Leakage and groutability

Ursula Sievänen

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Leakage and groutability

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LEAKAGE AND GROUTABILITY

ABSTRACT

Groundwater ingress into underground constructions may cause various technical and environmental problems. It is essential to localise and characterise potential leakages and to estimate the amount of water leakages, so that appropriate sealing works can be performed as pregrouting. Same problems with groundwater ingress are supposed to be met when constructing the final disposal facility for high level nuclear waste, for which purpose this work is performed.

Methods to estimate groundwater ingress are dealt in the first part of the report. Estimating the amount of water ingress into an excavation is a multi-phase process starting from qualitative and quantitative examination of bedrock and ending up to mathematical calculations based on earlier data. This literature study is focused on finding mathematical methods and studying their applicability. The second part of this literature study deals with rock groutability. The geological factors affecting groutability are discussed, and methods to estimate the groutability of rock and classify rock, fracture or fracture zone from grouting point of view are searched.

Two approaches are typically used when analytically calculating the amount of groundwater leakage. One is so called imaginary well method for tunnels and the other, which is suitable for excavations of any shape, is based on Thiem’s well equation. Several variations of these methods are presented as well as other solutions, too. Analytical equations are very simplified, but fast and easy to apply. There are many numerical methods of different character. In principle the capacity of numerical methods is unlimited, but they are slower, more expensive and more difficult to apply. There is no method which is proved to be good and reliable – the problem of calculating groundwater ingress culminates in obtaining qualitative and quantitative information of rock. The parameters of main importance are hydraulic conductivity and groundwater pressure.

Many geological factors affect the groutability of rock. Such are e.g. fracture aperture and the connectivity between fractures. Some essential but problematic factors are the channeling of flow and softening fracture fillings. Much information of rock is possible to gain by combining different existing investigation methods, but flow channels are not easy to localize or to grout. Softening and movable fracture fillings are problematic, too. Different rock classification systems for grouting purposes are presented in the literature. The most advanced are the Q-system, which is based on the well-known Q-system, and Hässler’s classification system. Both have advantages and disadvantages. The parameters used in the Q-system are familiar and relatively easy to determine, but the correlation with the grout take is found to be weak. Hässler’s system agrees best with the aim of this study, but the parameters are very theoretical and prospective fractures are presumed to be similar than previous ones. And, this system does not take into account the quality of fracture fillings, which greatly affects grouting.

Keywords: Nuclear waste, final disposal, groundwater, water leakage, water inflow, water ingress, groutability, estimation
VUOTOVESIVIRTAAMA JA INJEKTOITAVUUS

TIIVISTELMÄ

Kallioliotoihin tulevat vesivuodot voivat aiheuttaa monenlaisia teknisiä ja ympäristöllisiä ongelmia. Mahdollisten vuotokohtien tunnistaminen ja karakterisoiminen sekä vuotomääräen ennustaminen etukäteen on tärkeää, jotta voidaan varautua asianmukaisiin tiivistämistoimenpiteisiin. Nämä ongelmat nousevat esiin myös ydinjätteen loppusijoitusiloja rakennetaessa, jonka vuoksi tämä työ on tehty.

Työn ensimmäisessä osassa käsitellään menetelmiä, joilla voidaan arvioida kallioliotoihin tulevia vesivuotoja. Vuotovesimäärän arvioiminen on monivaiheinen prosessi alkaen kallion kvalitatiivisesta ja kvantitatiivisesta tutkimisesta päätyn matemaattisiin laskelmiin em. vaiheista saatujen tietojen perusteella. Työssä on keskitytty etsimään kirjallisuudesta matemaattisia menetelmiä ja arvioitui niiden sovellettavuutta. Työn toisessa osassa on kirjallisuuden avulla pohdittu geologisten tekijöiden vaikutusta injektointiin sekä etsitty menetelmiä arvioida ja luokitella kalliotaus, raoa tai rakovyöhykkeen injektioituva.


Avainsanat: Ydinjätte, loppusijoitus, pohjavesi, vuotovesi, injektioituvaus, arviointi
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LIST OF SYMBOLS

a = area of the surface S (m²)
a₁ = coefficient related to water pressure (constant) (-)
a₂ = coefficient related to the total stress (constant) (-)
a₃ = coefficient related to stress-independent factors (constant) (-)
A = hydraulic conductivity gradient (a constant) (-)
b = thickness of the fracture zone/confined groundwater layer/fracture plane opening (/aperture) (m)
C = constant
C_m = median of the specific capacity of the rock mass (m²/s)
d = depth of the excavation/centre of the opening/groundwater table (m)
D = characteristic diameter of the tunnel (m)
g = acceleration due to the gravity (9,81 m/s²)
h/hₓ = depth below the groundwater table
h_L = water head along the outer strip of the fracture plane
h_w = water head in the probe hole (m)
H = water pressure expressed as a height of the water column (m)
K = hydraulic conductivity (m/s)
K_g = gross hydraulic conductivity (m/s)
Kᵢ = hydraulic conductivity of the grouted zone (m/s)
K_m = equivalent mass permeability/hydraulic conductivity (m²)
Kₑ = surface intercept of the exponential conductivity gradient (m/s)
J = hydraulic gradient (-)
J_a = joint alteration number (-)
J_n = joint set number (-)
J_r = joint roughness number (-)
l = Lugeon value (-)
L = length of the tunnel / distance from the wall of the hole to the outer edge of the fracture plane (m)
L_max = the maximum penetration of the grout (m)
P_e = pressure inside the opening (kg/ ms²)
P_r = excess pressure at opening (kg/ms²)
q_b = median capacity of the borehole (m³/s)
q(ᵢ) = flow rate in point (ᵢ) (m/s)
P(ᵢ) = pressure in the point (ᵢ) (kg/ms²)
Q/Qₓ = water leakage to a tunnel/excavation (total m³/s or for a special length e.g. m³/s/100 m) / leakage through a fracture zone / radial flow into a well / flow through a certain surface (m³/s)
Qᵢ = quality of rock from grouting point of view (-)
Qᵢ(ᵢx) = transient flow into a tunnel from a certain part (m³/s)
r = radius of the borehole/well (m), distance from the well/from the center of a tunnel (m)
r_o = influence radius of the borehole/well (m)
r_probehole = circumference of the probehole (m)
R = radius of the excavation/tunnel cross section / equivalent radius of the opening (m)
R₀ = assumed influence radius of the excavation/flow (m)
\( R_e \) = radius of the imagined circle (m)
\( \text{RQD} \) = rock quality designation
\( s \) = drawdown of the groundwater table (m)
\( S \) = distance of the centre of the tunnel to the water injection borehole (m) / a surface through which the flow occurs
\( S_y \) = Specific yield (-)
\( t \) = thickness of the grouted zone (m)
\( t_x \) = time (s)
\( T \) = (gross)transmissivity (m\(^2\)/s)
\( \bar{v} \) = mean value of the seismic velocity (m/s) of the rock mass which has been penetrated by the boreholes
\( V \) = mean value of the seismic velocity (m/s) of the whole tunnel line
\( V_{\text{inj}} \) = the volume of grout that has penetrated (m\(^3\))
\( W \) = the circumference of the probe hole (m)
\( Z \) = z-coordinate (m)
\( \alpha \) = the propagation angle (rad)
\( \xi \) = skin-effect (-)
\( \phi(\bar{r}) \) = potential in point \( \bar{r} \) (m)
\( \phi(0,\bar{r}) \) = the potential difference between the tunnel surface and groundwater level
\( \mu \) = dynamic viscosity of water (kg/m/s)
\( \rho_m \) = density of the overburden (kg/m\(^3\))
\( \rho_w \) = density of water (kg/m\(^3\))
\( \tau_0 \) = shear strenght for Bingham fluid (N/m\(^2\))
1 INTRODUCTION

The final disposal of spent nuclear fuel in the Finnish bedrock has been studied since the early 1980s with the aim of selecting one site for the repository in the year 2000. The superterranenean phase of the investigations is coming to the end and Olkiluoto has been suggested as the site for further, subterranenean investigations. The Finnish parliament has to confirm or refuse the suggestion before the excavation of the underground research tunnel or shaft can be started. The excavation work of the repository is planned to start after 2010 and the disposal of the nuclear waste in 2020. Posiva Oy, a company jointly owned by two Finnish nuclear power companies (Teollisuuden Voima Oy and Fortum Power and Heat Oy), co-ordinates and manages the investigations, planning work and other related tasks.

After the closing of the repository the only way the radionuclides can get in contact with biosphere is migration with the groundwater flow after all the technical barriers have decayed. The long-term safety is not the only problem related with the groundwater. Water leakage into the repository also may cause technical problems during the construction work and the operation of the repository, and therefore the amount of leakage water should be controlled and restricted to a reasonable level. Prediction of the inflows is an essential task for optimizing construction operations. Groundwater leakage control can be regarded as a divaricate task. First the leakages and the amount of water ingress should be localized and estimated, and then the way of sealing should be determined, if it is needed. Grouting is the most commonly used sealing method.

The preliminary draft for sealing of the final disposal repository has been presented in a report by Riekkola (1990). The main aim is to control groundwater leakage into the repository during construction, operation and closing. It is uncertain if the intention is to use grouting as a technical barrier to restrict the migration of radionuclides, too (Pöllä et al. 1994). The sealing is planned to be solved by grouting the bedrock, bentonite buffers in the disposal holes and bentonite-based back-filling together with concrete plug structures in tunnels and shafts. Sealing of the disposal holes is especially challenging task; the holes must not be too dry, because bentonite buffers require water to swell. Grouting is planned to be carried out in water conductive fractures and fracture zones, in disposal holes if required, and also in the vicinity of the plug structures. The grouting materials are restricted to bentonite- and cement-based materials for the moment.

Bedrock is heterogeneous and anisotropic medium, therefore groundwater leakage controlling is not a simple task. Problems are for example where the sealing is needed, what is the amount of leakage after grouting, how to get adequate information of rock properties and what are suitable methods and materials to use? There is no grouting method or material that would be suitable for every kind of rock mass, and furthermore the small-scale properties of rock, which are very important in grouting, cause the biggest problems. The sealing work, especially the permanent sealing, should always be designed considering the site specific characteristics carefully. It is perhaps possible to avoid harmful and difficult fracture zones in layout-design if the bedrock properties are well known.
The watertightness is one of the major problems in underground construction and the subject is actively discussed abroad as well as in Finland in recent decade. Connected to the nuclear waste disposal, the water ingress problem arises again in evaluating the constructability of the nuclear waste repository in Finland. In that project rough estimations of the amount of leakage water and groutability of rock were made for every investigation site (Äikäs et al. 1999a, b, c and d). In the turn of decade 1990, extensive research work in grouting were carried out in Stripa, Sweden. One of the conclusions of field experiments in Stripa-project was that there are no adequate methods to localize a leaking zone exactly and to determine the hydrogeological and fracture characteristics and thus the groutability of rock. Both the grouting result and financial results would be improved by developing a procedure to determine the groutability characteristics of rock. Also for example Riekkola (Bäckblom & Svemar ed. 1994) has concluded from Finnish experiences that development of investigation methods is needed to get a more realistic picture of zones to be grouted.

The purpose of this literature study is to chart the methods to estimate the amount of leakage water into an excavation as well as to study and discuss the groutability of rock and chart the methods to estimate that. Although water leakages and groutability are closely related to each other, the scales of those problems are different. Water leakage estimations require large- or medium-scale knowledge of rock mass and fracture zones, and groutability estimation requires that of medium- to small-scale. Here the geological factors of these questions are of major interest, and the grouting material and methods are outlined when possible.
2 ESTIMATION OF GROUNDWATER LEAKAGE

2.1 General

Groundwater inflow is a common problem that tunnelers have to face when tunneling under groundwater table. Water can stop the construction operations for long periods, cause technical problems that lead to heavy economical losses. Cesano (1999a) has clarified the problem of water leakage generally with a diagram presented in Figure 2-1. Problems related to water inflow are many, concerning for example the excavation work, labour protection and maintenance of the repository during the utilization as well as stability, geotechnical and number of environmental reasons. For example a little water leakage into an excavation can cause a remarkable drawdown of groundwater table (Carlsson & Olsson 1978), which is a serious and common problem when tunneling in urban areas. If groundwater inflows can be predicted in advance, the best changes to control and restrict them are still available and many complications can certainly be avoided.

Traditionally in common tunneling cases in Finland the maximum limit for acceptable leakages has been determined on the basis of maximum allowable drawdown of groundwater table, and the amounts of groundwater leakage, if estimated, have usually been rough based on Lugeon tests made in few preinvestigation holes. Most of the literature in this work was found abroad, mainly from Sweden, where water leakage problem seems to be considered more important. The fact that in Finnish literature only one simplified equation to calculate leakage amounts has been presented (Holopainen 1977, Finnish Civil Engineers 1987), tells something about the traditional custom.

The amount of water leakage into an excavation is affected by many factors like the size and dimensions of an excavation/tunnel/water conductive zone, its depth below groundwater table, groundwater recharge and hydraulic conductivity of rock mass/water conductive zone (geological and structural conditions). For calculation of water leakage into a tunnel the most important property is the permeability of rock (before and after grouting) (Brantberger et al. 1998). Also the composition and thickness of the overburden has been statistically proved to have a clear effect on water leakage amount (Cesano 1999b, Cesano et al. 2000) According to one Finnish account of several excavations there is no correlation between the amount of leakage water and the size of the excavation (Pöllä & Ritola 1989). Also Tolppanen (1997) has made a literature study (not published) of water leakage amounts into excavations, but those numerical values are not directly comparable because of different purposes of use of excavations and different construction acts, and thus, the author of this report found only a weak correlation between leakages and sizes. Generally the lack of correlation is explained by the fact that in hard crystalline rock a great majority of leakage volumes occur in relatively small spots or lines, typically in fracture zones crossing the excavation. One example is the VLJ-repository for low and medium level nuclear waste at Olkiluoto, Finland (volume 90 000 m³, about 1 km long tunnelsystem) where one fracture zone conducts nearly 70 % of the total groundwater ingress into the whole excavation (Hakala 1998). The hydrogeological characterization
of one major fracture zone in Underground Research Laboratory URL in Canada revealed significant variations in hydraulic transmissivities along the zone (Bäckblom & Svemar ed. 1994). The variations ranged up to six orders of magnitude for locations within 10 m of another. Only a small proportion of all joints are hydraulically conductive according to investigations; Janson (1998) found in his field experiments that 10 - 15 % of total joints were water conductive and of those about half could be grouted. As well the groundwater pressure and other hydrogeological conditions affect the leakages strongly, it seems to be evident that the dimensions of an excavation are of minor importance. On the other hand, for example Carlsson and Olsson (1981a) has presented a diagram of water leakage as a function of depth and of area of tunnel. This may be actual for example in homogeneous porous rock masses.

The purpose of prediction is an essential thing to think about. It determines the extent and scale of investigations and measurings, as well as the accuracy of the results and interpretations. The purpose of the prediction is a function of various variables, among which the type of the underground construction, the economy of the project, and the kind of inflows to be predicted are the most relevant (Cesano 1999a): A very detailed hydrogeological study is usually not necessary in an ordinary tunnel project, where the construction techniques and the reinforcements are planned for the worst conditions. For underground repositories of toxic waste, like nuclear waste, where the prediction of inflows is not only related to the construction purposes but also to identify the minor and major fractures and fracture zones that can transport toxic matter to biosphere, a study of the rock mass on a number of scales is necessary.

Making prognoses of groundwater leakage into excavations is a multi-phase task. Cesano (1999a) has divided the predictive methods into groups of qualitative, quantitative and mathematical methods (see Table 2-1). Qualitative and quantitative methods are needed to localize where the leakage would happen and mathematical methods to define how much water will flow into the excavation. The problem and process is illustrated in Figure 2-2. In this context the mathematical methods are in main interest, and besides them the others are shortly discussed. The description of qualitative and quantitative methods, and above all, of numerical methods, is based on literature (of which the most important was the state of the art -report of Cesano (1999a)). The study of analytical methods is mainly derived and compiled by the author.

2.2 Qualitative and quantitative methods

The purpose of all field work is to find the significant features of the rock mass. Qualitative methods identify the components and geometry which control the groundwater flow and convey groundwater into the excavation. It is typically descriptive-type information and needed to focus those areas that require hydraulic characterization. Quantitative methods measure and quantify the properties, which control the groundwater flow.
Problems related to water inflows into tunnels and underground excavations

- Need for water pumping
- Stability problems
- Difficult construction conditions
- Deterioration of facility
- Impact on water supply
- Changes in groundwater chemistry
- Impact on vegetation
- Subsidence of ground

Figure 2-1. Main technical and environmental problems caused by groundwater leakage into underground constructions (Cesano 1999a).

Table 2-1. Subdivision of the predictive methods (Cesano 1999a).

<table>
<thead>
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<th>Qualitative methods</th>
<th>Quantitative methods</th>
<th>Mathematical methods</th>
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<td>Evaluating of existing material</td>
<td>Hydraulic tests</td>
<td>Analytical solutions</td>
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<tr>
<td>Remote sensing and aerophoto interpretation</td>
<td>Discharge hydrograph</td>
<td>Numerical solutions</td>
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<td>Outcrop and fracture mapping</td>
<td>Flow meter</td>
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<tr>
<td>Airborne and surface geophysical investigations</td>
<td>Other quantitative methods</td>
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<tr>
<td>Geochemical investigations</td>
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<td>Geographic information systems</td>
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<td>In-hole investigations</td>
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<tr>
<td>Stress measurements</td>
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<tr>
<td>Other qualitative methods</td>
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As mentioned before the scale upon which to collect information and thus selection of the most suitable methods are the main problems. They should be decided depending on the requirements of the project - for example investigations needed for nuclear waste repositories are many magnitudes more detailed than those for mines or sewer tunnels. The need of more detailed information increase the need of more detailed small-scale investigations but, however, don't decrease the need of large-scale investigations.

Qualitative methods for the estimation of groundwater leakages include visual inspection (maps, core recovery, outcrops), and geophysical and geochemical investigations. Proceeding from the regional to the local scales, the methods to be selected have to shift gradually from the most general methods, such as mapping, remote sensing and aerial geophysics, to more specific methods such as core recovery inspections, ground and in-hole geophysics and geochemical investigations.
INITIAL CONDITIONS, WHEN NO INFORMATION HAS BEEN ANALYSED

GEOLOGICAL AND HYDROGEOLOGICAL PICTURE AFTER THE USE OF QUALITATIVE METHODS

QUANTITATIVE METHODS SHOW THE AREAS THAT REGULATE MOST OF THE GROUNDWATER LEAKAGE

MATHEMATICAL METHODS ESTIMATE LEAKAGE AMOUNTS AT SPECIFIC LOCATIONS

Figure 2-2. The procedure to predict groundwater leakage into an excavation (Cesano 1999a, 1999b).
In addition to regional maps, information of large scale features, even in two or three dimensions, could be obtained from remote sensing method and aerial/satellite photo interpretation. Although depending on a scale of remote sensing images, only major lineaments may be able to be detected, but it is reasonable to consider these lineaments as major discontinuities that regulate groundwater flow (Cesano 1999a). However, among other problems related to interpretation, the method is very subjective. The combination of the earlier mentioned methods gives useful information of tectonic situation in the area. It should be remembered that based on those methods the water capacity of these geological features to give water problems to the excavation can not be estimated properly (Cesano 1999a).

Geological or hydrogeological mapping or for example tomographic or tectonic maps, if available, are of essential use - or in most cases, of vital use when planning and optimizing further investigations. Especially outcrop and fracture mapping as well as core recovery inspection give useful information for hydrogeological characterization of the most important fracture sets or zones. Local features may be able to be recognized as well as a general idea of geometry and properties of material may be gotten. The fracture properties usually mapped are for example strikes and dips, length, spacing, roughness and fillings and so on. However, in many times those are not enough for the identification of the water flowpaths (Cesano 1999a). To complete this shortage collection and analyzing of water flow information of fractures can be significant, but it is seldom available in outcrops.

Geophysical methods give indirect information of geometry and dimensions of geological features, even in three dimensions. Geophysical methods include for example magnetic-, geoelectric-, and seismic methods and ground penetrating radar. Usually relatively serviceable and relevant information for water leakage estimation purposes can be obtained by selecting some complementary ground geophysical methods in different scales. In-hole geophysics, for example seismic method and ground penetrating radar, produce more detailed small-scale information for hydraulic characterization. Useful information of hydraulic conductivities may be also available from rock stress measurements based on the logic that as rock stress increases, fracture apertures become smaller and thus hydraulic conductivity diminishes. The fracture orientations and the character of the stress field have naturally an effect on that phenomenon.

As well as core recovery inspection, borehole TV and other in-hole viewers offer direct information and are therefore more reliable and objective. They give reliable information just in the vicinity of the hole surface. With these methods the locations and dimensions of structures and zones, which are identified with the help of other methods, can be verified. In order to get as much information as possible from the boreholes, all available information should be used to find the best location and orientation of the holes (Moye 1967 referred by Fransson 1999).

Somewhere between qualitative and quantitative methods are geochemical investigations including especially isotopic and tracer analysis. Geochemical analysis can be of considerable help when estimating groundwater flow rates and directions as
well as the connectivity of conductive zones/discontinuities or even channeling of flow in fractures. They are exploited widely in advanced research projects concerning for example nuclear waste disposal (e.g. Laaksoharju 1999). It should be remembered also that the natural condition of the groundwater is easily disturbed by drilling and sampling.

Quantitative methods are mainly hydraulic tests, which estimate hydraulic characteristics such as hydraulic conductivity (K), and flowmeters, which detect the most hydraulically conductive fractures. For water leakage estimating purposes, the hydraulic measurements are of vital importance. Reliable measuring results are absolutely necessary. However, it's difficult to get representative results or interpretation and therefore the understanding of geology is necessary. The qualitative investigations must not be neglected.

Äikä vä (1984) has compiled a comprehensive summary of hydraulic tests. The names of hydraulic tests vary and are partly related to the measuring devices. According to the devices hydraulic tests can be divided to one-plug-tests and multi-plug-tests. Fransson (1999) divides the tests into pump-in and pump-out tests or single-hole and interference tests (cross-hole), and for example for grouting purposes Houlsby (1990) has divided tests to two types: exploratory testing and grout hole testing. Regardless the type, the following parameters are determined in all advanced tests: hydraulic pressure as a function of time, hydraulic conductivity of the medium, temperature of the groundwater and flow rate as a function of time (Äikä vä 1984).

The hydraulic tests are for example water loss measurements (constant head and constant flow rate tests), pressure fall-off tests, pulse response tests (e.g. drill stem test, slug-test, pressure pulse test) (Äikä vä 1984, Fransson 1999) and difference flow measurements (Öhberg & Rouhiainen 2000). Different methods are best to be regarded as complementary methods; combining some different methods the best results in understanding the hydraulic character of rock mass can be obtained.

Water loss measurements (includes constant head and constant flow rate tests) are much used in characterizing hard rock masses also for example in nuclear waste disposal investigations in Finland (constant head tests). Also Lugeon test, which is a common investigation method in conventional tunneling projects (e.g. Houlsby 1990), belongs to group of constant head measurements. Constant head tests of long term presuming transient state are discovered to correspond to real circumstances and it gives information of large rock volumes (Äikä vä 1984). In addition to hydraulic conductivity it is also possible to determine for example nature of fracturing and skin-effect. Constant flow rate tests gives as useful information as constant head tests but a constant flow rate is difficult to accomplish. In pressure fall-off tests a certain amount of water is injected to test section and the changes in pressure head are observed. Drill stem test is fast and cursory suitable for detecting water conductive parts in geological formations and slug-test is slow and limited only for confined aquifers. Pulse tests are meant for measuring hydraulic conductivity of single fractures (or short test lengths) of low conductivity, but the repeatability of test is poor. Difference flow measurement is under development and besides it is fast and thus relatively cheap method, it gives useful information on hydraulic conductivity and
flow rates and hydraulic head (Öhberg & Rouhiainen 2000). This method is also used in nuclear waste disposal site investigations together with water loss measurements.

In water loss measurements the length of test section can be varied depending on the purpose; with long test sections the information of the total hydraulic conductivity of rock mass is obtained, and with short test sections the values of hydraulic conductivity of fractures zones or even single fractures can be received. For water leakage estimations the test length should be focused to represent exact the leaking fracture/fracture zone or for example the uniform section of intact rock.

Understandably, but also referring to experiences from URL (Bäckblom & Svermar ed. 1994), if profound understanding of hydraulic properties of a fracture zone is needed, it would be necessary to drill and perform hydraulic tests on a large number of holes penetrating the zone at various locations.

2.3 Mathematical methods

Mathematical methods are needed for designing construction operations and are directly dependent on accuracy and adequacy of qualitative and quantitative methods. The most important uncertainties in water leakage estimations are not related to the mathematical equations, but in the understanding hydraulic properties and inhomogeneity of nearby rock mass as well as the values of those parameters. Here two basic approaches are derived. Besides, varieties of them presented in literature as well as some other methods are also shown.

2.3.1 Analytical methods

Well approach for shallow excavations/tunnels

Suitable and sometimes used approach for the situations when an excavation or a tunnel is situated relatively near below the groundwater table is based on the classical Thiem's well equation (Thiem 1929) for wells in confined aquifer, which is derived as follows (Airaksinen 1978):

\[ q(\vec{r}) = K \bar{J} = -K \nabla \phi(\vec{r}) \]

where \( q(\vec{r}) \) = flow rate in point \( \vec{r} \) (m/s), \( K \) = hydraulic conductivity (m/s), \( \bar{J} \) = hydraulic gradient (-) and \( \phi(\vec{r}) \) = potential in point \( \vec{r} \) (m), other basic equations for groundwater flow are applicable for radial flow into a well. The following assumptions have to be made for deriving the basic equations of well hydraulics (Airaksinen 1978): the area of aquifer is unlimited, aquifer is homogeneous and isotropic, the well extend to the bottom of conductive zone and losses of falls in perforated casing sections are insignificant. Even in exceptional cases these assumptions are not valid for real aquifers, but basic solutions got with them enable rough estimations.
Earlier mentioned assumptions and Dupuit's approximation (Airaksinen 1978) have to be made when deriving equations for flow into a well, which proclaim the interdependence between water extraction (flow rate $Q$) and drawdown of groundwater table ($s = h_0 - h_1$). According to Dupuit's approximation the flow per unit width of vertical surface is constant and according to Dupuit's flow equation the free groundwater surface is parabolic.

In confined aquifer Dupuit's approximation do not cause errors, because flow is horizontal. In polar coordinate system, where a well is situated in the midpoint of a round island, the radial flow into a well is (Airaksinen 1978):

$$Q = 2\pi rbK \frac{dh}{dr}$$

where $b$ = thickness of the confined groundwater layer (m), $r$ = distance from the well (m), $dh/dr = \text{gradient of groundwater table} (-)$. The situation is illustrated in Figure 2-3. After arrangements and integrating with the limits $h = h_w$, when $r = r_w$ and $h = h_0$, when $r = r_0$, we get

$$h_0 - h_w = \frac{Q}{2\pi Kb} \ln \left( \frac{r_o}{r_w} \right)$$  \hspace{1cm} 2-3$$

After rearranging and assuming that the well is situated in wide confined aquifer, the equation gets a form

$$Q = 2\pi Kb \frac{h - h_w}{\ln(\frac{r}{r_w})}$$  \hspace{1cm} 2-4$$

**Figure 2-3.** Constant flow into a well in confined aquifer in an imagined round island (Airaksinen 1978).
Shallow, round/broad shaped excavation can be approximated to be an enormous well with radial groundwater flow into it. The overlying rock mass (roof) is not considered, it's acting like a upper part of the imagined well. The radius of the well is meant to be fixed the same as the radius of the imagined circle covering the excavation. When the dimensions of the excavation are given for example as done in Figure 2-4 (Gustafson 1986), the equation gets familiar outlook:

\[
Q_T = \frac{2\pi T \cdot s}{\ln \left( \frac{R_o}{R_c} \right)}
\]

Where \(Q_T\) = total groundwater leakage into excavation \((m^3/s)\), \(T = K_b = \) grosstransmissivity \((m^2/s)\), \(s = h_o - h_w =\) drawdown of groundwater table \((m)\), \(R_o =\) assumed influence radius of the flow \((m)\) and \(R_c =\) radius of the imagined circle \((m)\).

About the applicability of the above mentioned equation 2-5 one should notice that the unusual variables mentioned above play a remarkable role in the equation. To estimate the drawdown detailed modeling or reciprocity analyses based on interference pumping tests are needed (Gustafson 1986). On the basis of the estimated drawdown and the understanding of material characteristics one can make assumptions of the influence radius. Airaksinen (1978) suggests that the estimation of the influence radius of the flow is arbitrary. He presents values varying between 500...1500 m for fractured hard rock. It depends on a type of aquifer as well as on the structure of the well (or an excavation) and on water extraction. The gradient of the groundwater table nearby a sink point has not been investigated much in practice. Observations of gradients of 1:100 have been made (Finnish Civil Engineers 1987). In that kind of a case the drawdown of 5 m induces an influence radius of 300 m. In materials with high hydraulic conductivity the gradient can be in order of 1:1000 or more. To give a value to grosstransmissivity one should have a comprehensive understanding about the distribution of hydraulic conductivity of rock mass and fracture zones. For investigation sites of spent nuclear fuel in Finland, the drawdown caused by tunnels has been estimated to be about 60 – 70 m and the induced influence radius of flow to be in order of 1 km (Posiva 1999).

One more thing is that the method does not regard the depth of the excavation though the pressure of groundwater has a strong influence to the leakage. It is reasonable to apply the method only for shallow cases. In spite of before mentioned negative properties, the method has been used quite successfully for example in Lyckebo: The estimated leakage was 103 l/min and the measured was 110 l/min (Gustafson 1986, Gustafson & Liedholm 1984).
Figure 2-4. The dimension parameters when calculating the total leakage into an excavation of any shape (Gustafson 1986).

Another approach of the same kind of an idea has also been presented (Gustafson & Liedholn 1984). Here the results of borehole observations has been simply scaled to correspond the dimensions and influences of an excavation (enormous well) and after reducing the following equation gives the leakage into the whole excavation:

\[
Q = q_b \frac{\ln \left( \frac{r_0}{r} \right)}{\ln \left( \frac{R_0}{R} \right)}
\]

Where \( q_b \) = median capacity of the borehole (\( m^3/s \)), \( r_0 \) = influence radius of borehole (m), \( r \) = borehole radius for which the capacity is estimated (m) and \( R \) = radius of the excavation (m).

Based on the same idea, if the excavation is empty to the depth of the floor of excavation at least under construction time the equation has been presented (Gustafson 1986):

\[
Q = d \cdot C_m \frac{\ln \left( \frac{r_0}{r} \right)}{\ln \left( \frac{R_0}{R} \right)}
\]

Where \( d \) = depth of the excavation (m) and \( C_m \) = median of the specific capacity of the rock mass (\( m^2/s \)).

Also for shallow tunnels (or oblong excavation) also the Thiem's well equation can be used. This approach is based on the idea of combined effect of several wells by summarizing the drawdown in infinitesimally narrow wells along the tunnel line (presented in Gustafson 1986, Olofsson 1991, Airaksinen 1978). The equation
includes the same parameters to be determined as the equation above (see also Figure 2-5), and it is derived to get following appearance (Gustafson 1986):

\[ Q = \frac{d \cdot C_n \cdot \ln \left( \frac{r_o}{r} \right)}{R_o} \tag{2-8} \]

The equation does not regard the size of the tunnel and it's suitable mainly for rough and conservative predictions. To apply this method the influence radius should be determined (rather estimated or guessed) carefully, because it affects directly to the result.

Leakage into a tunnel which intersects a steep fracture zone or a conductive rock type can be calculated by the Thiem's well equation assuming that the zone is not grouted and the tunnel is situated deep under the groundwater table (Figure 2-6) (Alberts & Gustafson 1983). The influence radius of the flow is assumed to be \( R_o = 2d \) and the drawdown of groundwater table is assumed to reach the tunnel (\( d \) corresponds \( s \) in the equation 2-5). This is a special situation and the equation gets the following appearance:

\[ Q = \frac{2\pi d \cdot K \cdot b}{\ln \left( \frac{2d}{R} \right)} \tag{2-9} \]

where \( Q \) = leakage through the fracture zone and \( b \) = thickness of the fracture zone and \( 2R \) (m) represents the characteristic diameter of the tunnel. The characteristics (thickness and its hydraulic conductivity) of the leaking zone should be determined carefully.

**Figure 2-5.** Factors used for the estimation of leakage into a tunnel using infinitesimally narrow wells (Olofsson 1991).
In this case of a leaking zone, to study the influence of the grouting one should take into account the hydraulic conductivity and "skin-effect" of the grouted zone and the altered drawdown (Alberts & Gustafson 1983) according to the equation 2-10. The grouting effect is illustrated in Figure 2-7.

$$Q = \frac{2\pi d \cdot K \cdot b}{\ln\left(\frac{4d}{D}\right) + 2\cdot \frac{K_1}{K} \cdot \frac{t}{D}}$$  \hspace{1cm} 2-10

Where $K_1$ = hydraulic conductivity of the grouted zone (m/s) and $t$ = thickness of the grouted zone (Figure 2-6). If $K_1 \ll K$ the equation can be simplified (Alberts & Gustafson 1983):

$$Q = \pi \cdot D \cdot b \cdot \frac{d}{t} \cdot K_1$$  \hspace{1cm} 2-11

_Imaginary well approach for tunnels_

The other method for tunnels is based on imaginary well approach, which is analogous to electrostatics of source-sink-system of two parallel linear conductor. The idea is the analogy of Darcy's and Ohm's laws. The method is also shortly described for example by Gustafson (1986), Olofsson (1991) and Cesano (1999a). The entire derivation of basic situation is presented in Appendix 1.

The derivation is based on three basic equations of groundwater flow: Darcy's law (eq. 2-1), the potential of flow field:

$$\phi(\vec{r}) = z + \frac{P(\vec{r})}{\gamma}$$  \hspace{1cm} 2-12

where $\phi(\vec{r})$ = potential in the point $\vec{r}$ (m), $z$ = z-coordinate (height) (m), $P(\vec{r})$ = pressure in the point $\vec{r}$ (kg/m$^2$s$^{-1}$) and $\gamma = \rho g$ = (kg/m$^2$s$^{-2}$), and the flow through a surface $S$:

$$Q_s = \int_{S} \vec{q}(\vec{r}) \cdot d\vec{S}$$  \hspace{1cm} 2-13

We assume a situation like in Figure 2-8, where the groundwater table is flat and the pressure on groundwater surface and in tunnel is same. We use the imaginary well method, where the real tunnel (a sink) and an imaginary tunnel (a source) are situated symmetrically at the same distance from the groundwater table. The same amount of water flows into the tunnel as "flows" out from the imaginary tunnel. The situation is analogous to the static electric field of two infinite long parallel conductor in different potentials. The potential field and as a consequence the flow pattern is like presented...
**Figure 2-6.** Leakage into a tunnel from fracture zone (Alberts & Gustafson 1983, Olofsson 1991).

**Figure 2-7.** Leakage through a grouted fracture zone (Alberts & Gustafson 1983).
in Figure 2-9. According to the analogy we get the potential $\phi$ of a source/sink, when the zero level is set on the groundwater table. When the potential fields of source and sink are combined, we got the total potential in a point $r$:

$$\phi(r) = \lambda \ln \left( \sqrt{\frac{x^2 + (2H - z)^2}{x^2 + z^2}} \right)$$  \hspace{1cm} 2-14

According to basic equation 2-12 the potential difference between the tunnel surface and groundwater level is:

$$\phi(0, R) = -H + R$$ \hspace{1cm} 2-15

Thus we get a value for the constant $\lambda$, which is put into the equation of gradient. If we assume that $R \ll H$ we get the gradient in the point $(0, R)$:

$$\nabla \phi(0, R) = \frac{H - R}{R \ln \left( \frac{2H}{R} \right)}$$ \hspace{1cm} 2-16

If we assume that the gradient is same in every point around the tunnel, we can use the assumption $q = $ constant and the flow is perpendicular against the surface, the flow rate per unit length of tunnel:

$$\frac{Q_s}{L} = \frac{2\pi K (H - R)}{\ln \left( \frac{2H}{R} \right)}$$ \hspace{1cm} 2-17

This equation has been presented (Vågverket 1993) for ungrouted tunnels of which the roof is situated about 1 tunnel radius or more below the groundwater table. If the radius of the tunnel is small in proportion to the depth of the tunnel under the groundwater table (the tunnel is situated at least 3-4 times deeper under the groundwater table than the tunnel radius), $R$ in the nominator can be neglected.

Based on the imaginary well approach the following variations has been presented (the summary presented in Vågverket-report (1993) which is based on for example Wiberg (1961), Reinius (1977) and Alberts & Gustafson (1983).
For shallow tunnels before grouting, the following equation based on same idea has been presented:

\[
Q = \frac{2\pi KL \cdot (H - R)}{\ln \left(\frac{H}{R} - \left(\frac{H}{R}\right)^2 - 1\right)}
\]

Where \( Q \) = leakage (m\(^3\)/s), \( K \) = hydraulic conductivity (m/s), \( H \) = water pressure expressed as a height of the water column (m), \( R \) = radius of the tunnel cross section (m). If the groundwater table is situated below the roof of the tunnel the special solutions are needed.

The following equations (Vägverket 1993) are based on the before mentioned considering also the skin-effect (\( \xi \)), which is a kind of correction factor, because the expected leakage is discovered to differ from the measured leakage.

The skin-factor has been used in groundwater technique and petroleum industry as an indication parameter to describe how well a well is connected to the surrounding formation, because of the before mentioned observation which diminishes the capacity of a well. The same phenomenon has been observed also between tunnel and rock. (Gustafson 1999).

\[\text{Figure 2-8. Estimation of leakage using an imaginary tunnel (Olofsson 1991 based on Gustafson 1986).}\]
The definition of skin-effect is variant and the explanation of the phenomenon is ambiguous. It has been thought that the effect is induced by the turbulent flow nearby in the tunnel periphery or for example due to the release of the dissolved gases in groundwater due to the fall of the water pressure. Grouting is also regarded as an artificial reason for the skin-effect by some researchers. The determination of the value of the skin-effect is undetermined and very difficult. Gustafsson presents a kind of solution for wells with the help of the drawdown. In the Åspö Hard Rock Laboratory the value of skin-effect is discovered to be 3...7, but it could vary more depending on for example the depth, geology and stress characteristics (Vägverket 1993).

The skin-correction is not needed for the tunnels situated right under the groundwater table (depth of the roof < tunnel radius) because of the low water pressure. For tunnels situated deeper (depth of the roof > tunnel radius) the equation 2-17 is modified as following when considering the skin-effect:

\[
Q = \frac{2\pi K L \cdot (H - R)}{\ln \left( \frac{2H}{R} \right) + \xi}
\]

The skin-effect problem is not studied for shallow tunnels, only for deep tunnels. Assuming that all the fall of the water pressure occurs in the grouted zone, which could be passable for shallow tunnels with normal high hydraulic conductivity of the rock, the following equation can be used:

\[
Q = \frac{2\pi K_1 L \cdot H}{\ln \left( \frac{R + t}{R} \right) + \xi}
\]

Where \( t = \) thickness of the grouted zone (m) and \( K_1 = \) the hydraulic conductivity of the grouted zone (m/s) (which is calculable according to Nonveiller (1989)). The value of
skin-effect is assumed to be about the same magnitude than in ungrouted zone, although the subject is not studied.

The following equation is suitable for the deep tunnels after grouting, where the tunnel is situated at least 3-4 times deeper below the groundwater table than the tunnel radius:

\[
Q = \frac{2\pi K L H}{\ln\left(\frac{R+t}{R}\right) + \frac{K t}{K} \ln\left(\frac{2H}{R+t}\right)} + \xi
\]

If the original hydraulic conductivity of the rock is high \((K \geq 10^{-6} \text{ m/s})\) and \(K\) after grouting is at least 10 times lesser, the equation can be modified to be the same than equation 2-20.

The infinitesimally narrow well method and the imaginary tunnel method have been used for the prognosis of leakage into the Bolmen tunnel at Staverhult in Sweden. Using the values of specific capacity and hydraulic conductivity from hydraulic tests in the area, the leakage was estimated to be 66-768 l/min/km. Measurements of water leakage into the tunnel after pregrouting gave 360-444 l/min/km (Cesano 1999a, Olofsson 1991).

A simplification of before mentioned equations to calculate the total water leakage into a tunnel has also been presented – also in Finnish literature (Holopainen 1977, Finnish Civil Engineers 1987). The equation is described shortly also in (Carlsson & Olsson 1977).

\[
Q = \pi KhL
\]

Equations 2-18 and 2-22 give the same answer for the leakage with the \(h/R\) ratio approximately equal to 4. For a tunnel with the \(h/R\) ratio equal to 10, equation 2-22 gives a value about 50 per cent larger than equation 2-18. (Carlsson & Olsson 1977).

If one takes into account the difficulty in quantifying the parameters, the difference between simple and complicated equations is not very significant.

Other analytical solutions

The following text is partly compiled by the author and partly by Cesano (1999a). If the original references are not seen they are not mentioned in reference list of this report, but in the reference list of Cesano's report (1999a).

Besides the equation for steady-state-flow into a tunnel, which is the same as equation 2-17, Goodman (Olofsson 1991 and Cesano 1999a, based on Goodman et al. 1965) has represented a formula for transient flow into a tunnel:

\[
Q(t_s) = \left(\frac{8C}{3} K d^3 S_y t_s^2\right)^{\frac{1}{2}}
\]
where $t_x$ = time (s), $S_y$ = specific yield (-) and $C$ = constant (0.5 - 0.75). The formula for the transient case is valid only if the Deapit-Forchheimer horizontal flow assumptions hold and the drawdown of groundwater level has reached the tunnel (Figure 2-10).

Bello-Maldonado (1974, 1983) extended Goodman's unsteady-state solution by considering the inclined initial water table common in mountainous areas.

Barton et al. (1986) tried to estimate the flow from (or to) a linear equipotential toward (or from) a horizontal circular opening of length $L$ by using Goodman's approach. The ground surface (and water table) represents a linear boundary midway between a "source" and an equal and opposite "sink" similarly that presented before in imaginary well approach, but the parameters differ from those presented in equations 2-17 and 2-18.

\[
Q = \frac{2\pi \cdot K_m \cdot L \cdot g \cdot P_r}{\mu \log\left(\frac{2d}{R}\right)}
\]

where $K_m$ = equivalent mass hydraulic conductivity ($m^2$), $g$ = acceleration due to gravity (9.81 m/s$^2$), $P_r$ = excess pressure at opening (kg/m$^2$), $\mu$ = dynamic viscosity of water (kg/m/s), $d$ = the depth below the groundwater table (m) and $R$ = equivalent radius of opening (m). This equation was used during the construction of the Oslo rail tunnel (Holmenkollenbanen) giving a theoretical value of 11.1 l/min/100 m length of tunnel. The measured value for the early ungrouted tunnel was maximum 80 l/min/100 m which was reduced to 13 to 50 l/min/100 m after the worst sections were lined with concrete (Karlsrud 1981).

![Figure 2-10. A transient case (Olofsson 1991 after Freeze & Cherry 1979).](image-url)
A similar approach has been used to forecast water ingress into the Seikan tunnel (Kitamura 1986, Ohshima 1984). The latter used the following formula:

\[ Q = \frac{2\pi \cdot K \cdot d}{\ln\left(\frac{4d}{R}\right)} \] 2-25

Ueda and Sugio (1971 in Komada et al. 1980) consider a two dimensional case of a water flow into the unlined underground cavern excavated in isotropic and homogenous rock mass, where seepage is supplied from a horizontal water injection borehole having a finite horizontal length 2E:

\[ Q = \frac{2\pi \cdot K \left[ h - \left( \frac{P_c}{\gamma_w} - S \right) \right]}{\ln \left( \frac{a + (S - R)/\sqrt{E^2 + (S - R)^2}}{a - (S - R)/\sqrt{E^2 + (S - R)^2}} \right)} \] 2-26

where

\[ a = \sqrt{\frac{S^2 + R^2}{\sqrt{(S + R)^2 + E} \left[(S - R)^2 + E^2\right]}} \]

and \( P_c \) = pressure inside the opening (kg/ms^2), \( \rho_w \) = density of water (kg/m^3) and \( S \) = distance to the centre of the tunnel to the water injection borehole (m).

A complex solution to the problem has been given by Zhang and Franklin (1993), who point out that underestimation of the leakages is usually due to the assumption of a constant average hydraulic conductivity that is independent of the depth. The solution has the following expression:

\[ Q = \frac{2K_s \pi (\gamma_m - \gamma_w) \cdot \exp\left[ -\left( \frac{\gamma_m A}{\gamma_m - \gamma_w} \right) \frac{d}{\gamma_m - \gamma_w} \right] \cdot \exp\left( \frac{\gamma_w (A - a^3) H_o}{\gamma_m - \gamma_w} - 1 \right) \cdot \gamma_w (A - a^3) \cdot K_s \left( \frac{A}{4D} \right) - K_s (Ad) }{\gamma_w (A - a^3) \cdot K_s (Ad)} \] 2-27

Where \( \rho_m \) = density of overburden (kg/m^3), \( \rho_w \) = density of water (kg/m^3), \( K_s \) = surface intercept of the exponential conductivity gradient (m/s), \( A = -a_1 + a_2 + a_3 = \) contribution of stress dependent \( (a_1, a_2) \) and stress independent factors \( (a_3) \). Zhang and Franklin (1993) estimate that, depending on the value of \( A \), the formula can usually give predictions within ±15 %.
More about peculiar solutions is presented in Cesano (1999a).

A totally different approach from the other is suggested by Carlsson and Olsson (1977). A rough estimation about the amounts of leakage water can be made based on seismic data and borehole tests. The idea of the method is that we usually obtain the first information of the rock mass by geophysical exploration. However, the seismic velocity of rock mass does not give a direct measure of its water bearing capacity, though there is probably a certain synchronization between the seismic velocity and water bearing capacity. After certain assumptions Carlsson and Olsson presents

\[ Q_x = Q \left( \frac{\bar{v}}{V} \right)^{10} \]  

where \( Q_x \) = leakage into a certain section of a tunnel quantified according to the seismic velocity (m\(^3\)/s), \( Q \) = estimated total leakage (m\(^3\)/s), \( \bar{v} \) = mean value of the seismic velocity (m/s) of the rock mass which has been penetrated by the boreholes and \( V \) = mean value of the seismic velocity (m/s) of the whole tunnel line. The method was tested in the Forsmark tunnel, in Sweden (Carlsson & Olsson 1977). The calculated leakage exceeded the measured by about 50 %. A comparison between the intensities of the leakage in the seismic zones and some rock-mechanical zones shows a fairly good agreement. However, there are a number of zones in the calculation with very high leak intensities which do not have any correspondences in the tunnel. The suspected reasons are many described by Carlsson and Olsson (1977).

Assumptions, limitations and applicability of analytical solutions

The following text is compiled with the help of the state of the art report of Cesano (1999a).

Analytical solutions are a simple and rapid as well as easy to understand and sometimes possible to be used by non experts in hydrogeology. However, very many assumptions are considered in the analytical solutions, and thus they may often underestimate the complexity of the problem and can seldom be considered representative of the real hydrogeological conditions. The user should investigate the degree of reliability and range of applicability of these solutions, and to always use them with skepticism. The major assumptions usually adopted in the solutions are:

1. Countless different factors and processes affects to the groundwater leakages, but just a limited number of parameters is (can be) taken into account.
2. Rock mass is considered homogeneous and isotropic, although in the scales to be examined, hard crystalline rock is never that in regard to fracturing, which governs the groundwater flow.
3. Groundwater flow may have turbulent or in general a non-linear character, which is not taken into account.
4. Hydraulic conductivity is assumed to have a constant value in volumes to be examined, although it varies strongly in hard rock masses.
5. The groundwater table is given a constant level and the potential field is strongly simplified. Also rock mass is considered to be completely saturated.

6. The hard rock mass and the potential overburden above it are two distinct elements. The water reservoir in overburden is not taken into account in analytical solutions.

7. The solutions are one- and two-dimensional but the studied volume is three-dimensional.

Considering all the limitations related to analytical solutions and as well the problems in measuring hydraulic properties of rock mass, it is concluded that analytical solutions can be applied only for (Cesano 1999a):

- very general assessments of leakage amounts
- to support numerical estimations
- to get a general idea of the pregrouting requirements.

2.3.2 Numerical methods

Computers have remarkably affected modeling groundwater flow numerically, and the use of modeling approach is widening as the practicableness and easiness of computers is developing. Nowadays numerical methods can be regarded increasingly an equal possibility to make predictions as any conventional method, because of increasing amount of specialists and decreasing costs of computers and software. As modeling has facilitated the predictions, it has as well contributed to better understanding of groundwater flow systems.

Numerical modeling programs are many, and they are based on statistical and geometrical conceptualization of data (Figure 2-11). Figure 2-12 shows the protocol that is for prediction of the groundwater leakages by numerical modeling. The following text is compiled based on a considerable part of a treatise of Cesano (1999a) (in cases the original reference is not studied it is not mentioned in the reference list here, but in the reference list of (Cesano 1999a).

![Figure 2-11. Mineral composition, origins of discontinuities and methods of characterization may be used to conceptualize a rock mass (Fransson 1999).](image)

Rock mass & Discontinuities

Mineral composition & Alteration

Origins of discontinuities

Methods of characterization

Conceptual model
Steps in a protocol for numerical modelling

1) Scale for prediction: a) aim; b) geometry and depth of the excavation; c) homogeneity and anisotropy of HR terrain

Qualitative methods: collection and analysis of qualitative data for a geometrical characterization and location of main groundwater bearing features in the overburden and rock mass

Quantitative methods: collection and analysis of quantitative data for a hydraulic characterization and an estimation of the capacity of the features identified by qualitative methods to transmit water

2) Conceptual model

3) Mathematical model and code selection

4) Model design

5) Calibration

6) PREDICTION

7a) Validation not consistent with prediction

7a) Validation consistent with prediction

8) Conclusions (e.g. good knowledge of the hydrogeological conditions, reliable hydrogeological characterization, reliable modelling protocol, etc...)

Figure 2-12. A modeling protocol for estimation of groundwater leakage (Cesano 1999a).

The first step is to select a scale for prediction according to the aim/purpose of prediction, the heterogeneity and anisotropy of the rock mass, and the geometry and depth of the construction. A rock volume can be regarded homogeneous and/or isotropic in certain scales, while heterogeneous and anisotropic on other scales.

Next the qualitative and quantitative information is integrated to form a conceptual model, which describes the features that regulate the water flow. A mathematical model is then selected and adjusted to express the conceptual model in mathematical terms.

Calibration reassesses the model, while validation is necessary to estimate the reliability of predictions. The prediction output is then an assessment on the leakage at a certain location. If the leakage is consistent with the measurement, the modeling protocol can be considered as reliable, otherwise it is necessary to review the whole procedure or at least part of it.

The conceptualization and selection of the numerical code is a chain of approximations. The validity of a conceptual model is dependent on the number and quality of assumptions and/or simplifications that are made, and the following work is based on it. The properties of fractures and the rock mass must be simplified and then
described statistically/numerically, hence the model is simple enough for practical calculations. The representativeness of data available is important to understand. It should be also reminded that approximations at each stage affect also the other stages, and therefore have serious consequences on the final result.

The homogeneity and anisotropy are important factors also in selecting mathematical code, which are many. Different flow equations with different simplifications and assumptions can be used in models, and the ways of representing discontinuities in fractured media are many. Generally used classes of methods for accommodating the heterogeneity of hard rock masses are the following three: discontinuum, continuum and dual continuum approaches. In discontinuum approach the water flow occurs in defined net of discontinuities. In continuum approach the rock mass is regarded to have certain throughout conductivity or fracturing. Dual continuum approach is a combination of those two approaches so that it is built up of a network of discontinuities certain conductivities (fracture zones and major fracturing) and the medium of different conductivity between them (minor fracturing and porosity). Different approaches are illustrated in Figure 2-13.

The choice of the most suitable approach depends on the local geological and hydrogeological conditions, the available data as well as on the skill of the modeler. The dual continuum approach requires a considerable amount of information on the property of the media, but in tunnel-scale it seems to correspond best the hard rock masses. About the applicability of other approaches, for example Nonveiller (1989) criticizes the model of porous medium (continuum approach) applied to fissured rock to be a very crude approximation of the real conditions; fractured and fissured rock is substantially different from porous media. The factors governing the flow of water through fissures are: direction and position of discontinuities, their spacing, degree of fracturing of the medium, width of fractures, roughness and shape of the walls of fractures, material filling the fractures, permeability of the soils between fractures.

Many modeling procedures are two-dimensional (2D), while the real situation is three-dimensional (3D), or even four-dimensional (4D) when time is also involved. Therefore, 3D models are more desirable since they are more representative of the local field situation.

One problem is also to give representative values to input data, which consists always of a small amount of samples and can seldom be regarded statistically random, when it is a question of geological features. Certain properties are not so available for investigations and particularly those properties may be most important in regard to water flow in rock mass. Two different approaches can generally be used to enter parametric values depending on the quality of data and parameters: probabilistic approach and deterministic approach. In the former approach variables or parameters used to describe the input-output relationships are not precisely known, and in the latter a definite fixed value is assigned to each variable. In general, if precise predictions are required, e.g. in modeling for underground repositories for nuclear wastes, the deterministic approach is more suitable than the probabilistic (Cesano 1999a).
Validation is best to perform as a comparison between predictions derived from the model and from empirical observation. It is not always possible, and if it is, there are some problems related to measuring the water leakages accurately. Water leakages are possible to be measured only for sections of the excavation, which are typically relatively long (or even the whole excavation) and the leakage rates are often a kind of average in certain time period. This should be paid an attention to validation.

Considering the before mentioned problem, if validation shows that the observations are not consistent with the results of the model, the model protocol might be revised. If unacceptable inconsistencies persist even after accurate reviews, some serious mistake or assumptions have been made both during data collection and/or in the modeling protocol. However, sometimes the geological and hydrogeological conditions are so complex that an accurate hydrogeological characterization would result too expensive. It should be also understood that in a case the model is consistent with measured data, the applicability of it in some other areas remains still somehow uncertain because of strong heterogeneity typical to hard rock masses.

The opinion about reliability and usefulness of groundwater models is naturally subjective and varies a lot. The modeling procedure involves many stages that are
interrelated to each other, and this might be one of the main reason why predictions of groundwater leakages are easily and sensibly underestimated or overestimated (Cesano 1999a). He also suggests that a prosperous model may give a good description of large scale flow and average and/or approximately leakage into the whole excavation, but the details concerning, i.e. an individual fracture zone or one short part of the tunnel need extremely accurate description of rock characteristics. Cesano sees numerical models more reliable than analytical methods and emphasizes the importance of large amount of good quality data, but for example Rojstaczer (1994) consider the ability of numerical models poor and estimate them to be suitable only for qualitative predictions with order of-magnitude accuracy.

2.3.3 Comparison between analytical and numerical methods

A general and rough comparison between analytical and numerical methods is presented in Table 2-2. (Table is based on the perusal of Cesano (1999a) with some modifications and additional comments).

Considering the advantages of both analytical and numerical methods it seems that in demanding tunneling projects the best results in reliability and usefulness could be obtained by using both methods complementary. The idea for layout- and preliminary grouting design may be obtained with numerical estimations and the model can be improved during the work using latest in situ information, but analytical methods should be in use when rapid decision for example in grouting are needed. Also continuous comparison between numerical and analytical methods, and of course with observations, are of big help in estimating the reliability of different methods in a certain rock environment. In conventional tunneling projects analytical methods are of major importance and the use of numerical methods should be evaluated depending on the requirements of the watertightness of a facility.
Table 2-2. Comparison between analytical and numerical method (with modifications and additional comments to Cesano 1999a).

<table>
<thead>
<tr>
<th>ANALYTICAL METHODS</th>
<th>NUMERICAL METHODS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>To be used</strong></td>
<td>At any stage</td>
</tr>
<tr>
<td><strong>Aim</strong></td>
<td>Quantification of leakage amounts on the most general scale</td>
</tr>
<tr>
<td><strong>Specific tasks</strong></td>
<td>Provides a measure of the hazard for groundwater leakages on a general basis as a rough assessment in order to optimize the construction techniques and operations</td>
</tr>
<tr>
<td><strong>Time required for data collection and data analyses</strong></td>
<td>Short time for computation; long time for data gathering</td>
</tr>
<tr>
<td><strong>Costs of money required for data collection and data analyses</strong></td>
<td>Moderate – high, mainly due to estimations of K values</td>
</tr>
<tr>
<td><strong>Usefulness of the prediction</strong></td>
<td>Assessment with low reliability</td>
</tr>
<tr>
<td><strong>Time</strong></td>
<td>Fast and rapidly applied when excavation is in progress and fast decisions are needed</td>
</tr>
<tr>
<td><strong>Relation between costs and usefulness</strong></td>
<td>Low?</td>
</tr>
<tr>
<td><strong>Reliability</strong></td>
<td>The results are dependent on the amount and quality of the input data (geological, hydrogeological, and hydraulic information). Errors in order of magnitude of 100 are possible</td>
</tr>
<tr>
<td><strong>Use of experts for reliable analysis</strong></td>
<td>Not necessary</td>
</tr>
<tr>
<td></td>
<td>Necessary</td>
</tr>
</tbody>
</table>
3 ESTIMATION OF GROUTABILITY OF ROCK

3.1 General

According to the requirement of the allowable water leakage into the excavation and the results of pre-investigations one should determine if sealing of the rock in the vicinity of the excavation is needed, and if sealing is needed, how extensive it should be and how to do it. Rock grouting is one possible and much used method for sealing. In order to get rock mass sealed by grouting, it is required that the fractures and fracture zones to be grouted can be localized and the grout will penetrate them, fill them and stay there.

There are many different things affecting to grouting performance. As well as geological factors the properties of grout and many other things have an influence to the grouting result. Despite relatively wide-scale experience, the task is not easy at all because rock as a construction material is usually very heterogeneous and anisotropic, and grout does not flow through cracks like water. The difference comes from the presence of cement particles, from density currents in the grout, from thixotropic stiffening of it and from a few other things (Houlsby 1990). This study here is concentrated on the geological factors even though it is not possible to limit totally outside the other factors.

A successful grouting operation means that (Gustafson & Stille 1996):

- the groundwater leakage is reduced to acceptable levels
- the grout penetrates far enough out into the rock to build a sufficiently thick barrier
- the grout is thin enough that it also penetrates the fine joints
- the grout is thick enough that refusal can be achieved by reasonable grout volumes (no uncontrolled grout pumping)
- the packers can be taken out after a short time without a back-flow of grout.

General opinion, which has appeared repeatedly for example in recent seminars concerning water leakages in excavations (Nordic Symposium on Rock Grouting 1998 (Helsinki University of Technology 1998, Finland (Eloranta ed. 2000)) and Water Leakage Controlling in Excavations 1999, (Technical Research Center of Finland)) as well as in some grouting research projects and experiences, seems to be that the biggest problems in tunneling are related to water leakages, which are often more or less difficult to control. Good investigations are widely regarded to lead to considerably better results. Investigations can be expensive but usually save money overall because they can allow the grouting to be more accurately tailored to conditions and therefore not wasteful. Besides the unsolved leakage problems the growing demand for the watertightness has arisen the question about grouting issue.

Among the others Houlsby (1990) presents that it is wise to investigate enough of the foundation to know how it will relate to the grouting and to decide how much grouting to do. Generally a good investigation preceding grouting design should be able to provide for example:
• reduction of uncertainties present in all grouting works
• information for deciding whether grouting is really needed
• information indicating which type of grouting is suitable
• data for designing grouting
• data for compiling an appropriate contract specification
• information for preparing realistic bids
• a basis for comparison with what eventuates during the progress of the work.

Commonly the grouting design has been mainly based on Lugeon test (Widmann ed. 1993). A water pressure test (WPT) -value of 1 Lugeon (= 1.3E-7 m/s) has been widely acknowledged limit of rock permeability for decades (Houlsby 1990, Ewert 1992); if permeability testing indicates higher values, a grouting program is considered necessary. Anyway, Lugeon test does not take the flow geometry and fracture apertures into account and can therefore be questionable (Widmann ed. 1993, Pöllä et al. 1994). Also the definition of Lugeon criterion logically presupposes that rocks of higher permeabilities are groutable and thus can be sealed by grouting to a defined degree, and that different rocks should be equally groutable (Ewert 1992). According to Gustafson, Rhén, Stanfors and Stille (Backblom & Svemar ed. 1994), several attempts have been made to correlate grout take with the measured water loss from packer tests and all of them have given a very poor correlation - actually 1980s the fact that there is not sufficient correlation between Lugeon values/groutability and grout takes has been realized (Ewert 1985; Lombardi 1985 and Houlsby 1990), Naturally, many specialists criticize the use of water pressure tests as the only method to evaluate the necessity of grouting and the groutability of rock (e.g. Ewert 1992, Weaver 1992, Blinde et al. 1983, Carlsson & Olsson 1981b, Widmann (ed.) 1993, Nonveiller 1989, Shroff & Shah 1993). For example Hässler’s (1991) opinion is that hydraulic conductivity as a parameter to describe rock mass from a water-flow point of view is relevant for calculations of net flows of water on a large scale. Practical experiences (Ewert 1992) indicate that

• the water absorption rates are frequently not in harmony with grout takes
• sometimes it is impossible to achieve a substantial reduction in the permeability, although the original WPT values were quite high and substantial amounts of grout were injected
• it can also happen that very little seepage occurs, although the original WPT values were high and grout takes were low
• it happens that no overproportionate head reduction takes place in spite of high grout takes
• rock types are not equally groutable, and they have their own individual groutability characteristics.

Considering the almost endless variety of different geological conditions, each with their specific water conductivity, it is evident that the basic presupposition for the definition of a Lugeon criterion general applicability does not exist (Ewert 1992). In spite of this, the criterion has been established and used. Nowadays the general opinion seems to be that, if possible, technical specifications, sealing materials and
placement methods should be derived and selected for a specific "seal location", i.e. considering the tightness of the criterion, the dimensions of the tunnel, the depth of the seals, characteristics of surrounding rock mass, and in situ fluid pressures and flows, rock stresses and temperatures and so on.

Naturally, as Houlsby (1990) maintain, a good knowledge of permeability is essential for grouting purposes. Among the others, for example Shroff & Shah (1993) emphasizes the following information to be obtained for adequate grouting design: boundaries and contacts of the different geologic units to be treated, dips and strikes of sedimentary of metamorphic rocks, bedrock contours obtained from surface and subsurface data, physical condition of the rocks, prominent joint system, their spacing, opening and character of the material of infilling if any, location, strike and dip of prominent faults, shear zones faults etc., lines of geologic cross-sections and location and logs of drill and auger holes, exploration tunnels and shafts.

One thing should be paid an attention to. The term "groutability" means different thing to people. Some regard groutability of rock as its capacity to take grout or as penetration of grout. Other sees it as combined effect of fracture properties which defines the maximum grain size of grout. Some consider it as quality of rock in regard to easiness/possibilities to seal it by grouting. In this report the latest interpretation is adopted, but the other interpretations are taken into account, too.

3.2 Geological factors

The characteristics of surrounding rock normally have direct impacts on the seal performance. Usually there is no need to have detailed geological knowledge; most times a general idea of the subject is sufficient, perhaps helped with the details by a geologist in special cases (Houlsby 1990). However, for example Nonveiller (1989) emphasize also the importance of understanding of geologic and tectonic processes and units. The need of investigations is of course a question about the quality of grouting one wants to get.

The geological factors which have an effect on the grouting performance are many and most of them are closely related to each other and often derived from few basic characteristics of discontinuities. The main differences in grouting different rock masses with different hydraulic properties culminates to the differences of properties of discontinuities. Different fractures are different to grout, hence the principal thing to understand is the fracturing (or void) systems. The next point of the interest is the physical properties of the rock. Although rock identification and characteristics are not prime factors most times, they shouldn't be ignored. Factors that more or less affect to the grouting performance are discussed below. Naturally one can never take most of them into account in grouting design. Here the main factors are divided into three groups: rock mass characteristics, fracture characteristics and hydrogeological characteristics. The grouts and grouting techniques are not here at main interest because they should be derived from the geological characteristics as one affecting main factor. The other phenomena, occurrences and terms associated to grouting event are described more detailed for example in (Houlsby 1990). Here the following

3.2.1 Rock mass characteristics

Rock type itself does not usually affect grouting, but many rock types have special characteristics in jointing system, porosity, weathering as well as physical and hydraulic properties, so the identification of rock type and its characteristics can give a rough general idea about the things which affect grouting. The appearance of joints depends on the origin and the tectonic history of the rock. The homogeneity of rock affects directly predictability and grouting design and isotropy, including orientation and it's grade or stratification may give preliminary idea about jointing system and directions in which water and grout flows. Uniformity (mainly of jointing) permits a regular layout of grout holes, whereas irregular jointing, dykes, disconformities, and so on may require placement of holes at various orientations and spacings. Apart from bulk fracturing, individual fracture or shear zones as well as other structures should be recognized and examined individually. They may differ significantly from the environment and have their own special characteristics. The various geological features and various grouting considerations are illustrated in Figure 3-1.

Physical properties of rock such as rock soundness, strength and stress have an effect on grouting and thus should be paid an attention to. Rock soundness may be taken into account, because holes that don't collapse permit grout easier than those that do collapse. Strenght affects directly so that grouting of strong, massive, well-anchored rock is usually easier than when working in weak, broken, loose materials where holes repeatedly collapse, or where blocks move (Houlsby 1990). If a foundation contains tectonic stress, the stability matters will need to take account of this. The stress also logically correlates with the hydraulic conductivity of rock mass - the increase of rock stress can be seen also as the decrease of hydraulic conductivity of rock mass. This correlation has been observed clearly for example in Underground Research Laboratory in Canada (Bäckblom & Svemar ed. 1994): high normal stress across the fracture zone under examination correspond to regions of low hydraulic transmissivities, while low normal stresses correspond to regions of high hydraulic transmissivities.

It is rare but possible that a certain chemical composition of rock type causes chemical incompatibility with grout and surroundings. Or that certain products of disintegration are not compatible with grout. Such chemically aggressive attack can develop for example in carbonaceous environment and can cause seepage (Houlsby 1990). An other thing is also the temperature of rock mass and the possible changes of it. That question arouses especially in nuclear waste disposal.

It is widely adopted that solely rock type is an incompetent and unreliable indicator about the groutability of rock, although rock types have often some congruent characteristics in porosity and fracture features due to a certain origin. However, for example Ewert (1985) has presented a rock classification system for grouting purposes based on rock type.
SPACING OF OPEN JOINTS

LEFT: EASIER GROUTING, RIGHT: HARDER GROUTING

SIZE OF OPEN JOINTS

LEFT: EASIER GROUTING, RIGHT: HARDER GROUTING

DIRECTIONS OF OPEN JOINTS

THE JOINTING ON THE LEFT REQUIRES DIFFERENT HOLE INCLINATIONS THAT THE SET ON THE RIGHT

UNIFORMITY

REGULAR HOLES SUIT THE LEFT JOINTING, BUT NOT THE RIGHT

ROCK SOUNDNESS

COLLAPSING HOLES CAN BE A NUISANCE

ROCK STRENGTH

STRONG ROCK IS EASIER TO GROUT

ROCK STRESSES

LEFT: TYPICAL UNSTRESSED ROCK. RIGHT: A CANYON IN STRESSED ROCK

PROMENESS TO PIPING

JOINTS CONTAINING SOFT MATERIAL THAT CAN BE PIPED (WAShed AWAY BY SEEPAGE)

Figure 3-1. Various geological features and various grouting considerations (Houlsby 1990).

3.2.2 Fracturing characteristics

In addition to the hydraulic properties of rock the fracturing characteristics are/should be of importance in grouting design. The fracturing characteristics could be divided into large-scale and small-scale characteristics, but the limit between them is at least indistinguishable. The large-scale characteristics could include for example spacing of open joints, joint continuity, joint directions and inclination. The small-scale
characteristics could be for example joint aperture, width, roughness, fillings, channeling and so on.

Spacing of (open) joints and their direction and inclination are important features to find out. Open, groutable joints are of interest, especially in cement grouting. If they are widely spaced, the grouting is usually easier than if closely spaced, when troubles such as frequent surface leaks, collapsing holes and patchy penetrations can happen (Houlsby 1990). More grout holes will usually be needed for close spacing than for wide spacing (Shroff & Shah 1993). Spacing of the grout holes is also controlled by other geological features. The whole joint system with directions and inclinations of joint sets determines the optimal spacing and orientations of grouting holes. The more right-angled the grouting hole cuts the jointing the better, even though it is seldom possible to cut several joint sets optimally. The attention must be paid to the importance of the different joint sets. Houlsby (1990) suggests that the easiest cracks to grout are near vertical and the hardest are those near horizontal.

Continuity of open joints affects penetration: lack of continuity means that there could be lack of flow paths for water, but it also means that if grouting is needed more grout holes will be needed, because of lack of penetration and ventilation paths. Short and sparse fractures in few directions are not as probable conduits for water or grout as long and dense fractures with several fracture directions. The continuity is related to joint length and directions. It has been given a linear relationship between joint length and maximum joint aperture (Janson 1998). This would suggest that long joints are more receptive to grout.

Joint width/aperture defines the main flow paths, is connected to grouted volumes and sets limits to the entrance and penetration of grout (especially grout with particles like in cement). It is the most essential parameter for the movement of fluid in rock (Janson 1998). But the relation between these is not straightforward; it is very complex relation because of rheology and other properties of grouts (Figure 3-2) and it can be estimated only theoretically. According to Houlsby (1990) the easiest joints to grout have widths in the range between 6...0.5 mm. The problems arise in both borders. Houlsby also presents a very arbitrary dividing line: cracks and spaces wider than 2.5 mm and extending 25 mm or more across the direction of grout flow can often pass grout more freely. In a report by Janson (1998) it has been presented that joints finer than 0...2 mm are not usually penetrated by conventional grout, but an extremely finely ground grout with admixtures can penetrate apertures down to 0.02 – 0.05 mm. It has been suggested also that in hard rocks largest apertures are often found in vertical joints and thus they are most water conductive (Janson 1998). Related to joint width such factors like roughness of fracture surfaces, spatial correlation, stiffness, tortuosity (Figure 3-3) as well as fracture fillings and so on also determines the possible flow paths/channels and penetration of grout. Profound research work of fracture properties has been done by Hakami (1995) and random modeling by Lanaro (1999). Together with the grout properties these things define the spaces where grout can or cannot (theoretically) penetrate. It should remind that because the changes in parallelism of opposite fracture surfaces (spatial correlation) the information got from ground surface of excavated surface may not be reliable, because usually only one cross section of a fracture can be seen and the information of aperture and roughness
can be distorted. Also such term as contact area has been adopted to describe fractures. It is the proportion of "tight" surfaces in the joints and it is also important factor. The investigations also have shown that only a small proportion of all joints conduct water (Doe & Geier 1990 and MacLeod et al. 1992, both referred by Janson 1998). All those factors above are also the reason for the channeling of flow; it has been widely discovered and acknowledged that remarkable part of water flows occurs in tubular channels instead of planar flow between planar fracture surfaces. The channeling of flow sets hard difficulties in determination of flow paths and thus in focusing the grouting. Those channels should be localized for favourable and effective grouting, but the task is extremely demanding, practically even impossible. Even a systematic grouting with very high hole density may not lead to the reduction of water leakage at all.

Kutter (2000) has studied the concentration of flow in branching and intersecting flow channels in small-scale hydraulic experiments on joint models with different apertures, channel geometries and fracture roughness. According to him, the interesting observations also in regard to the groutability question are many: even fractures under very high compressive stress can be expected to transmit water if the walls are slightly shear-displaced walls creating channeling - the chance for channeling effects increases with progressing shear displacement between fracture walls and the increasing roughness of fracture surfaces. The channeling of flow and strong tortuosity of channels enhance the transition from laminar to turbulent flow, and high speed flow in channels seems not to be significantly affected by the low speed flow elsewhere. Even the validity of cubic law does not hold for narrow fractures with flow channeling.

Figure 3-2. Illustration of what happens in cracks (Houlsby 1990).
Fracture fillings can cause unexpected behavior. Alteration and movements of soft and loose fracture fillings can happen suddenly or imperceptibly long time after grouting. Longer joints often contain a combination of different materials (Janson 1998). Also incompatibility between fracture fillings and grout can lead to surprising events. Material in joints can be moved by seepage, either by taking the material into solution or by eroding; Houlsby call this piping. According to Simmons (Bäckblom & Sverma ed. 1994) jet washing can be used to clean infilling material out of fractures to improve the effectiveness of grouting but that has not confirmed in practice. Some successful experience is got from URL.

At any hand, complete filling of fractures with cement grouts, is impossible. Cement grains and any fluid with lower viscosity than that of water can not crowd into such small openings that water do. Figure 3-4 illustrates this.

Figure 3-3. Fracture properties determined by fracture void geometry (Hakami 1995).
3.2.3 Hydrogeological characteristics

Apart from hydrogeological conditions and groundwater table, the hydraulic characteristics of rock are closely connected to fracture characteristics. Hydraulic conductivity/transmissivity depends on the amount, sizes and connectivity of the openings in the rock mass. Hydraulic conductivity is measured for a certain length and thus it represents an average hydraulic conductivity of that certain section. The determination of representative length of test section is difficult and depends on purpose of measuring as well as for example on the requirements for grouting results. The volume of representative element (REV) and thereby the representative test section will be derived from the amount of fractures and standard deviation of transmissivity distribution (Gustafson 1986). Fracture widths and spacings and thus hydraulic conductivity have been observed to be distributed lognormally in crystalline bedrock (Gustafson 1986). Traditionally hydraulic conductivity is considered to be in relation with grout takes, but it should always be remembered that the rheology of grout is not like that of water and so the results of water tests can not be directly applied (Figure 3-2). Also the water flow may be turbulent as well as linear, and that affects to the results.

Hydrogeological conditions and groundwater table need to be understood too. As the depth grows the hydrostatic pressure grows. High water pressures causes stability problems in addition to grouting problems. To get the grout to penetrate the grouting pressure must exceed the hydrostatic pressure - from theoretical point of view it has been found that the grouting pressure must exceed twice the groundwater pressure to get a good refusal and prevent fingering of the grout (Bäckblom & Svemar ed. 1994). The more the grout has to penetrate the more additional grouting pressure is required and that may lead to stability problems.

3.3 Short review on groutability research methods

Among many hydraulic tests the Lugeon test is widely accepted to be suitable for grouting explorations. Lugeon test is a water pressure test (WPT) which indicates the flow radially from the test hole (Houlsby 1990). It is relatively relevant and it is also
very simple to carry out. Not everyone uses the Lugeon test; alternatives are many (e.g. TRH-test, EIPD-test and ED-test (Houlsby 1990)).

Water pressure tests give precious information on hydraulic properties of rock and are thus of great importance in grouting design (see also Section 2.2). But as emphasized earlier, alone they are not enough for grouting design - among others, Houlsby (1990) gives a warning that Lugeon values must never be used on their own without some idea of the fracture sizes and spacing to which they apply. If at least moderately quality of grouting is needed, geological exploration is necessary for such purposes as site suitability and structural assessment for the grouting. Fransson (1999) suggests in her licenciate thesis that for the grouting prediction purposes the water pressure tests of short duration seem to be useful and effective method when combined with geological characterization. Also, for example, the researchers in former Stripa-project seemed to get quite satisfying results and experience about the grouting predictions by modeling on the basis of Lugeon testing and fracture mapping (Pusch et al. 1988). Opinion and experiences of the extent of geological characterization varies a lot.

According to Fransson (1999) the transmissivity and the specific capacity obtained by water pressure tests of short time duration should be sufficient to predict the geometry and grout take in a fracture. Also when characterizing a rock mass for grouting purposes, the rock in the vicinity of tunnel front is of major interest, and hence a short duration test should be sufficient. Fransson has named also some other advantages of this method: consistency throughout the different stages of grouting project, reasonable amount of data, the test is not especially time-consuming, the choice of grouting strategy is easier and predictions are improved. The distribution of specific capacities gives an idea of the geometrical variations within the fracture and should be useful as a guide in choosing grouting strategy.

In addition to careful determination of hydraulic conductivity and hydrogeologic conditions, following exploration methods can be suitable/recommended to understand the geological features of rock to be sealed (Houlsby 1990, Nonveiller 1989):

- surface inspection from ground surface, shafts and adits (geologic mapping and analysis of tectonic processes and units)
- geophysical measurements (e.g. electric resistivity and seismic velocity)
- core drilling
- borehole TV/periscope
- impression packer.

Geological exploration of the site to be grouted should commence with regional geological mapping, leading through to detailed examination of the site in particular. As mentioned earlier, thorough identification of rock types is not imperative for grouting purposes. Strength and other physical features are more important. Joint details need to be carefully sorted out. (Houlsby 1990).

An overall understanding of jointing system, structures and fracture zones can be received only from large-scale visual inspection from ground surface or tunnel. Recording and presentation of joint patterns for the various parts can be done in a
number of ways. Often jointing details cannot be properly seen this way and other exploration methods are needed. Core drilling helps with finding rock properties and some of the jointing details but cannot reliably indicate joint widths or strikes, and may underestimate some joint strikes or dips. However full knowledge about those is vital to planning of effective grouting. Also if core recovery is decreased, there arises uncertainty as to whether the unrecorded section of hole is passing through a cavity or through weathered materials. Furthermore the nature of such poor material needs to be known. Many other problems are possible with core recovery exploration and thus borehole TV, borehole periscope or impression packer could be of remarkable importance. A full circumferential view of borehole TV does not necessarily pick up cracks well and an optional sideways viewing mirror should be available in the field of view of camera. The sideways viewing option permits looking into cracks, and their width and dip can be measured. Also a borehole periscope enables the measuring of crack widths, strikes and dips, but in can be used only in dry parts of holes. Fiber optics present a potential method for improved downhole examination. Impression devices are long, soft rubber packers which are planned to present an impression of the wall of the hole including cracking. However, considerable doubt exists about whether they effectively do this. (Houlsby 1990).

It should not be forgotten the in-situ tracer experiments which can be beyond comparison in understanding the channeling of flow in single fractures, fracture zones as well in averagely fractured rock.

Some designers prefer a policy of almost no investigations and a "find out as you grout" approach. It can not be maintained that it is totally wrong way; site grouting tests can be also very beneficial. To resolve doubts, they can also help in finding the best drilling method, checking proposed w:c ratios, trying various injection techniques, finding safe pressures and comparing equipment (Houlsby 1990). The general idea is to investigate the properties of the test area before grouting it and later to find out the changes in these properties produced by the grouting. Site testing should be extensive to be of any use, many holes are needed to get a good understanding.

In principle, when drilling it is possible to get information about rock type, fracture density, RQD-value, content and grain size of certain ore minerals, unit weight, strength, drilling rate index, etc. and thus to characterize properties of rock (Kuivamäki et al. 1995). This was studied in the research project "Rock Engineering 2000 Technology Program", where "Interpretation of continuous data registered during percussive drilling and core drilling" was one subproject (Kuivamäki et al. 1995). Analyses of the project were partly concentrated mainly for mining purposes, but it was revealed that it's possible to delineate the relatively tough wall rocks immediately surrounding the ore deposit. Likewise, it is generally possible to identify areas of compact ore, as well as intensively fractured zones. Other results indicated that penetration rate and compressive stress and their derivative parameters correlated best with variations of fracture density. A method based on R index (RQI-ratio - penetration rate) allowed both orientation and distribution of fracture zones to be defined. The main conclusion of the study from the viewpoint of rock engineering applications is that in areas that are lithologically homogeneous, it is possible to use a
range of percussion drilling parameters to delineate zones of varying fracture density. Thus it could be promising to combine drilling information to water pressure tests for rock classification for groutability estimation purposes.

3.4 Experiences of rock characterization and groutability

3.4.1 General

There is experience of grouting from a thousands of cases and during long time but usually they have not been analyzed or even reported properly. It is mainly saved in brains of experienced contractors. Fortunately some experiences have been written down and some conclusions are drawn. Also there has been some scientific grouting experiments, of which the OECD international Stripa-project has produced much useful knowledge.

3.4.2 Stripa experiences

OECD Nuclear Energy Agency NEA carried out a research project considering groundwater flow and rock sealing in former Stripa mine in Central-Sweden in the 1980's (Figure 3-5). Project was an international co-operative project and as many as nine countries participated in it. From Finland both nuclear power companies and the Ministry of Trade and Industry of Finland took part in the project.

The primary goal of the Stripa project was to develop techniques to scientifically characterize a granitic rock mass with regards to its hydrological and geochemical properties, to understand the influences of heating a granitic rock mass and develop techniques to seal various parts of a repository system (OECD 1992, Pusch 1992, Pöllä et al. 1994). Phase I of the Stripa project involved among other things the development of methods for determining the in situ hydraulic conductivity of a fractured rock mass by use of both single and multiple boreholes. Phase II was concerned with the development of crosshole geophysical techniques, including radar, seismic and hydraulic methods for detection and characterization of fracture zones. Phase III, also called Stripa Sealing Project, consisted of material and field investigations of grouting. The objective of this phase was to find ways of sealing finely fractured rock by grouting. During this phase groundwater flow in rock fractures was studied and modeled, and comparison of predictions with data collected by the use of improved methods were made. As well the devices and methods for bedrock studies as the suitability of various materials for the long-term sealing of fractures and fracture zones in crystalline rock were tested. Field investigations concerned for example grouting of the rock around disposal holes and grouting of the excavation damaged zones around the tunnel and grouting of the natural fracture zone crossing the tunnel. Interesting Stripa experiments as well as the general conclusions are briefly discussed below.
Site characterization and validation project

The Site Characterization and Validation (SCV) project (Figure 3-5) was set up to test the ability to characterize the structural features of a site and to quantify groundwater flow and transport through the site (OECD 1992). The characterization program was applied from boreholes and drifts and it included fracture mapping, borehole radar measurements, borehole seismic measurements, single borehole geophysics, hydrogeological characterization, hydrochemical characterization and rock mechanical characterization. A Fracture Zone Index (FZI) was used to define objectively the occurrence and width of fracture zones where they intersected the boreholes. The conceptual model of the site was used as input to four different numerical models used to predict the inflow and tracer transport.

Groundwater flow at the SCV-site was concentrated within the fracture zones. Within the fracture zones a large variability in hydraulic transmissivity over small scale was found. Occasionally highly transmissive fractures were found outside fracture zones. Predicted inflows to the drift were generally larger than measured due to unaccounted for drift excavation effects. The reduction in flow through the "averagely fractured rock" was found to be significantly greater than the reduction in flow through the "fracture zone". Two-phase flow conditions are considered to be the principal cause for reduction.

According to the interpretations the geological and hydrogeological conceptual model was in good agreement with observations (OECD 1992). It was thus concluded that the combination of single-hole and cross-hole investigations which were applied can be used to give reliable and sufficiently detailed description of a site at the 100 m scale. A prerequisite to obtain a reliable geometrical model is the use of remote sensing techniques such as radar and seismics. The models were regarded to provide realistic predictions of groundwater flow. General agreement was obtained between
predicted and measured inflows to boreholes. The attempts to model the inflow to the drift were not successful due to inadequate understanding of the hydrology of the disturbed zone around drifts.

Also it was regarded that borehole seismic and radar data provided consistent description (large-scale) of the fracture zones at Stripa in agreement with geological, geophysical and hydrogeological observations.

*Sealing of disposal holes*

Sealing of disposal holes with bentonite was studied with field investigations (Pöllä et al. 1994, Pusch 1992, Börgesson et al. 1991). The main objectives of the investigation was to search how large volume of rock around tunnels or disposal holes can be sealed. Other aims were to characterize hydrologically the rock around disposal holes, find out the limit of hydraulic conductivity which can be reached with grouting and examine the effect of heating into hydraulic characteristics and fracture geometry in the vicinity of the holes (simulating of the thermal effect of capsules).

Before grouting the geometry of holes and the grade level of tunnel floor were measured and the hydraulically conductive fractures in holes were mapped. Hydraulic measurements were made with Large borehole Injection Device (LID) and the amount of leakage water were measured. Dynamic grouting was used as sealing method and grouting material was a mixture of fine grained quartz sand, Tixoton bentonite and natriumchloride. After grouting the rock was warmed.

Several measurings and investigations were made in holes after grouting. According to the measurements it could be discovered that after grouting the hydraulic conductivity decreased significantly. However, the heating of rock up to almost 100 °C increased hydraulic conductivity clearly. Also on the basis of the investigations grout penetrated quite well into horizontal fractures without chlorite fillings. Into fractures with chlorite fillings the grout didn't penetrate. Dynamic grouting method and used grout were discovered to be appropriate at least in circumstances in question. Also with the theoretical model developed in the project the penetration of grout in simple fractures can be estimated.

The moving of rock due to grouting occurred in existing fractures. The heating of rock caused permanent widening of fractures and impaired the effect of grouting. However, time dependent decrease in hydraulic conductivity can be seen. The estimation of penetration of grout is difficult especially because of fracture fillings. The grouting performance with the used technique in a rock volume of 1 x 1 x 1 m³ can be estimated as follows:

\[
K < 10^{-10} \text{ m/s} - \text{not groutable}
\]

\[
10^{-10} \text{ m/s} < K < 10^{-8} \text{ m/s} - \text{fractures groutable if the water flows in channels or slots with a width < 1-10 cm and if the fracture is free from debris}
\]

\[
K > 10^{-8} \text{ m/s} - \text{groutable in all fractures free from debris.}
\]
Bentonite intruded into fractures with an aperture/width of ≥ 20 - 30 μm. Grout intruded into fractures without incompact fillings well and according to predictions. Into fractures with a hydraulic opening of less than 50 - 100 μm and with incompact fillings, the grout didn't intrude nearly at all.

**Characterization and sealing of excavation damaged zone**

One purpose of investigations was to study the hydraulic conductivity of excavation damaged zone around a tunnel and to find out if it is possible to diminish that by cement-grouting (Pöllä et al. 1994).

Initial conditions in this experiment were not similar to normal excavation because studies concerning grouting of disposal holes (including drilling, grouting, heating and pressurizing) had been made in this tunnel. Before grouting the amount of water leaking from drillholes and the hydraulic conductivity of every fifth hole were measured. In most of the holes the hydraulic conductivity \( K < 10^{-10} \text{ m/s} \). The prevailing state of stress in some points was also determined and maximum stress was horizontal and perpendicular to the tunnel axis.

The calculations and modeling before grouting were meant to clarify for example the amount of water flow and its distribution in excavation damaged zone and the reliability on Lugeon tests. The deviations in modeled pressures and hydraulic conductivities were small and insignificant. The reliability of Lugeon tests was estimated with calculations and the result was that in this case the calculated hydraulic conductivity has to be multiplied with the correlation factor 1.6. The excavation damaged zone was examined using water loss measurements done in grouting holes. The measured hydraulic conductivities were small, in more than half of the holes \( K < 1E^{-9} \text{ m/s} \).

On the basis of the measurements and modeling a three dimensional model of rock around the tunnel was created. According to the calculations it was concluded for example that:

- the average hydraulic conductivity of the excavation damaged zone was 1.1E-8 m/s in the depths 0 - 0.7 m from the tunnel surface. Near the surface (0.1 - 0.2 m) K is 5E-8 - 1E-7 m/s.
- the hydraulic conductivity in the excavation damage zone caused by blasting is higher in the tunnel floor (2E-8 m/s) than in walls (1-3E-9 m/s), and it is lowest in the tunnel arch
- the hydraulic conductivity of intact rock is 3E-11 - 1E-10 m/s. The determination of the axial hydraulic conductivity in the depths 0.8 – 3.0 m is unreliable with both the models used in the case (about ten times higher than in intact rock).

As a conclusion it was noticed that the effects of blasting to the fractures near the tunnel surface depends strongly from the detailed fracture geometry.

Different grouts were used and according to experiments the penetration of every grout into a fracture of 100 μm aperture was good. It was possible to estimate the
penetration beforehand fairly accurately with calculations in both dynamic and static grouting method. One cement (Alofix-microcement) could penetrate even into fractures of 20 μm.

According to the theoretical study before grouting it was concluded that grouting depends mainly on three things:

- fracture geometry (most important are aperture/width, proportion of open fractures and continuity)
- fracture fillings (sort and proportion)
- grouting technique and properties of grout.

In regard to the penetration it is not the most important how deep the grout intrudes but how does it intrude into fractures with small aperture. The following restrictions which depend on the hydraulic conductivity can be set to the groutability of rock on the basis of the restricting aperture (20 μm):

- \[ K = 1 \times 10^{-10} \text{ m/s} \] only tubular channels for example in the intersection of two fractures can be grouted
- \[ K = 1 \times 10^{-9} \text{ m/s} \] groutable if there is only one fracture with unfilled volume less than 20% per a meter
- \[ K = 1 \times 10^{-8} \text{ m/s} \] groutable if there is only one open fracture per a meter
- \[ K = 1 \times 10^{-7} \text{ m/s} \] nearly all fracture types are groutable.

Investigations were made after grouting and the results were not encouraging. The average hydraulic conductivities (arithmetic and geometric) changed only a little, possibly because the grouting caused movements in rock and thus fractures opened and hydraulic conductivity increased. It is even possible that the effect of grouting is negative. It was also discovered that probably hole spacing of one meter is not enough to seal all flow paths in this kind of rock.

The penetration of grouts was examined by removing the surface parts of rock and all the revealed fracture surfaces were investigated. The following conclusions were drawn:

- grout was not found in wide hydraulically conductive fractures, which all contained fracture fillings
- grout was not found in fractures which were grouted from boreholes if there was a circle of drill dust/cuttings in the fractures.

As a conclusion it was found out that the grouting of excavation damaged zone from short grouting holes didn't succeed to diminish the hydraulic conductivity. The reason to this is poor groutability because of fracture fillings. Particularly the most fractured zone near the tunnel periphery was extremely difficult to grout.
Main findings:

- the excavation damaged zone extend to about 1 meters depth from the tunnel periphery. The hydraulic conductivity of that zone is about 100 times more than that of intact rock. Deeper than this, to about 3 meters depth, there is a disturbance zone caused by secondary stresses. The radial hydraulic conductivity is clearly decreased, but the estimation of axial hydraulic conductivity is unreliable.
- Alofix was found to be the most suitable cement for grouting fractures with a small aperture, mainly because of its small grain size
- estimations of penetration in a static and dynamic grouting succeeded fairly well with the help of modeling and calculations
- the tests of grouting an artificial fracture showed that the narrowest fracture, which can be grouted is about 20 μm of its aperture
- on the basis of theoretical examination the floor and walls of tunnel could be groutable and grouting will diminish hydraulic conductivity significantly. However, it was not possible to perceive the reduction of hydraulic conductivity in the tests subsequent to the grouting.

Sealing of a natural fracture zone

The intention of one experiment was to study if a natural hydraulically very conductive fracture zone can be sealed effectively, prove that sealing changes flow paths of groundwater and give new information on flow paths into 3D-tunnel (Pöllä et al. 1994).

Before the first grouting water movements were estimated. In the first phase 75 holes were grouted using the dynamic grouting method and Alofix cement. The measurements afterwards showed that the amount of leakage water didn't diminish but the flow paths directed in new ways. Evidently only 3 holes hit the fracture zone to be sealed.

In the second grouting phase 10 holes were bored and the fractures in coring samples were examined. The leakage waters and the hydraulic conductivity were measured. On the basis on the fracture mapping a new understanding of the location of fracture zone was got. Very similar grout mixture and grouting technique as in the first phase were used.

According to the measurements done after the second phase it was noticed that the grouting succeeded well. Some special observations were made:

- leakage waters were directed to new flow paths as it was supposed
- cement was channeled in fractures with an aperture of more than 50 μm. The fracture channels into which the grout intruded, were tortuous and continuing. Fractures with an aperture of 20 - 30 μm were succeeded to get sealed with the grouting method and grout used in this case. Grout was found also in the fractures with a maximum aperture of 10 - 20 μm. Those samples were porous and heterogeneous
- deposition in regard to particle size or shape did not occur.
On the basis of the experiment it can be concluded that hydraulically conductive zones can be grouted to $K = 1 \times 10^{-9}$ m/s ($K$ of the zone in the case was $1 \times 10^{-8}$ before grouting), but it requires much work. Because sealing of this kind of a zone was successful, it will probably be possible to seal a more conductive zone for example at several stages.

Determination of the location of a hydraulically conductive zone for grouting was difficult in the experiment. Clear need for development was perceived in the technique in question.

The experiment proves that the hydrogeological properties of rock should be well understood so that the amount of total leakage water which flows into a tunnel could be reduced and not only to move to a new places. However, it is not appropriate to put up a model of the area to be sealed but more reasonable is to make the grouting in an area wide enough.

To get the above mentioned grouting result, spacing between grouting holes must be about $0.5 - 0.7$ m.

**General conclusions**

The Stripa Rock Sealing Project is concluded to have yielded a large amount of very valuable information of sealing finely fractured rock (Pusch 1992). But still it remains questions, some because of uncertainties in the sealing strategy and aims in final disposal repository. In regard to groutability (Pusch 1992, Pöllä et al. 1994, OECD 1992) the following conclusions were made:

- Natural fracture zones, except regional and local ones, can be rather effectively sealed by using cement injected under high pressure applying dynamic injection technique. The spacing of the injection holes determines the effectiveness of the grouting.
- The field tests showed grouting of blasted rock ineffective (excavation damaged zone). Narrow fractures in deposition holes and in a natural fracture zone were sealed rather effectively.
- Rock near the ground with fine fractures coated with fracture minerals like chlorite is not effectively groutable. In general, fracture fillings affect significantly how grout penetrates.
- The restrictive limit of a fracture aperture able to be grouted is about $20 \mu$m. The channels with a maximum aperture of less than $20 - 30 \mu$m are not groutable while continuous wider channels can be sealed. Fine-grained grouts can be effectively grouted in relatively fine fractures.
- Stripa experiments supported the well-known matter that initially very conductive rock can be more effectively sealed than initially less conductive rock.
- The penetration depth and thereby the extension of the grouted zone can be predicted with acceptable accuracy if the channel geometry is derived from the basic hydraulic data obtained by Lugeon testing and the location and number of hydraulically active fractures in injection holes are determined by core of drillhole inspection. The groutability of rock can be predicted by hydraulic testing and
evaluation of the rock structure provided that debris is not present in the fractures. The possibility of reducing the average hydraulic conductivity effectively by grouting depends on the structure of the rock.

- Determination of location of a zone to be sealed is difficult and the properties of fractures affect remarkably penetration and thereby need to be known.
- The technical and economical results would probably be better if methods were developed to determine the groutability of rock. Potential developing subjects are for example detailed hydrogeological characterization methods of rock to be grouted, characterization methods of fracture apertures and fillings, and methods to check the grouting result.

3.4.3 Other experiences

Some someway "scientific" and/or well documented/analyzed grouting experiences are collected in the following text. It includes sporadic experiences based on appropriate research work made in existing nuclear waste repositories and underground rock laboratories, as well as some experiences from ordinary tunneling projects.

**Åspö Hard Rock Laboratory**

Hedman (1999) has compiled a report of experiences from design and construction of Åspö Hard Rock Laboratory (HRL) in Southern Sweden (Figure 3-6). High groundwater pressures and highly conductive fracture zones crossing the tunnel made grouting important in the Åspö tunnel. Grouting was the main interest in testing the tunneling methods.

The locations of discontinuities were identified by drilling done from the ground level, and from information gained when constructing the tunnel (Hedman 1999). The general requirements of grouting work were: grouting materials must have a limited impact on the groundwater chemistry (cement with bentonite and calcium chloride and certain chemical grouts e.g. TACCS), the spreading of grout in the rock mass must be limited (= limited grouting volumes) and on the other hand, the total inflow must be restricted to a certain level. The grouting activities in the access tunnel of the Åspö HRL have to a great extent followed normal practice in Swedish tunneling (Bäckblom and Svemar ed. 1994). Based on pilot boreholes or inflow through blast-holes or the tunnel front, pre-grouting is performed when considered necessary. Typically, a fan of 10 - 25 grouting boreholes is drilled, tested by water loss measurements and grouted. The basis for the procedures is a grouting program which defines the objectives, rock classification, guidelines and limitations. A special condition for this project, as mentioned before, was that limited grout penetration was desired. The final decision on how and when grouting had to be done, was made when the holes drilled in the tunnel face had a greater flow than a previously set limit (Hedman 1999). The final placing of grouting holes and their lengths was decided depending on local conditions. Documentation of geology, hydrogeology and of the grouting activities was done very carefully during the whole process of grouting. A number of grouting concepts were tested, but the most of results were unsatisfactory. Later on, the restrictions were eased, and satisfactory grouting results followed shortly. The only grout found suitable
when grouting in wide fractures as well as in clay filled fractures, against a high groundwater pressure and with strict limitations on the spreading of the grout was cement (w/c-ratio 1.0) with addition of bentonite (2 %) and salt (8 – 15 % CaCl₂). The injection pressure was limited to 20 bars over the hydrostatic pressure. There was a certain suspicion that it would be difficult to grout at great depths, due to the high pressures, but the grouting works actually went quite well. For the grouting of the TBM tunnel a special program was issued. The contractor carried out pre-grouting in the raise-bored shafts without beforehand made plans. Down to the -220 level, the shafts were pre-grouted via the pilot hole and the results were not satisfying. A number of post-groutings were then carried out via the shafts but the leakage is still too large. In deeper levels pre-groutings were carried out via several drilled holes outside the shaft's periphery and results were very good.

In the Äspö HRL the collection system for leakage waters is advanced (Hedman 1999). High standards have been imposed on the measuring the water leakage, which has worked well. The total groundwater leakage into the excavation after (restricted) grouting is 2100 l/min ≈ 60 l/min/100 m.

Gustafson, Rhén, Stanfors and Stille has studied the passage of a regional fracture zone (NE-I) at the Äspö HRL (Bäckblom & Svenmar ed. 1994), where a series of experiments has been performed in order to test and develop methods for passing through a major water-bearing zones, especially with regard to location, characterization and controlled pregrouting. The tunnel crosses the zone at a depth of about 190 m below the sea (SKB 1996). It was estimated that without sealing, the inflow to the tunnel would have been up to 6000 l/min from this zone (SKB 1996). The passage of the access tunnel through that zone was both time-consuming and difficult, mainly due to the restrictions concerning grouting (Hedman 1999). The main objectives of the experiments on passage through waterbearing fracture zones was to develop and evaluate methods for passing through such zones under controlled conditions as regards the following activities (Bäckblom & Svenmar ed. 1994): Exact location of the predicted fracture zone, step-by-step characterization of the fracture zone, controlled pre-grouting (primarily the spreading of grout), registration and analysis of the hydraulic pressure responses and changes in groundwater chemistry, assessment of the course of events whenever the tunnel passes through the zone, documentation of achieved sealing effect and spread of grout, initiation of long-term monitoring of the zone's groundwater pressure, chemistry etc.

Before entering the fracture zone in question, the zone was characterized from the ground surface and from the tunnel. The zone was attempted to be located exactly with several cored and bored holes from the tunnel as well as from the ground surface, and with measurements made in those holes (spinner, borehole radar, flow meter, interference tests). Also other possible water-bearing fractures in an other direction connected to the zone was attempted to be found. Flow meter survey in the cored boreholes and the interference tests in the cored boreholes and the percussion holes drilled through the south part of the zone all indicated a very conductive structure with a transmissivity of an order of about 1E-4 m²/s. The responses indicated that the major conductive structure should be the zone in question but it could also be seen that the fractures in
the other direction were probably in contact with the zone. After passage of the zone, it proved to be highly water-bearing and assumed to be steep.

The most intensive part of the zone, which is intersected by the tunnel at about 130 m depth is approximately 5 m wide, highly fractured or crushed and in an approximately 1 m wide partly clay-altered section. The tunnel intersects the zone along a length of approximately 30 m. The main rock type in the zone is Småland granite with minor inclusions of greenstone and mylonite where the most intensive part of the zone is located in fine-grained granite. The intensive part of the zone with open, centimeter-wide fractures and cavities is surrounded by 10 - 15 cm wide sections of more or less fractured rock. The gouge material in the zone include fragments of all sizes and the fragments are sharp angled, more or less tectonised granite and mylonite. Older fracture formations are also found as fragments, indicating that the gouge formation is a reactivation of pre-existing zone which developed under ductile, semiductile conditions. Some fragments are penetratively oxidised, probably before the fragmentation took place, and post-fragmentation precipitation of pyrite on the grain surfaces is visible, indicating that reducing conditions prevail. The main clay minerals present in the gouge material are mixed-layer illite/smectite.

The grouting acts were not unambiguously clarified. Stabilodur F1 (a mix of cement, bentonite, plasticizer and silicate) with altering the amount of additional calcium chloride was used as the grout. Such rarely used grouts as polyurethane (TACCS) and Mauring were used in desperation when the sealing work seemed more or less hopeless, but they did not have the desired effect and were soon abandoned. Some fans were grouted with high water-cement ratio. It was found that this type of grout was not stable and also that no sealing effect was observed.

Figure 3-6. The location and outline of Åspö Hard Rock Laboratory (Bäckblom & Svenmar ed. 1994, Emsley et al. 1997).
Hedman (1999) suggests that experiences from the construction of the Åspö Hard Rock Laboratory shows that it may be difficult to control the grouting in order to achieve the required results in final disposal repository. Gustafson, Rhén, Stanfors and Stille (Bäckblom & Svemar ed. 1994) concluded from the experiences gathered from NE-1-zone that a development of a grouting system for grouting must be compatible with the theory of grouting. A classification system must thus be divided into three parts that take into account: grouting factors (e.g. grouting pressure, yield stress and viscosity and grain size of the grout), geological factors (e.g. number of joints, joint geometry and joint aperture) and hydrogeological factors (e.g. transmissivity, groundwater pressure and porosity). A classification system must also be practical and simple so that it can also be used at tunnel front during the grouting work. The system must give advice about type of grout, grout take and penetration as well as grouting methodology.

**AECL Underground Research Laboratory**

In the AECL Underground Research Laboratory URL in Canada (Figure 3-7) a throughout hydraulic characterization for locating the ventilation raise was carried out (Bäckblom & Svemar ed. 1994). One major fracture zone was about to be penetrated. The characterization revealed strong variations of transmissivities along the zone, and the vent raise was excavated through a section with no need of grouting and final measured water inflows were negligible. It was concluded as well that if a vent raise had been located in a high transmissivity portion of the fracture zone, a significant grouting and sealing effort would certainly have been required.

![Figure 3-7. Isometric view of the URL (Bäckblom & Svemar ed. 1994).](image-url)
Other experiences

Riekkola (Bäckblom & Svemar ed. 1994) has compiled Finnish case histories of sealing significant fractured zones in Harjunsuo sea sewer tunnel in Vantaa, VLJ Repository at Olkiluoto, Vuosaari-Pasila heating tunnel and Viikinmäki central wastewater treatment plant in Helsinki.

One case was the sealing work in sea sewer tunnel in the Harjunsuo area in Vantaa. The tunnel penetrated a 60 m zone which was partly chemically weathered in a way that made rock very porous. The zone also consisted partly of nearly horizontal clay-filled fractures. The zone conducted water considerably but the grout take was poor. Groundwater pressure was about 0.3 MPa. The zone was pregrouted with cement and resins, but the inflows into the tunnel remained at a level that made postgrouting with special solutions necessary before shotcreting was possible. Postgrouting was completed after shotcreting. Total cement take was about 150 tons corresponding about 1250 kg/grouted tunnel-m. Consumption of chemical grouts varied from 20 to 116 kg/grouted tunnel-m. The final inflows from the zone were about 100 m$^3$/day = 70 l/min.

The access tunnel of the VLJ Repository for low and medium level nuclear waste at Olkiluoto in Eurajoki in Western Finland penetrates one inclined fracture zone at two points. The lower part of the zone conducted water into the tunnel at a rate of 30 - 35 l/min. The zone was postgrouted with cement (w:c ratio 0.5 and melamine based superplasticizer). The grouting was performed in 18 holes with a length of 5 - 20 m. Cement intake varied from 0 to 1120 kg/hole. The inflows were first reduced to about 8 l/min. After redirection of the flowpaths, the inflows returned to their original level.

The Vuosaari-Pasila heating tunnel in Helsinki was situated in granite and mica gneiss and one nearly horizontal water-conducting fractured zone cut the tunnel. The fractures are mainly open with apertures on the order of few centimeters. In part of the tunnel, a 10 - 20 cm wide fracture with sandy filling were able to be seen. The highest inflows were several hundred litres per minute. Grouting was carried out with cement and normally started with high w:c ratios. A lower w:c ratio (0.5) was used in connection with the open fractures. Pre- and postgrouting were done in the zone and the inflow after pregouting was about 200 l/min. Groundwater pressure is about 0.4 MPa. The results of post-grouting are not reported.

In the Viikinmäki central wastewater treatment plant the field investigations and measurements showed some tight and some heavily water-conducting areas in a fractured zone under interests. According to those investigations pregouting was planned, but the grout intake in the zone was unexpectedly proved to be minimal and the grouting operations were reduced.
According to before mentioned cases Riekkola (Bäckblom & Svemar ed. 1994) seemed to have concluded that development of investigation methods is needed to get a more realistic picture of zones to be grouted.

The VLJ Repository at Hästholmen in Loviisa in Southeastern Finland is the other nuclear waste repository in Finland for low and medium level radioactive waste (total volume 110 000 m³). The sealing was mainly carried as pre-grouting and partly as post-grouting (Auvinen 1997). Portland- and Rapid-cements were used as grouts and grouting pressure varied between 0 - 6 MPa. The grouting work was mainly succeeded (Anttila 1998). The set tightness criteria was not achieved in the access tunnel (Auvinen 1997). In the final disposal facilities, where systematic grouting was performed, the result was good and satisfying. According to Anttila (1999), the horizontal fractures was easiest to grout. However, two gently dipping fracture zones that are intersected by the access tunnel (and the other also by two shafts) conduct great part of the leakage waters into the repository. The biggest difficulties in grouting were related to these zones. Although the pre-grouting seemed to be succeeded and the excavation was continued, these zones started to leak into the tunnel again. Those fracture zones included considerably clay-filled fractures and Anttila (1999) assumes that particularly those fractures were not filled by grout and thus started to leak later again. With three other vertical zones of few fractures in the facility and one nearly vertical zone in the access tunnel, special problems did not arised. Immediately after the excavation the total leakage amount were at the highest level, about 300 l/min mainly due to the leakage into the access tunnel, which was measured to be about 250 l/min (Auvinen 1997). Later it has decreased steadily, and in the end of 1999 it was about 140 l/min, of which about half was measured from access tunnel and half from VLJ-facilities (Saari 2000). It is assumed that the decrease of the water ingress has occurred for example because the fractures have been precipitated tighter and the air conditioning has been improved (Auvinen 1997, Saari 2000).

Dalmalm (1999) has studied the correlation between grouted volume and geology in Arlandabanan in Stockholm area. From each grout hole the following data has been registered: tunnel section, pressure, flow, stop pressure, time, w:c-ratio, total grout volume. This information was combined with geology data (RQD, SRF, Jn, Jn, Jr and Jw) from which the Q-values were calculated. According to his studies the correlation with the Q-value is weak, because of the inapplicability of the variables for grouting purposes. He found following dependence between different geological factors and grout take:

- higher RQD $\Rightarrow$ grout take decreases
- higher $J_{w}$ $\Rightarrow$ grout take decreases
- higher $J_{n}$ $\Rightarrow$ grout take increases
- higher $J_{r}$ $\Rightarrow$ grout take increases
- higher SFR $\Rightarrow$ grout take increases

He has also concluded that the grout volume gives a good correlation if the hydraulic conductivity value is correctly measured. However, the hydraulic conductivity value measured from pre-investigation holes could not be used for grouted volume
predictions, because it is not the same as in grouted hole. Dalmalm emphasizes that when estimating groutability, the measurements have to be carried out also in grout holes.

**General overview on grouting experiences**

Fracture fillings, especially clays and softening fillings, seem to cause most difficult problems in grouting. They are very difficult to avoid or control, and thus they often cause unexpected problems. The other thing which was emphasized was the demand for appropriate investigation methods to localize the leakages and to estimate the groutability of rock.

### 3.5 Groutability estimations

At least in principle, groutability (often simply grout take and/or penetration) of rock can be estimated using expert judgement, or analytical or numerical studies. Expert judgement has been a common "method" in conventional tunneling projects; no "scientific" method has been adopted to customary use. The need of a systematic and workable approach to the problem has arisen in many contexts, for example such a demanding project as nuclear waste disposal (e.g. Pöllä et al. 1994).

Analytical and numerical methods are under developing and improving all the time. Problems arise in both; with analytical methods large amount of factors must be neglected for enough simple calculations and it is very complex to take into account the fracturing, the rheological properties of grout and the grouting technique, simultaneously. With numerical methods the handling of more numerous parameters could be possible, but only very simplified systems of one or a few fractures have been modeled for the moment. Analytical and numerical calculations have been presented for example by Hässler (1991), Janson (1998) and Fransson (1999).

For the development of mathematical methods the characteristics of fractures and fracturing systems should be given numerical values. This may need some kind of a rock classification system for grouting purposes. Some attempts of classification as well as the mathematical methods to estimate groutability of rock has been presented.

Expert judgement is based on experiences and common sense, and may not be underestimated. Experience based estimations may be completely as adequate as any other existing method. There is much knowledge among contractors; probably the most valuable experience is somewhere else than in researcher’s conscious.

Many methods to classify rock mass and proposals how to characterize fractures have been presented, but there is only few classification systems for grouting purposes, in addition to the traditional one Houlsby (1977, referred by Singh & Goel 1999) has presented. A working system for classifying rock from a grouting point of view is definitely necessary to achieve more efficient treatment and optimization of the grouting process. In this literature study the existing rock classification and estimation systems for grouting purposes are examined.
A simple rock classification system for grouting strategy (above all for grouting need purposes but also considered to give primary information about groutability) based on simple water pressure tests has been suggested by Houlsby (1977, 1982, presented in Singh & Goel 1999) (Table 3-1). Water injection tests are typically made to predict where and what quantities of grout to inject. One Lugeon is the degree of permeability encountered in those nearly tight foundations, where grouting is hardly necessary (Houlsby 1990, Shroff & Shah 1993). Ten Lugeons usually warrant grouting for most seepage reduction jobs and one hundred Lugeons are encountered in heavily jointed sites with relatively open joints or in sparsely fractured joints which are very widely open (see p. 34).

Based on Lugeon tests Houlsby (1990) presents grouting instructions. Also other rough technical instructions for grouting of dam foundations and underground excavations in different hydrogeological circumstances have been presented (e.g. Ewert 1985, Nonveiller 1989, Shroff & Shah 1993, Widmann (ed.) 1993, Verfel 1989, Lombardi & Deere 1993).

There have been some attempts to create a classification system of groutability of rock: for example Ewert (1985) has presented one based on rock type but also with no accuracy enough for today's requirements. In the following text, some interesting attempts have been presented.

According to Johansen et al. (1991) the estimation of groutability (which in this case means grout take) can be based on engineering geological mapping. They have presented an attempt to systematize the experience based on grouting field by introducing the alteration of the well-known formula in the Q-system, see equation 3-1. Originally the Q-system was designed to make prognoses for the required support of tunnels in different rock types (Barton et al. 1974). The equation for grout take estimations is:

\[ Q_i = \frac{RQD}{J_n} \cdot J_r \cdot J_a \cdot \frac{1}{1 + l} \]  

where \( Q_i \) = quality of rock from grouting point of view, \( RQD \) = rock quality designation (-), \( J_n \) = joint set number (-), \( J_r \) = joint roughness number (-), \( J_a \) = joint alteration number and \( l \) = Lugeon value (-). Observe that the joint alteration number was regarded as positive in connection with grouting as compared with stability, i.e. if the joints were partly filled with clay, for example, less grouting would be required.

Johansen et al. (1991) presents diagram relation between grout take and \( Q_i \)-value like presented in Figure 3-8. The relation is regarded as curvy-shaped and it is site-specific. Johansen et al. (1991) has noticed a relation between rock type and grout take. Schists and phyllites differ clearly from other common rock types.
Table 3-1. Classification of rock masses on the basis of Lugeon values (Singh & Goel 1999 based on Houlsby 1977).

<table>
<thead>
<tr>
<th>Lugeon value</th>
<th>Strong, massive rock with continuous jointing</th>
<th>Weak, heavily jointed rock</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>completely tight</td>
<td>completely tight</td>
</tr>
<tr>
<td>1</td>
<td>sometimes open joints up to about 1 mm</td>
<td>sometimes open to hair crack size of 0.3 mm</td>
</tr>
<tr>
<td>3,5</td>
<td>occasionally open to 2.5 mm</td>
<td>occasionally open to 1.2 mm</td>
</tr>
<tr>
<td>20</td>
<td>often open to 1.2 mm</td>
<td>often open to 1.2 mm</td>
</tr>
<tr>
<td>50</td>
<td>often open to 2.5 mm</td>
<td>often open to 2.5 mm</td>
</tr>
<tr>
<td>100</td>
<td>often open to 6.2 mm</td>
<td>often open to 6.2 mm</td>
</tr>
</tbody>
</table>

Note: Joint measurements are in mm; 1 Lugeon =1.3E-7 m/s. Local variation in permeability is probable due to locally open fractures.

Figure 3-8. The relation between grout take (kg/tunnel-m) and $Q_r$-value. Limit curves are from Holmen Vest and NSB tunnels in Norway (Johansen et al. 1991).
The correlation between the Qi-value and grout take has proved to be weak in practice (Dalmalm 1999, Andersson 1998). Dalmalm (1999) has studied the correlation between geology and grouted volume in Arlandabanan. He suggests that neither Q-method nor Qi-method are accurate this way, because the fracture characteristics have a different influence in the rock support matter than to grouting design. This classification could be an adequate solution if the parameters were, for example, renumbered and/or new variables were taken along.

Holopainen (1977) and Dalmalm (1999) have suggested that, for example, RQD-system or some other classification system could be potential foundation for a rock classification system for grouting purposes with some modifications (as mentioned before about the Qi-system). There might be some correlation between groutability and for example RQD$^2$ or log(RQD), but the subject is not studied.

Barton & de Quardos (1997) has discussed that groutability of rock masses (which in this case refers to the volume of rock that can be grouted and to the grout take) can be estimated from joint aperture and roughness using core logging and packer tests. The volume of rock mass that can be grouted is related both to the hydraulic conductivity K of the network of the interconnected discontinuities and to the physical aperture E of the joints in each fracture family. In this method the joint roughness coefficient JRC is determined for example from tilt tests in natural joints selected from drill cores and hydraulic conductivity K is determined from in situ water pressure or suction tests. The theoretical hydraulic apertures $e$ in the model have to be converted to physical apertures $E$ using data from field hydraulic tests and the $E/e$-diagram or equation $e = JRC^{2.5} / (E/e)^2$. Groutability which in this case actually means the maximum grain size of the grout (Barton 1999a), can be estimated from JRC estimation and $E/e$ calculation (effective physical aperture/theoretical aperture). This ratio has been used in modeling as an essential parameter when assessing groutability with particulate grouts (Barton 1999b).

Hässler has proposed a method to classify rock from a grouting point of view as a part of his doctoral thesis (Hässler 1991). Actually, the suggestion to improve sealing performance concerns the whole grouting process, not only the characterization, and according to him to get satisfying results active designing is needed when the grouting is in progress. He also emphasizes that when planning the grouting process one should have good understanding also on grouting techniques and materials. His ideas of possible grouting process in future are presented in Figure 3-9. During the execution stage it is theoretically possible to produce an estimation of the geometrical classification parameters from water pressure tests and measurement of the grouting process. During the planning and design stage, however, classification will probably have to be done empirically from pure geological data. When the geometrical classification parameters have been produced, the selection of the method and grout can be done by means of calculations or possibly nomograms, in which the geometrical parameters are input data.
Hässler emphasizes that to work a classification system must have some basic properties. It must be relevant, reproducible and consist of measurable components, it should not be unnecessarily complicated. It may be founded on purely empirically factors based on some more or less well-planned model or based on a combination of the two. From this starting point he has built up a classification system purely based on empirical approach which is meant to function by geological and hydraulic data collected from large number of grouted areas. Correlations between the collected data and result of grouting could then provide information about the parameters that are important in grouting. These parameters would then be used in the classification, which would form the basis for the selection of a method of grouting. Besides this way, Hässler has presented other ways as potential basement for a classification system.

The classification system that Hässler has proposed is based on a generalization of the rock mass to a simple geometry in which the flows of both water and grout can be calculated. He suggests that when the generalization has been done, the grouting
process can be calculated in the model. The research work and modeling continues in Royal Institute of Technology in Stockholm and Chalmers University of Technology in Göteborg (see Section 3.7).

In the method a very idealized fracture plane penetrated by a hole is considered to be described in plane-parallel and circular-symmetrical terms (Figure 3-10) (Hässler 1991). A fracture plane can be described by using four parameters: \( L = \) distance from the wall of the probe hole to the outer edge of the fracture plane (m), \( b = \) opening (m) (corresponds to the aperture of a fracture), \( \alpha = \) propagation angle (rad) and \( W = \alpha r_{\text{probe hole}} = \) circumference of the probe hole (m) (Figures 3-11 and 3-12). The propagation angle describes the conductive parts of the fracture plane; the angle of an entire conductive fracture plane in a cylindrical case is \( 2\pi \).

First the classification should presumably be done from pure geological data. When the process proceeds, one can define the parameters better according to the previous experience. In reality, the parameters, especially the propagation angle, are quite difficult to determine and to apply the method correctly one needs to get acquainted with it by large experience. Janson (1998) has estimated the values for his modeling purposes according to the RMR-value (rock mass rating number) of the rock.

Hässler (1991) has presented the equations to calculate maximum penetration and the volume of a Bingham fluid. Under laminar flow, cement grouts and cement-bentonite grouts are usually characterized as Bingham fluids – cement-based grouts are some of the best-known Bingham materials encountered in rock mechanics and rock engineering (Amadei 2000). According to the Bingham model (Bingham 1916) the fluid starts to flow when the shear strength, \( \tau_0 \) (the yield value of the shear stress), is exceeded by the shear stress (caused by the grouting pressure) and as long as the shear stress is higher than \( \tau_0 \) the behaviour is linear (Verfel 1889, Håkansson 1993) - similar as with Newtonian fluids. In general, as the Bingham material flows in the fracture, a viscoplastic layer develops over time along each fracture boundary (Amadei 2000). In that layer, the shear stress exceeds the yield stress of the material and flow occurs. The central part of the fracture is characterized by rigid plug flow with constant velocity and there the shear stress is less than the yield stress. As the flow front moves ahead, the velocity of the fluid decreases and the viscoplastic layer gets thinner, and as soon as it is zero, the flow stops because the shear stress on the fracture surface becomes smaller than the yield stress.

Hässler (1991) has presented the derivation of the maximum penetration of the grout \( (L_{\text{max}}) \) on the basis of the velocity profile of the grout (described above). The equation gets similar form in both one-dimensional and cylindrical symmetrical cases. In the latter case, when the flow occurs in a radial direction, the maximum penetration of the grout is independent of \( \alpha \). The maximum penetration of the grout can be calculated as follows:
where \( L_{\text{max}} \) = the maximum penetration of the grout (m), \( \rho_w \) = density of water (kg/m\(^3\)), \( h_w \) = water head in the probe hole (m) and \( h_L \) = water head along the outer strip of the fracture plane (m) and \( \tau_0 \) = critical shear strength (yield stress) for Bingham fluid (N/m\(^2\)).

As the volume of the space into which the grout has penetrated can be described as a sector of a circular symmetrical plane, the volume of the grout (\( V_{\text{inj}} \)) can be calculated:

\[
V_{\text{inj}} = \frac{a}{2} \left( r_{\text{inj}}^2 - r_w^2 \right) b = \frac{L_{\text{max}}}{2} \left( L_{\text{max}} + 2 \frac{W}{\alpha} \right) b
\]

where \( V_{\text{inj}} \) = the volume of the grout (m\(^3\)).

Classification of the rock mass on the basis of grouting results may be done at several levels of ambition, depending on the grouting data measured and the geological information available. Good knowledge of the properties of the grout used is required. It is possible for a very simple classification to ignore the fracturing of the rock mass and provide values per meter of borehole. Measured values (water-loss measurement, grout volume, etc.) are then split up per meter of borehole before the classification calculations are performed. This simple classification naturally imposes difficulties in adjusting a suitable method of grouting. A more suitable level of ambition takes into account the fracture frequency and local fracture plane variations. The measured values are then split up onto individual fracture planes before the classification calculations are performed. Alternatively, special investigatory grouting can be done where individual fracture planes have been isolated by packers.

The quantity of information collected from grouting also steers the level of ambition of the classification. If pressures and flows are not recorded during grouting, two variables must be set before the classification calculation is performed. It is probably easiest to set a value for \( W \) and \( L \) and, by means of a simple calculation, estimate \( \alpha \) and \( b \) on the basis of the quantity of grout injected and water-loss measurements. In cases where pressures and flows were recorded during grouting, it is theoretically possible to assess all the component parameters. But the whole thing is not so simple as variations in time can be expected to affect both the applied pressure and grout properties. Using the parameters produced that describe the rock, suitable measures to improve grouting results can be calculated in numerical simulation models and/or be obtained from earlier experience. Simple nomograms may possibly also be developed as a practical aid.
The Piece of Cake model

Figure 3-10. Simplification of the rock mass for grouting classification (Hässler 1991).

Figure 3-11. Circular-symmetrical, plane-parallel fracture plane. The angle of propagation $\alpha = 2\pi$ (Hässler 1991).
Hässler (1991) discusses also about the selection of the method of grouting on the basis of the classification parameters. Components of the method consists of grout, grout admixtures, extension of grouting time, location and orientation of grouting holes and grouting pressure and technique. The selection of a method is complex.

The four classification parameters contain condensed geometrical information about the conductive parts of the rock mass and the coupling between the grouting holes and the rock mass conductors. But it is not possible to list the significance of the parameters on the results of grouting, as all parameters, together with the rheological properties and hardening process of the grout, affect both the penetration and the course of events with time. For a given grout of Bingham type, the following simplified principles may be said to apply.

- The fracture plane opening (aperture) together with the shear strength of the grout and the grouting pressure used, steers the maximum penetration of the grout. Variations in these things affect directly the distance between grout holes for a given rock mass.
- The propagation angle, $\alpha$, affects the volume of grout necessary to reach a certain distance from the grouting hole (penetration). The propagation angle, combined with geological data can also certainly be used to assess the coupling between the rock mass's various fracture systems and, thereby, provide basic information for the selection of the distance between the holes.
- The contact length, $W$, which describes the connection between the grouting hole and the rock mass’s conductors, affects the time required to inject a certain volume of grout. As it takes longer to grout a large volume, a high $\alpha$, together with a low $W$, may justify the use of grout that takes longer to harden or has a lower viscosity.
- The distance influence, $L$, together with $b$ and $\alpha$, controls the resistance to water flow beyond the grout front. $L$ therefore affects the time required to reach a certain distance.

Hässler (1991) presents also a very simplified design example for a tunnel job based on his ideas and some calculation examples of grouting results from the numerical simulation model developed in the same context.

**Figure 3-12.** A partly conductive fracture plane $\alpha = \pi / 2$, $W = \alpha r_{\text{probehole}} = $ circumference of the probehole (Hässler 1991).
General overview on the groutability estimation/classification methods

Among the methods described above the Qi-method and the Hässler’s method are the most advanced. They take into account several different factors. However, both have significant deficiencies. The Qi-method includes familiar parameters which are easy to determine but the parameters should be modified or renumbered for this purpose again. The method Hässler has proposed agrees best with the aim of this study, but the parameters are very theoretical and prospective fractures are presumed to be similar that previous ones, which is not usually realistic. Other methods do not agree with the aim of the study.

3.6 Recent development work in Sweden

The grouting subject is widely under interest in Sweden and this has lead to co-operation between Royal Institute of Technology (KTH), Chalmers University of Technology (CTH), industry (e.g. NCC, Stabilator) and Swedish Nuclear Fuel and Waste Management Co (SKB). The grouting project is under way for the time being; and besides large amount of academic research, it has produced few doctoral thesis (e.g. Hässler 1991, Håkansson 1993, Andersson 1998, Janson 1998) and licenciate thesis (e.g. Fransson 1999 and Eriksson 1999) and few are under proceeding (by Fransson, Brantberger, Eriksson and Dalmalm).

This comprehensive grouting project consists of sub-projects joined to each other concerning widely the subject: rock characterization, grouts, laboratory and field tests, analysis of grouting results and modeling. The research work follows the ideas of Hässler (1991) (see Section 3.6 and Figure 3-9). In this context themes concerning geological factors in grouting and things related to that are in the interest.

Rock characterization problem has been examined with analytical, numerical and experimental approaches in CTH in Goteborg by Fransson (1999) together with prof. Gustafson. The aims of Fransson’s licenciate thesis (1999) were to investigate usefulness of water pressure tests of short duration as a tool to characterize the geometry of a fracture and to enable a more informed assessment of the resulting specific capacities, or transmissivities, for grouting predictions. In the first phase, which has produced a licenciate thesis, the work has been concentrated primarily on one fracture system in two dimension; later research is planned to treat a rock mass in three dimensions – several fracture systems and several boreholes. The results of Fransson’s studies up to the licenciate degree support the supposition of usefulness of water pressure test in characterization and choosing grouting strategy. There are plans to investigate four specific aspects of grouting prediction and strategy: characterization of fracture replica with hydraulic tests and grouting design, the effect of fracture fillings, distribution of specific capacities along a borehole. The project is planned to end up to field experiments involving all parts of the grouting project at Äspö HRL in the year 2000.

Extensive research is under work also in KTH in Stockholm (Prof. Stille and doctorands Brantberger, Eriksson and Dalmalm). The work contains grouting research by using laboratory experiments, numerical and analytical modeling and case analyses.
More or less associated to geologic factors in grouting the following research projects are under way (personal announcements and website: http://www.ce.kth.se/aom/AJOB/OTHER/personal.htm):

- Optimizing of grouting system, based on active design - design of a model which identifies and classifies the parameters which will affect the grouting process.
- Mechanisms controlling spreading of grout in rock. The focus on the attention is to measure and incorporate the limited penetration ability of grout. The work includes complementary laboratory tests and probably field tests during this year.

As a part of the research work in KTH, Eriksson has developed/proposed a method to calculate sealing efficiency and to predict ingress to a tunnel. The models and methods need in some cases to be further developed and as well tested on site. That is the scope in the future work.
4 DISCUSSION AND CONCLUSIONS

Water leakages are one of the major problems in tunneling - as well in the construction as in the operation phases. Uncontrolled water leakages can cause technical, occupational safety and environmental problems, and thereby also economical losses may be significant. The water problems arise frequently in tunneling projects - and will probably arise also in the final disposal facility of the high level nuclear waste.

This study concentrates on water leakages into excavations and on grouting as a method to seal rock mass; how to estimate the amount of water leakage and thus the need of rock sealing, and how to estimate groutability of rock from geological point of view? Reliable methods are desired to improve the grouting design and to optimize the process. To focus the sealing works accurately the presumable leakages have to be localized and estimated beforehand, so that the best possible sealing techniques can be applied.

This study is primarily based on a literature review. In the first part the methods to estimate groundwater leakages are described. The focus is on mathematical methods, but also qualitative and quantitative methods are shortly presented. The second part treats rock groutability and it contains a review of geological factors in grouting, research methods, experiences and suggestions to classify rock from grouting point of view and/or methods to estimate groutability of rock. In the following text these two topics are dealt separately.

Groundwater leakage estimations

Many analytical and numerical methods have been presented in the literature and occasionally used for groundwater leakage estimation purposes. Analytical solutions are many, based on different mathematical approaches and for different situations: for underground excavations of different shape, at different depths, in stable and transient groundwater flow conditions, before and after grouting, and also for single fracture zones. Because of this reason the comparison of them is not appropriate. All of them include naturally a lot of assumptions and simplifications, because water flow in fractured hard rock is very complex and the methods presume significant generalization of rock mass. The results are directly dependent on the available geological and hydrogeological information and the skills of the user. Some experiences of estimations are also shown - the results are variable. It is estimated that the analytical methods seem to be potential for estimations with order of magnitude of $10^{10}$ accuracy. However, two approaches seem to be the most relevant and practical: well approach based on Thiem's well equation for shallow excavations and imaginary well approach for tunnels at different depths.

Using computer aided numerical methods the water flow may be described and quantified more detailed than with rough analytical methods. The use of them is increasing and the development is fast. The problems with the numerical methods are naturally similar than with the analytical methods - to understand the hydrogeological system and conceptualize it, selection of the best mathematical model, large amount of data and its representativeness and validation (which can be made after the
excavation) - but the capability of them is nearly unlimited. Reliability and results depend highly on investing in creating a proper model and on data gathering.

Experiences of water leakage estimations are variable and there exists relatively little information in the literature about correlation between calculated and measured leakages.

The analytical solutions can be applied to general assessments of leakage amounts. In principle there is not such a limit for numerical methods. The choice how to estimate the amount of leakage water should be decided according to the requirements of the project. Both have remarkable advantages and disadvantages. Analytical methods are cheap and fast to use, but the reliability must be regarded quite weak. Numerical methods are more expensive and slow to apply, but the reliability is nearly unlimited depending highly on the gathered investigation data. For best results and idea of water leakages it seems justified to use numerical modeling as basis for design and improve the model as the tunneling proceeds. Analytical methods applied to fractured rock might be acceptable if they are used for very general estimations, and the biggest advantage of them may be got when rapid estimations are needed. The general fact is that the amount and quality of the input data determines strongly the reliability of estimations in both approaches, analytical and numerical.

It could be worthwhile to test systematically different analytical and numerical methods in the field and to compare the results with the measured leakages.

**Groutability estimations**

The other subject in this study was to study the factors that affects groutability of rock and search for methods to estimate and/or classify rock from groutability point of view. Grouting is a complicated task. Here the main interest is paid on geological and hydrogeological factors, which should be the basis for selecting suitable grouting strategy, technique and materials. The study here includes a review on geological factors in grouting, investigation methods, some experiences and proposed classification and estimation methods for grouting purposes.

Numerous factors affect groutability of rock. Among geological factors, the main properties are fracture aperture, spacing and connectivity of fractures, direction of fracture sets, fracture fillings and channeling of flow. The most significant uncertainties and difficulties seem to concern channeling of flow and fracture fillings, which may behave uncontrollably. Problematic fracture fillings, especially altering and softening ones like clays or chlorite, are so common that they may be impossible to avoid in practice. As well the variations in fracture characteristics along fracture zones can be very strong. These three factors appeared to be supposed reasons in most cases where the grouting result was not satisfying.

Conventional investigation methods seem to be sufficient for grouting purposes in tunneling projects, where the requirements are not high. The requirements of desirable result define the extent of preinvestigations – the methods, amount of investigation holes etc. Combining hydraulic tests together with detailed geological characterization
(including for example core recovery inspection and borehole scanning) seem to give usually enough information for groutability estimations if accurately done and interpreted. If the requirements of water tightness are strict the demands for special characterization methods arise. Channeling is extremely problematic to determine. There are no practical and reliable methods to find the channels of flow or methods to control the movable fracture fillings.

To get satisfying grouting results the specialists emphasize the importance of geological investigations in addition to traditional hydraulic testing. But the opinion how extent preinvestigations are needed/recommended depends as well on the desirable grouting results as the specialist. Reliable methods to investigate and classify rock and to estimate its groutability are widely desired. Also the opinion on the adequacy of existing grouting techniques and materials varies: some see that today's technique does not set limits to results and some see that there is much need for development in that sector. However, according to some profound grouting research projects, where much effort has been put on grouting design, it seems that most problems can be controlled by using suitable grout and spacing of grout holes.

Some proposals of rock classification and groutability estimations were found in the literature. Most of them are very rough and do not correspond to the idea of groutability meant in this report – the use of the term is not established. And experiences of any method are nearly non-existent. No satisfying classification and/or estimation method exists.

One proposed method is based on the Q-system, which is modified and take into account also the results of lugeon measurement. For grouting purposes a $Q_i$-value is calculated and from it the grout take could be estimated. However, the correlation has proved to be weak in practice and renumbering or remodification of the equation has been suggested. The advantage of $Q_i$-system is that the parameters are familiar and quite easy to determine.

Another interesting classification and estimation system has been proposed by Hässler (1991). The purpose of it corresponds to the idea what was desired in this context. In that method the fracturing is meant to be described with simple axialsymmetric geometry and as grouting proceeds the values of parameters will be checked and corrected according to earlier experiences for later calculations for grouting design. Thus it is supposed that following fractures are quite similar than previously grouted, which is not often realistic. One problem is the parameters in the method. They are not able to be given explicit values from ordinary geological data. Another lack of the method is that it does not pay attention to quality of fracture fillings.

Hässler’s method has not been established in use, but it has been basement for following research and development in Sweden as co-operation between universities (KTH and CTH), industry and Swedish Nuclear Fuel and Waste Management Co (SKB). The research work covers the whole grouting process and several academic thesis are published. The work continues and it is worth of following or even taking part into.
General conclusions are: there are methods but none proved to be practical and reliable enough for groundwater leakage estimations and there is no sufficiently satisfying method to estimate groutability of rock.

*Few points of interest*

There are some main questions, also concerning the Olkiluoto investigation site, which are not clear:

- are major water leakages presumable in deep excavations in crystalline bedrock?
- can those cause problems to safety and constructability?
- can those be grouted by using today's technique to avoid possible problems?
- If not, do we assume that we can develop the grouting technique significantly to solve the problems?

Before these questions are thoroughly answered, which may need further research, the usefulness of the development of a grouting procedure remains unclear.
REFERENCES


Finnish Civil Engineers (RIL) 1987. Tunneling and Construction in Rock 1. RIL 154-1 pp. 119, Helsinki, Finland. In Finnish.


SKB 1996. Äspö Hard Rock Laboratory. 10 years of research. Swedish Nuclear Fuel and Waste Management Company, SKB.


References referred by another but not seen by the author:


APPENDICES

Appendix 1:
The derivation of an equation to calculate groundwater leakage into a tunnel by using an imaginary well approach

The groundwater leakage into a tunnel can be calculated by using imaginary well approach, which is analogical for electrostatics of source-sink-system of two parallel linear conductor. The idea is the analogy of Darcy’s and Ohm’s laws.

The basic equations of groundwater flow (Airaksinen 1978) are:
1) Darcy’s law:
   \[ q(\vec{r}) = K \vec{J} = -K \nabla \phi(\vec{r}) \]
   where \( q(\vec{r}) \) = flow rate point in \( \vec{r} \) (m/s), \( K \) = hydraulic conductivity (m/s), \( \vec{J} \) = hydraulic gradient (-) and \( \phi(\vec{r}) \) = potential in point \( \vec{r} \),
   2) the potential of flow field:
       \[ \phi(\vec{r}) = z + P(\vec{r})/\gamma \]
       where \( z = z\)-coordinate (height) (m), \( P(\vec{r}) \) = pressure in the point \( \vec{r} \) (kg/ms\(^2\)) and \( \gamma = \rho g \) (kg/m\(^2\)s\(^2\)), and
   3) the flow through a surface \( S \):
       \[ Q_s = \int_{S} q(\vec{r}) \cdot d\vec{S} \]

If \( q \) = constant and flow occur perpendicularly against the surface, the integral can be written
   \[ Q_s = qa = -KaJ \]
   where \( a \) = area of the surface \( S \) (m\(^2\))

Next we assume a situation like in Figure 2-8 in Chapter 2 and the coordinate system presented in the follows; the groundwater table is flat and the pressure on groundwater surface \((z = H)\) and in tunnel \((z = 0)\) is same (here marked \( P_A = P_0 = 0 \)) We use the imaginary well method, where the real tunnel (a sink) is situated on a level \( z = 0 \), and an imaginary tunnel (a source) is situated on a level \( H \) above the groundwater surface \((z = 2H)\). The same amount of water flows into the tunnel as "flows" out from the imaginary tunnel. The situation in analogical to the static electric field of two infinite long parallel conductor in different potentials. The potential field and as a consequence the flow pattern is like presented in Figure 2-9 in Chapter 2. According to the analogy the potential \( \phi \) of a source/sink, when the zero level is set in the groundwater table, is
\[ \phi = \lambda \ln \left( \frac{H}{r} \right) \]

where \( \lambda \) = constant, \( H \) = height (m) and \( r \) = distance from the center of a tunnel (m).

Then for the source:

\[ \phi_s = -\lambda \ln \left( \frac{H}{\sqrt{r^2 - 2Hk}} \right) \]

where \( 2Hk \) = position vector in coordinate system. And for the sink:

\[ \phi_A = \lambda \ln \left( \frac{H}{|r|} \right) \]

When these potential fields are combined, we got the total potential in point \( \vec{r} \):

\[ \phi(\vec{r}) = \lambda \ln \left( \frac{H}{\sqrt{x^2 + z^2}} \right) - \lambda \ln \left( \frac{H}{\sqrt{x^2 + (2H - z)^2}} \right) = \lambda \ln \left( \frac{\sqrt{x^2 + (2H - z)^2}}{x^2 + z^2} \right) \]
When the gradient of potential is

$$\nabla \phi(r) = \frac{\partial \phi}{\partial x} + \frac{\partial \phi}{\partial z} \bar{k} = \frac{2 \lambda H}{r^2 (2H - r)^2} \left[ 2x(2z - H) \bar{k} + (2z^2 - 2Hz - r^2) \bar{k} \right]$$

In the point \(x = 0\) and \(z = R\), the gradient is:

$$\nabla \phi(0, R) = \frac{2 \lambda H}{R^2 (2H - R)^2} [R - 2HR] \bar{k} = \frac{2 \lambda}{R \left( 2 - \frac{R}{H} \right)}$$

According to basic equation 2 the potential difference between the point \(x = 0\), \(z = R\) and groundwater level is:

$$\phi(0, R) = -H + R$$

The equation 8 in point \((0, R)\) gets a form:

$$\phi(0, R) = \lambda \left( \frac{2H - R}{R} \right) = -H + R$$

$$\Rightarrow \lambda = \frac{-H + R}{\ln \left( \frac{2H}{R} - 1 \right)}$$

When \(\lambda\) is put in the equation of gradient

$$\nabla \phi(0, R) = \frac{-2(\lambda - H + R)}{R \left( 2 - \frac{R}{H} \right) \ln \left( \frac{2H}{R} - 1 \right)} \bar{k}$$

If we assume that \(R \ll H\) we got

$$\nabla \phi(0, R) = \frac{H - R}{R \ln \left( \frac{2H}{R} \right)}$$

which is the gradient in the point \((0, R)\)
If we assume that the gradient is same in every point around the tunnel, we can use the assumption \( q = \text{constant} \) and the flow is perpendicular against the surface, we can use the equation 4, when the flow rate is

\[
Q_s = -K_a J = K_a \nabla \phi = 2K \pi R L \frac{H - R}{R \ln \left( \frac{2H}{R} \right)}
\]

Thus the flow rate per unit length of tunnel is:

\[
\frac{Q_s}{L} = \frac{2\pi K (H - R)}{\ln \left( \frac{2H}{R} \right)}
\]