1 Introduction

Sealing of tunnels by grouting is made by injection of grout into the fractures to stop (limit) water inflow into the tunnel. In Sweden, a design method for grouting based on and adapted to geology and hydrogeology has been developed. A compilation of some of the projects where the method has been used is found in Fransson (2008) [1]. Using the design method, tunnel inflow is estimated, and the type of grout and the extent of grouting are suggested. During the design process, a grouting fan design that results in an estimate of inflow to the tunnel that fulfills the inflow requirements is accepted. The grouting design is based on hydraulic tests and geological mapping and the data are used for estimation of hydraulic apertures, tunnel inflow and selection of grout. The hydraulic aperture, \( b \), is estimated using the cubic law:

\[
T = \frac{\rho gb^3}{12 \mu} \approx \frac{Q}{dh}
\]  

(Eq. 1)

In Fransson (1999) [2], it is shown that the specific capacity, \( Q/dh \), can be used as an estimation of the transmissivity, \( T \), for hydraulic tests of short duration. In (Eq. 1), \( Q \) is the flow, \( dh \) is the change in hydraulic head, \( \rho \) and \( \mu \) are the density and viscosity of the fluid and \( g \) the acceleration due to gravity. Considering the performance of grouting materials, apertures no smaller than about 50 – 100 \( \mu \)m are expected to be groutable using a cement-based grout [3,4]. For fine aperture sealing materials such as e.g. Silica sol [5] or polyurethane [6] are needed.

An inflow into the grouted tunnel can be estimated using the following expression [7]:

\[
q_{gr} = \frac{2\pi T_{tot} H / L}{\ln(2H / r_i) + (T_{tot} / T_{gr} - 1) \cdot \ln(1 + t / r_i) + \xi}
\]  

(Eq. 2)

Included in the equation are the total transmissivity (ability of fractures or sections to transmit water) along an investigated borehole \( T_{tot} \), the residual transmissivity for fractures not sealed \( T_{gr} \), the depth (or hydraulic head) \( H \), the radius and length of tunnel, \( r_i \) and \( L \), the thickness of the grouted zone \( t \), and the skin factor \( \xi \), see e.g. [8]. The estimated inflow is related to the requirement on a maximum total inflow after grouting. Based on the estimated inflow into the tunnel and the inflow requirement, it is decided what interval of hydraulic apertures needs to be sealed. The remaining, not sealed fractures add up to the transmissivity following grouting, \( T_{gr} \). Results from hydraulic tests in grouting or control boreholes can also be used for estimating the transmissivity values \( T_{tot} \) and \( T_{gr} \). For one of the case studies the parameter \( T_{tot} \) was estimated...
using transient tests investigating pressure and flow as a function of time. The early inflow prognosis (Table 2) assumes radial flow in independent, stacked fractures or sections.

Analyses presented above describe grouting design based on hydraulic tests and geological mapping. Before these data are available a general (early) assessment of rock conditions should be useful. Existing methods for site characterization and tunnel design are e.g. the Rock Mass index (RMi) [9] and the Q-system [10]. For the Q-system the RQD-value (percentage of drill core > 100 mm) is of importance. For grouting, the geometry of the water-bearing fracture system is of most importance and therefore one could argue that the RQD, or preferably fracture frequency or intensity, see e.g. [11], in combination with hydraulic tests are important sources of information. Suggested parameters for describing the water-bearing fracture system for grouting are fracture intensity (frequency), fracture set orientations and distribution of hydraulic apertures (transmissivity), see [12].

In the paper "Fault zone architecture and permeability structure", Caine et al. (1996) [13] describe the fluid flow properties of fault zones in various geological settings. Fig. 1 presents a conceptual model of a fault zone consisting of fault core and damage zone surrounded by relatively undeformed protolith or host rock [13]. Gouge, cataclasite and mylonite are included within the description of the fault core. Small faults, fractures, veins and folds are included within the description of a damage zone.

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![Figure 1](image.png)

**Figure 1** Conceptual model of fault zone including fault core and damage zone (modified after [13]). Caine et al. (1996) [13] include a description of fault zone permeability structures; in this paper similar host rock (protolith) permeability structures are suggested.

Caine et al. (1996) [13] include a description of fault zone permeability structures and in the right hand side of Fig. 2 four different types (end-members) of conduits are presented: Localized conduit; Distributed conduit; Combined conduit-barrier and Localized barrier (see description in Table 1). The hydraulic function will differ depending upon the amount of fault core or damage zone present in a particular fault.

The intention in this paper is to use the conceptual scheme of fault zones presented in Fig. 2 as a basis for a description of the water-bearing fracture system or the permeability structure for the host rock (protolith), fracture zones (locally increased fracture frequency) and fault zones for grouting purposes. As suggested in Fig. 2, both the host rock (protolith) and the localized and distributed conduits are described as Type I (2D flow) or Type II (3D flow) permeability structures. The right hand side of the figure describes the fault zones and is modified after [13]. The left hand side of the figure is new suggesting similar host rock (protolith) permeability structures.
Figure 2 Conceptual scheme of permeability structures for fault-related fluid flow (right hand side of figure modified after [13]). In addition, permeability structures for the host rock (protolith) are suggested. Both the host rock and the localized and distributed conduits are described as Type I (2D flow) or Type II (3D flow) permeability structures.

Table 1 Description of fault zone permeability structures and the applicable flow model according to Caine et al. (1996) [13] and suggested, similar host rock permeability structures and possible implications for grouting.

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<tbody>
<tr>
<td>Localized conduit (absent to poorly developed fault core and damage zone)</td>
<td>Discrete fractures</td>
<td>Type I Mainly one conductive fracture set, not well connected network. A higher fracture frequency describes a zone (similar to Localized conduit end-member).</td>
<td>Grouting in selected areas possible. Inflow prognosis equation (2D*) may overestimate tunnel inflow (see case study).</td>
</tr>
<tr>
<td>Distributed conduit (absent to poorly developed fault core, well developed damage zone)</td>
<td>Equivalent porous medium</td>
<td>Type II More than one conductive fracture set, well connected network. A higher fracture frequency describes a zone (similar to Distributed conduit end-member).</td>
<td>Systematic grouting. Inflow prognosis equation (2D*) may underestimate tunnel inflow (see case study).</td>
</tr>
<tr>
<td>Combined conduit-barrier (well developed fault core and damage zone) Fault core consists of e.g. gouge, cataclasite, mylonite.</td>
<td>Aquitard (fault core) between two aquifers (damage zone)</td>
<td>Type III Generally a fault zone.</td>
<td>Focus on damage zone (Type II permeability structure). The core is difficult to grout.</td>
</tr>
<tr>
<td>Localized barrier (well developed fault core, absent to poorly developed damage zone). Fault core consists of e.g. gouge, cataclasite, mylonite.</td>
<td>Aquitard (fault core) within a higher permeability aquifer (protolith)</td>
<td>Type IV Generally a fault zone.</td>
<td>The core is difficult to grout.</td>
</tr>
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</table>

* The 2D inflow prognosis is based on stacked independent fractures.
Table 1 compiles a description of the fault zone permeability structures and the applicable flow models according to Caine et al. (1996) [13]. In addition we suggest host rock (protolith) permeability structures similar to the fault zone structures and comment on possible implications for grouting. In the following we present examples of rock permeability structures of Types I and II using two case studies and comment on the tunnel inflow prognosis, the extent of the grouting and the selection of grout.

2 Case studies

2.1 Geological and hydrogeological conditions: permeability structures

Two tunnel sections from two different tunnels in crystalline rock were used as case studies:

- TASQ-tunnel, Äspö Hard Rock Laboratory (Äspö HRL): deep tunnel, sparsely fractured rock, suggested Type I permeability structure, see Fig. 2 and principle sketch Fig. 3.
- Hallandsås: Excavated though a horst (a parallel block remaining between two grabens), well connected fracture network, suggested Type II permeability structure, see Fig. 2 and principle sketch Fig. 4.

The first tunnel section, the TASQ-tunnel [14], is 70 m long and located at 450 m depth in south-east Sweden, see Table 2. The main rock type is a diorite. The second tunnel section in Hallandsås, south-west Sweden is 36 m long and at approximately 100 m depth. In this section the rock consists of gneiss and amphibolite.

Fig. 3 presents a principal sketch of the TASQ-tunnel and a suggested Type I permeability structure, see Fig. 2, including host rock consisting of sparsely fractured rock mass and a fracture zone (locally increased fracture frequency). Further, Fig. 4 presents a principal sketch of the Hallandsås tunnel section which is a suggested Type II permeability structure including host rock with a well connected fracture network and a fracture zone (locally increased fracture frequency at a gneiss – amphibolite contact).

**Figure 3** Principal sketch of TASQ-tunnel, Type I permeability structure: host rock (sparsely fractured rock mass) and a fracture zone (locally increased fracture frequency). The section where the inflow was measured \( q_{\text{measured}} \) is 70 meters long.
2.2 Prognosis of tunnel inflow

The fractured rock mass in the TASQ-tunnel is sparsely fractured (RQD: 100) and the identified fracture zone has a higher fracture frequency (slightly lower RQD), see Fig. 3. The tunnel rock mass could be described as a Type I permeability structure: Mainly one conductive fracture set and the network is not well connected. The higher fracture frequency identified within a section of the tunnel is a zone (similar to Localized conduit end-member). Following pre-grouting and excavation, the measured inflow, \( q_{\text{measured}} \), was 5 liters/min (Table 2) and low compared to an early prognosis of inflow, \( q_{\text{gr}} \) of 25 liters/min assuming stacked, independent fractures with 2D flow. One reason for the difference could be sealing of the fracture zone limiting the flow in connected fractures (deviation from the stacked fracture 2D flow assumption of the early prognosis). Locally measured hydraulic properties using natural inflow measurements in control boreholes and tests of longer duration (transient) resulted in values of \( T_{\text{gr}} \) and \( T_{\text{tot}} \) giving a good agreement between measured and estimated inflow (5 liters/min compared to 4 liters/min), see Revised inflow prognosis (Table 2).

For the section of the Hallandsås tunnel the fractured rock mass is expected to be well connected (low RQD compared to above), see Fig. 4. A zone (still lower RQD) is found in an area with contacts between gneiss and amphibolite. The section could be described as a Type II permeability structure: More than one conductive fracture set and a well connected network. A higher fracture frequency describes a zone (similar to Distributed conduit end-member). The measured inflow following pre-grouting and excavation, \( q_{\text{measured}} \), was 144 liters/min (Table 2) and large compared to an early prognosis of the inflow (16 liters/min). The circumstances that the section where inflow was measured was small compared to the extent of the zone, and that the local hydraulic properties were not representative for the section, are likely explanations for the deviation between measured and estimated inflow. Table 2 presents examples of values of \( T_{\text{tot}} \) and \( T_{\text{gr}} \) resulting in an inflow of 144 liters/min.
Table 2 Case studies TASQ-tunnel [12,14] and Hallandsås: Depth, Rock type, RQD, input data and estimated tunnel inflow, $q_{gr}$ (Eq. 2) and measured tunnel inflow, $q_{measured}$. The early inflow prognosis assumes radial flow in independent, stacked fractures or sections.

<table>
<thead>
<tr>
<th>Tunnel</th>
<th>TASQ tunnel (Äspö HRL)</th>
<th>Hallandsås section</th>
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<tbody>
<tr>
<td>Depth</td>
<td>450 m</td>
<td>100 m</td>
</tr>
<tr>
<td>Rock type and RQD</td>
<td>Äspö diorite</td>
<td>Gneiss and Amphibolite</td>
</tr>
<tr>
<td>$RQD$</td>
<td>95 - 100</td>
<td>25 - 70</td>
</tr>
<tr>
<td>$H$ [m]</td>
<td>340</td>
<td>75</td>
</tr>
<tr>
<td>$L$ [m]</td>
<td>70</td>
<td>36</td>
</tr>
<tr>
<td>$n$ [m]</td>
<td>2.8</td>
<td>4.3</td>
</tr>
<tr>
<td>$t$ [m]</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Inflow prognosis</td>
<td>Early</td>
<td>Revised</td>
</tr>
<tr>
<td>$T_{tot}$ [m$^2$/s]</td>
<td>3.8·10$^{-6}$</td>
<td>1.5·10$^{-5}$</td>
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<tr>
<td>Transient test</td>
<td>1.5·10$^{-5}$</td>
<td>Pre-grouting</td>
</tr>
<tr>
<td>boreholes</td>
<td></td>
<td>boreholes</td>
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<tr>
<td>$T_{gr}$ [m$^2$/s]</td>
<td>2.6·10$^{-7}$</td>
<td>3.4·10$^{-8}$</td>
</tr>
<tr>
<td>Inflow cored</td>
<td></td>
<td>Control</td>
</tr>
<tr>
<td>borehole (for b &lt; 50 µm)</td>
<td></td>
<td>boreholes</td>
</tr>
<tr>
<td>$q_{gr}$ [liters/min] Eq. 2</td>
<td>25 (ξ set to 0)</td>
<td>4 (ξ set to 0)</td>
</tr>
<tr>
<td>$q_{measured}$ [liters/min]</td>
<td>5 Measured tunnel inflow after pre-grouting and excavation, 70 m.</td>
<td>144 Measured tunnel inflow after pre-grouting and excavation, 36 m.</td>
</tr>
</tbody>
</table>

* Examples of values of $T_{tot}$ and $T_{gr}$ resulting in an inflow of 144 liters/min.

2.3 Extent of grouting and selection of grouting material

Based on an 80 meter long cored borehole drilled before excavation of the TASQ-tunnel, it was decided that two grouting fans should be made at the location of increased inflow (and higher fracture frequency), see principal sketch Fig. 3. The fracture mapping and the hydraulic tests made up the basis for the decision. Sealing was performed using a cement-based grout only and therefore sealing of fractures down to 50 µm, but not smaller, was expected. Investigations performed after grouting and excavation later confirmed this [8].

The section of the Hallandsås tunnel presented here is a part of a longer tunnel section (130 m) where systematic pre-grouting had not been sufficient and additional systematic post-grouting was therefore made. Due to a high fracture frequency both large and small aperture fractures were expected and two types of grout were selected: a cement-based grout and a grout for fine aperture sealing (Silica sol). Following the post-grouting, the inflow requirement was fulfilled.

In this paper it is suggested that both the host rock (protolith) and the localized and distributed conduits (fault zones) are described as Type I or Type II permeability structures based on geological mapping and hydraulic tests, see Fig. 2. According to Table 1, a Type I (permeability structure) mainly consists of one conductive fracture set resulting in a fracture network that is not well connected. A higher fracture frequency describes a zone (similar to Localized conduit end-member, Fig. 2). Depending upon the inflow requirement grouting in selected areas (zones) can be sufficient. This was the case for the grouting in the TASQ-tunnel. Here the early inflow prognosis equation based on individual fracture apertures along the borehole overestimated the tunnel inflow. Sealing of the fracture zone limiting the flow in connected fractures could be one
reason. A revised inflow estimate based on a transient test and tests in control boreholes after pre-grouting gave a good agreement between estimated and measured inflow.

For a Type II (permeability structure) there is more than one conductive fracture set, and the fracture network is expected to be well connected. As for Type I, a higher fracture frequency describes a zone (similar to Distributed conduit end-member, Fig. 2). Due to the well-connected network systematic grouting is reasonable to avoid moving the water from one section to another. Systematic (pre- and) post-grouting was used for the Hallandsås tunnel section. In this case, the inflow prognosis equation using data from the pre- and post-grouting boreholes underestimated the tunnel inflow. Likely explanations are that the section where inflow was measured was small compared to the extent of the zone transmitting the water, and the local hydraulic properties not being representative for the section.

Fracture mapping and hydraulic tests (both short duration tests and transient or time dependent tests) are important sources of data for the design process. Compiling the information into fracture stereo plots gives important information for assessment of permeability structures and for decisions on grouting borehole orientations for the grouting fan design.

3 Conclusions

A conclusion based on the two case studies is that the principal descriptions of the geology and hydrogeology for the two tunnel sections identify general differences between the two tunnels. This explains in part the deviation found between early inflow prognoses and the measured inflows. Further, the estimated hydraulic aperture from hydraulic tests can be used as a basis for selection of grout. Finally, the fracture frequency or the variation in fracture frequency (e.g. between neighboring borehole- or tunnel sections) in combination with hydraulic tests indicate what areas to focus on when grouting.

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References


