obtained in this thesis. However, the literature review clearly indicates that a systematic, science-based design of soil grouting would be valuable in order to improve the sealing efficiency.

- Erosion is identified within the literature review as a risk at hydraulic gradient lower than 1.0, particularly in erosion susceptible soils where the critical gradient for onset of erosion is significantly lower. Erosion is also identified as a risk if the retaining walls are backward anchored through the frictional soil and high pressure is used during isntallation. Additionally, critical gradients for onset of erosion can be as low as one fifth of the critical gradient for buoyancy, and erosion can be more easily initiated along material borders. Erosion and out-wash of material was identified at one soil rock interface in the case study as the cause of large vertical deformation. Out-wash of material occurred in coincidence with large infiltration rates and installation of retaining wall anchors. Out-wash of fines was observed also at another soil rock interface excavation in the case study.

- Large inflows occurred at two of the studied soil rock interfaces, Järntorget in Götatunneln and Tomteboda in Citybanan. In both cases unexpectedly high bedrock transmissivity in combination with insufficient sealing of the excavations seem to have been the cause of the inflow.

- Deformation due to erosion is mentioned as a risk in one planned large-scale Swedish tunnel project in sensitive urban area. It is not described in the railway study of the projects what measures shall be taken to handle the risk of erosion. In another similar project in similar geological environment erosion is not mentioned in the railway study, the Environmental Impacts Assessment document or in the railway plan. In the case study erosion was seen as a potential problem during investigation and construction phase in one project but not in another. When erosion was seen as a risk it was however unclear what stakeholder that should investigate and handle the risk.

- The literature review and the case study identifies soil grouting in combination with rock grouting as important in order to obtain enough sealing of the excavations at soil rock interfaces. It is however not described explicitly in literature how the grouting design shall be based on the material characteristics at the site. Within the case study the interviewed contractors had not insight into the sealing design.

- Soil rock interfaces seem not to be explicitly dealt with in the regulatory inflow requirements. Inflow to excavations at soil rock interfaces will lead to reduced amount of allowable inflow to the rest of the tunnel stretch (if the requirement is expressed as an inflow). Thus large inflow to excavation can potentially increase the sealing demand to the bedrock tunnel or even delay the whole project. In combination with future stricter inflow requirements, unexpectedly large inflows to excavations can be a growing problem with significant costs.

- The prognoses used for suggestion of regulatory inflow requirement often deviates from actual inflow, it is often unclear what system parts the prognoses are based on and conceptual models are seldom used. It is unclear how the excavations at soil rock interfaces including temporary constructions are handled within the inflow prognoses. Large inflow as well as large gradient can be a problem. Large inflows can give difficulties with fulfilling the regulatory inflow requirement. Large gradients can give problems with erosion and also problems with performing additional sealing measures.

- Generally in large-scale projects communication and transmission of information and knowledge is difficult, both between project phases and between stakeholders, companies and personnel.

## 14.2 Conceptual model and early assessment of soil rock interface sensitivity

Conclusions in the list below concern the use of a conceptual model and the suitability of the suggested conceptual model for early sensitivity assessment of a soil rock interface. Findings concerning initial suggestions on the use of grain size distribution curves for assessment of erosion susceptibility, groutability and hydraulic conductivity are also concluded.

- Theory in the literature review indicates that local properties of the frictional material can be evaluated or indicated at early project phases from a grain size distribution curves. Such properties are hydraulic conductivity, erosion susceptibility and groutability. An example of assessment of erosion susceptibility and grouatbility was given in chapter 13.1.4. Grain size distribution curves from Götatunneln have not been obtained and not made within Citybanan, so the exemplified assessment has not been compared to real outcomes. Investigation of erosion susceptibility and groutability is not custom within the investigatory phase of infrastructure projects like the case projects. Grain size distribution curves of a frictional material however allows for a qualitative assessment of these local material properties at the excavation location.

- If grain size distribution curves shall be used in the sensitivity assessment effort shall be put on performing the sampling in a representative manner. It is important not to loose the fine share of the materials. It is also important to identify potential large particle sizes such as stones and boulder, which however cannot be done within sampling but instead observed during intrusive investigations.

- A number of points of investigations are suggested around the location of the excavation in order to evaluate local material properties and to get a picture of the system variability. Such local material properties are considered to be hydraulic conductivity, erosion susceptibility and groutability. Hydraulic tests of short duration (slug tests) are primarily suggested for evaluation of local transissivity. Transient pumping test is suggested for evaluation of overall hydrogeological system behavior (effective aquifer transmissivity, aquifer coefficient of storage, aquifer hydraulic diffusivity, hydraulic boundaries). All geotechnical and hydrogeological investigations are preferably planned in a cooperative manner to derive as much information as possible about the subsurface geometry such as material layers, presence of stones/boulder and bedrock surface. Special effort might also be valuable to put on investigation of bedrock surface, bedrock transmissivity and identifying presence of potential deformation zones in the vicinity of a soil rock interface.

- A sensitivity assessment, together with an assessment of the sensitivity of the surrounding area can indicate needed effort of investigations at the soil rock interface. The model can be updated throughout the project to a suitable level of detail and certainty depending on sensitivity of surroundings or strictness of inflow requirement.

- Using a conceptual model like the one presented in Chapter 12.8 provides a structured and transparent way to deal with site investigations and to share information between stakeholders and project phases about relevant processes and material properties. Using a conceptual model can also be helpful in estimates of project costs, be a basis for procurement documents and contractual risk allocation. Furthermore, using a conceptual model can be helpful when the observational method is used as design method and a catalogue of predicted system behavior shall be established at relatively early project phases when not very much site-specific data is available.

- Continuous meetings throughout construction seem to provide a suitable opportunity for updating of conceptual models and to transmit new information and knowledge concerning the soil rock interface between stakeholders.

- Further studies are needed in order to confirm whether the local material properties included in the conceptual model provided in this thesis are relevant, and how they shall be evaluated in a suitable way. The suggestions in this thesis however provide an opportunity to qualitatively assess potentially important material properties at an early project phase, which can be important to the subsequent choices of investigations and technical solutions.

## 14.3 Future studies/research

- In order to verify or reject the hypothesis about important local material properties (i.e. hydraulic conductivity, erosion susceptibility and groutability) a larger case study is firstly suggested and secondly lab tests on erosion and groutability if the case study still indicates that these properties are important. It would be valuable to follow real-time examples of excavations at soil rock interfaces and to perform grain size analyses, evaluate the local properties and compare to observations and outcomes. One suitable project would be the Gothenburg tunnel Västlänken, which includes a large number of soil rock interfaces.

- In order to further verify whether erosion and subsequent out-wash of material is a problem a larger case study would be valuable. Comparing measured ground water levels, vertical deformation and rates of infiltration can indicate erosion and outwash. Since infiltration often is used as a countermeasure to groundwater drawdown it is also interesting to look more upon how the infiltration shall be designed in order to avoid erosion of the frictional material. Theory and research from dam construction is useful but it needs to be investigated how suitable it is to apply this theory on the frictional soil types typically encountered in Swedish tunneling projects.

- An extended case study is also beneficial in order to see how well soil grouting works and how high sealing efficiency of the soil rock interface excavations typically is achieved. Also sealing methods needs to be studied; why and when is either soil permeation grouting or jet grouting chosen? How to decide what soil grouting method is the most optimal for a specific project site? It is also interesting to look further on how to assess the groutability and chose suitable grout material (if permeation grouting is used). In addition to hydraulic and mechanical processes now included in the suggested conceptual models chemical processes needs to be considered when choosing suitable grout material.

- How to evaluate the expected penetration length of grout into a frictional soil is beneficial to study further in order to obtain a more reliable grouting design. Lab tests could be performed in order to study grout penetration depending on soil type and content, and how to describe this relationship between soil properties and grout properties. Possible ways to characterize the soil for grouting purpose could be for example the grain size distribution curve, the specific surface, the porosity, or the hydraulic conductivity according to Kozeny-Carman. Perhaps the equivalent hydraulic aperture of sand, introduced by Axelsson *et al* (2009) can be further developed. It is also interesting to investigate if the grouting performance at construction should be related to for example used pressure or used volume of grout. These suggestions apply to permeation grouting.

- When it comes to forms of organization and contracts, it is important the division of responsibility between stakeholders is clear. It is also desirable that the economic compensation is made in a way that offers potential of economic benefits to a contractor who makes a successful job.

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*Personal Communication*

Johansson, I. 041514, Swedish Land and Environment Court, Nacka Tingsrätt, Sweden

Nilsson, N. G. 041614, Swedish Land and Environment Court, Vänersborgs Tingsrätt, Sweden

# Appendix 1 Original framework for conceptual model

*Original framework for the conceptual model used in this thesis, table introduced by Olsson et al (1994).*

| **SOIL ROCK INTERFACE MODEL** | |
|---|---|
| Model scope or purpose<br>Specify the intended use of the model | |
| Process description<br>Specification of the processes accounted for in the model, definitions of constitutive equations | |
| **CONCEPTS** | **DATA** |
| **Geometric framework and parameters** | |
| dimensionality and/or symmetry of model<br>specification of what the geometric (structural) units of the model are and the geometric parameters (the ones fixed implicitly in the model and the variable parameters) | specify size of modelled volume<br>specify source of data for geometric parameters (or geometric structure)<br>specify size of units resolution |
| **Material properties** | |
| specification of the material parameters contained in the model (should be possible to derive from the process and structural descriptions) | specify source of data for material parameters (should normally be derived from output of some other model) |
| **Spatial assignment method** | |
| specification of the principles for how material (and is applicable geometric) parameters are assigned throughout the modelled volume | specify source of data for model, material and geometric parameters as well as stochastic parameters |
| **Boundary conditions** | |
| specifications of (type of) boundary conditions for the modelled volume | specify source of data on boundary and initial conditions |
| **Numerical tool**<br>Computer code used | |
| **Output parameters**<br>Specify computed parameters and possibly derived parameters of interest | |

# Appendix 2 Geological maps Gothenburg

Screen shots of the geological maps provided by the Geological Survey of Sweden. More detailed maps and legends are seen in the map viewer at www.sgu.se.

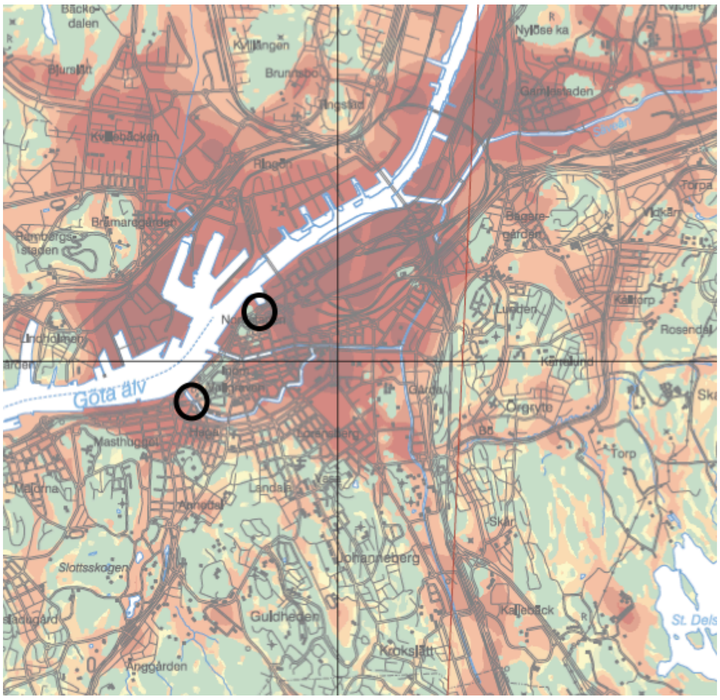


*Figure 1* ***Soil depth map****, Geological Survey of Sweden. Location of soil rock interfaces marked with black circles The darker red, the deeper; the greener, the thinner.*

*Figure 2* ***Bedrock type and tectonic map****. Black stripes are deformation zones. Orange - granite type bedrock. Green - mafic rock type. Grey - sedimentary greywacke type.*

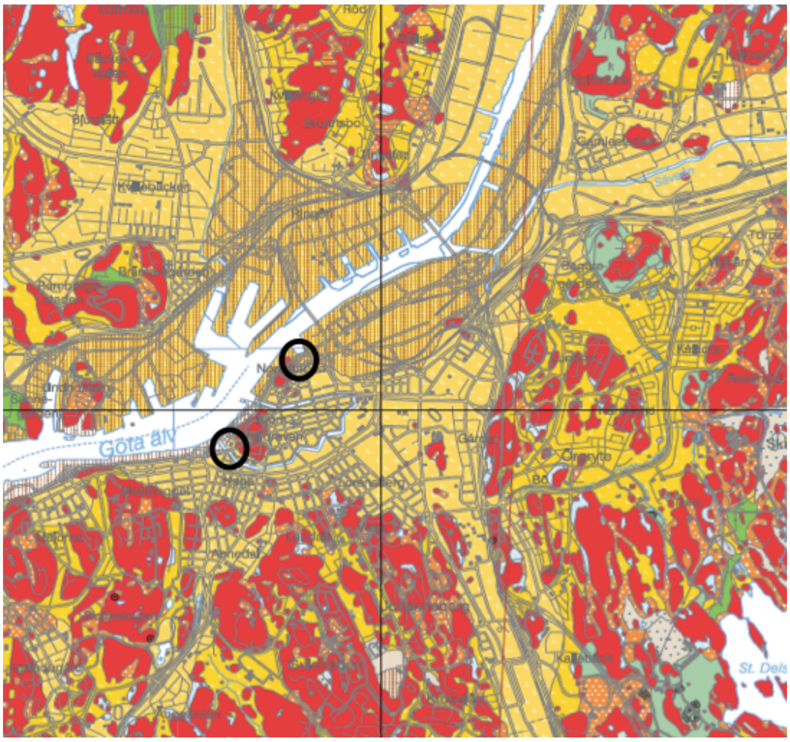



*Figure 3* ***Soil type map****. Red - bedrock outcrops. Green - glaciofluvial. Blue - till. Yellow – clay (glacial and postglacial). Orange - post glacial sediments.*

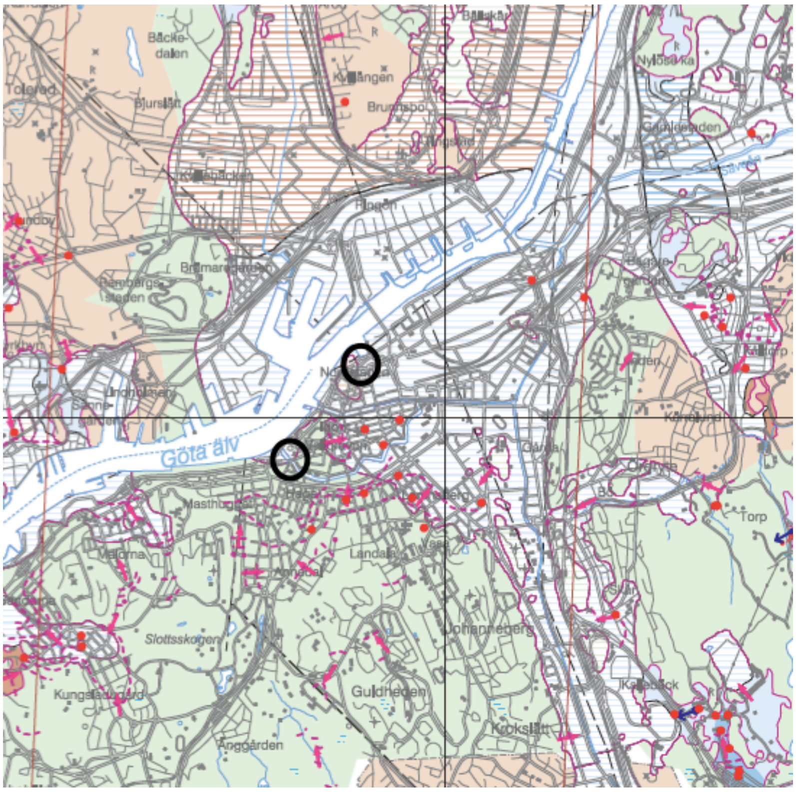



*Figure 4* ***Groundwater map****. Green and orange - bedrock aquifers. Lightly blue striped and brown striped - soil aquifers covered by lowpermeable soil. Pink lines indicates distinct aquifers. Pink arrows groundwater flow direction.*

# Appendix 3 Geological maps Stockholm


*Figure 5* ***Soil depth map****, Geological Survey of Sweden. Location of soil rock interfaces marked with black circles The darker red, the deeper; the greener, the thinner.*


*Figure 6* ***Bedrock type and tectonic map****. Black stripes are deformation zones. Orange - granite type bedrock. Green - mafic rock type. Grey - sedimentary greywacke type.*

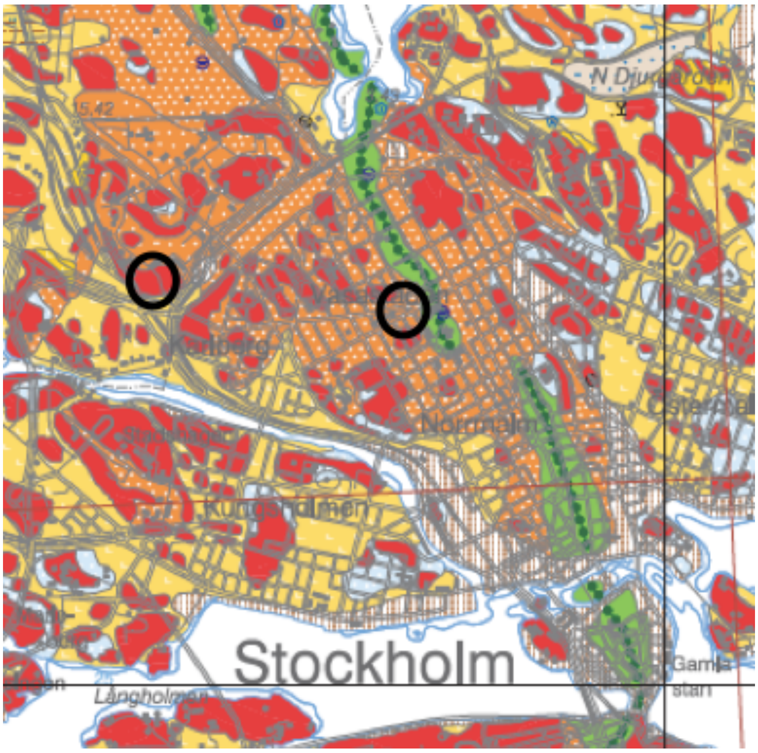
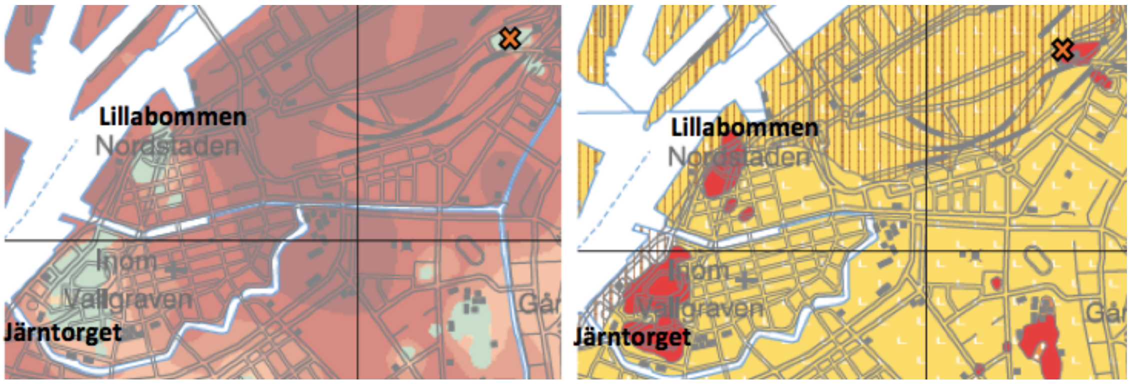



*Figure 7* ***Soil type map****. Red - bedrock outcrops. Green - glaciofluvial. Blue - till. Yellow – clay (glacial and postglacial). Orange - post glacial sediment.*


*Figure 8* ***Groundwater map****. Green and orange - bedrock aquifers. Lightly blue striped and brown striped - soil aquifers covered by lowpermeable soil. Pink lines indicates distinct aquifers. Pink arrows groundwater flow direction.*

# Appendix 4 Location of samples used in Chapter 13.1.4

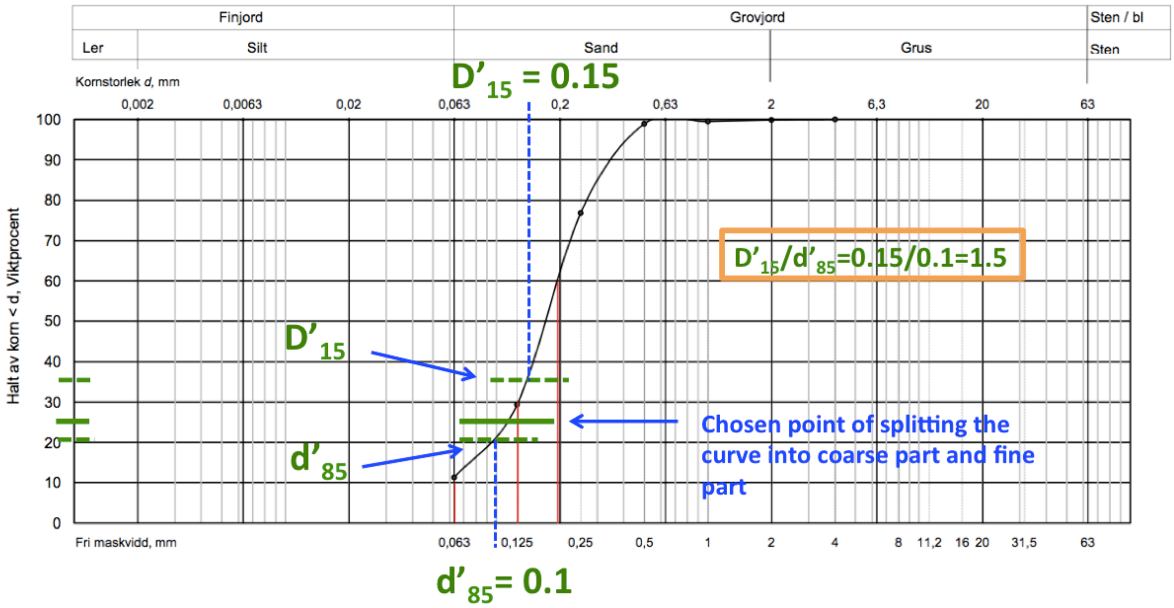

*Map from google.se. Orange cross indicates location of Skansen Lejonet and boreholes 4002 and 4003. The distance between Skansen Lejonet and Lillabommen is about 1.5 km.*



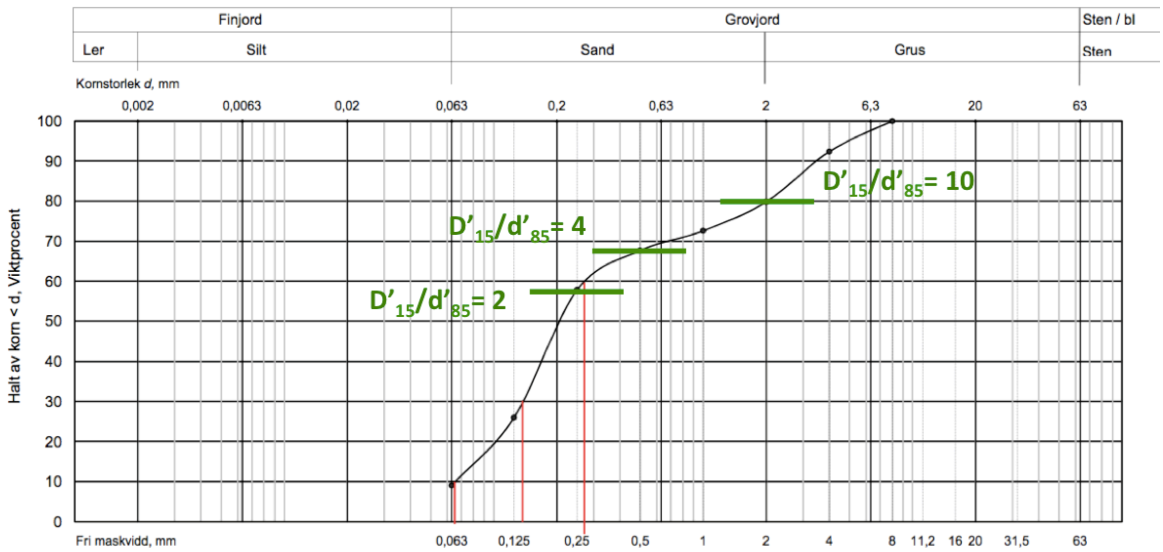

*Skansen Lejonet and Götatunneln marked at soil depth map (to the left) and soil type map (to the right). Maps from Geological Survey of Sweden.*

# Appendix 5 Grain size distributions and Kezdi's criterion

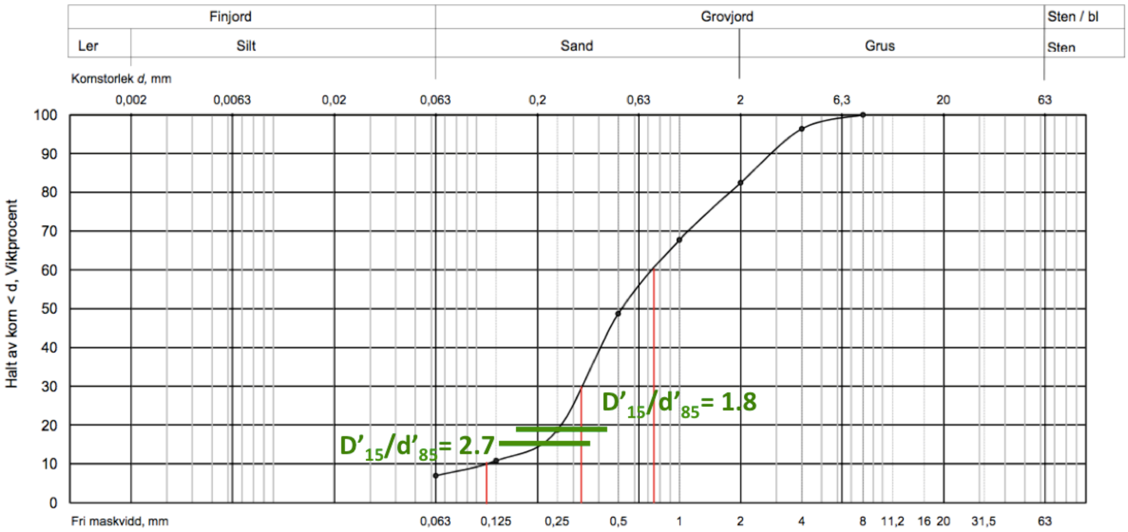
Grain size distribution curves from SWECO (2013), samples from Skansen Lejonet used for exemplification of assessment of frictional material's stability to internal erosion. Kezdi's criterion, material stable if $\frac{D'_{15}}{d'_{85}} \leq 4$ (Equation 19).



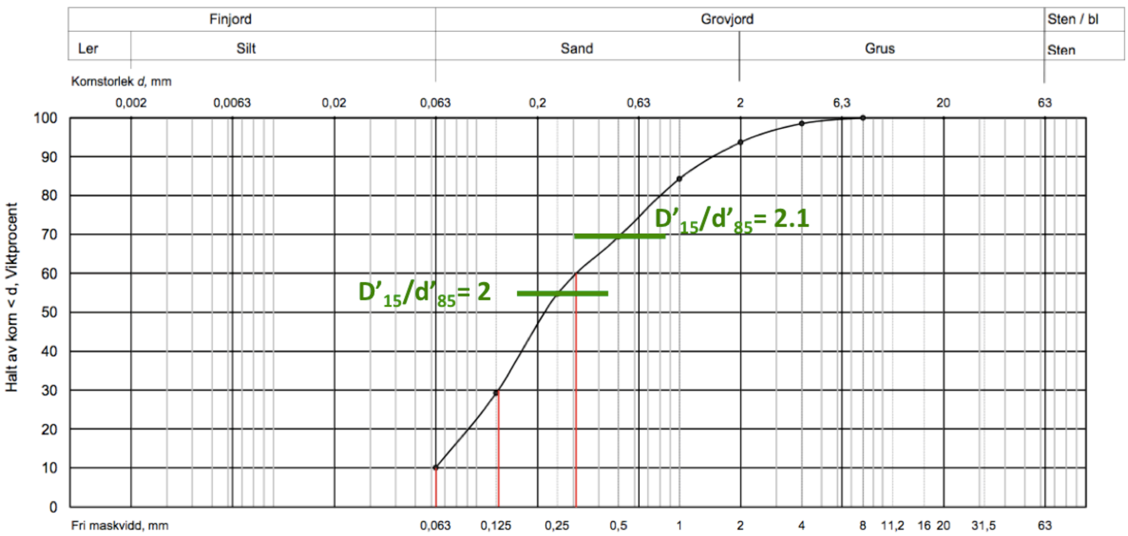

*Sample nr 1, Borehole 4002 depth 14-15 m, evaluation of erosion susceptiblity with Kezdi's criterion.*



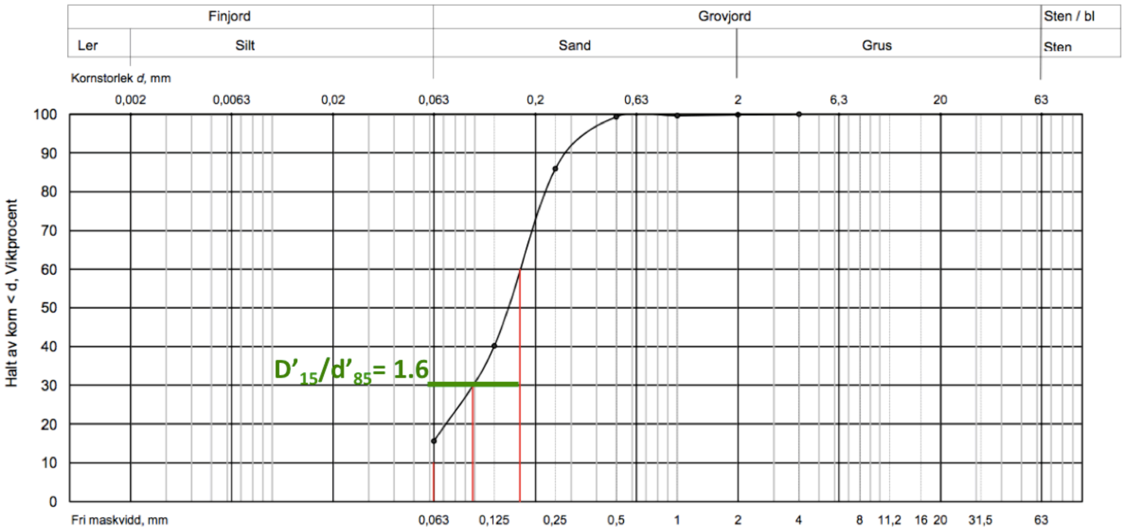

*Sample nr 2, Borehole 4002 depth 16-17 m.*



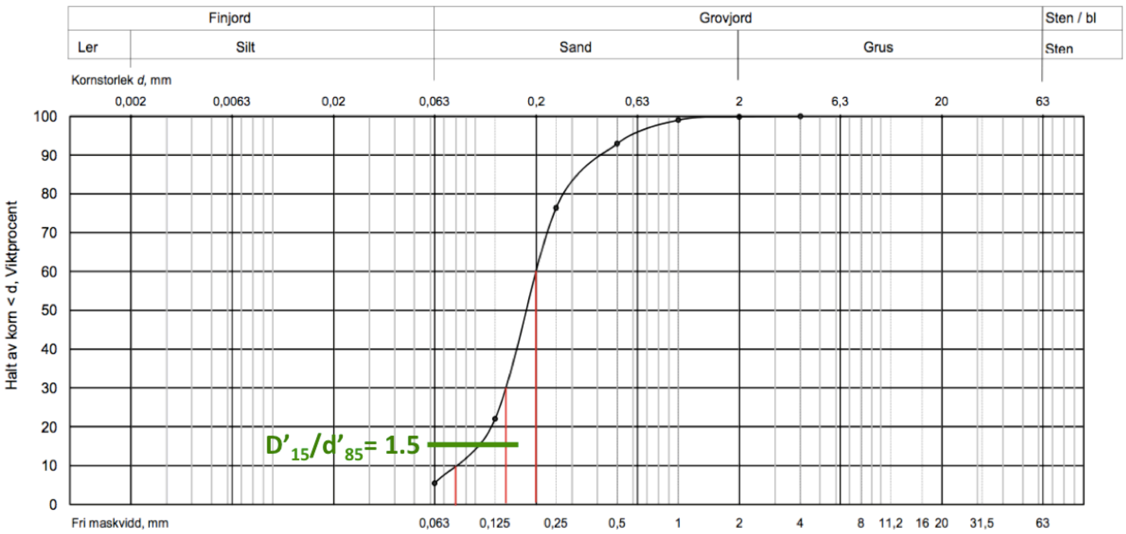

*Sample nr 3, Borehole 4002 depth 17-18 m.*




*Sample nr 4, Borehole 4002 depth 18-19 m.*



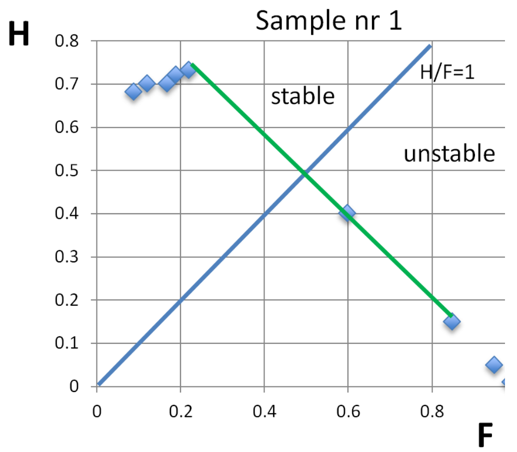

*Sample nr 5, Borehole 4002 depth 12-13 m.*



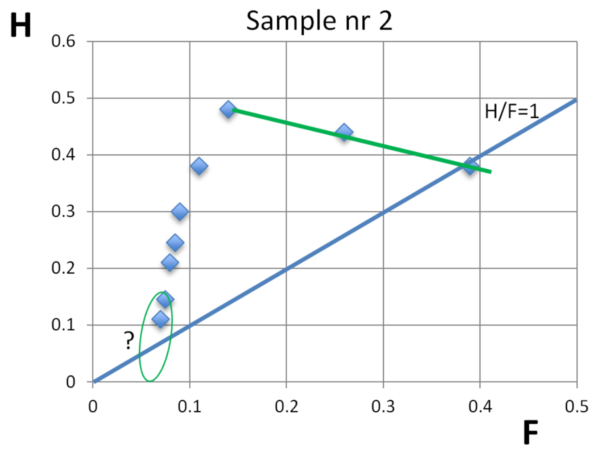

*Sample nr 6, Borehole 4003 depth 13-14 m.*

# Appendix 6 Kenney and Lau criterion

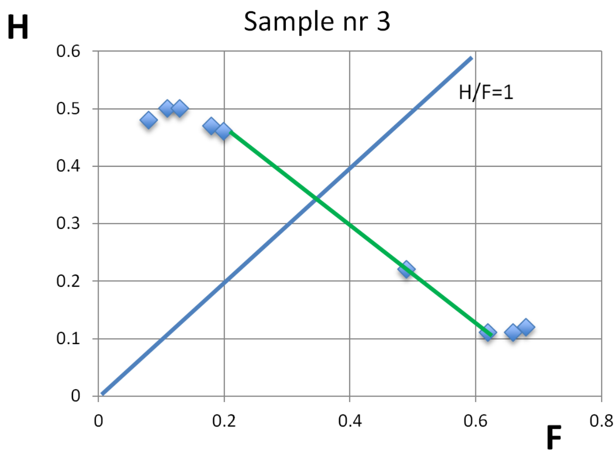
Assessed internal stability against erosion according to Kenney and Lau (see e.g. Kenney and Lau, 1985; Li, 2008; Rönnqvist, 2012). The material is considered stable if $H/F \geq 1$ for $0 < F < 0.2$ for well-graded materials and $0 < F < 0.3$ for poorly graded materials (Ahlinhan *et al*, 2012).



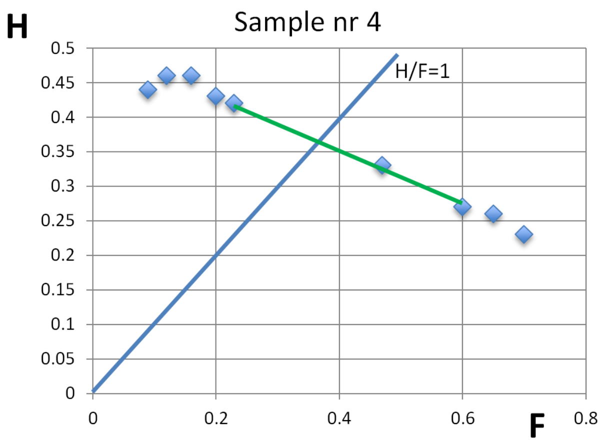

*Sample number 1, stable.*



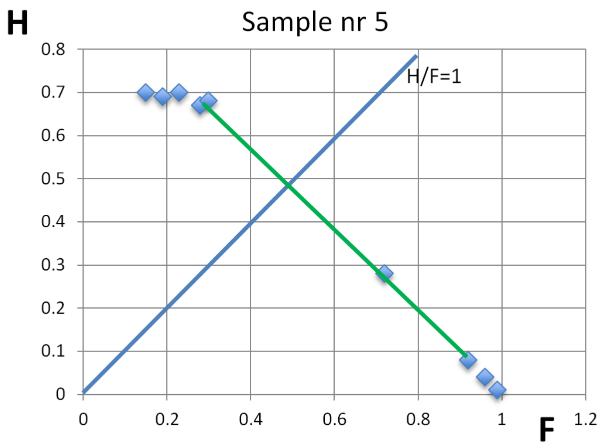

*Sample number 2, possibly unstable depending on the fines.*



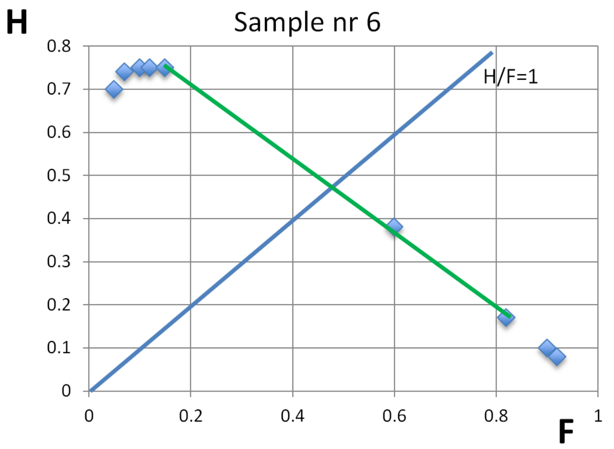

*Sample number 3, stable.*




*Sample number 4, stable.*




*Sample number 5, stable.*




*Sample number 6, stable.*