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Preface

The scope of this report is cement-based grouting for sealing of soil and rock formations in normal civil engineering projects. It does *not address hydraulic fracturing* at great depths of the kind practised in the Petroleum Industry, where the objectives are contrary to grouting for reduction of permeability.

Grouting with the aim of tightening and reinforcing the sub-ground holds a rather special position among the specialities of civil engineering for the simple reason that the result of grouting work cannot usually be readily inspected. Hence, the ways in which the sub-ground is actually affected by the treatment largely remains unknown. Although injection of fluids under *high pressure* into the ground may seem straightforward, the interpretation of how the zone subject to treatment is affected by the grouting work presents many difficulties.

Tightness and closure of a treated area may of course be checked by pumping tests, but the *actual physical impact and change,* to which a soil or rock formation is subjected, are seldom observed or documented, and therefore not very well understood even by those responsible for the grouting operations.

For instance, in many case records known to me, where excavation subsequent to grouting work has actually been carried out, only a *minute fraction* of the injected volumes of grout (as estimated often less than 1 %) have been identified within the treated area. A vital question is then: “Where is the balance of the grout consumed to be found?”
The answer may be a complex matter but evidently the grout is somewhere outside - mostly far outside the zone intended to be treated.

Grouting strategies embrace several spheres of civil engineering, such as:

- Geology - for structure and crack pattern of a rock formation;
- Soil mechanics - for soil structure, stress/strain properties and permeability;
- Structural mechanics - for the assessment of deformations, grout propagation and stresses related to the impact of subjecting ground to the action of a fluid under high pressure, usually far in excess of in situ states of stress;
- Hydraulics - not only covering Newtonian fluids but also the transient consistency properties of stiffening grouts - for the prediction of grout propagation and spread;
- Detailed knowledge of the rheological properties of different grouts, a vast subject deserving to be regarded as a discipline in its own right.

The complexity generated by these interacting factors, and the fact that the actual ***physical modification of the ground*** by the grouting work is hardly ever observed or known in its entirety, leaves the interpretation of results achieved open to guesswork and often to *unfounded speculation*. There are few other domains in civil engineering, where the engineers involved allow themselves such a range of different opinions and conceptual understandings, as when interpreting the effects of grouting in soil or rock formations.

The issues are admittedly complicated, but the only reliable and rational way to optimize grouting strategies is in my view to ***resort*** to the ***basic general laws*** known to engineering science, particularly in the fields of structural mechanics, soil mechanics and hydraulics.

In a former capacity as head of the engineering department of a major contractor, and later as consulting engineer, I have in the past been involved in grouting operations in connection with important civil engineering projects. Grouting work, as it is practiced and represented in related technical literature and research, has in my view largely been focused on grout consistency and the ability of grouts to penetrate the voids and cracks in ground formations in their primordial states - i.e. when still unaffected by grouting pressures.

Insufficient attention has - in my opinion - been given to a number of other vital aspects of the grouting process. Notably, little effort has been made on the study of how pressure-induced deformations in the ground affect - and often decisively control - the outcome of grouting operations.

It is hoped that the structure-mechanical applications to this effect, which are presented in this report will increase the appreciation of the importance of analyzing pressure-induced deformations in connection with grouting work.

As a rule, grouting work turns out to be adequately successful, although noteworthy failures are reported from time to time. However, with a better understanding of the mechanical response of the ground to *high grouting pressure*, and of the way injected grout is "accommodated" in the formation, it must be possible to achieve improved results in respect of both economy and target objectives. Importantly, the impact on the environment can be predicted with greater certainty.

Among other things I have found that the following aspects of grouting work are often overlooked or disregarded in current practice:

– The practicability of low pressure grouting according the principle of permeation (impregnation) in natural soil deposits and rock formations by cement-based grouts is often considerably overrated. The actual penetrability of suspension grouts in natural sediments and fissured rock is in my opinion almost consistently overestimated in grouting practice. The aim to attain target volumes of injected grout thus often results in inducing the operator to raise pressures to levels leading to the opening of the medium by hydraulic fracturing.

In my experience, closer studies of case records of penetration grouting usually reveal clear evidence of a high frequency of hydraulic fracture events with associated mechanical deformation. In the writers' opinion, few cases of **intended** *genuine* permeation grouting would in fact be successful without **unintended** hydraulic fracturing.

For instance, Ewert (1996 a,b) shows convincingly how several case records of penetration grouting according to the GIN principle in dam construction, actually to a major extent describe hydraulic fracture events. (GIN= Grouting Intensity Number = pressure × injected volume, Cf Section 5.3)

Garshol (2001) advocates using extremely high grouting pressures in combination with limitation of grout take. (Cf Section 5.3)

- Time and again, in connection with grouting work, discussion arises, in which experienced engineers claim to be performing 'permeation grouting' even when injection pressures in the order of 5 to 15 times the in situ overburden stresses are being used. Under such conditions, the incidence of hydraulic fracturing is obviously a very likely event.
- As mentioned above, the importance of the issue as to how pressure-induced deformations affect grouting has – in the writers' opinion – been notoriously underestimated in grouting practice. It is, for instance, rather symptomatic of the state-of-the-art in this field of engineering that a report such as Pettersson & Molin (1989), which constitutes a comprehensive and excellent review of objectives, techniques and procedures in the field of grouting, hardly comments upon the importance of deformation to grouting success. The need for making at least rough assessments of the magnitude of pressure induced displacements and their implications with respect to grout spread is not dealt with although reference is made to 37 items of work related to grouting technique and research. In Vägverket (2000), dealing with grouting specifications for tunnelling issued by the Swedish National Road Administration, the effect of deformation on the penetrability of suspension grouts is not even mentioned. This is indeed remarkable, as deformation of the sub-ground is an inevitable and quantifiable reality, constituting a powerful mechanism for grouting success. In the authors opinion it constitutes the main reason why grouting - in most instances – can actually be depended on as a viable method for reducing permeability and reinforcing the sub-ground.

- When grouting according to the principle of permeation as defined in Section 3 of this report, the stability of grout and its viscous properties are crucial for achieving optimum results. However, more attention should be paid to the question as to how the deformations in the ground, associated with the injection pressures actually applied in practice, affect the requirements regarding the properties of grouts. Clearly, when deformations tend to increase the initial width of a crack many times over, this must be a vital factor to be considered in this context.
- Grouting engineers often prescribe injection pressure limits with the good intent of controlling heave of the ground surface and environmental damage. However, the fact is that locally applied high temporary pressure as such, deep down in a bore hole, is not likely to have much effect in terms of lift or other damage at the surface. Instead, the decisive factor, generating heave of ground and related damage, is the quantity of grout actually forced into the ground, and the manner in which it has been injected.
- It is often argued that hydraulic fracturing must be avoided because of the risk of heave of the ground surface. Yet, in reality, there is a general tendency for hydraulic fractures to manifest themselves initially in vertical or sub-vertical planes, and therefore - especially in the beginning – to generate horizontal displacements, which are usually not even monitored. Normally, heave tends to occur in the final stages of a grouting program when, as a result of horizontal stress build-up, the horizontal stresses eventually may exceed those of the overburden considerably.
- As stated above, grouting operations for the most part attain the objectives set. Yet, forcing more grout into a formation than can be accommodated in the available volume of cracks and voids at a given pressure, serves no good purpose. I firmly believe that grouting according to solely pressure related stop criteria, as is very often the case in current practice, mostly leads to waste of grout and unnecessary risks of damage to the environment.

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In this report, the term '*hydraulic* fracture' is applied to the phenomenon, which some authors in the profession refer to as *'hydro* fracture'. Hence in the following, the term 'hydro fracture' is reserved for the case, where the hydraulic fracture is implemented with *water* acting as the pressure transfer medium.

The report is only concerned with the use of suspension grouts based on cement with or without additives such as bentonite, micro-silica, water reducing agents and similar products. The advantages and problems associated with various chemical grouts are not within the scope of this report.

In my view, the observations made above merit serious consideration by all parties engaged in the success and economy of grouting operations. It is my hope that this paper will stimulate *further scientific research* in this field of civil engineering as well as discussion on a number of topics regarding grouting techniques and grouting strategies.

The subsequent report was in essence completed in October 2003 but has been subject to minor editorial changes and supplements between this date and December 2004. Bernander (2003), represents a brief review of the October version.
Appendix (Exemplifications) was added in March 2004. An article on the same subject in Swedish was published in Bernander (2004).

Mölndal in December 2004

Stig Bernander

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The figures have been worked through by Lic. Tech. Håkan Thun, LTU, who has also edited the various versions of the report.

Abstract

After a short introduction in ***Chapter 1***, some typical properties of sedimentary rocks are given in ***Chapter 2***, exemplified with limestone formations in the Malmö Region in southern Sweden.

Two main grouting techniques are defined in ***Chapter 3***, grouting by *permeation* (pressure not causing fracture in the rock) and grouting by *hydraulic fracturing* (pressure causing opening of existing fissures or tensile fractures in the rock). The effect of deformations generated by grout pressure is discussed.

Permeation grouting, crack volume and the permeability of soil materials as well as different crack patterns in rock are reviewed in ***Chapter 4***. The permeability of cement based grouts in soil and rock is often overestimated. In a diagram, a relationship is given between Darcy's coefficient of permeability, k [m/s]; and a crack pattern defined by the number of cracks per meter, n [1/m] and the crack widths, t [mm]. The crack volume ratio is expressed as v_c / V [%].

Hydraulic fracturing is treated in ***Chapter 5***. For *confined* conditions, equations and diagrams are given for the maximum gap deformation in the cracks, δ_A [mm], and for the extension of the grouted zones. The equations and the diagrams are given as functions of the injected grout volume per round V [m^3/round] and the ratio E/p_o of the mean modulus of Elasticity E [MPa] and the injection overpressure p_o [MPa].

Three loading cases are treated:

a) two-dimensional loading with a grouted zone of length L_S [m];

b) conical loading with a grouted zone of diameter D_S [m];

c) concentric constant loading of diameter D [m].

A major issue here is that a defined relationship is established between the width of the grouted zone and the injected grout quantity per round, thus providing a rational basis for the prediction of grout spread into the environment.

For *unconfined* conditions, the risk of uncontrolled spread of the grout is discussed.

The importance of considering the deformations is illustrated with case studies.

Final remarks are given in ***Chapter 6***. One main conclusion is that the injection pressure as such is not a satisfactory stop criterion. Instead, the volume of grout injected per round should be defined *without limitation* of grouting pressure. It is

recommended to inject small amounts of grout, in several rounds, allowing the grout to stiffen between the rounds, rather than injecting large quantities in one or two stages. This procedure is illustrated by the examples in the *Appendix.*

The report is limited to grouting for tightening of soil and rock material in the neighbourhood of the bed-rock surface (say less than 200 to 300 m). It does not therefore deal with the type of deep-seated hydraulic fracturing practised in the Petroleum Industry aiming at promoting drainage of petroleum reservoirs.

Sammanfattning

Efter en kort introduktion i ***kapitel 1***, presenteras några typiska egenskaper hos sedimentära bergarter i ***kapitel 2***. Dessa exemplifieras med kalkstensformationerna i Malmö-regionen i södra Sverige.

Två injekteringsmetoder presenteras i ***kapitel 3***, *permeation*s*injektering* (där injekteringstrycket avsiktligt hålls så lågt att det inte ger upphov till deformationer och uppsprickning i berggrunden) och *hydraulisk uppspräckning* (där injekteringstrycket hålles så högt att det ger upphov till dragbrott eller utvidgning av sprickor i berget). De deformationer som uppstår vid de två metoderna diskuteras.

Permeationsinjektering, sprickvolym och permeabilitet för olika spricksystem i berg diskuteras i ***kapitel 4***.

Permeabiliteten hos cementbaserade bruk i jord och berg synes ofta vara överskattad. I ett diagram ges sambanden mellan

- Darcys permeabilitets-koefficient, k [m/s]
- ett sprickmönster definierat genom antalet sprickor per meter, n [1/m]
- sprickbredd, t [mm]
- och relativ sprickvolym v_c / V [%]

Hydraulisk sprängning (hydraulic fracturing) behandlas i ***kapitel 5***. För randvillkor tillämpliga på *slutna system* (confined conditions) ges ekvationer och diagram för maximal sprickbredd δ_A [mm] och för utsträckningen hos den injekterade zonen. Ekvationerna och diagrammen ges som funktion av injekteringsvolymen per injekteringsomgång V [m^3/omgång] och förhållandet E/p_o mellan bergets elasticitetsmodul E [MPa] och injekteringstrycket p_o [MPa]. Tre lastfall behandlas:

a) tvådimensionell belastning med spridningen L_S [m] i målområdet för injekteringen;

b) koniskt formad belastning med spridningen D_S [m] i målområdet för injekteringen;

c) konstant cirkulärt formad belastning

Analysen ger således ett rationellt samband mellan förväntad utsträckning hos den behandlade zonen och injekterad bruksmängd per etapp. Detta möjliggör bedömning av risken för icke önskad spridning av bruk i omgivningen.

För randvillkor under förhållanden *som medger läckage* ut i omgivningen (här benämnda *'unconfined conditions'*) diskuteras risken för bruksspridning utanför den avsedda injekteringszonen.

Betydelsen av den av injekteringstrycket orsakade utvidgningen av sprickor och deformationerna i omgivande bergmassor belyses vidare med praktikfall.

Avslutande kommentarer ges i ***kapitel 6***. En huvudslutsats är att enbart injekteringstryck inte utgör ett nöjaktigt stoppkriterium. Rekommenderat stoppkriterium är i stället injekterad mängd bruk per etapp utan tryckbegränsning. Det är således ofta bättre att injektera en liten mängd bruk per omgång i ett ökat antal etapper och låta bruket styvna mellan de olika injekteringsomgångarna. Detta förfaringssätt belyses med två exempel i ett *Appendix*.

Föreliggande rapport är begränsad till injektering för tätande av den övre berggrunden (säg mindre än 200 à 300 m under bergytan) och behandlar således *inte* hydraulisk uppspräckning på mycket stora djup av den art som förekommer inom petroleumindustrin. Avsikten där är för övrigt att *öppna* formationen i syfte att öka flödet av produkt från petroleumfyndigheten.

Table of Contents

Notations and Symbols

In this report, the following denotations are met with.

Symbols used in more than one sense are explained in more detail as well as being defined in the context, where they appear. Inconsequent notation of parameters results mainly from references being made to diagrams, figures and expressions by other workers or to previous work by the author of this report.

Roman letters

a	=	Distance between injection holes (Appendix).
b	=	Width of a considered section, [m].
C_1, C_2, C_3	=	Proportional constants related to Eq. 5.5b, 5.8b and 5.10a.
d	=	Distance - perpendicular to an action plane - to where the deformation induced by the 'jacking' pressure in the action plane may be regarded as negligible, [m].
d_{10}	=	Effective grain size in respect of permeability, [mm].
D	=	Diameter of even concentric load in the action plane as defined in Figure 5.2.
D_o	=	Nominal diameter of loaded area in the action plane – conical loading.
D_S	=	Extension of radial grout spread in the action plane – conical loading as defined in Figure 5.5a.
e	=	Crack volume ratio, i.e. porosity in terms of volume of the crack cavities related to conductivity or transmissivity - i.e. not considering closed isolated cavities. $e = v/V$ [m^3/m^3]. *Exception:* In figure 4.1, referring to work of Cambefort & Back (1968), e denotes the width of the considered flow channel.
e_o	=	Initial 'groutable' crack volume ratio related to conductivity. (Appendix)

E = Mean effective modulus of elasticity of a rock formation (unless defined differently in the local context). In rock mechanics this parameter is often denoted as E_m

$f_{t,\ rock}$ = Tensile strength of rock material around bore hole.

F_j = Jacking force in action plane induced by grout pressure.

g = Earth acceleration - gravitational constant.

h = Vertical extension or distance, as defined in the current contexts, such as:
a) Height of studied flow area in the derivation of Equation 4.1, Section 4.
b) Distance, over which the number of cracks N is defined in Figure 4.3 - i.e. $n = N/h$ cracks per metre.
c) Height or length of grouting stages (Section 5. and Appendix). E.g. injected quantity of grout per round and meter ($v = V/h$) in Equation 5.5a and Figure 5.4b.

H = Height or vertical distance as defined in the current contexts, such as
a) Head of water column, (piezometric height), in Section 4.
b) Total depth of zone subject to grout treatment, (Appendix).

H_G Depth from pressure guage to grouting stage.

H_R Depth from bed-rock surface to grouting stage.

H_S Depth of soil overburden (Section 5.2, unconfined conditions)

k = Hydraulic conductivity of water (hydroconductivity) or Darcy's coefficient of permeability in respect of water, [m/s].
Exception: In figures 4.1 & 4.2, referring to work of Cambefort & Back (1968) K_o and K are used for denoting Darcy's coefficient.

k_o = Ratio between the computed extension L_S (respectively D_S) of grout spread and the length (width) L (resp. diameter D_o) of assumed nominally loaded area.

K_o = Ratio of σ_h to σ_v - i.e. $K_o = \sigma_h/\sigma_v$.

L, l = Length or distance as defined in current contexts, such as a) Length (width) of nominally loaded area, ($L = 2\ l$) - triangular loading. b) Length of zone subject to grout treatment, (Appendix). c) Length of flow channels (Regarding unconfined conditions and Bingham behaviour).

L_S = Extension of grout spread, (width of grouted zone) – triangular loading.

l_S = Grout spread from injection hole $l_S = L_S/2$ – triangular loading.

m = Metre

Mwl = MWL = Mean water level

N = Newton, [kg·m/s^2].
Note: In Section 4 and in the Appendix, N also signifies the number of parallel cracks in a rock formation over a distance h.

n = Number of parallel cracks per metre in a rock formation, i.e. $n = N/h$ [1/m].

p = General denotation for pressure, [kN/m^2]. Pumps sometimes show pressure in bar. 1 bar = 100 kN/m^2 = 0.1 MPa.

Δp = Pressure difference, [kN/m^2].

p_i = Injection pressure in bore hole, [kN/m^2].

p_c = 'Claquage' pressure, [kN/m^2], (Cf Equation 5.1).

$p_{in\ situ}$ = Pressure required to restore the original in situ state of ground stress at the grouting stage level - i.e. the stress condition existing prior to the drilling of the hole for injection.

p_o = Effective injection pressure (overpressure) in relation to the prevailing pressure in the considered action plane, [kN/m^2] - i.e. $p_o = p_i - \sigma_{in\ situ}$.

p_G = Guage pressure, [kN/m^2].

p_f, p = Pressure in action plane, [kN/m^2].

p_{front} = Pressure at flow front - unconfined conditions, [kN/m^2].

$p_{f,\ r}$ = Pressure resistance at flow front - unconfined conditions, [kN/m^2].

$p_{i,\ flow}$ = Injection pressure in the final stage of an injection round, [kN/m^2]. Note: As grouting stop is based on a *pre-set grout volume*, $p_{i,\ flow}$ *is not* a stop criterion.

p_τ = Pressure loss in flow channel due to friction.

PL = Piezometric level in artesian aquifer.

r = Radial coordinate in polar coordinate system, [m].

r_o = Nominal radius of loaded area in the action plane, (conical loading).
Exception: In Equation 5.1 and Figure 5.1, r_o also denotes the radius of the injection hole.

r_s = Radius of grout spread, [m] – conical loading.

R = Radius defining the extent of the considered ground formation.

s = Second (time).
In the Appendix, s signifies the number of grouting stages per bore hole.

t = Mean crack width as determined from permeability according to Fig. 4.3, [mm].
Alternatively, t stands for the thickness of the flow channel in the assessment of Bingham flow. In general, the parameter t may also denote 'time'.

v = Crack volume in rock, [m^3] in Section 4. (Crack volume ratio $e = v/V$).
Exception: In Section 5 and in Appendix, v designates the injected

quantity of grout per round and metre of stage, i.e. $v = V/h$ [m^3/m].

V = Volume in general, m^3. V may refer to volume of treated soil/rock mass or to the volume of injected grout per round.

ΔV = Partial grout take per round in Appendix with reference to grout absorbed by pressure-induced deformation of the action plane.

x = Horizontal coordinate.

z = Vertical coordinate.

Greek letters

δ = Gap width of action plane, [m or mm].

δ_A = Maximum gap width of action plane, [m or mm].

γ = Bulk density of rock/soil material, [kN/m^3].

γ' = Bulk density in submerged state, [kN/m^3].

Δ = Signifies differential of variable such as $\Delta\sigma$, ΔV, $\Delta\delta$ etc.

ε_o = Coefficient defining Bingham behaviour. ($\varepsilon_o = 2\cdot\tau L/(t\cdot \Delta p)$).

ρ = Specific gravity, [kg/m^3 or ton/m^3].

μ = Dynamic viscosity of water, [centipois = $Ns/m^2\cdot10^{-3}$].

μ_o = Dynamic viscosity of water at 0 ^oC, [centipois = $Ns/m^2\cdot10^{-3}$].

$\mu_{H20,\,8°C}$ = Dynamic viscosity of water at 8 ^oC, [centipois = $Ns/m^2\cdot10^{-3}$].

ν = Poissons ratio.

σ_t, $\sigma_{r,t}$ = Tangential stress in a polar coordinate system. (In the current context normally a tensile stress).

σ_r, $\sigma_{r,r}$ = Radial compressive stress in a polar coordinate system.

$\sigma_{in\ situ}$ = Stress in the considered section prior to the application of injection overpressure (p_o) in the bore hole, [kN/m^2].

φ = Angle of internal friction.

σ_h = Horizontal principal stress in the formation, ($\sigma_h / \sigma_v = K_o$).

σ_v = Vertical principal stress in the formation.

τ_r = Pressure-induced shear stress at a distance (radius) r from the injection hole in a polar coordinate system.

τ = Shear stress in general.

τ_o = Flow resistance of a Bingham fluid in Section 5.1.4.3 regarding ε_o.

1 Introduction

The construction of tunnels, cut and cover structures, dams, coffer dams, excavations, etc, often necessitate prevention of excess water leakage during the working stages as well as in the operational phase. One method frequently used to achieve this objective is sealing the ground by grouting.

The success and economy of grouting work is, however, dependent on a valid understanding of how the individual ground formation responds to forcing ***large volumes*** of grout under ***high pressure*** into it. Grouting strategies and interpretations of the results obtained thus require good knowledge of the intrinsic properties of the ground material and how the formation will respond to high injection pressure. Of vital importance is furthermore knowledge of the way that other factors affect the process such as *injected volume of grout in each round*, *number of rounds, grout properties (consistency etc)* and *rate of grout flow*. Stop criteria must be set in accordance with the ambient conditions and with the objectives of the ongoing project.

Grouting holds a rather special position among the disciplines of civil engineering due to the fact that the result of grouting work cannot normally be observed or documented. The way the sub-ground is actually affected by the treatment is therefore often not very well understood.

In the following sections, a general grouting philosophy - by which is meant the conditions under which grouting operations should be planned and their results interpreted - is briefly presented. The scope of the report is limited to the use of *cement based* emulsified grouts with or without additives. The advantages and problems linked with various chemical grouts are not within the scope of this report.

It is important to note that the current report does ***not address*** the kind of *deep-seated hydraulic fracturing technology* practised in the Petroleum Industry, where the over all purpose is to open up a formation in order to promote drainage of petroleum reservoirs. This objective is effectuated by using grouts of high initial viscosity containing so called 'propping' agents. Contrary to ordinary grouts, the viscosity of the grouts used in this context *abates radically* with time thus promoting backwash of the fluid grout components - the propping objects being left behind.

Furthermore, the presentation is focused on grouting within a zone of not more than a few hundred meters below the bed rock surface. This is because at great depths, the relationships between the in situ overburden stresses and the shear strengths of the rock materials involved are *markedly different* from those in the vicinity of the bed rock/soil interface. For instance, at a depth of 2000 m the vertical in situ effective stress may be in the order 33 MN/m^2, i.e. far in excess of the shear strengths of a wide range of sedimentary and igneous rocks. Under such conditions, plasticity and creep can have far reaching effects bearing on the incidence of certain hydraulic fracturing phenomena.

Presentations of practice and research in Sweden can be found in e.g. Hässler (1991), Håkansson (1993), Gustafson & Stille (1996), Graad & Hedlund (1996), Hässler & Forshaug (1997), Stille (1997), Jansson (1998), Pettersson & Molin (1999), Vägverket (2000), Gustafson et al. (2000, 2004), Eriksson & Stille (2003), Dalmalm (2004) and Eklund (2005).

International practise and research are reviewed in e.g. Chadeisson (1962), Cambefort et. al. (1969, 1977), Lombardi (1985), Houlsby (1990), Ewert (1996), Henn (1996), Garshol (2001), Karlsrud (2001) and Warner (2004). (See also References.)

2 Ground conditions involving sedimentary rocks in Sweden

In Sweden, sedimentary rock is scarce occurring mainly in formations of Cambrian, Ordovician, Silurian, Jurassic and Cretaceous (Danian) origin. As grouting in sedimentary rock is sometimes considered as being radically different from grouting in igneous rock it may be of interest to comment briefly on past experience of grouting in sedimentary rock in Sweden

As is evident from Bernander (2000, 2002, 2003), Graad & Hedlund (1996) and VBB-COWI (2000), a number of grouting case records happen to relate to the geological conditions in the Malmö area. These case records are in time sequence: The Kockum Dry Dock (1967), Grouting tests in the Limhamn Quarry (1998), Grouting tests at Västra Station (1999), Grouting tests at Bagers plats (2001 - 02) and grouting for the foundation pit of the Turning Torso Tower (2002). The geological conditions in Malmö may therefore in this context conveniently serve as reference conditions for discussion in respect of grouting in sedimentary rock. Extensive grouting work has also been conducted in limestone formations for the Copenhagen Metro.

However, here only a brief description - exemplifying relevant ground conditions for the sedimentary rock structure in the Malmö area - will be given. This may of course be of particular interest, as tunnelling and cut and cover structures in Danian formations are presently being foreseen for the City Tunnel Project in Malmö. An example of the conductivity and transmissivity of the limestone in the Malmö area is shown in Table 2.1.

Table 2.1 Representative examples of conductivity and transmissivity of the limestone formations in Malmö. VBB-COWI (2000).

Formation	Zone	Conductivity [m/s]	Transmissivity (mean) [m^2/s]
Copenhagen Limestone	Hydraulic zone I	$5\cdot10^{-3}$	$5\cdot10^{-4} – 1\cdot10^{-2}$
Bryozoan Limestone	Hydraulic zone II	$1\cdot10^{-5} – 5\cdot10^{-4}$	$4\cdot10^{-4}$
Bryozoan Limestone	Hydraulic zone III	$1\cdot10^{-4} – 5\cdot10^{-3}$	$2\cdot10^{-3}$

The limestone base rock in the Malmö area consists of Bryozoan Limestone extending to considerable depth, but is in many places overlain by the so called Copenhagen Limestone having a depth often ranging between 0 to 8 meters. The Bryozoan Limestone located above 40 to 30 m below ground level is intrinsically of low permeability, sometimes being vertically jointed or having isolated vertical or sub-vertical bands of more permeable material. Below these levels, the limestone is more permeable. The vertical conductivity of the limestone is considered to be about 1/10th of the horizontal conductivity, implying that the given values of transmissivity in Table 2.1 are mainly related to horizontal flow – i.e. in the direction of the layered structure. This is not an unusual condition in sedimentary deposits.

The Copenhagen Limestone overlying the Bryozoan Limestone is assumed to be rich in fractures and horizontal permeable layers of considerable lateral extension but with regard to the random fracture system, the derived values of conductivity in the table above may be regarded as being equal in all directions. However, it is important to note that the limestone as such, like most rock material, inherently has low permeability to water and may be regarded as virtually *impermeable* to suspension grouts based on cement and cement/bentonite mixtures. The low intrinsic permeability of the limestone thus indicates that the documented transmissivity of the formation may be ascribed to local horizons of permeable material, water-conductive fissures or cracks and isolated leakage veins. The total porosity accessible to cement-based grout is likely to be extremely small, which is a factor to be considered when deciding on grouting techniques. (Cf Section 4.2.)

The limestone formations are built up of layers with variable degrees of induration classified as H1, H2, H3, H4 and H5, VBB-COWI (2000). This applies in particular to the Copenhagen Limestone exhibiting large portions of both of the extreme indurations H1 and H5. According to VBB-COWI (2000), the effective modulus of elasticity for the limestone rock mass is in the range of 200 to 2400 MN/m^2.

The progression of grout in sedimentary rock is of course influenced by its stratified structure but is, as far as concerns *hydraulic fracture grouting*, in principle not very different from other randomly built-up and heavily fissured rock formations. The action planes generated under fully developed hydraulic fracture conditions may *locally* follow an existing crack orientation but tend on a larger scale to adopt a more or less straight planar shape as illustrated on Figure 5.1 in Section 5. This topic is dealt with in more detail in Section 3.2. (Cf Figures 3.1 and 3.2).

However, in view of the fact that the *E*-modulus of igneous rocks often exhibits values greater than 30 000 MN/m^2, many sedimentary rock formations may be considered as consisting of material in between competent igneous rock and stiff soils in as far as strain and deformation analyses are concerned. Attention should be paid to this important condition, when assessing the response of this type of sedimentary rock to high grouting pressures.

3 Grouting techniques – definitions

The grouting techniques, which may be considered relevant to the reduction water conductivity in ground, are defined below.

The following presentation is focused only on the use of emulsified grouts based on cement with or without additives. The advantages and problems linked with various chemical grouts are not within the scope of this report.

3.1 Grouting by permeation - penetration of pores, open fissures and leakage paths

Grouting by permeation or impregnation may be defined as a method, by which the grout penetrates the cavities, the pore system, the open cracks and leakage channels of a formation at low, or under the circumstances, insignificant overpressure. A major issue in this context is that *deformations* in the rock or soil structure due to the grouting pressure are disregarded, the possible minor effects thereof not being considered as a means of achieving the sealing objectives. Generally, when grouting by the principle of permeation, the grouting pressure around the bore hole should not significantly exceed that of the overburden. Clearly, if this condition is not maintained*, hydraulic fracturing phenomena are prone to intervene*. However, the desire to attain target volumes of injected grout often inspires the operator to raise the pressure to levels conducive to the opening of the medium by hydraulic fracturing.

3.2 Grouting by hydraulic fracturing

Grouting by hydraulic fracturing is a technique, by which the ground structure is deliberately - but not seldom unknowingly - subjected to deformation and - in the immediate vicinity of the bore hole - often to fracturing (claquage) by the application of grouting-induced stresses *substantially in excess* of the local overburden vertical stress or the tensile resistance of the rock material. The objectives here are listed in items a) through d) below:

a) Opening of fine fissures and widening of existing cracks, thus creating enhanced possibilities for penetration, propagation and spread of the grout. In soils, wide seams of grout tend to form but also in igneous rock, groutstone intrusions more than 10 to 20 mm thick have been documented. In addition, new cracks may develop.
   Thus, in bore holes, which are *initially* not connected to existing open cracks and fissures in the immediate neighbourhood, fracturing and split-

ting of the rock near the bore hole can provide access to such crack systems. The fact that claquage pressures intentionally induce tensile stresses around a bore hole that are far in excess of the tensile strength of – in particular - sedimentary rock means that hydraulic fracture is normally locally initiated, and is in many instances actually a *requisite feature* of the grouting procedure.

b) Establishing *long range* access of grout to cavities, cracks and leakage paths that would be *far out of reach* if grouting were carried out by the principles of permeation and impregnation.
   Objective b) is realised by grout spreading into those cracks that, at each separate grouting stage, are most accessible to grout penetration, thus forming *action planes* likely to promote intersection with permeable structures even at considerable distance from the point of injection. Hence, contrary to what is sometimes argued, hydraulic fracturing does not in any way *exclude permeation* of porous material and open cracks, wherever this is possible.

c) Closing of finer cracks and fissure systems, which are inaccessible to grout penetration, due to *horizontal stress build-up*.

d) Consolidation and compaction of sub-grounds of soil, and in the case of rock formations with soil-filled crevices - also as a result of stress build-up.

As shown in Section 5 below, high pressure grouting inevitably entails the widening of existing cracks many times over, effectively promoting penetration and spread of grout. Obviously, this circumstance must have significant implications for the requirements in respect of stability and penetrability of grouts in connection with hydraulic fracture grouting. It stands to reason that these requirements are then not as crucial as when low pressure permeation grouting is at stake.

3.2.1 Vertical or horizontal hydraulic fracturing - lateral displacement or lift?

The likelihood of *vertical* hydraulic fracture is enhanced by *long stage* 'Tube à Manchette' (TaM) or 'End of Casing' (EOC) grouting, as the injection pressure then forms an *effective line load inducing additional concentric states of horizontal tensile stress*. The probability of *horizontal* fracturing, on the other hand, is favoured by concentrated short stage grouting.

Condition a) Grouting stage confined in sound, not fissured rock

When a grouting stage is totally contained in rock without cracks or fissures, fracturing is governed by the levels of tensile stress and the resistance to tension of the rock material. Even for grouting stages of moderate length, the material close to the bore hole is subjected to a 2-dimensional state of tangential concentric tensile stress.

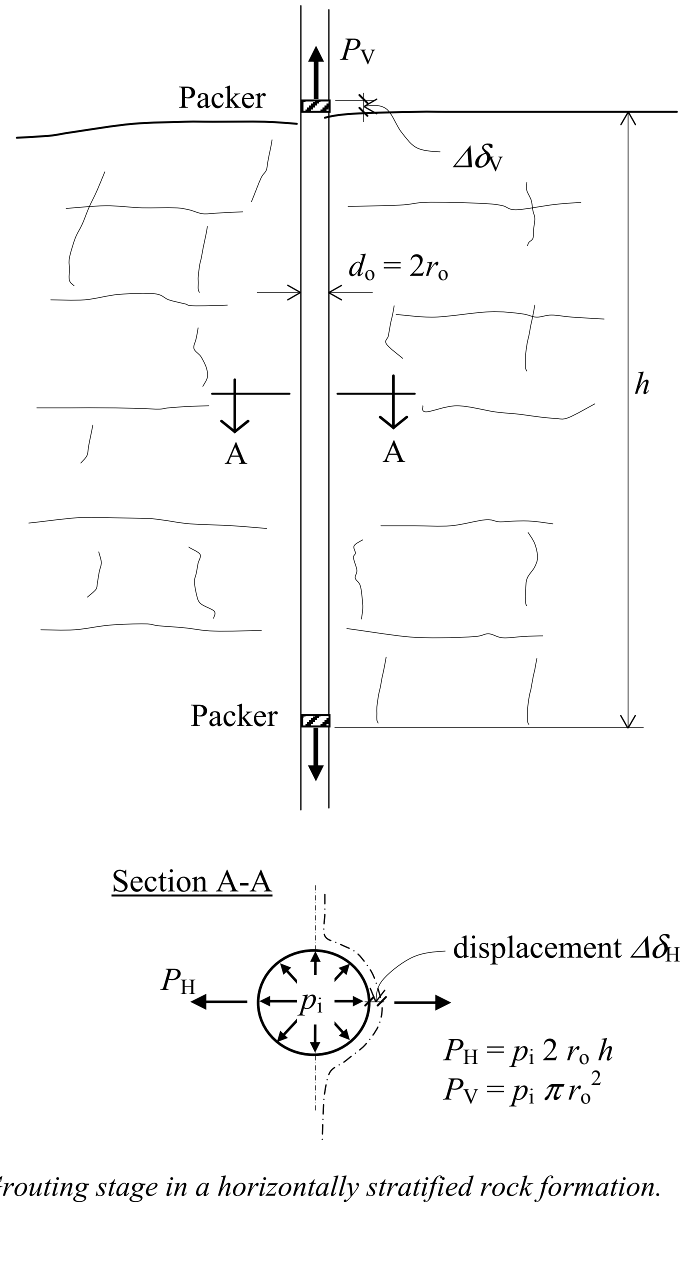


Figure 3.1 Grouting stage in a horizontally stratified rock formation.

As shown in Section 5.1.1, the stress induced by the claquage pressure $\Delta\sigma_c$ initiates 'by definition' vertical fracturing close to the bore hole. The considerable length of the zone subjected to tension, as well as the likely presence of micro cracks and varying tensile strength, *enhances* the probability of fracturing in planes parallel to the orientation of the bore hole.

By contrast, although it may be argued that the stresses at the extreme ends of the grouting stage are of similar magnitude, fracturing in the direction of the bore hole is normally restrained by the casing installed. This applies in particular when grouting with 'tube à manchette' (TaM) but is – due to a somewhat different effect–applicable also at the upper end of an EOC–stage provided the gap between casing and bore hole wall is sealed.

Condition b) Grouting stage in cracked or fissured rock

When grouting in fissured and densely jointed rock, the orientation of fracture planes is no longer governed by stress or by the strength of the rock material. Instead, the probability of vertical fracturing as compared to the incidence of horizontal fracturing, is more related to the following relationships - representing the degrees of mobilisation of the overall ground resistance to force and deformation.

Figure 3.1 illustrates a grouting stage of height (h) in a horizontally jointed rock formation. The bore hole diameter is $d_o = 2r_o$.

Using the denotations in Figure 3.1, the vertical pressure-induced force at the packers is $P_V = p_i \pi r_o^2$. The horizontal thrust acting between the packers amounts to $P_H = p_i 2 r_o h$. Hence, the ratio between horizontal thrust and uplift can be defined as $P_H/P_V = 2h/\pi r_o$. Inserting, for instance, $h = 2$ m and $d_o = 2r_o = 0.1$ m, then $P_H/P_V = 2{\cdot}4/(0.1{\cdot}\pi) \approx 25$ implying that, in the example, the initial horizontal splitting force is 25 times greater than the force acting upwards.

A relationship of even more interest in this context is the ratio between the horizontal and vertical displacements induced by the grouting pressure. For loads of equal intensity acting on the surface of a 3-dimensional elastic half-space, there exists a known condition implying that the deformations generated by such loads are approximately proportional to the extension of each individual load (i.e. to the total load), see Note below. It may thus be concluded that the ratio between the horizontal and vertical displacements δ_H/δ_V is in the order of $h/(2r_o) = h/(d_o)$.

In the example given above, the ratio of potential horizontal displacement to vertical displacement will be $2/(0.1) = 20$.

In sum, the study of grouting induced stresses, forces and displacements above indicates an overwhelming tendency to grouting induced fracturing in the direction of the bore hole, especially in the early phases of grouting work. This tendency is - in already fissured rock (Condition b) - to an important extent not much affected by the orientation of weak joints and tight cracks. For instance, even in a horizontally stratified formation of varying induration as depicted in Figure 3.1, fracturing in vertical planes will be a dominant feature. The reinforcing effect of the casing enhances this condition.

Case record: The tendency to fracturing in vertical planes has often been observed in practice. For instance, at Bagers plats in Malmö, some 80 m^3 of grout were injected into horizontally jointed and fractured sedimentary rock, without any heave of the ground surface being recorded until in the final phases of the grouting work. (In fact, even small settlements were observed initially).

Case record: Graad & Hedlund (1996) performed grouting trials in jointed limestone below the bottom of the Limhamn quarry using differently dyed grouts in the separate grouting stages. The coloured grout in core samples retrieved from special inspection holes revealed that grout injected at certain TaM levels was encountered several metres *below* the level of injection – a condition clearly indicating vertical fracturing in the *marked* horizontally jointed and fissured rock.

Furthermore, the very fact that extremely high 'claquage' pressures are often applied, mostly without any heave being recorded, is also indicative of the predominance of vertical fracturing. (Cf quotation from Cambefort & Back (1969) in Section 5.1).

Note: The law mentioned above concerning proportionality between the deformations and the extensions of areas with equal load intensity is related to the concept generally known as the 'pressure bulb' implying that a larger load area mobilises more of the subground than a smaller one. This relationship constitutes the basis for the well-known law applied in Winkler analysis of deflections y of beams on elastic foundation. In the basic equation, $p = c \cdot y$, valid for a slab with load p (kN/m^3), the bedding constant c (kN/m^3) may be regarded as being independent of its width. Hence, in terms of a line load q (kN/m) on a beam (or slab) having a width b (m), the equation is written $q/b = p = c \cdot y$ or $q = c \cdot b \cdot y$.

3.2.2 Orientation and straightness of action planes

In excavations of ground subjected to grouting treatment, one may observe a remarkable straightness in the orientation of action planes in soil, sedimentary and jointed rock. This may of course be due to the presence of planar weakness characteristics in the structure of the formation but there are, as will be demonstrated below in section 3.4, powerful structure-mechanical factors, which favour a tendency to straight planar propagation of action planes.

Figure 3.2 depicts an action plane, which owing to local features in the intrinsic crack pattern, tends to deflect in direction BC - away from the dominant direction AB. It may be established already at this point in a qualitative way that the principal stresses on the 'acute side' of the plane are compressive in both directions, thus involving *little shear*.

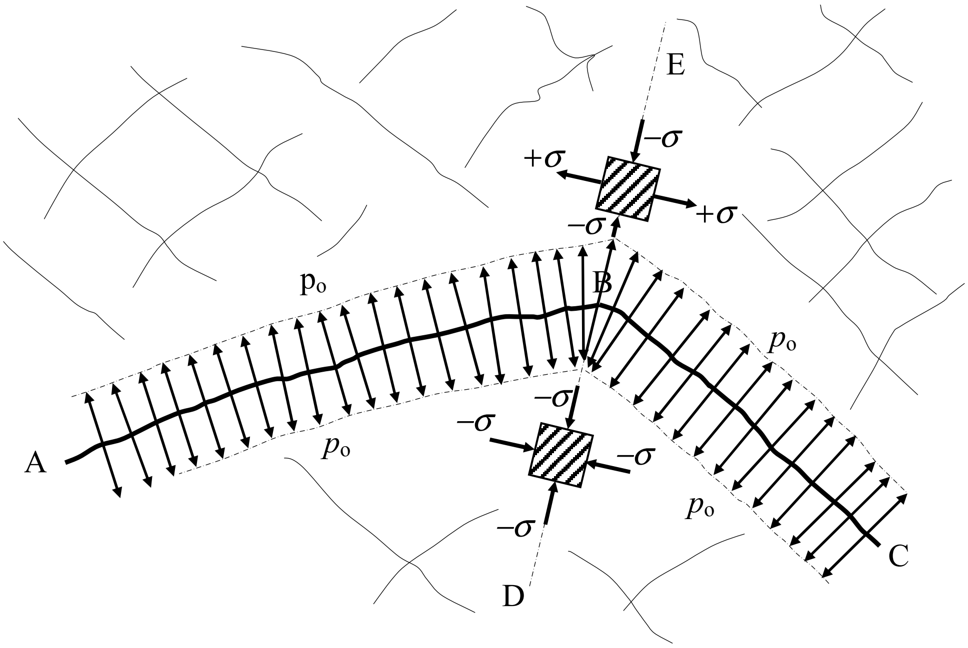


Figure 3.2 Stress conditions at an assumed change of direction of the crack plane from AB to BC.

By contrast, the state of stress on the 'obtuse side' of the plane is characterised by compression in direction DBE, and tension in the perpendicular direction - i.e. a state of *high tension* and *shear*. This counteracts the change and the crack action plane prefers to stay straight, see further Section 3.4 below.

3.2.3 About initiation of hydraulic fracture

Among many engineers in the grouting profession, there exists a belief that the occurrence of hydraulic fracturing requires that the force exerted by the grouting pressure must exceed the weight of a cone shaped body of overlying material. It is, therefore, assumed that the injection pressure may be allowed to surpass the vertical overburden stress substantially, without risking the incidence of hydraulic fracturing.

Hence, for instance, when ostensibly performing permeation grouting according to the so-called ‘GIN method’, grouting pressures up to *three times* the ambient overburden pressure are allowed, (Lombardi (1985) and Lombardi & Deere (1973)).

This ‘belief’ is questionable for at least two reasons:

- Firstly, hydraulic fractures generally initiate in vertical or sub-vertical action planes and cannot therefore have any relevance whatsoever to the weight of a cone shaped body of overburden material.
- Secondly, the criteria governing the incidence of hydraulic fracture are not solely related to force or stress but also to strain and deformation. Hence, once (after claquage) the local stresses due to the grouting pressure exceed the in situ stresses by some measure, the rock (or soil material) around the point of injection is subject to deformation, whereby radial or horizontal cracks begin to form and open up.

Even when these cracks are thin, such displacement may facilitate penetration of grout exerting increased tangential tension or lift, which again leads to more penetration, and further crack propagation. The likely result is a progressive failure typical of hydraulic fracturing, which for structure-mechanical reasons therefore may occur even at lower injection pressure than three times the vertical overburden stress.

In the estimation of the author, any grouting method allowing injection pressures of more than 1.5 times the overburden vertical stress has the potential of resulting in hydraulic fracturing, especially when grouting at some depth. For instance, at a depth of 30 m, the overburden vertical stress (in terms of total pressure) amounts to some 800 kN/m^2. Multiplying this value by a factor 1.5 represents an overpressure of 1200-800 = 400 kN/m^2, which means that, when grout penetrates say 1 m in each direction into a crack, the splitting force is already in the order of $400 \cdot \pi \cdot 1^2 \approx 1300$ kN. Assuming for instance an E-modulus for fissured rock of 2000 MN/m^2, the equation 5.7a (section 5.1.4) predicts a crack width growth of 0.3 mm (with E/p_o = 5000 and D_S = 2 m) – i.e. a gap growth well sufficient to enhance further grout propagation - even disregarding the claquage effect. Moreover, the effects of *hydro-fracture* due to excess pore water pressure, induced by the advancing grout, are likely to boost these tendencies. In fact, the build up of *high excess water pressure in cracks* during grouting may actually be an essential ***primary mechanism*** *in hydraulic fracturing*.

3.2.4 Disadvantages of hydraulic fracture grouting??

It is often maintained, especially by engineers involved in dam construction that high pressure grouting might be harmful to the closure of a rock mass because of the formation of new fractures.

Clearly, as for instance stressed by Ewert (1996 a,b), there is little advantage in fracturing a rock mass of inherently low permeability, especially if the transmissivity of the formation is characterised by well distributed systems of fine cracks.

In contrast, if transmissivity is unacceptably high, especially when due to discrete and randomly distributed leakage paths, it may well be worth while to fracture or deform the rock mass with the precise aim of intersecting such concentrated permeable features.

However, for several reasons, the author of this report does not generally subscribe to the notion regarding the highly deleterious nature of hydraulic fracturing phenomena. One important reason for this is that close studies of case records of intended penetration grouting usually reveal evidence of a high frequency of *hydraulic fracture events*. Of the many successful grouting operations investigated or controlled by the author, none can actually be categorised as '*genuine*' permeation grouting. This is largely due to the fact that the legitimate attempt of the operator to attain the target volumes of injected grout often results in raising the pressure to a point leading to an opening of the medium by hydro-fracture and hydraulic fracturing.

This circumstance is, for instance, convincingly demonstrated by Ewert (1996 a,b), where several case records of penetration grouting according to the GIN principle in dam construction (Lombardi & Deere (1985, 1993)), actually document a *high frequency* of hydraulic fracture events.

Furthermore, from structure-mechanical points of view, the formation of serious new fractures is not a likely result, as repeated grouting tends to build up considerable horizontal pre-stress in the ground (significantly increasing the values of $K_o = \sigma_h/\sigma_v$), closing other finer cracks that would otherwise be inaccessible to grout.

Moreover, the shear deformations generated in the rock along the action planes are normally small, a fact, that can be readily documented in excavations subsequent to grouting simply by observing the rate of change in thickness of groutstone intrusions, i.e. the small taper of the same. In one such case observed, the intrusions were of almost constant width over a distance of 6 to 8 metres, which by the way, is consistent with wide spread of grout.

The GIN method is commented upon in more detail in Section 5.3.

3.3 Compaction grouting

Another grouting technique, used mostly for the purpose of rectifying the effects of settlement in buildings and undesired subsidence, is ‘hydraulic jacking’. By this method very thick grouts, i.e. of a consistency similar to that of normal fresh concrete are employed, whereby the desired lift action can be confined to a restricted area of treatment. However, this technique is not relevant in the current context.

3.4 Orientation and straightness of cracks

Below follows some derivations regarding the straightness of cracks. They complement the presentation in Section 3.2.2 above.

Curved deflection of action plane

Figure 3.3 shows the states of stress in a case, where the deflection of the action plane is curved. If the radius of curvature is r_o, the stresses on the ‘inside’ of ABC are:

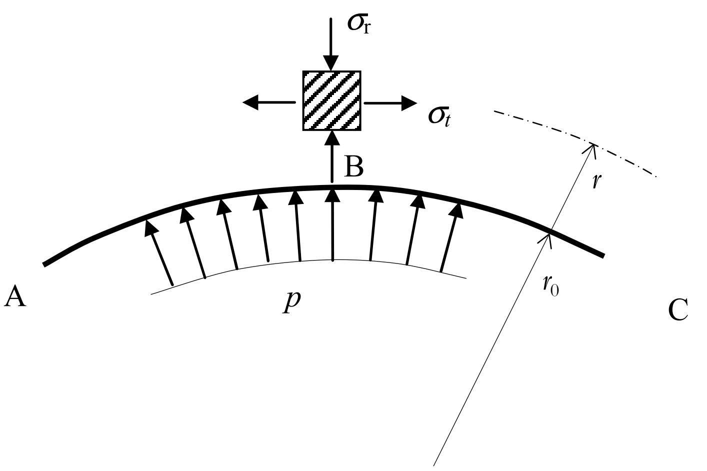


Figure 3.3 Curved deflection of action plane – stress conditions

$$\sigma_t = \sigma_r = -p \quad \text{- i.e. a state of compression in both directions} \qquad (3.1a)$$

Immediately ‘outside’ of ABC, the principal stresses are as given by Equation (5.1) in Section 5.1.1:

$$\sigma_r = -p \text{ and } \sigma_t = +p \quad \text{- i.e. radial compression and tangential tension} \qquad (3.1b)$$

rendering a state of pure shear.

Sharp angular deflection of action plane

Figure 3.4 on the other hand illustrates the stress conditions in a case, where the change of the orientation of the action plane is pointed and defined by an angle α.

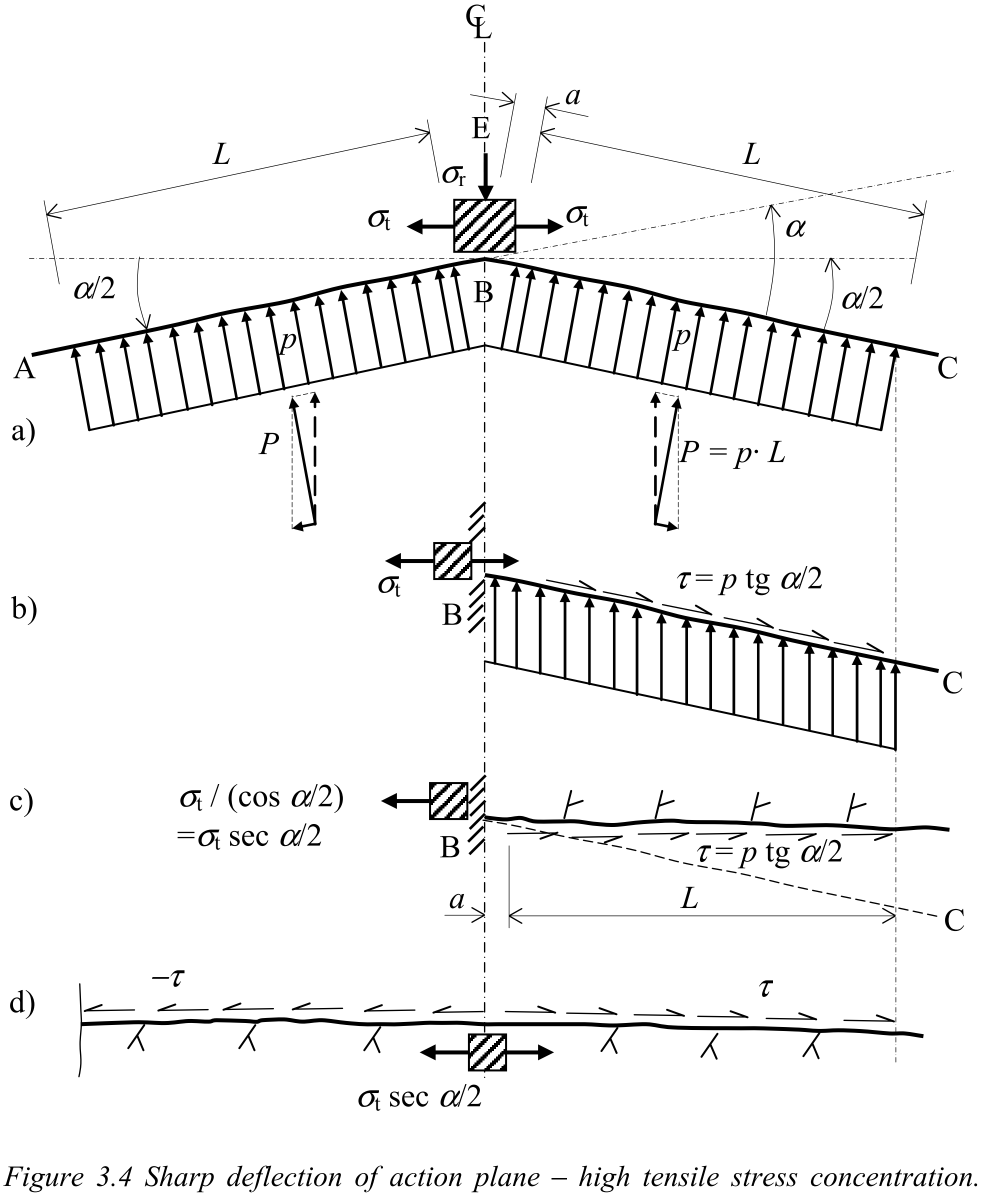


Figure 3.4 Sharp deflection of action plane – high tensile stress concentration. a) Stresses when a crack changes direction from AB to BC. b) Equivalent shear and vertical stresses. c) Equivalent horizontal stresses, d) Tensile horizontal stresses.

In this situation, there will be a high concentration of tensile stress at the point of sharp deflection generated by the pressure component ($\tau = p \cdot \tan\alpha/2$) acting away from the line of symmetry DBE on the ‘obtuse side’. This can be shown by the following derivation.

Derivation of horizontal stresses σ_r

Figure 3.5 shows a section through an elastic 2-dimensional half-space, the surface of which is loaded by two horizontal line loads working in opposite directions.

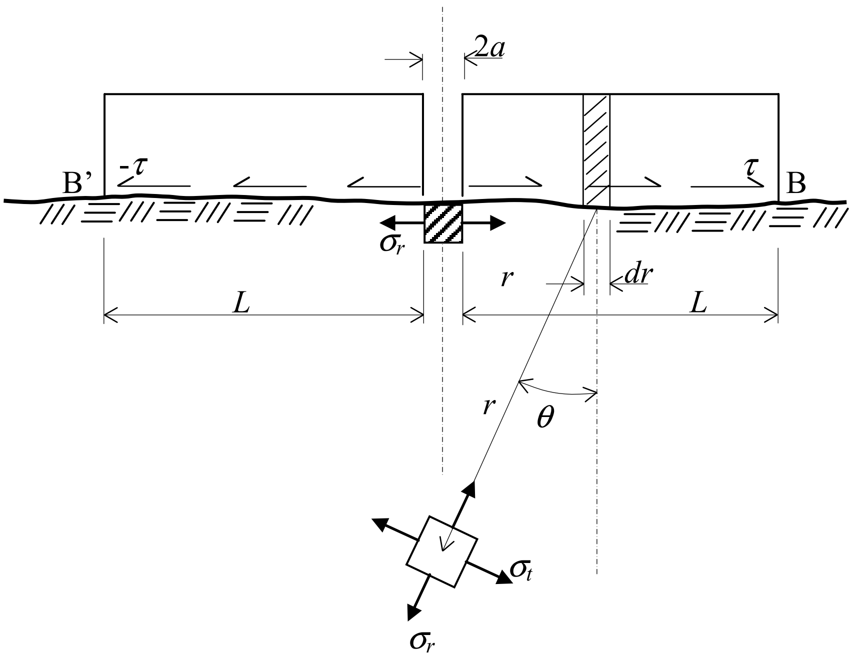


Figure 3.5 Elastic half-space subjected to horizontal surface loading.

The radial stress σ_r at a point (r, θ) in the half-space due to a horizontal point load of $\tau \cdot dr$ may be expressed as, see Timoshenko & Goodier (1970).

$$d\sigma_r = \frac{2 \cdot \tau \cdot dr \cdot \sin\theta}{(\pi r)}$$

For points near the surface $\theta = \pi/2$, whence $\sin\theta = 1$ and

$$d\sigma_r = \frac{2 \cdot \tau \cdot dr}{(\pi r)}$$

Integration from $r = (a)$ to $r = (L+a)$ gives

$$\sigma_r = \int_a^{L+a} \frac{2 \cdot \tau \cdot dr}{(\pi r)} = \frac{2\tau}{\pi} \cdot \left[\ln(L+a) - \ln a\right] = \frac{2\tau}{\pi} \cdot \ln\left(\frac{L+a}{a}\right)$$

Adding the effect of the load $-\tau{\cdot}dr$, working in the opposite direction the value of σ_r at the centre line is

$$\sigma_r = \frac{4\tau}{\pi} \cdot \ln\left(\frac{L+a}{a}\right) \qquad 3.2$$

In a brittle elastic material, the tensile stresses at the line of symmetry would approach infinity for infinitely small values of the distance *a*.

The horizontal ***tensile*** stress σ_t can be estimated at

$$\sigma_r = \frac{4p}{\pi} \cdot \tan\left(\frac{\alpha}{2}\right) \cdot \ln\left(\frac{L+a}{a}\right) \qquad (3.2)$$

Putting $\sigma_r = \sigma_t / \cos(\alpha/2)$, as in Figure 3.4c, the tensile stress at B is

$$\sigma_t = \frac{4p}{\pi} \cdot \sin\left(\frac{\alpha}{2}\right) \cdot \ln\left(\frac{L+a}{a}\right) \qquad (3.2a)$$

where α, *a* and *L* are defined in Figure 3.4.

On the ‘acute side’ there will be a corresponding concentration of high compression i.e.

$$\sigma_t = -\frac{4p}{\pi} \cdot \sin\left(\frac{\alpha}{2}\right) \cdot \ln\left(\frac{L+a}{a}\right) \qquad (3.2b)$$

It is thus evident from Equation (3.2a) that the sharper the angular deflection of the action plane is, the higher are the concentrated shear stresses at the point of deflection. In fact, if the distance (*a*) approaches zero, tension and shear in a brittle (non-plastic) elastic material would virtually become ***infinitely great***. This indicates a high proneness for failure initiation at the point of deflection thus favouring from then on further straight propagation in the direction of the main action plane.

It may here be argued that the tensile strength σ_t might favour fracturing perpendicularly to the main crack orientation – i.e. in the direction BE according to Figures 3.2 and Figures 3.4. However, in this context it must be born in mind that the tensile stress according to Eq. 3.2b is an extremely local effect on account of the smallness of the distance *a* in relation to the distance *L* and to the total spread of grout.

Furthermore, as demonstrated in Figure 3.6, the overall effect of the ***total*** extended load on the half-space results in significant compression (parallel to the half-space surface) in the area where the loads ends. This stress condition is related to the shape of what in soil mechanics is often termed as the '*pressure bulb*'. The compressive stresses, which extends far from the loaded surface effectively counteract crack development along the direction BE.

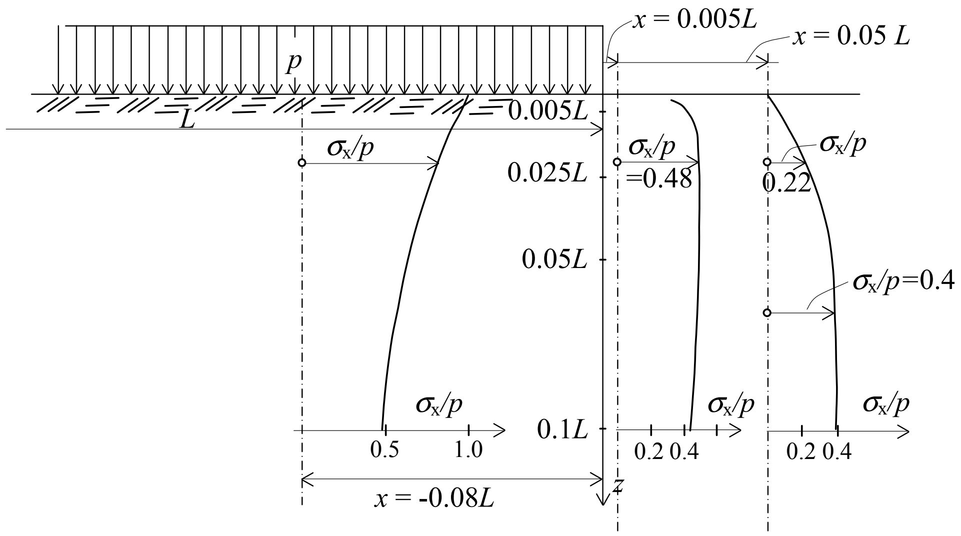


Figure 3.6 Detail at the end of a constant distributed load on an elastic half-space. The diagram focuses on the compressive stresses parallel to the surface (or the direction of load extension).Compare with the inclination of the pressure trajectories of what, in the current context, is often termed as the 'pressure bulb'.

To sum up, the important conclusion to be drawn from the study of the stress state generated by the deflection of pressurised action planes is therefore that there are strong ***structure-mechanical grounds*** for action planes to ***abhor deflection***, thus tending to remain in more or less straight planes. Once an action plane is well established, its ***direction of propagation*** is largely independent of the local crack or fissure system.

Considering that most rock is jointed with inherent weak seams and fissures, this conclusion has rather general validity but applies in particular to soil, weak and tightly fissured rock formations.

Note: It may be observed in this context that the structure-mechanical conditions explained above may constitute an important factor, accounting also for the circumstance that intrusive ‘hypabyssal’ rocks in the earth crust tend to occur as straight planes. Intrusive dykes – such as diabase dykes – may, for instance, run straight through a landscape for tenths of kilometres. (Confer for instance Figure 5.9).

3.5 Conclusions

The forced deformation of the ground structure induced by grouting pressure is the ***principal means,*** by which the desired sealing effect is achieved, when applying hydraulic fracturing.
In view of the inability of cement based grouts to penetrate into fine particle soils, rock material and fine crack systems in a rock mass*,* hydraulic fracturing often constitutes the ***only viable technique*** for reducing the transmissivity of most types of ground.

Yet, although the prospects of success are greatly enhanced by hydraulic fracturing, it is important to keep the *probabilistic* nature of all grouting methods in mind. Practical experience shows that repeated injections are usually needed for attaining the desired sealing effect.

In hydraulic fracture grouting, fracturing in planes oriented in the direction of the injection hole is a predominant feature, especially in the early phases of grouting work. Hence, with vertical bore holes, the incidence of fracturing in vertical or sub-vertical planes must be anticipated and the consequences thereof considered. This applies even to horizontally jointed formations - unless of course *open*, *horizontal crack planes* are present.

In addition, attention should be given to the fact that most rocks also display fissuring and seams of poor tensile resistance in directions other than the orientation of the dominant system of weakness joints.

Furthermore, in superficial layers of bedrock, the occurrence of vertical initially *open* cracks is generally more frequent than horizontal ones due to compression from the weight of overburden.

Once dominant action planes have formed there are, apart from the orientation of the current crack system, strong structure-mechanical factors ***promoting a straight planar progression*** of action planes.

The consideration of deformations in hydraulic fracture grouting is *exemplified* for two different scenarios in an Appendix .

4 Permeation grouting - permeability of sub-ground to grout

It is evident from the definitions in Section 3 above that permeation and impregnation grouting techniques are not designed to subject the ground to excessive strain by the grouting process. Instead, the grout is meant to permeate cavities, pores and cracks in the ground structure at pressures *sufficiently low to avoid hydraulic fracturing*.

The radius of grout penetration from the point of injection depends primarily on the width of open cracks in the rock or of the gradation of soil. It is also strongly conditioned by the rate of grout flow (i.e. the time factor), the rheological properties (e.g. viscosity) and the stability of the grout. Local leakage paths, encountered by advancing grout, may randomly be sealed by grout penetrating further into veins, channels and other markedly porous structures. (Incidentally, an important question in this context to those who do not recommend high pressure grouting is in what way permeation of discrete leakage paths is likely to be enhanced by low pressure as such).

However, an *inherent* ***adverse feature*** of *genuine* permeation (impregnation) grouting is the ***limited radius of action*** of cement-based grouts in most natural deposits. This entails among other things a substantial risk of *not striking* important leakage paths, even when these are located at a small distance from the injection hole.

Similarly, in rock formations, *thin cracks* are difficult to reach and to seal solely by permeation under low pressure using cement-based grouts.

Moreover, the basic material of most rock structures may be regarded as having inherently *zero* permeability in as much cement-based suspension grouts are concerned – i.e. a fact totally ruling out impregnation of the same. Further, the probability of encountering isolated permeable features with bore holes that are often spaced several metres apart, is adversely low.

It should be emphasised in this context that the penetrability of many types of chemical grout is superior to that of emulsified grouts. However, the advantages and the potential problems in terms of toxicity linked with various chemical grouts are not dealt with in this report.

4.1 Limit criteria for penetration grouting in soil

The limiting criteria for impregnation grouting are related to grouting pressure, permeability, crack widths, rheological properties and flow rate of grout. In soils, the effective grain size d_{10} is a crucial parameter.

Advanced studies by Henri Cambefort (1969, 1977) deal, for instance, with the limiting criteria regarding permeability, viscosity and flow rates, when grouting by impregnation in porous granular materials, see Figure 4.1. The figure is based on the following deduction. According to Darcy's law (1856) the flow rate Q [m^3/s] through an area A [m^2] of a medium with the permeability k [m/s] and the pressure gradient $\Delta H/\Delta x$ (usually negative) can be written as

$$Q = -A\,k\frac{\Delta H}{\Delta x}$$

During injection of a hole at a depth H [m] in a section/stage of height h [m] with a grout with an overpressure p_o [MPa], a density ρ [kg/m^3] and the gravitational acceleration $g \approx 9.81$ m/s^2, the pressure gradient $\Delta H/\Delta x$ can be written as $\Delta p/(\rho g \Delta r)$. The flow Q can then be written as

$$Q = -2\pi r h k\frac{\Delta p}{\rho g \Delta r} \Rightarrow Q\int_{r_1}^{r_2}\frac{dr}{r} = -2\pi h k\int_{p_o}^{0}\frac{dp}{\rho g}$$

$$Q\left(\ln r_2 - \ln r_1\right) = 2\pi h k\frac{p_o}{\rho g} \Rightarrow Q = 2\pi h k\frac{p_o}{\rho g \ln\frac{r_2}{r_1}}$$

In order to study the influence of the permeability of the foundation to water k_o, the permeability to grout k can be written as $k = k_o\,\mu_o/\mu$, where μ_o and μ [Ns/m^2] are the dynamic viscosities of water and grout respectively. For a case with r_1 = 0.05 m, r_2 = 2 m and $H \approx 10$ m we may assume $p_o = 0.59\,p_{ref}$, where p_{ref} = 1 MPa, which gives

$$Q = k_o\frac{h\mu_o}{\rho g \mu}p_{ref}$$

The formula is illustrated in Figure 4.1 for h = 1 m, $\mu_o \approx 2\cdot10^{-3}$ Ns/m^2 at 0 ^{o}C, ρg = 10 kN/m^3 and for three values of $\mu = 2\cdot10^{-3}$, $20\cdot10^{-3}$ and $100\cdot10^{-3}$ Ns/m^2 respectively, corresponding to three values of $\rho g\mu/(h\mu_o)$ = 10, 100 and 500 kN/m^4 respectively. As an example $k_o = 10^{-4}$ m/s and $\rho g\mu/(h\mu_o)$ = 100 kN/m^4 gives Q = 0.001 m³/s = 1 l/s. (This is 10 times more than what is given in Cambeforts original diagram. The mistake in the original diagram is apparently due to an error in defining the units of measurement).

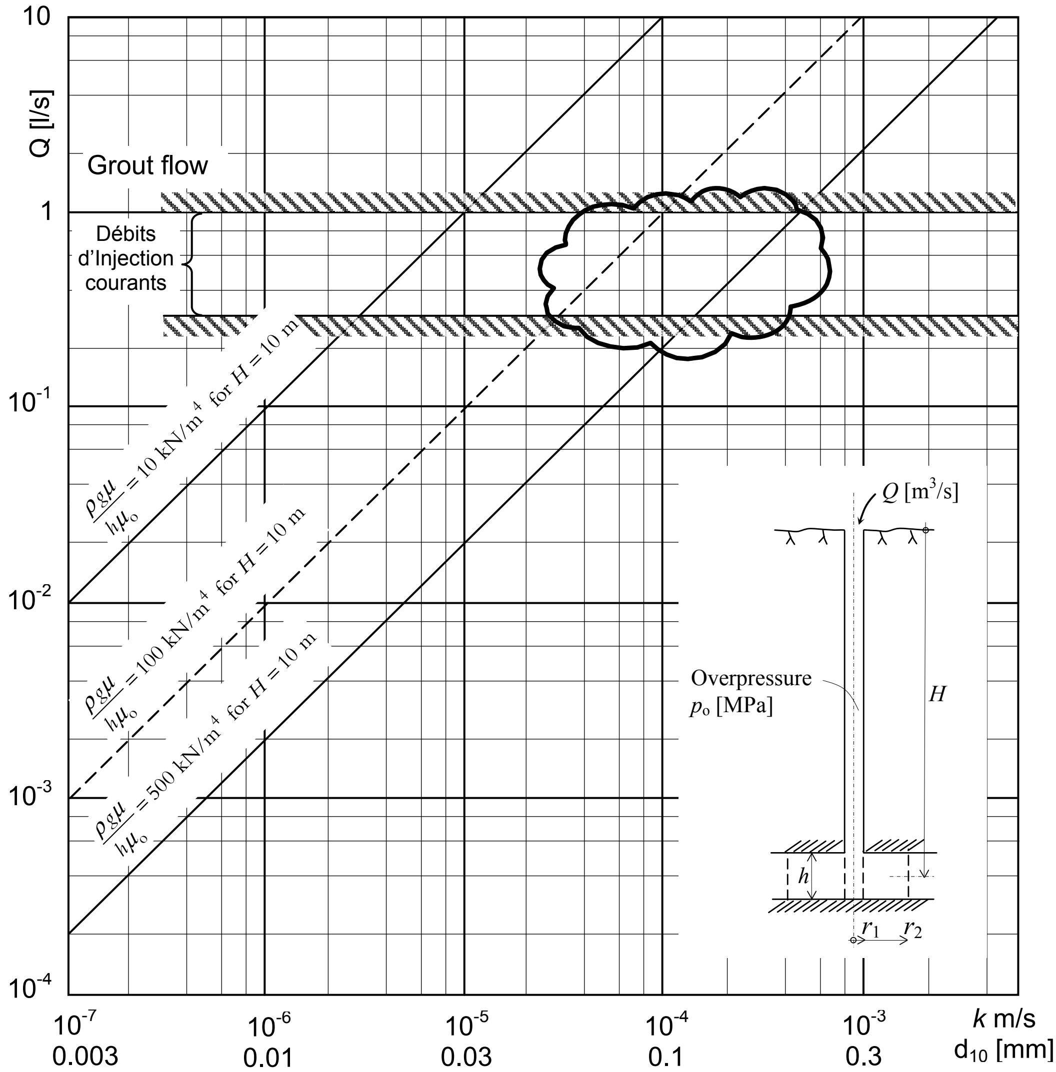


Figure 4.1 Diagram showing the limiting relationships between permeability k [m/s], dynamic viscosity μ [Ns/m^2] and flow rate Q [l/s] lest hydro-fracturing occur at a depth of H = 10 m, when grouting by the principle of impregnation in porous granular, ρ = density [ton/m^3], μ_0 = dynamic viscosity of water at 0 ℃ ≈ 2 centipois = $2 \cdot 10^{-3}$ Ns/m^2, h = the width of flow path = 1 m. The two lines in the diagram represent viscosities of 2 and 100 centipois [$10^{-3} \cdot$Ns/m^2]. The dashed line, representing a viscosity of 20 centipois [$10^{-3} \cdot$Ns/m^2], has been added to the diagram by the author of this report. Diagram published by Back (1969) in a translation of the works of Cambefort (1964). Using cement-based grouts, hydraulic fracturing is likely to occur under these conditions for k between $5 \cdot 10^{-5}$ and $2 \cdot 10^{-4}$ m/s (i.e. for $d_{10} \approx$ 0.05 to 0.2 mm with d_{10} according to Hazen (1925)).

In respect of cement-based grouts figure 4.1 indicates that the permeability coefficient k should be considerably *in excess* of $0.5 \cdot 10^{-4}$ to $2 \cdot 10^{-4}$ m/s for any prospects of successful permeation grouting with cement based grouts.

Applying relationships established by Hazen (1925), see e.g. Jansa & Isgård (1961), this may be interpreted to the effect that, using normal flow rates, the corresponding effective grain sizes d_{10} with regard to permeability should be greater than 0.05 to 0.2 mm.

This implies, of course, that for smaller values of d_{10}, hydraulic fracturing phenomena are prone to manifest themselves, whether they are intended or not. (In soil mechanics, by definition, 90 % of the grains by weight are larger than d_{10} and 10 % smaller or equal to d_{10}).

In conclusion, at low injection pressure, permeability coefficients and grain sizes have to be *at least* within the mentioned ranges, if penetration of *any practical significance* is to be achieved. In this context, one must take into account the fact that natural soil deposits or soil filled gaps in rock (even when consisting of coarse sand and gravel), normally contain significant and *unpredictable* amounts of finer particles. They therefore often exhibit values of d_{10} less than the about 0.2 mm given in Figure 4.2. In fact, using emulsified cement grouts, very *few natural soils* are *sufficiently permeable* to be readily grouted solely by the principles of permeation and impregnation.

Type de COULIS	Ciment	Bentonite + Ciment	Bentonite défloculée	Produits chimiques			Mousses	
				Silicate de soude		Résines organiques		
				Gel dilué	Gel dur		Ciment	organiques
NATURE	SUSPENSIONS			LIQUIDES			EMULSIONS gazeuses	
	instables	stables						
DOMAINE d'utilisation	Fissures	Sables et graviers					Cavités	Fortes circulations d'eau
		$k>5.10^{-4}$ m/s	$>10^{-4}$ m/s	$>10^{-5}$ m/s	$>10^{-4}$ m/s	$>10^{-6}$ m/s		
Conduite de l'INJECTION	Pression de REFUS	QUANTITÉS limitées					Remplissage	
PRIX Relatif des produits pour 1 m^3 de vides	4.2 Dépót avec γ_d = 1.5	1 Ciment 2kN Bentonite 300 N	0.8 á 1	2 á 4	6	10 á 500	1.2	10
d_{10} [mm]		Gravels and Sands		Silty sands, sandy silts				
		> 0.18	> 0.10	> 0.032	> 0.10	> 0.01		

Figure 4.2 Table showing areas of application and recommended stop criteria for commonly used grouts in terms of permeability coefficients of the treated ground from Cambefort (1977). The table has been supplemented with a bottom row indicating the effective grain sizes d_{10}, which according to Hazen (1925) correspond to the different values of permeability shown in the table, (Bernander 2002).

4.2 Grout penetration in rock formations

Grouting involves the introduction of considerable volumes of grout under high pressure into the sub-ground to be treated and different types of geological structures may respond differently to this mostly violent activity.

In Figure 4.3, a relationship is shown between the width t [mm] and the number of parallel cracks $n = N/h$ [1/m] on one hand and the permeability k [m/s] on the other. The relationship is based on a combination of Darcy's law (1856) and Poiseuille's law (1846).

Darcy's law gives the flow rate Q [m^3/s] through a cross-sectional area A [m^2] of a medium with the permeability k [m/s] and the hydraulic gradient $I = dH/dx$:

$$Q = A\,k\,I = A\,k\frac{dH}{dx}$$

Poiseuille's law expresses the flow rate q [m^3/s] in a slit-shaped channel between two parallel horizontal plates at a distance t [m] for laminar flow of a fluid with the dynamic viscosity μ [Ns/m^2], width b and a pressure gradient dp/dx [N/m^3]

$$q = \frac{t^3}{12\mu}\cdot\frac{dp}{dx}\cdot b$$

Combining the two equations for an area $A = hb$ with N cracks of width t within the distance h gives

$$Q = hbk\frac{dH}{dx} = q = Nb\frac{t^3}{12\mu}\cdot\frac{dp}{dx}$$

Using the relationship between the pressure p, the piezometric head H and the density of the fluid γ[N/m^3], the pressure gradient can be rewritten as $dp/dx = \gamma\ dH/dx$. Inserting $n = N/h$, the expression for t becomes

$$t = \sqrt[3]{\frac{12k\mu}{n\gamma}} \tag{4.1}$$

For water at 8°C, $\mu = 1.387\cdot10^{-3}$ Ns/m^2 and $\gamma = 10^4$ N/m^3, Eq. (4.1) gives

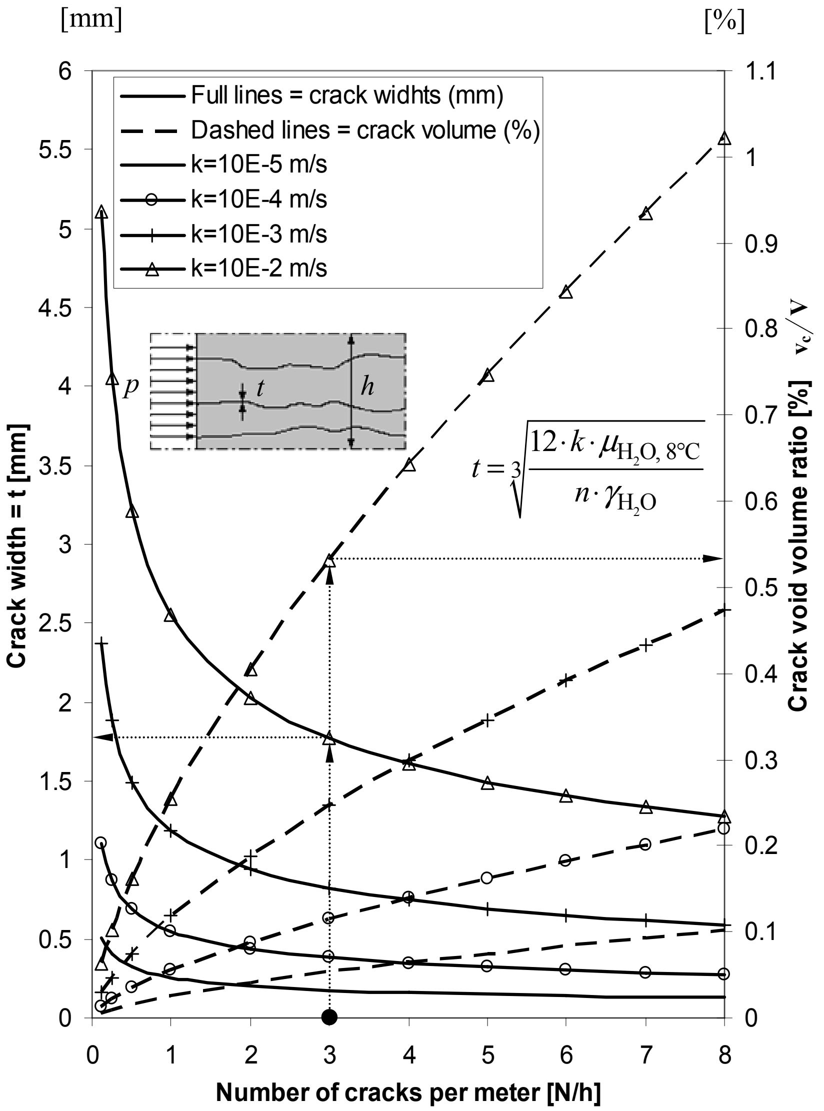
$$t = 11.85\cdot10^{-3}\cdot\sqrt[3]{\frac{k}{n}}\quad[\mathrm{m}] \tag{4.1a}$$

Figure 4.3 shows a graphical presentation of Equation (4.1a). In the diagram, the crack void volume ratio v_c/V [%] is also given as a function of of the number of cracks $n = N/h$ [1/m] and the permeability k [m/s].

In other words, Figure 4.3 represents an example of the relationship between a certain crack pattern in a rock formation and a corresponding coefficient for potential flow according to Darcy.

In Figure 4.3, the permeability is taken to be 2-dimensional, (i.e. all cracks are parallel to *one* plane. However, if the cracks are oriented in two mutually perpendicular planes, the permeability may be said to be 3-dimensional, the formation being conductive in all three directions. If this is the case, then the void ratios according to Figure 4.3 should be suitably adjusted.

Assume for example that the water conductivity in the horizontal direction is $k = 10^{-3}$ m/s and that the crack spacing is 2 m. Then, with a conductivity in the vertical direction of $k = 10^{-4}$ m/s and a related crack spacing of 0.4 m, the *total void ratio* according to Figure 4.3 would amount to 0.100+0.070 = 0.170 %.



$\mu_{H_2 0,\,8°C}$ = dynamic viscosity of water at 8°C $\approx 1.387 \cdot 10^{-3}$ Ns/m^2

Figure 4.3 Relationship between Darcy's coefficient of permeability, k [m/s], number of cracks per metre, n = N/h [1/m] and crack widths, t [mm] for a specific crack pattern. The corresponding crack volume ratio v_c/V [%] is also shown. For a case with n = N/h = 3 cracks per metre and a permeability of k = 10^{-2} m/s the diagram gives a crack width t = 1.8 mm and a crack void volume v_c/V = 0.53%.

Successful impregnation grouting in *soils* generally requires a high consumption of grout. If, for instance, a fluid stable grout is injected into a formation of coarse gravel with a penetrable porosity of say 30 %, then in theory a volume of some 300 litres of grout would be needed to seal a target volume of 1 m^3.

By contrast, in a heavily fissured and *highly permeable* formation of igneous *rock* with, say 5 open through cracks per m^3 and each having a width of 1.0 mm, the nominal void ratio amounts to only 0.5 %. According to Figure 4.3, this crack pattern correlates with a permeability coefficient of $3 \cdot 10^{-3}$ m/s. The volume of grout, which can be injected at low pressure into a target volume of 1 m^3 is then only 5 litres as compared to the 300 litres.

The example thus clearly illustrates the *fundamental condition* that the capacity of rock to absorb injected grout volumes *radically* deviates from that when impregnating soils. Little wonder, therefore, that the overwhelming evidence from the field is that the injected grout, owing to *overrated permeability*, often migrates *far away* from the intended areas of treatment.

Assessment of permeability in the field

The potential of rock formations to absorb a given volume of grout is in fact often systematically overrated for various reasons. It is evident from the exemplification above, that a reasonable prediction of the grout volume that can be accommodated in the treated zone constitutes a major efficiency and cost factor. The volume of grout absorbed is usually defined as a percentage of the rock volume to be treated. This percentage is often based on water loss tests (Lugeon tests), on core drilling or on other determinations of the permeability coefficient (*k* m/s).

However, the results of water loss tests measured in Lugeon (1 Lugeon = 1 liter/minute and meter bore hole at an overpressure of 10 bar) and Darcy's permeability coefficient *k* are not compatible, even in theory. In large masses of rock, where the whole length of a bore hole is pressurised, the Lugeon value may provide a measure of the mean permeability of the formation. But, when water loss is recorded over short lengths, there can be no unambiguous relationship whatsoever between Lugeon values and hydro-conductivity.

Moreover, the permeability of rock formations is often based also on core drilling. The method of retrieving core samples usually necessitates flushing by water, whereby fine particles, loose fragments from the core and soil filled crevices tend to be poorly represented in the boring logs. Because of this, there is a notorious tendency to overestimate the void volume and the permeability of the rock mass.

This in turn is likely to entail a corresponding overestimate of the expected grout take.

Although it is often practised, the assessment of grout take on the basis of water permeability in terms of Lugeon values or permeability coefficients, is actually a highly questionable approach. This is due to

– the fact that the capacity of cement-based emulsified grouts to penetrate systems of thin cracks in a rock mass may have *little* (often nothing) in common with permeability to water.

– the fact that there cannot possibly be a fixed relationship between the mean permeability of a rock mass and random crack systems in the rock. As is evident from Figure 4.3, such a relationship will vary strongly with the *nature of the crack pattern*.

– the fact that the pressures applied in Lugeon tests are clearly sufficient to bring about *hydro fracturing*, in which case the test results will not be valid at all for penetration grouting purposes.

For instance, in a case known to the author *no* grout consumption was registered when grouting at levels where high Lugeon values had been measured. On the other hand, large grout takes were recorded at other levels in the same bore hole with Lugeon values of almost *zero*. These inconsistencies evidently reflect the incidence of hydro fracturing phenomena in the Lugeon tests.

Nevertheless, as the permeability valid for 'potential flow' of water is often used as a measure of 'groutability' in current practice, it may be of interest to study more closely in what way a given permeability coefficient can be translated into a penetrable or 'groutable' crack volume. In Figure 4.3, for instance, the widths and frequency of parallel systems of cracks are studied.

It stands to reason that a water flow rate based on a certain k-value represents a very different void ratio if there is, for instance, **one** crack over a distance of 10 m or if there are **twenty** cracks per 10 m, (i.e. 0.5 m on centres). (Cf Figure 4.3)

In the first instance, a k-value of say 10^{-4} m/s would correspond to one, ca 1.2 mm wide readily penetrable crack, making a total porosity of only 0.012 %. In the second case, the same k-value is consistent with 20 cracks, each being 0.44 mm wide. If all of the cracks are presumed penetrable by grout, they represent a volume of 0.088 %, consuming about 7 times more grout than in the first case.

By comparison, if grout in the order of 1 % of the treated rock mass is to be accommodated in a treated zone, the permeability of the formation has to be about

10^{-2} m/s. (1 % is a figure often specified in practice.) In such a case, the *k*-value corresponds to a rather unlikely situation with 8 cracks per meter, each being 1.25 mm wide. (Cf Figure 4.3).

Clearly, the nature of the crack pattern must have a decisive influence on the choice of grouting strategy. Hence, if a formation is made up of tight blocks separated by a few highly permeable faults, grouting of *small volumes* by permeation at low pressure in the faults may be a feasible and economical approach. (Cf Example 1A in Appendix).

Conversely, the closer the crack pattern corresponding to a certain *k*-value is, the thinner the cracks and the greater the difficulties to accomplish permeation with cement-based grouts. Thus, if the volume of rock to be treated constitutes a zone with fine closely spaced cracks, grouting by permeation at low or moderate pressure holds little prospects of success. Hydraulic fracture (expansion) grouting is then likely to be the only viable option.

If the nature of the crack pattern and the permeability of a rock formation are known, Figure 4.3 may provide guidance when assessing a suitable quantity of grout to be injected. For more details on this issue, see Appendix "Exemplification of deformation analysis".

It may be mentioned here, that the importance of the geologic features and crack pattern is strongly emphasised by Ewert (1966 a, b). Confer in this context also Section 5.3, where grouting according to the GIN principle is briefly discussed.

Note: In the reasoning above in respect of grout take, the incidental effect of filling possible closed cavities *not contributing* to the transmissivity of the rock mass, has been disregarded.

4.3 Conclusions

- The practicability of grouting following the principle of permeation (impregnation) in natural soil deposits and rock formations using cement-based grouts is usually significantly *overrated* in current practice. This is mostly due to the tendency to *overestimate* the true penetrability of suspension grouts in natural sediments and fissured rock.
- Considering that the transmissivity of rock formations is often due to *discrete water conductive leakage paths,* and that the inherent porosity of most rock material is virtually inaccessible to cement-based grouts, grouting by penetration (impregnation) is for obvious reasons generally not a viable approach. This depends to a great extent on the fact that - *with bore holes at normal spacing -*

the prospects of striking and/or reaching discrete water- conductive features in the rock mass by low pressure grouting may be very unfavourable in terms of probability.

- Because of this, grouting work intended to be carried out as genuine *penetration grouting* is often in practice actually *realised* by *hydraulic fracturing.* Time and again, in connection with grouting projects, the writer has experienced how knowledgeable engineers claim to be performing ‘permeation grouting’ despite the manifest fact that *injection pressures* in the order of *5* to *10 times* the in situ overburden stresses are being applied. Under such conditions, structural mechanics as a matter of course predict the likely incidence of hydraulic fracturing.

5 Hydraulic fracturing - structure-mechanical response of the sub-ground

The response of a rock formation to high injection pressure is different depending on whether the grouting pressure is applied under ***confined*** *conditions* deep down in the base rock or under ***unconfined*** *conditions* in the vicinity of the soil/bedrock interface.

In the former scenario, the spread and absorption of grout is – apart from filling of possible directly accessible open cracks and cavities - governed by deformations in the rock mass induced by the grouting pressure, which inevitably results in the widening of some of the cracks penetrated by grout. Often, also new cracks may form, especially in the neighbourhood of the injection hole when applying the 'claquage' pressure.

Hence, the use of high injection pressure brings about an expansion of the cracks in which penetration and spread of grout takes place, entailing greater consumption of grout than what corresponds to the initially penetrable crack volume. On the other hand, the increase of horizontal thrust in the ground may close adjacent crack systems making them less accessible to grout penetration - at least during the ongoing injection round. (Cf examples in Appendix).

Yet, once the maximum volume of grout that can be accommodated in the formation at *a certain pressure* has been injected in the course of a grouting stage, the risk of resurgence of grout at the soil/bedrock interface or at the ground surface is imminent.

As soon as the grout front has invaded the rock/soil interface over a distance greater than about the depth of the soil cover, there is virtually no limit to further progression for a Newtonian fluid under a sufficiently high, sustained pressure. (Regarding the Bingham effect confer Section 5.1.4.3 and Figure 5.8 below). The limiting condition here is whether the pressure at the front of the advancing layer of grout is in excess of or less than the ambient vertical overburden stress.

In this situation the grout propagates under *unconfined conditions*, whereby the spread of grout, although still conditioned by pressure, is no longer in direct proportion to the same. Instead, grout spread depends on pumping rate and duration of the current grouting operation – i.e. to the *injected volume of grout per round*.

5.1 Response of sub-ground to Hydraulic Fracturing – confined conditions – ‘action planes’

As opposed to penetration and impregnation grouting, the hydraulic fracturing technique implies subjecting the ground to appreciable stress and deformation. By *static necessity*, the application of high hydraulic pressure in a bore hole initially induces radial compression and tangential tension in the form of principal stresses. The additional radial compressive stresses and tangential tensile stresses being numerically equal, a state of pure shear is induced around the hole. The grouting induced stresses are superimposed on the existing in situ states of stress, see Figure 5.1.

Formation of ‘action planes’

However, the moment grout - even to a minor extent - has spread into the widest or *most accessible* crack, the expanding pressurised area in this crack (soon forming an immense jacking force) generates a radically **changing regime** of stress and deformation.

As a result of pressure-induced elastic deformation and closure of possible adjacent open cracks, the dominating active crack widens, and may propagate progressively far into the surrounding rock mass forming a dominant ‘*action plane*’. Hence, the state of stress may be fundamentally altered at considerable distance from an injection hole.
For example, if grout has spread r = 5 m in all directions from the TaM packers in a bore hole, at say a mean pressure of p =10 bar = 1 MPa, the ‘jacking force’ may be in the order of $F_j \approx p\pi r^2 \approx$ 75 000 kN. The loaded area would then importantly affect stress and deformation in the rock mass up to some 30 to 40 meters away.

The dimensions of *action planes* depend mainly on the quantity of *grout injected per round, grouting pressure, rock stiffness, depth and length of the individual injection stages* as well as on the *structural characteristics* of the formation. Hence, the spread of grout may vary widely depending on prevailing conditions from a few meters to several tens of meters – in fact spread of grout amounting to several hundred meters have been recorded now and again.

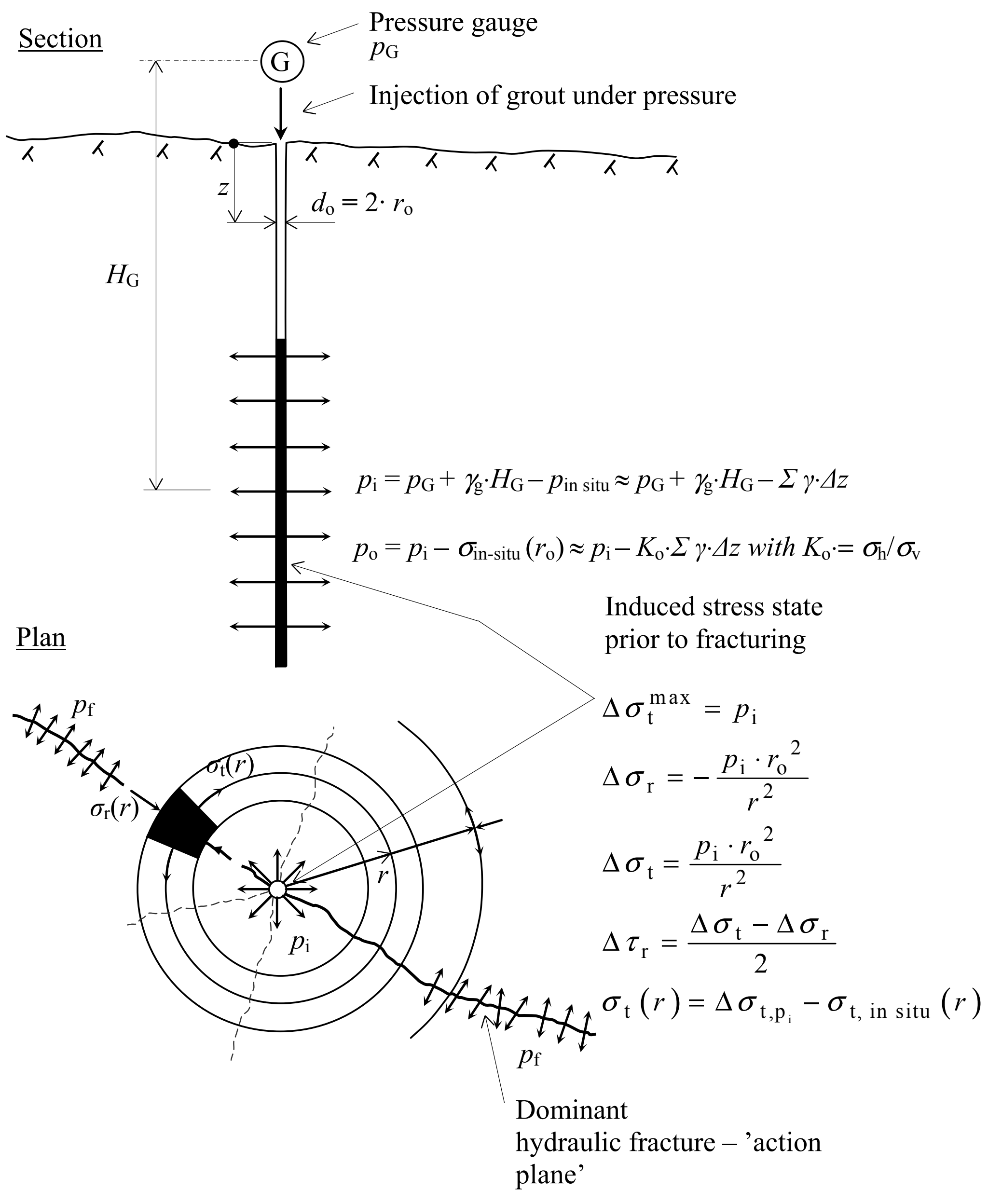


Figure 5.1 Initial states of stress – formation of hydraulic fracture – development of grout spread in an action plane. Note that the stress induced by the active injection pressure p_i has to exceed the prevailing local stress $\sigma_{in\ situ}(r_o)$ before any deformation takes place. Hence, the overpressure p_o is defined as $p_o = p_i - \sigma_{in\ situ}(r_o)$, where $p_i = p_G + \gamma_g \cdot H_G - p_{in\ situ}$ and where $p_{in\ situ} \approx \Sigma\gamma\Delta z$ is the pressure required to restore the original state of stress around the bore hole, i.e. to what it was before any hole was drilled, see Section 5.1.2 below.

On grounds given previously in Section 3.2, fracturing and widening of cracks – especially in softer or densely fissured rock– initially tend to manifest themselves in vertical or sub-vertical planes, generating horizontal thrust and displacements. Cf also Cambefort (1977). This is the reason for the *absence of ground heave* in the initial stages of high pressure grouting, which is a frequently observed phenomenon. Then, as grouting proceeds the horizontal stresses gradually increase, and when they eventually exceed the vertical stresses from overburden, by some measure horizontal fracturing and ground heave are likely to intervene, explaining why heave of ground tends to occur when grouting has been going on for some time.

Quotation from Klaus Back's edition of Cambefort (1969) (from German):
"The first hydraulic fractures always occur in radial vertical planes through the sediments as well as in weak rock. Accordingly, P. Lévêque (1954) injected a marl sandstone formation at a depth of a few meters with a pressure of 100 bar without heaving the ground - apparently because of fracturing in vertical planes."
(Confer also the Bagers Plats grouting trials, Section 5.4.4.3)

The likelihood of *vertical* hydraulic fracturing is enhanced by *long stage* TaM or end of casing (EOC) grouting, as the injection pressure then forms an *effective line load* inducing additional concentric states of horizontal tensile stress. The corresponding probability of *horizontal* fracturing is favoured by concentrated very short stage TaM grouting or by end of casing grouting (EOC), where the casing extends right down to the bottom of the bore hole. (Cf Section 3.2.1 and the Västra station trials in 5.4.3)

Configuration of action planes

When, as mentioned, the grout under pressure spreads in vertical *action planes* 'preferred' by the advancing grout (the jacking effect), the horizontal thrust increases dramatically, thus expanding active cracks by *elastic deformation* of the rock structure. Also, widening of the active crack may take place partly at the expense of the width and permeability of neighbouring crack systems.
Horizontal fracturing may result in both elastic deformation and heave. The width of an active crack will tend to grow progressively with increasing spread of grout, which in turn results in failure propagation and further widening of the dominant active crack.

Moreover, the very fact that the initial width of a crack is likely to increase many times over by the penetrating grout greatly enhances the prospects of further grout propagation. This allows, for instance, unstable grouts with poor penetrability to reach far from the point of injection.

In permeation grouting, the spread of grout is often described as being *dendritic*, i.e. as a tree-shaped branching propagation system. From the above it is evident that, in principle, grout spread in hydraulic fracturing is *not likely* to be dendritical. The author has observed this phenomenon several times in excavations, where the ground had previously been subjected to grouting treatment.

With stable grouts, the process often causes resurgence at the bedrock surface or develops progressively into a widespread *slow* deformation controlled dynamic failure, extending far away from the bore hole. Using stable grout and sufficiently high injection pressure, the action radius is virtually *unlimited*, especially if Newtonian fluid properties are assumed. In fact, the ***only way*** to confine grout to the ***intended area of treatment****,* and to gain ***guaranteed control*** of *undesired spread* of grout to the environment, is ***limiting the volume of grout injected in each round or pass*****!** By contrast, limiting the pressure is *not a reliable criterion* for controlling grout spread. (Regarding Bingham behaviour confer Section 5.1.4.3).

The formation of large action planes greatly enhances the possibility of reaching isolated cracks and other water-conductive features in the ground structure. Hence, in view of the inability of cement-based grouts to penetrate into *fine particle soils*, *rock material* and *fine crack systems in rock,* hydraulic fracturing constitutes the most ***viable technique*** for reducing the transmissivity in most types of ground.

In this context, the following quotation from the works of the illustrious pioneer and expert in the art of grouting - Henri Cambefort (1969) - may be appropriate (in translation from French):

"Thus injection is only possible to achieve if the ground is subjected to deformation, and if one tries to reduce, or even eliminate deformations, for instance by a reduction of the grouting pressure, only a very unsatisfactory treatment can be accomplished."

Further it should be emphasised here that grouting by hydraulic fracturing – like permeation grouting - is a *probabilistic approach* requiring repeated injection stages. Permeable features, which are not sealed in the first 'try' may well be blocked by the second or third round. Practical experience shows clearly that satisfactory closure of a formation is rarely achieved by a single injection, but develops gradually as the number of grouting passes increase. This phenomenon applies particularly to soils and sedimentary formations of the types encountered in, for instance, the Malmö area.

5.1.1 'Claquage' – fracturing close to the bore-hole

When injecting at high pressure, the material close to the injection hole may obviously fail in tension. The tangential tensile stress ($\Delta\sigma$) induced by a pressure (p) in a bore hole is according to Timoshenko & Goodier (1970):

$$\Delta\sigma_{\mathrm{t}} = \frac{p \cdot \left(\frac{r_{\mathrm{o}}^2}{r^2} + \frac{r_{\mathrm{o}}^2}{R^2} \right)}{\left(1 - \frac{r_{\mathrm{o}}^2}{R^2} \right)}$$

The pressure required to initiate tensile fracture is often denoted as the "claquage" pressure. The tangential tensile stress immediately at the wall of the bore hole where $r = r_{\mathrm{o}}$ then amounts to:

$$\Delta\sigma_{\mathrm{c,t}} = \frac{p_{\mathrm{c}} \cdot \left(1 + \frac{r_{\mathrm{o}}^2}{R^2} \right)}{\left(1 - \frac{r_{\mathrm{o}}^2}{R^2} \right)} \qquad (5.1)$$

where p_{c} is the active 'claquage' pressure in the bore hole, r_{o} is the radius of bore hole and R denotes the radial extension of the formation. For values of $R >> r_{\mathrm{o}}$, $(\Delta\sigma_{c,\mathrm{t}})_{\mathrm{r=ro}} \approx p_{\mathrm{c}}$.

The claquage condition may then be expressed as:

$$\sigma_{\mathrm{t}} = \Delta\sigma_{\mathrm{c,\,r=r_o}} - \sigma_{\text{in-situ}} = p_{\mathrm{c}} - \sigma_{\text{in-situ}} \geq f_{\mathrm{t,\,rock}}$$

where

p_{c} is the active claquage pressure. (Cf definition of p_{i} in Sect. 5.1.2. below) $\sigma_{\text{in situ}}$ is the ambient compressive stress due to overburden in the considered location (in this case the wall of the bore hole) as defined below in Section 5.1.2.

$f_{\mathrm{t,\,rock}}$ is the tensile resistance of the rock material.

Hence, with a pressure of for instance p_{c} = 20 bar, the induced tensile stress is almost 2 MN/m^2, i.e. a level of stress, which – if $\sigma_{\text{in situ}}$ is small - few sedimentary rocks will resist in tension even in the absence of weak joints. (Cf Figure 5.1 and 5.7)

5.1.2 Pressure induced deformations

The 'active injection pressure' p_i is here defined as the recorded gauge pressure p_G corrected for differences in hydraulic head and stress changes around the bore hole caused by the removal of material by the drilling operation.

Hence, $p_i = p_G + \gamma_g \cdot H_G - p_{in\ situ}$

where H_G is the vertical distance between the pressure gauge and the grouting stage, and $p_{in\ situ}$ is to be understood as *the pressure required to restore the original in situ state of ground stress* at the grouting stage level – i.e. the stress condition existing prior to the excavation of the hole for injection.
In principle $p_{in\ situ} \approx {}_1\Sigma^N \gamma_n \cdot \Delta H_n$.

In the subsequent analysis, it is important to observe that the pressure denoted as p_o is defined as the *overpressure,* i.e.

- in permeation grouting, the part of the injection pressure required to maintain penetration flow

  and

- in case of hydraulic fracturing, the part of the 'active injection pressure' p_i, that exceeds the prevailing in situ stress $\sigma_{in\ situ}$, and which will generate tension in a studied fracture plane.

Furthermore, as dissipation of excess pore water pressures is not likely to take place within the duration of one injection round, computations are carried out in terms of *total stresses.* Effective stress analysis would presume significant water transport through the formation in a short time span and cannot be depended upon.

Hence the overpressure in terms of total stress is

$$p_o = p_i - \sigma_{in\ situ}$$

At the wall of a vertical bore hole $\sigma_{in\ situ}(r_o) \approx K_o \cdot {}_1\Sigma^N \gamma_n \cdot \Delta H_n$, where $K_o = \sigma_h / \sigma_v$
i.e. the ratio between horizontal and vertical principal stresses in the formation.
ΔH_n is the depth of an individual soil or rock layer.

To summarise:

$$p_o = p_i - \sigma_{in\ situ} = p_G + \gamma_g \cdot H_G - p_{in\ situ} - \sigma_{in\ situ} \approx$$

$$p_G + \gamma_g \cdot H_G - (1+K_o) \cdot {}_1\Sigma^N \gamma_n \cdot \Delta H_n$$

The term $p_{in\ situ}$ adjusts the pressure to its original value before drilling the hole and the term $\sigma_{in\ situ}$ represents the pressure required to level out prevailing compressive stresses in the ground.

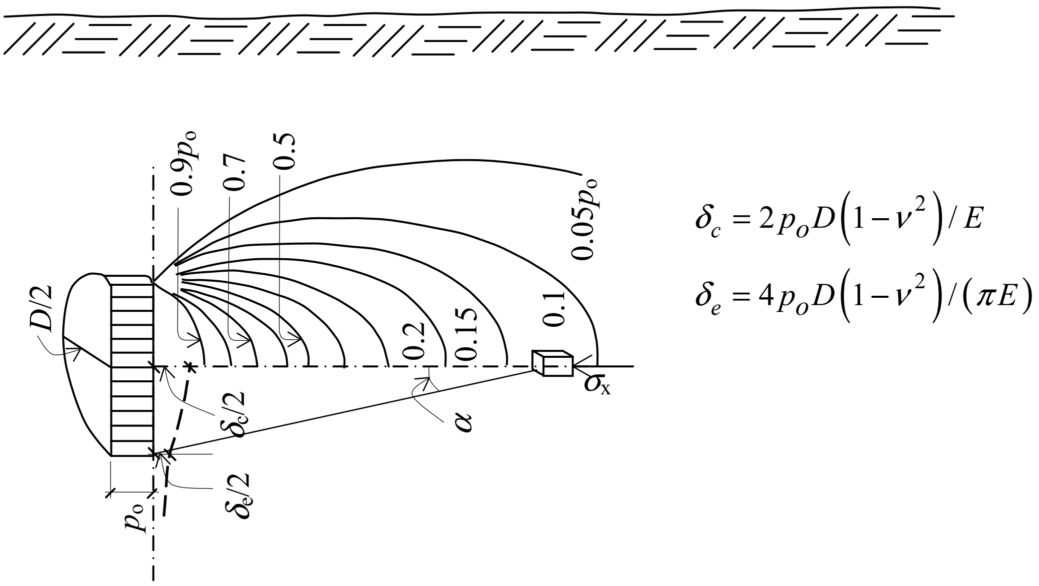


Figure 5.2 Approximate assessment of induced stress and deformation in rock due to mean grouting pressure in an action plane. (Modified application by the author of a diagram originally derived for a load on the surface of an elastic half-space according to Boussinesq (1885), (Cf Sahlberg (1961)).

The diagram in Figure 5.2 shows an approach to estimating the *order of magnitude* of stresses and displacements. Modifying expressions according to Boussinesq (1885), valid for a circular load on the surface of an elastic half-space we get:

Width of gap at the centre of the pressure plane:

$$\delta_C = 2p_o \cdot D \cdot (1-\nu^2)/E \tag{5.2}$$

Width of gap at the edge of the pressure plane:

$$\delta_C = 4p_o \cdot D \cdot (1-\nu^2)/(\pi \cdot E) \tag{5.3}$$

For instance, the *widening* of an existing crack in a rock formation with an E-modulus of 4000 MN/m^2, ν = 0.15 and with a diameter $D = D_S$ of the grout spread of 10 m, will amount to some 5 mm, at a *mean* pressure of 10 bar (= 1.0 MN/m^2).

Applying instead the expression in Figure 5.5c (or Equation 5.7a), valid for conical pressure distribution, and using a *peak* overpressure of p_o = 25 bar at the injection hole, the corresponding displacement will be almost the same i.e. 4.5 mm.

The example clearly illustrates that the forced displacement induced in rock, as well as in ground of soil, constitutes a ***determining factor*** for the propagation of grout during the injection process. In the choice of grouting strategies one should, therefore, not ignore the fact that a crack, having ***an initial width*** of say 0.5 mm, may ***widen tenfold*** to some 5 mm during the grouting operation.

***Conclusion*:** Strain and deformation in the rock mass must *not* be overlooked or disregarded in the planning of grouting operations or when interpreting the results thereof.

It is often maintained that high pressure grouting may be harmful to the closure of a structure because of the formation of new fractures. For comments on this issue see Section 3.2.3 above, and Section 5.3 below, where among other the GIN method is briefly commented upon.

5.1.3 Pressure distribution in the jacking plane

The pressure distribution in the action plane is difficult to define or predict accurately by computational analysis. Therefore, in order to be able to make at least approximate assessments of the magnitude and effects of pressure induced displacements some reasonable assumptions regarding the pressure distribution in the jacking plane have to be made.

*Flow conditions at the **termination** of a grouting step*

A cardinal issue when considering the relevant pressure distribution for estimating the volume of grout that can be accommodated in an action plane are the flow conditions at the termination of the grouting round. Obviously, if the grout flow at ***grouting stop*** is ***insignificant*** or close to zero, then the pressure is likely to be more or less constant over a major part of the action plane on account of small pressure losses due to flow friction. In such a case, the deformations may be estimated on the basis of the data given in Figure 5.2

By contrast, if the grouting operation is terminated at a ***pre-set grout take,*** then frictional flow losses are to be considered. Under such conditions, the pressure at grout stop will abate with increasing distance from the injection hole.

When considering pressure distribution in this context, the length of the grouting stage is, as demonstrated in the following, also of major importance.

Conclusions: When considering the grout pressure distribution for estimating the grout volume consumed in an action plane, the *grout stop criteria* and the *length of the grouting stage* are factors of decisive importance. E.g. stopping at *insignifi-*

cant flow at a *pre-set pressure* gives an entirely different grout pressure distribution than stopping at a *pre-set grout volume* with *no pressure limit*.

5.1.4 Assessment of grout spread and displacements

In the course of the author's involvement as an outside adviser in the City Tunnel Project in Malmö, (Bernander (2000, 2002, 2003)), a special assessment of the magnitude of the ground deformations generated by hydraulic fracturing and the associated grout spread were carried out.

The analyses described below are based on the concept that ***maximum grout spread*** can be determined assuming that all of the injected grout *in one injection round* corresponds to the volume of the slit-shaped gap generated by the pressurised grout in the current action plane.

This assumption obviously disregards the grout take required to fill possible initially open cracks, cavities or 'karst' formations but, as may be concluded from Figure 4.3, even very pervious rock structures exhibit remarkably small void ratios.

In any case, if open cavities – not relating to transmissivity - do exist they are likely to be filled in the initial stages of the grouting process. Furthermore, such initial voids will only reduce the real action range of an injection round as compared to the calculated value - i.e. a factor on the safe side in respect of environmental damage.

5.1.4.1 Assumptions regarding pressure distribution in the jacking plane

<u>Concentric 'short stage' loading</u>

Extensive side analyses made by the author assuming varying degrees of tapering crack width in the action plane indicate curves of the type given in Figure 5.3a for *concentric loading.*

As stated above the question of defining the pressure distribution in the gap generated by the hydraulic fracture presents some difficulties. Nevertheless, the overpressure p_0 in the bore hole is known, while the pressure at a Point E, defining the *far end* of the hydraulic fracture gap must, at least be close to zero, see Figure 5.3a. This follows from the fact that, if the grout pressure were significant at E, then the deformation at this location cannot possibly be nil.

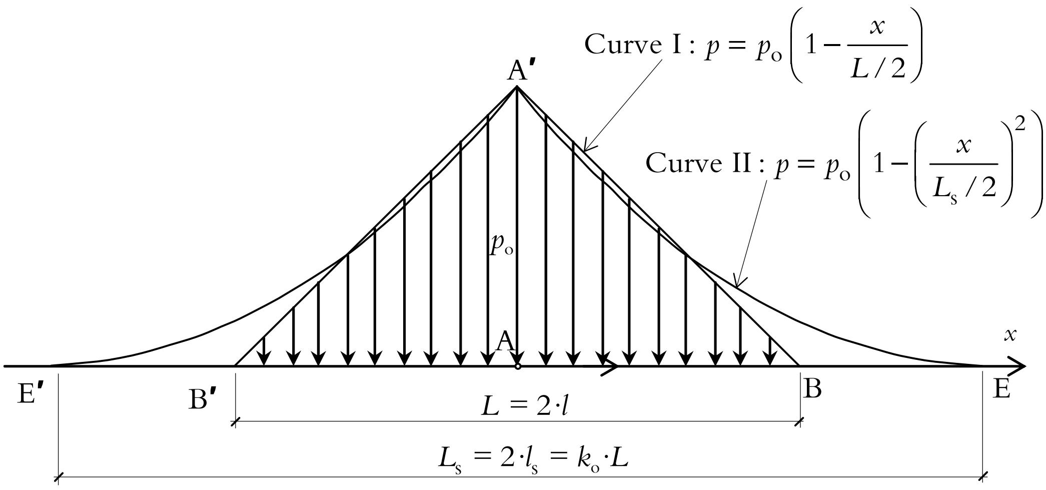


Figure 5.3a Assumed pressure distributions in the jacking plane for conical concentric loading. L_S is the width of grout spread zone and equal to $k_o\cdot L$. L is the width of the nominally loaded area (B'B), on which the mathematical analysis of deformations is based. The overpressure is defined as $p_o = p_i - \sigma_{in\ situ}(r_o) \approx p_i - K_o\cdot\sum\gamma\cdot\Delta z$, where $K_o = \sigma_h/\sigma_v$. Note: If linear distribution from A to E is assumed, then k_o in the applicable equations approaches 1.0 i.e. $L_S \approx L$.

A reasonable assumption, therefore, is that the pressure in the tapering slit abates parabolically between the points A and E, e.g. according to Curve II (AE) in Figure 5.3a.

However, in order to simplify the mathematical analysis, a linear pressure distribution as per Curve I (AB) is instead used as input for the computation of deformations in the fracture plane. The point B is then selected in such a way that the displacement generated by the 'nominal' linear load as per Curve I (AB) is *zero* at Point *E*, i.e. $\delta_E = 0$.

Since the displacements resulting from the two loading alternatives (i.e. according to Curves I and II) cannot differ significantly in magnitude, it follows that the extension of the grout filled fracture gap is roughly compatible with the actual extension of the 'real' parabolic load.

<u>Long stage strip loading</u>

For extended strip loading - applicable for instance to long stage EOC grouting - the *pressure distribution under flow* is likely to be more linear or in a strongly tapering slot even somewhat convex. In such a case, the Point B in Figure 5.3a approaches Point E, rendering a value of k_o in the applicable equations close to 1.

5.1.4.2 Deformation analysis

As already mentioned above in Section 5.1.2 p_o denotes the *overpressure*, i.e. the active injection pressure p_i minus the stress $\sigma_{\text{in situ}}(r_o)$ required for balancing the existing local stresses due to overburden.

Furthermore as previously stated, $p_{\text{in situ}} = {}_1\Sigma^N \gamma_n \cdot \Delta H_n$ signifies in the current context the pressure required for compensating the change of stress in the ground caused by the installation of the drill hole.
At the bore hole wall, the value of $\sigma_{\text{in situ}}(r_o)$ is approximately equal to $K_o \cdot {}_1\Sigma^N \gamma_n \cdot \Delta H_n$ where $K_o = \sigma_h/\sigma_v$.

Thus in terms of the gauge pressure p_G:

$$p_o = p_i - \sigma_{\text{in situ}}(r_o) = p_G + \gamma_g \cdot H_G - p_{\text{in situ}} - \sigma_{\text{in situ}} \approx$$
$$p_G + \gamma_g \cdot H_G - (1+K_o) \cdot {}_1\Sigma^N \gamma_n \cdot \Delta H_n$$

Note: It may be observed in this context that when a pre-set grout volume per round is used as the grouting stop criterion, the pressures registered at the *attainment* of the *pre-set grout take* may be regarded as relevant to the analysis.

In the investigation, two types of linear pressure distribution were analysed:

a) Triangular strip load compatible with 'long stage' injections – such as, for instance, long stage end of casing (EOC) grouting.

b) Cone shaped load distribution applying to injections with little distance between packers, as is often the case in the 'Tube à Manchette' (TaM) grouting method.

The closed form expressions for the deformations in the rock mass have been based on the known differential equations for a concentrated line load acting on the surface of a 2-dimensional *elastic half space*, Timoshenko & Goodier (1970). However, as solutions for neither a single triangular load distribution nor a symmetric one, are given, the author of this report has derived new relationships valid for both triangular and conical loading. It may be noted that the expressions for the triangular strip loading in Figure 5.4 are *strictly accurate* in relation to the assumptions made, whereas the equations for conical load distribution according to Figure 5.5 represent an approximate but for all practical purposes sufficiently accurate approach, compare with Bernander (2001).

The analysis thus presumes that the nominally assumed grout pressure abates linearly with the radius (or distance) from the point of pressure application. This is evidently on the safe side in respect of the spread of grout as regards concentrated TaM grouting, because the pressure volume then actually forms more of a truncated cone rather than the pointed cone, on which the analysis is based. Such a load configuration would generate more accommodation by displacement for the pre-set volume of grout to be injected resulting in less computed spread.

As for long stage '2-Dimensional' grouting (e.g. EOC), the assumption may be reasonably accurate for a Newtonian fluid under flow conditions. However, by varying the value of k_o, (as defined in Figure 5.3a), different assumptions with regard to pressure distribution can be accommodated. For linear distribution between the points A and E (Figure 5.3a) $k_o \approx 1$.

Results of the analysis

Results of the analyses are presented in Figures 5.4 and 5.5, where formulae and diagrams show the likely extensions of grout spread for given volumes of injected grout per round for certain values of k_o. Figure 5.4c and 5.5c indicate the corresponding maximum displacements induced by the applied grouting pressure under homogeneous conditions for $k_o = 1.2$ and $k_o = 1.646$ respectively.

For constant circular load according to Figure 5.2 the equations 5.10 and 5.11 apply.

<u>Triangular pressure distribution</u>

We start with an expression derived by Timoshenko & Goodier (1970), based on the works of Boussinesq (1885). Accordingly, the vertical deformation $d\delta$ of the surface of an elastic isotropic half-space at the distance r from a vertical line load pdr may be written, see Figure 5.3b

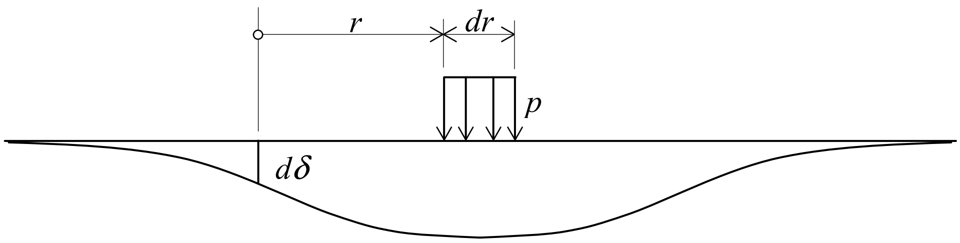


Figure 5.3b Vertical displacement $d\delta$ of the surface of an elastic isotropic half-space at a distance r from the vertical line load pdr. At distance d in the direction of load action the displacements are assumed to be negligible.

$$d\delta = \frac{pdr}{\pi E}\left[2\ln\frac{d}{r} - (1+\nu)\right]$$

Hence, in the derivation of the formulae, an infinite 2-dimensional half space, having a modulus of elasticity, E, and Poisson´s ratio, ν, is considered. The integration constants are, following Timoshenko & Goodier, determined by three boundary conditions:

a) Horizontal displacement along the line of symmetry is zero.

b) Rotation along the line of symmetry is zero.

c) Radial displacement is negligible at a distance d in the direction of load action.

Comprehensive computations, given in detail in Bernander (2002), show that the surface deformation δ_A at the centre of a double-symmetric *triangular load* with a peak intensity of p_o and extending over a length of $L = 2l$ can be written as

$$\delta_A = \frac{2p_0\ell}{\pi E}(2\ln\frac{d}{\ell} + 2 - \nu)$$

Inserting $d/l = 6$ and $\nu = 0.15$ gives $\delta_A = 5.4335 p_o 2l/\pi E = 1.73 p_o L/E$

The deformations can also be expressed as functions of the real grout spread L_S, as shown in Figures 5.3a & 5.4:

$L_S = k_o \cdot L = k_o \cdot 2l$, where k_o is defined in Figure 5.3

$L = 2l$ = Width of the 'nominally' loaded area = L_S/k_o

According to the above mentioned computations, the gap width at centre of the pressure plane is

$$\delta_A = \frac{p_o \cdot L_s}{k_o \cdot \pi \cdot E}\left(2\ln\frac{2d}{L} + 2 - \nu\right) \tag{5.4}$$

Gap width at the edge of the pressure plane:

$$\delta_B = \frac{p_o \cdot L_s}{k_o \cdot \pi \cdot E}\left(4\ln\frac{d}{L} - 2\ln\frac{2d}{L} + 2 - \nu\right)$$

Gap width at $x = L/4$ in the pressure plane:

$$\delta_C = \frac{p_o \cdot L_s}{k_o \cdot \pi \cdot E}\left(\frac{9}{4}\ln\frac{4d}{3L} - \frac{1}{4}\ln\frac{4d}{L} + 2 - \nu\right)$$

Width of grout spread zone:

$$L_S = 1.319\sqrt{\frac{k_o \pi E}{p_o} \cdot \frac{V/h}{2\ln\frac{2d}{L} + 2 - \nu}} \tag{5.5}$$

Putting for instance: $d = 3.0 \cdot L$, $L_S = k_o \cdot L = 1.20 \cdot L$ and $\nu = 0.15$, then:

$\delta_A = 4.53 \cdot p_o \cdot L_S / \pi E$

$\delta_C = 3.62 \cdot p_o \cdot L_S / \pi E$

$\delta_B = 2.22 \cdot p_o \cdot L_S / \pi E$

In Figure 5.4c:

$$\delta_A = 1.44 \cdot p_o \cdot L_S / E \tag{5.4a}$$

In Figure 5.4b:

$$L_S = 1.100 \cdot [E \cdot (V/h)/p_o]^{1/2} \text{ i.e.} \tag{5.5a}$$

$$V/h = 0.828 \cdot L_S{}^2 \cdot p_o / E \tag{5.6}$$

Here p_o denotes the injection overpressure in the bore hole and E denotes the mean modulus of elasticity of the ground. d designates a distance perpendicular to the 'action plane' from centre of load application to a point, where the displacement is considered negligible. ν = Poissons ratio. V/h = injected volume per meter bore hole (m^3/m).

<u>Conical distribution of pressure</u>

Assessment of induced deformation and grout spread D_S in an elastic half-space for *conical distribution of grouting pressure* in an action plane gives:

$$\delta_A = \left(\frac{2 \cdot p_o \left(1 - \nu^2\right) D_s}{k_o \cdot E}\right) \cdot \left(\frac{\ln\frac{2d}{D_o} + 1 - \frac{\nu}{2}}{2\ln\frac{2d}{D_o} + 1 - \nu}\right) \tag{5.7}$$

$$D_s = 1.0744 \cdot \sqrt[3]{\left(\frac{4 \cdot E \cdot V \cdot k_o}{\pi \cdot p_o \left(1 - \nu^2\right)}\right) \cdot \left(\frac{2\ln\frac{2d}{D_o} + 1 - \nu}{\ln\frac{2d}{D_o} + 1 - \frac{\nu}{2}}\right)} \tag{5.8}$$

where δ_A is the gap width at the centre of the pressure plane. V = injected volume of grout per round, m^3. p_o is the injection overpressure in the bore hole and E denotes the mean modulus of elasticity of the ground formation. Again, d designates the distance from centre of load application to a point, where displacement may be considered negligible. ν = Poissons ratio, see Figure 5.5.

D_S = Diameter of grout spread zone = k_oD_o, where $D_o = D_S/k_o$ = Diameter of the 'nominally' loaded area. k_o is defined in Figure 5.3.

For $d = 3.0 \cdot D$, $k_o = 1.646$ and $\nu = 0.15$

$$\delta_A = 0.728 p_o D_S / E \qquad (5.7a)$$

$$D_S = 1.62 \cdot [V \cdot E / p_o(1 - \nu^2)]^{1/3} \qquad (5.8a)$$

$$V = 0.230\, D_S^3\, p_o / E \qquad (5.9)$$

<u>Even concentric pressure distribution – constant circular load</u>

As mentioned in Section 5.1.3, the *pressure at grout stop* under *insignificant flow* is likely to be constant in a major portion of the action plane. Basing the displacement for constant load on the expressions given in Figure 5.2, the following relationships between grout spread and grout take have been derived.

Thus, approximating the deflection curve between the displacements defined at the centre and at the periphery of the circular load in the figure as a parabolic function, the following relationships between grout spread and grout take are valid:

$$V = \frac{2.57 p_o D^3 \left(1-\nu^2\right)}{E} \text{ or inversely } D = 0.73 \sqrt[3]{\frac{EV}{p_o\left(1-\nu^2\right)}} \qquad (5.10)$$

Maximum displacement as a function of injected grout volume is then

$$\delta_c = \frac{0.78V}{D^2} \qquad (5.11)$$

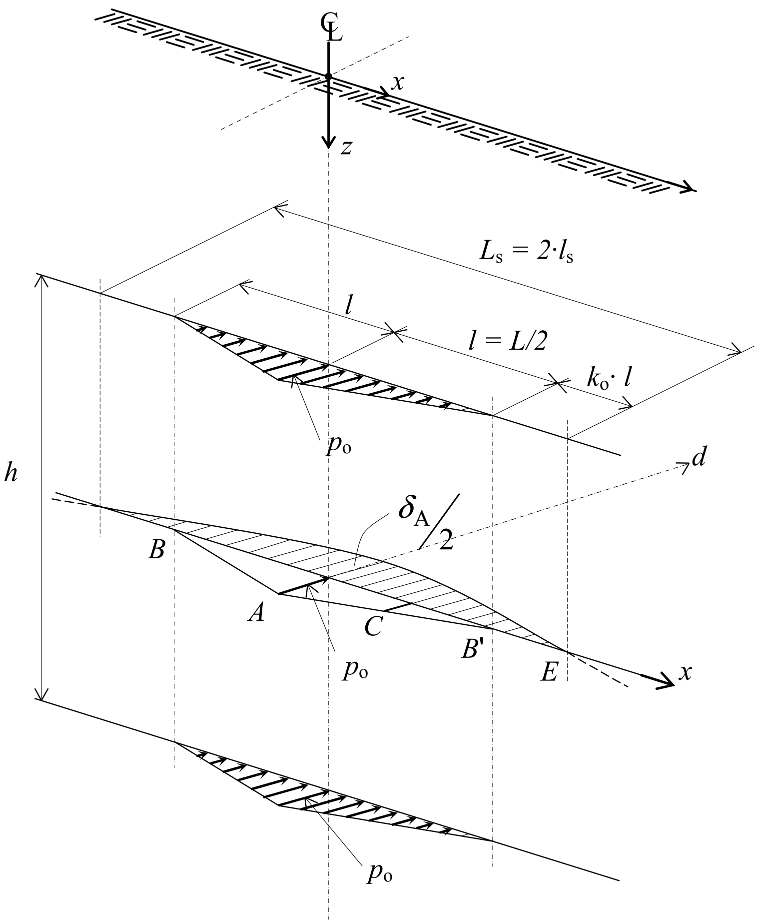
It should be observed that the analyses performed address hydraulic fracturing or opening of existing fissures in vertical or sub-vertical planes. However, although the fracture planes are assumed to be evenly planar, they may – for large action planes - still considered to be valid for irregular and jagged fracture planes adapting locally to existing fissure patterns. The occurrence of vertical crack opening or fracturing is sometimes an issue under discussion among practising engineers. Nevertheless, theoretical prediction and practical evidence in the field of the incidence of vertical fracturing is well established. (Cf e.g. Cambefort (1969, 1977) and Section 3.2.1).

However, in view of the erratic nature of ground structures, it is *not a primary objective* of this section to cover all types of conceivable loading situations induced by grouting pressure or even to claim high precision of the analysis made.

The aim of the analysis is to demonstrate *the order of magnitude of pressure-induced deformations in rock* and *how these affect the sub-ground subject to treatment*. Another aim is to demonstrate that *reasonable assessments of grout take and spread of grout can be performed,* and that predictions can be made with a degree of reliability commensurate with those of many standard evaluation methods in the field of soil mechanics.

5.1.4.3 The Bingham effect in the context of hydraulic fracturing

It can readily be shown that the Bingham effect is normally insignificant under hydraulic fracturing conditions owing partly to the high pressure gradients at stake and partly to the radically increased width of expanded cracks, see Figure 5.8. Assume for example that the flow gap is 1 mm thick and 5 m long with a pressure difference of 20 bar. For a normal grout (τ_o= 5 N/m^2), the coefficient ε_o= $2\tau L/(t\Delta p)$ would then be in the order of 0.025 giving a Bingham effect of reduced flow-rate by a factor of f = (1- (4/3)$\cdot\varepsilon_o+\varepsilon_o^4$/3) = 0.967, i.e. about 3.5 %. Cf. Håkansson (1993).



Maximum grout spread L_s under 2-dimensional conditions as a function of injected volume of grout per pass $v = V/h$ (m^3/m). Range of action $l_s = L_s / 2$

Triangular strip loading

$$L_s = 1.319\sqrt{\frac{\pi \cdot E \cdot k_o}{p_o} \cdot \frac{V/h}{\left(2\ln\left(\frac{2d}{L}\right) + 2 - \nu\right)}} \quad \text{[m]} \quad (5.5)$$

Maximum gap δ_A (m) due to pressure Induced deformation as a function of the width L_s (m) of grout spread

$$\delta_A = \frac{p_o \cdot L_s}{\pi \cdot E \cdot k_o}\left[2 \cdot \ln\frac{2 \cdot d \cdot k_o}{L_s} + 2 - \nu\right] \quad (5.4)$$

where:

E = Mean elastic rock mass modulus [Pa]

L = Width of loaded strip [m]

$L_s = 2l_s = k_o \cdot L$ = Width of grouted zone [m]

V/h = Injected volume [m^3/m/round]

p_o = Applied maximum injection overpressure [Pa]

ν = Poissons ratio

d = Distance from centre of bore hole to where deflection is negligible [m]

Figure 5.4a Assessment of grout spread, L_S, and maximum crack gap width, δ_A, for a triangular distribution of grouting pressure (2-dimensional load).

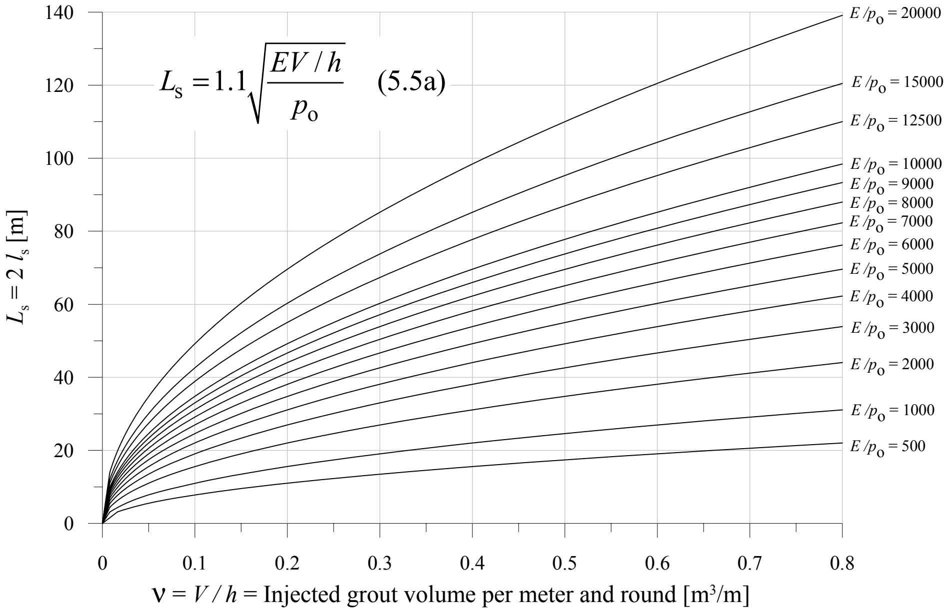


Figure 5.4b Grout spread L for two-dimensional triangular strip load. The diagram is valid for d = 3.0·L and ν = 0.15, $L_S = k_o{\cdot}L = 1.20{\cdot}L$. d = distance to where deformation is negligible.

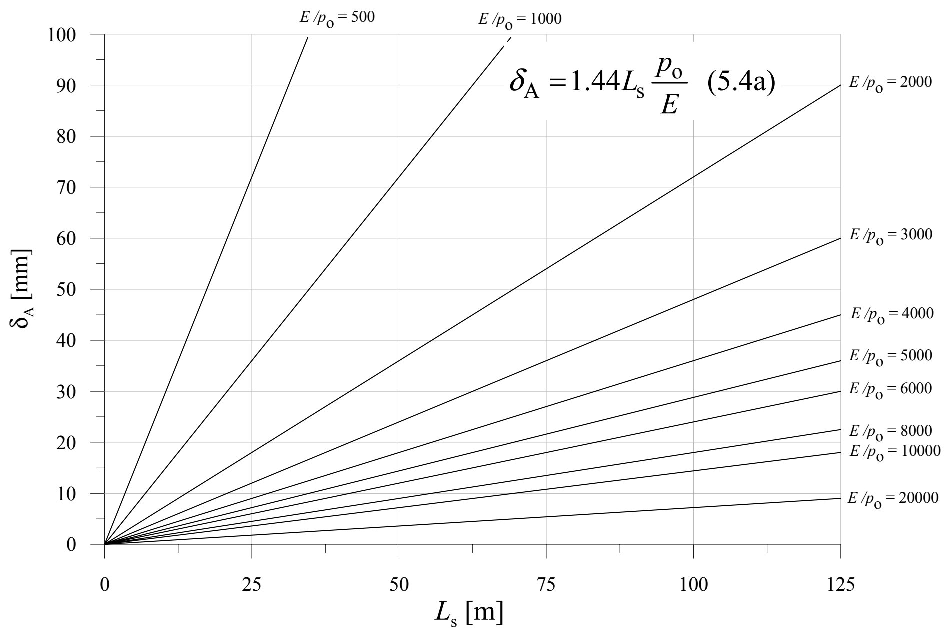
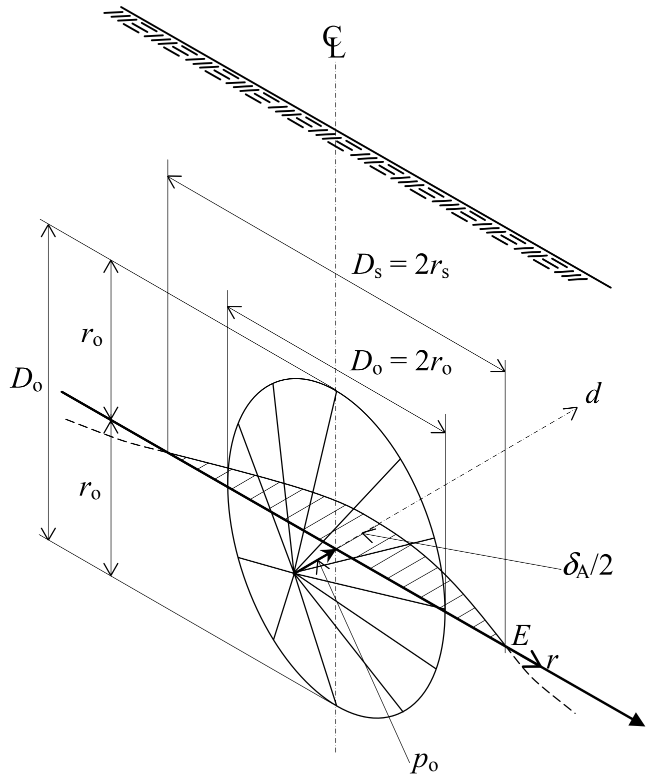


Figure 5.4c Maximum crack gap width δ_A for two-dimensional triangular strip load. The diagram is valid for d = 3.0L and ν = 0.15, $L = L_S/k_o = L_S/1.20$.



Maximum diameter of grout spread D_s as a function of injected volume of grout V (m3/ round).

$$D_s = 1.074 \cdot \sqrt[3]{\frac{4 \cdot k_o \cdot E \cdot V \cdot \left(2\ln\frac{2 \cdot d}{D_o} + 1 - \nu\right)}{\pi \cdot p_o \cdot \left(1-\nu^2\right)\left(\ln\frac{2 \cdot d}{D_o} + 1 - \frac{\nu}{2}\right)}} \quad (5.8)$$

$$D_s = 1.62 \cdot \sqrt[3]{\frac{E \cdot V}{p_o \cdot \left(1-\nu^2\right)}} \quad (5.8a)$$

Maximum pressure induced deformation δ_A [mm] as a function of the diameter D_s [m] of grout spread

$$\delta_A = \frac{2 \cdot p_o \cdot D_s \left(1-\nu^2\right)\left(\ln\frac{2 \cdot d}{D_o} + 1 - \frac{\nu}{2}\right)}{E_o \cdot k_o \left(2\ln\frac{2 \cdot d}{D_o} + 1 - \nu\right)} \quad (5.7)$$

where:

V = Injected grout volume [m^3/round]

p_o = Max. injection overpressure

E = Mean modulus of elasticity of rock formation

d = distance to where deformation is negligible

ν = Poissons ratio

$k_o = D_s / D$

Figure 5.5a Assessment of grout spread, D_S, and maximum crack gap width, δ_A, for a conical distribution of grouting pressure in an action plane.

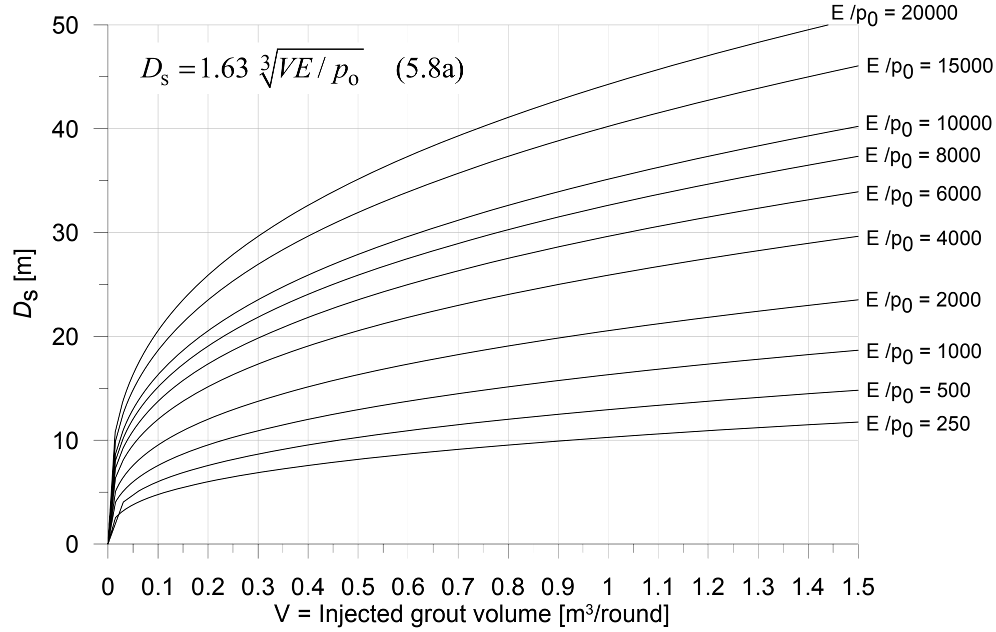


Figure 5.5b Grout spread D_S for conical load. The diagram presumes $d = 3.0{\cdot}D_o$, $\nu = 0.15$ and $k_o = D_S/D_o = 1.646$.

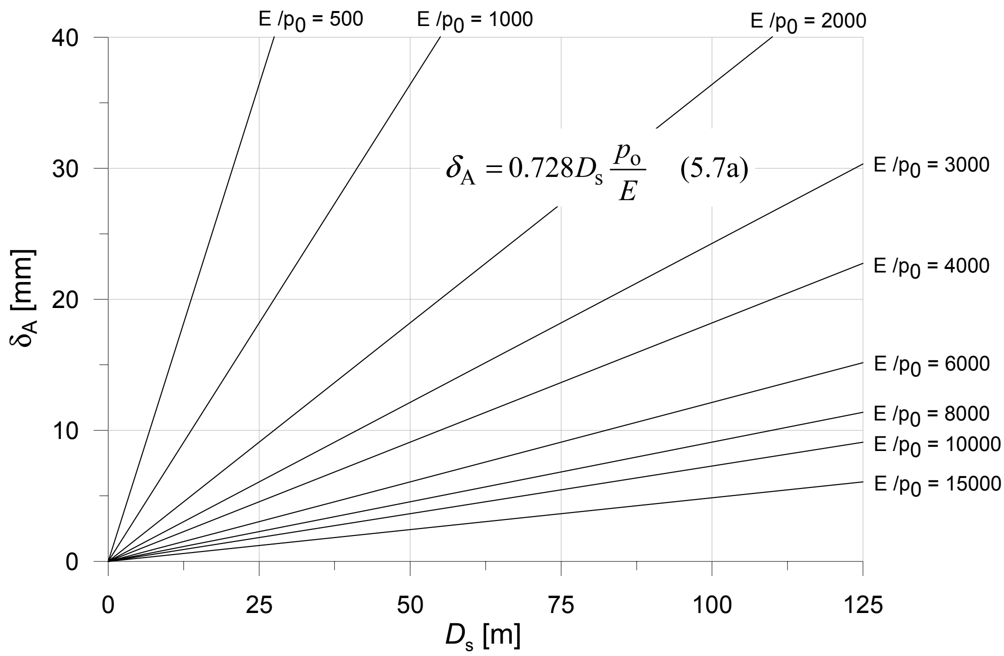


Figure 5.5c Maximum crack gap width δ_A for conical load. Diagram valid for $d = 3.0{\cdot}D_o$, $v = 0.15$ and $k_o = 1.646$.

5.1.5 Conclusions that may be drawn from the deformation analysis under confined conditions

(Cf diagrams in Figures 5.4 and 5.5 and Equations (5.4), (5.5), (5.7), (5.8), (5.10) and (5.11)).

1) As mentioned above, the analysis most importantly demonstrates the considerable magnitude of the deformations inevitably linked with the grouting pressures applied and the volumes of grout actually injected.

The calculated deformations demonstrate the ***dramatic impact*** of injection pressure on the grouting process and the outcome thereof – i.e. strongly confirming the observation of H. Cambefort quoted previously in Section 5.1.

Furthermore, as already pointed out, the deformations in the ground may expand the width of an existing crack many times over, effectively promoting further grout propagation. The requirements with regard to grout stability and its capacity to penetrate pores and fissures in the ground are therefore likely to be very different for hydraulic fracture grouting as compared to low pressure permeation grouting.

It is sometimes maintained that high pressure grouting may be harmful to the closure of a formation because of the formation of new fractures. However, this is hardly an inevitable result from structure-mechanical points of view, as repeated grouting tends to build up considerable horizontal pre-stress and elastic rebound in the ground, significantly increasing K_o values. (Cf. further discussion in Section 5.3 below.)

2) Hence, when applying hydraulic fracturing, the width of the zone affected by grouting in a single hole - deep down in a rock or soil formation - is mainly a function of the stiffness of the ground E, the injection overpressure p_o and the volume V of grout injected per round. The effect of the time factor is briefly touched upon in Item 4) below.

- In the case of ‘long stage’ EOC grouting, the width of the grouted zone L_S (i.e. two times the radius of action) is *in principle* proportional to the square root of the ratio of E-modulus to injection pressure times the injected volume/round i.e.

$$L_S = C_1 \sqrt{\frac{V}{h} \cdot \frac{E}{p_o}} \qquad (5.5b)$$

where, h denotes the length of the stage and V is the *totally* injected volume of grout/pass.

For instance, if the E-modulus of an igneous rock formation is 16 times that of a soft sedimentary structure, the *potential spread* of a defined volume of injected grout in the stiff rock will be *4 times* greater than in the sedimentary rock.

- In the case of more locally concentrated TaM grouting, the diameter of the grouted zone D_S (i.e. two times the radius of action) is, again *in principle,* proportional to the third root of the ratio of E-modulus to injection pressure times the injected volume/pass, i.e.

$$D_S = C_2 \cdot \sqrt[3]{\frac{VE}{p_o}} \tag{5.8b}$$

For constant concentric load according to Figure 5.2

$$D = C_3 \cdot \sqrt[3]{\frac{VE}{p_o \cdot \left(1-\nu^2\right)}} \tag{5.10a}$$

The values of the coefficients C_1, C_2 and C_3 depend essentially on load distribution, Poissons ratio and a distance d from the centre of loading to a point where the deformation can be considered to be negligible. (Cf Figures 5.4 and 5.5).
Hence, assuming for instance E = 2000 MN/m^2 for a sedimentary rock and p_o = 1 MN/m^2, then E/p_o = 2000. Grouting a volume of 100 litres/round at each of 5 TaM levels at 1 m on centres, (i.e. a total volume of 0.5 m^3), results in a **9.5** m wide treated zone according to Figure 5.5b (or Eq. (5.8a)). Figure 5.5c (or Eq. (5.7a)) then gives the corresponding displacement per round of about 3.5 mm, amounting in total to some 15 to 17 mm.

By contrast, if the *same amount* (0.5 m^3) were to be grouted in *one long* EOC stage, with V/h = 0.1 m^3/m, the width of the grouted zone would instead be **15.6** m according to Equ. (5.5a), i.e. a *dilution* of the grouting intensity by a ratio of about 1:2 (It may be noted in this context that *compaction* due to lateral displacement contributes to the *closure* of the target zone.). The peak displacement is now only ≈ 11 mm (Equ. 5.4a), which is appreciably less than the 15 – 17 mm in the previous case.
On the other hand, if only 25 litres/m of grout per round are injected, the corresponding grout spread will be **7.8** m (EOC) instead of 15.6 m.

Furthermore, it may be of interest to note that in a *stiffer rock formation* of igneous rock with, for instance, E = 30000 MN/m^2, the *equivalent spread* of grout would have been about $15.6\sqrt{30/2} = 60$ m as compared to 15.6 m.

The analysis thus underscores the fact that focusing of grouting treatment to a target area is best achieved by injecting ***smaller batches*** of grout ***in several rounds*** rather than injecting the same total volume in, for instance, one or two steps.

3) It should further be stressed in the current context that, when grouting according to the principles of hydraulic fracturing, *stop criteria* based solely on *pressure* are ***unsuitable*** for restricting the grout spread. Moreover, contrary to a general notion in the trade, *pressure as a stop criterion* is by no means a *valid or reliable measure of achieved tightness or closur*e in the area subject to treatment. This applies especially in the initial stages of grouting and when grouting under unconfined conditions.

4) In view of the fact that the applied so called 'claquage' pressure is mostly of a magnitude far in excess of that sufficient to generate tangential tensile fracture in the rock around a bore hole, hydraulic fracturing may occur irrespective of the strength characteristics of the ground. In fact, the practical purpose of applying claquage pressure is actually to create access from the bore hole to adjacent crack systems.

5) The relationships given above state that the decisive factors with regard to the deformations and the extension of the grouted zone by hydraulic fracturing are:

a) The ratio E/p_0 of the effective E-modulus of the formation to the injection pressure at grout volume stop;

This implies for instance (presuming stable grout) that the deformations and the grout spread may be the same in a *soft formation at low* injection pressure as in a *more rigid formation at higher* pressure.

b) The volume of injected grout per round – being the only parameter directly controlled by the operator - must always constitute ***the main stop criterion***. (The pressure required to inject a given quantity of grout can namely vary widely for a number of unpredictable reasons).

6) Interestingly, the analysis performed reveals that, if a *defined volume* of stable grout is injected at a lower pressure, it will tend to migrate further away from the injection site than if the same amount were injected at higher pressure and at a correspondingly higher flow rate. This may appear paradoxical but follows from the fact that higher pressure loading generates a *larger gap* in the 'action plane' thus instantaneously providing *more ample accommodation* for the pre-set *limited*

grout take. In addition, the *time factor* is likely to accentuate this phenomenon provided the grout remains reasonably stable during the studied time interval.

7) It is often maintained that grouting at low pressure is preferable because of better permeation owing to the time factor. This may be true in a sense, especially in case of genuine permeation grouting according to Section 4.1 above. However, when hydraulic fracturing is at stake, the conditions are radically *different*. The elastic deformations induced by the impact of the enormous jacking forces are transmitted instantaneously through the rock mass, (or to be more exact at the speed of sound i.e. = $(E{\cdot}g/\gamma)^{1/2}$).

This means that the associated dilatation of existing cracks and fissures must generate *negative pore water pressure changes* (suction) in crack locations ahead of the leading front of grout. In due course, the expanding crack volume has to be filled by pore water from the surroundings and/or by the advancing grout. Hence, the pore water changes in the dilating fissure systems are likely to assist the migration of grout in the action plane to surrounding areas rather than being a hindrance. The time factor, therefore, does *not* work in the same way when grouting by hydraulic fracturing as when grouting by permeation.

8) When grouting, the first injection round is to a greater extent than the subsequent rounds, likely to fill possible initially open larger cracks and voids. In addition, adjacent cracks that are more or less parallel to the current action plane tend to decrease in width due horizontal pressure build-up, a phenomenon inevitably affecting the effective E-modulus applicable to ensuing injections.

The pressure required to attain pre-set volumes of grout per round is therefore likely to increase considerably in the later injection steps, while the calculated grout takes designed to attain the target spread tend to decrease.

Therefore, in hydraulic fracture grouting, due attention must be paid to the increasing stiffness of the sub-ground as the grouting operations progress. (see examples in Appendix).

9) Consideration of deformations in hydraulic fracture grouting is *exemplified* in two different scenarios in Appendix.

5.2 Response of sub-ground to Hydraulic Fracturing – unconfined conditions

As mentioned in the beginning of this section, grout spread and grout consumption under *unconfined* conditions are likely to be related to pumping rates and to the duration of the grouting operation rather than being in proportion to grouting pressure or to the mechanical properties of the ground. Provided the pressure in an advancing grout layer - at the top of the bed-rock surface – exceeds the vertical stress due to the weight of overlying soil by some measure, there is always an imminent risk of grout spreading to the extent that heave of the soil cover takes place. Once this condition is established, there is - for a Newtonian fluid - virtually no limit to further propagation if the pressure p_0 is maintained at a sufficiently high level - i.e. above the overburden pressure σ_v. In fact, when grouting is carried out using only pressure stop criteria, large quantities of grout are likely to go to waste - especially into the bedrock/soil interface. It may be noted that the Bingham effect is likely to be of even less importance in this context due to the reduced confinement.

5.2.1 Spreading behaviour of grout when injecting at high pressure – unconfined conditions

Although the spread of grout in connection with grouting operations may be the result of complex interaction between the procedure used on one hand and geological, geotechnical, structural as well as hydrological conditions on the other, practical experience shows that the propagation of grout in sub-ground of soil tends to follow certain *predictable rules*.

Special reference in this context is made to the general rule that grout advancing under pressure in natural soil deposits tends to follow closely the interfaces between stiff and softer structures. When the leading grout front encounters hard surfaces such as those of bedrock, larger stones, hard pan, horizons of hardened grout, sheet pile walls or the like, the grout always tends to spread along the harder surface.

Another circumstance related to this rule is the observation that larger stones in soil, which come in contact with advancing grout, tend to become embedded or wrapped in layers of grout.

The author of this report has frequently experienced this phenomenon in practical engineering. Actually it constitutes a physical law, which is for instance exploited in engineering when establishing anchors for stays in soil – in which case a body of grout is systematically built up in layers around the stay in the soil by *repetitive rounds* of grouting. Suitably limited volumes of grout in each step are then used.

A consequence of the mentioned phenomena is that, when a grout front emerges from a crack at bedrock surface, it will tend to follow the interface between the hard rock (or hard moraine) surface and overlying softer material. Resurgence to the ground surface occurs if

a) open connections through the soil layers exist or

b) the grout front encounters hard objects in direct contact with the bedrock projecting through the soil cover such as a house foundation, a retaining wall, sheet piles driven to firm bottom or pronounced boundaries between soil layers of markedly different properties.

The fact that advancing grout follows firm contours in the ground is evident from structure–mechanical analysis, and was postulated already in the 1970-ties by H. Cambefort.

As opposed to grouting under confined conditions the consumption and spread of grout under *unconfined* conditions are – although conditioned by the current ratio between overburden pressure and injection pressure - nevertheless in no *direct proportion* to the same injection pressure or to the *stiffness* and *strength properties* of the sub-ground.

Figures 5.6 and 5.7 illustrate typical scenarios where grout, emerging from vertical cracks or fractures in the bedrock under considerable pressure, spreads laterally along the rock/soil interface, eventually causing heave of the ground.

The process may be understood as follows:
When resurgent grout encounters the overlying soil, the flow is momentarily impeded, resulting in a substantial pressure growth at the point of contact with the soil. This follows from the fact that, when the flow stops, *pressure losses* generated by friction against the rock surfaces of the crack *vanish*. The pressure at the advancing grout front then rises temporarily and may even approach the relevant pressure in the bore hole $p_o - \gamma_g \cdot H_R$, where H_R is the depth of the grouting stage below the bedrock surface, compare with Figure 5.7.

Now, if the grouting pressure at the leading front - for instance $p_{S/R}$ - exceeds the prevailing overburden pressure by some measure at the bedrock surface, the soil around the point of contact between grout and soil will be deformed. This happens in a way similar to that occurring when grout under pressure deforms and splits the soil around a regular injection hole.

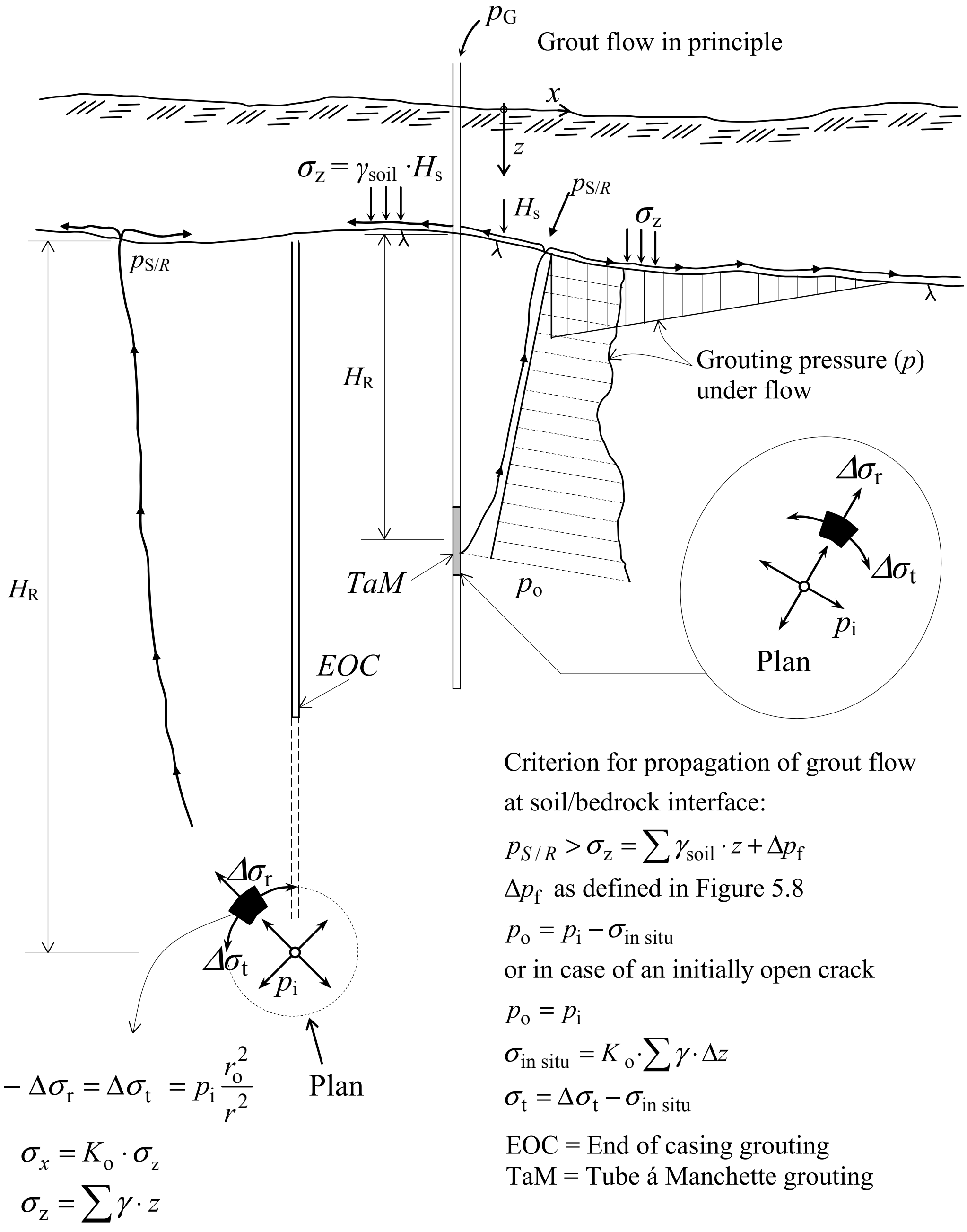


Figure 5.6 Grout propagation. Examples of flow and pressure development under unconfined conditions. Note: The overpressure at the soil/bed-rock interface is $p_{S/R} = p_o - \gamma_g \cdot H_R - p_\tau$, where H_R denotes the depth of the grouting stage below the bed-rock surface and p_τ the pressure loss due to flow friction.(Cf Figure 5.1).

Stress states generating fracture
Applies in principle to any orientation of bore hole

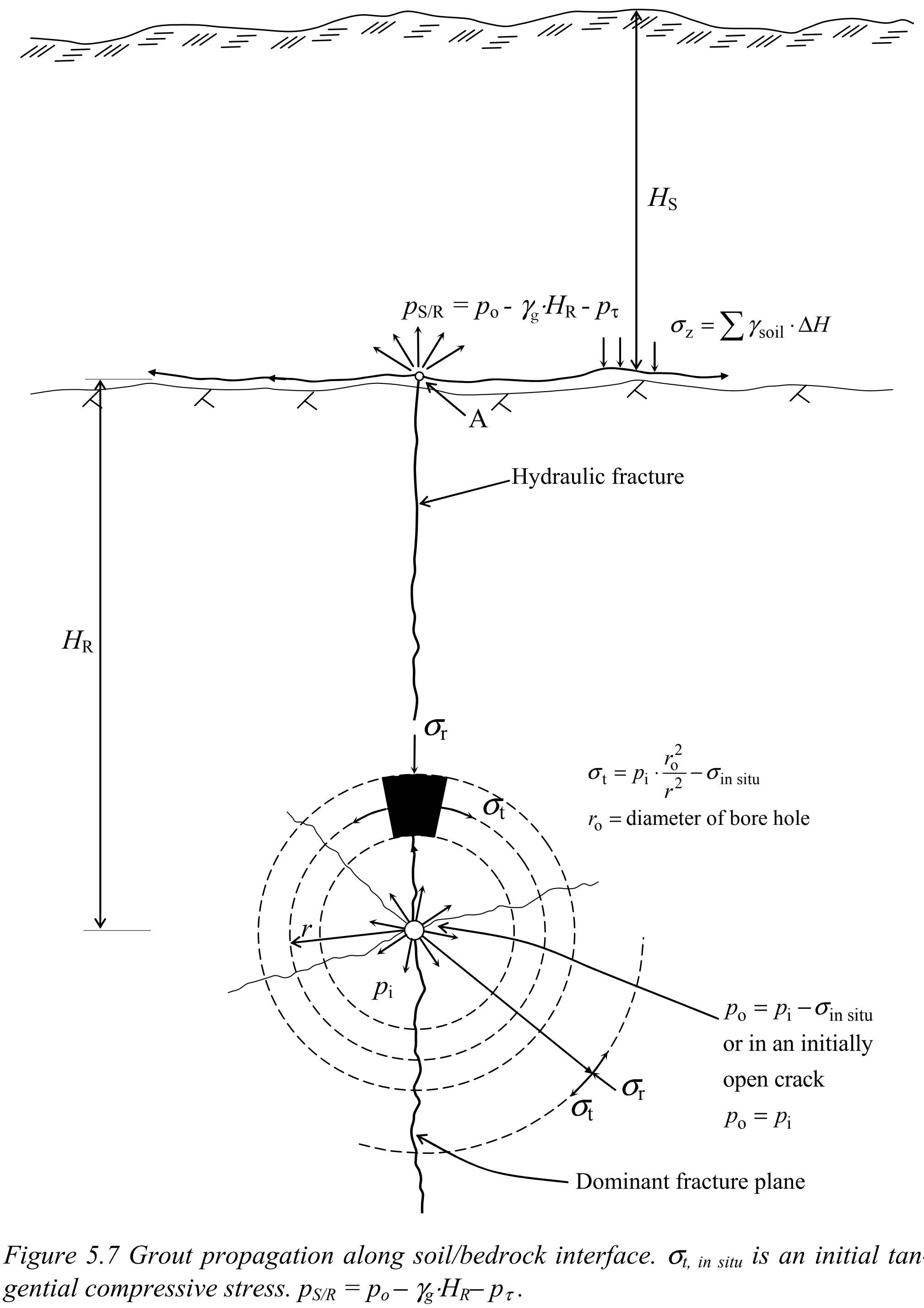


Figure 5.7 Grout propagation along soil/bedrock interface. $\sigma_{t,\ in\ situ}$ is an initial tangential compressive stress. $p_{S/R} = p_o - \gamma_g \cdot H_R - p_\tau$.

At sufficient overpressure, local stress and deformation at the grout front will generate an initial fracture or slit along the rock/soil interface allowing penetration of grout as shown in Figures 5.6 and 5.7. The growing uplift force then instantly results in more deformation of the immediately overlying soil. This entails additional grout penetration and so on, whereby a deformation controlled 'progressive failure' is likely to develop along the surface of the stiffer material - i.e. provided the grout pressure by some measure continues to exceed that of the overburden.

As the advancing grout layer spreads over a larger area, heave of the ground takes place under abating resistance from vertical shear forces. Eventually, the soil may actually *float* on a blanket of grout.

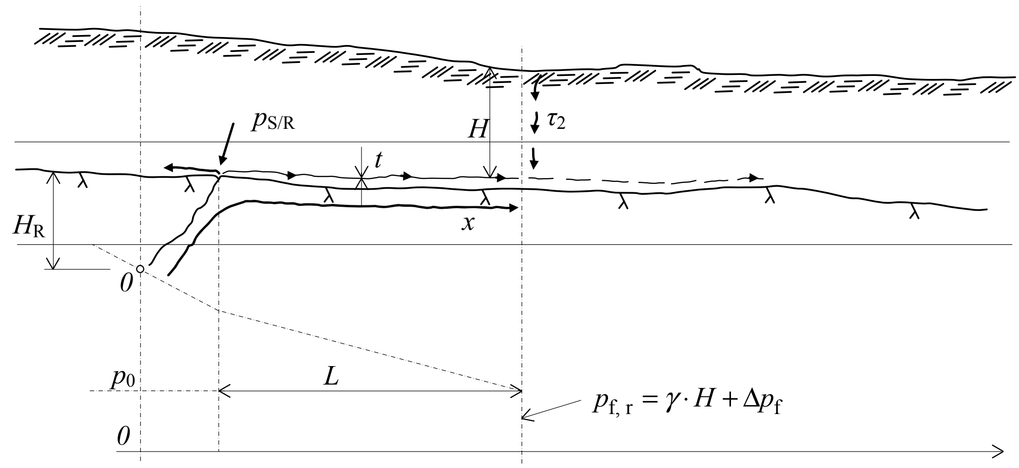
If sufficiently high pressure is maintained, there is - for a Newtonian fluid - in principle no limit to the further propagation of the grout. If the progress of the grout front is temporarily blocked for some reason by increased front resistance p_{fr} (e.g. by increased weight of overburden), then - provided (p_o- $\gamma_g \cdot H_R$) is still greater than $p_{f,r}$ - the grout front pressure p_{front} may, as a result of reduced flow rate, rise again so that

$$p_{front} = p_o - \gamma_g \cdot H_R - p_\tau > p_{f,\,r} \approx \gamma_S H_S + \Delta p_f \qquad (5.12)$$

where p_τ denotes the pressure loss due to friction at a certain flow rate and H_R is the vertical distance from the front to where p_i is applied. p_o denotes the overpressure in the crack at stage level i.e. $p_o = p_i - \sigma_{in\ situ}$.

Hence, when $p_\tau = 0$, then $p_{front} \approx p_o - \gamma_g \cdot H_R > p_{f,\,r} \approx \gamma_S H_S + \Delta p_f$

When again Equation (5.12) is satisfied, the grout can propagate further, yet at a lower rate. A rough assessment of the added front resistance Δp_f (2-dimensional conditions), originating from vertical shear as the grout front *initially* forces its way, is shown on Figure 5.8.



Newtonian flow

$$\tau = -\mu \frac{du}{dz}$$

$$v = \frac{\left(p_{S/R} - p_{f,r}\right) \cdot t^2}{\left(12\mu \cdot L\right)}$$

$$q = \frac{b \cdot \left(p_{S/R} - p_{f,r}\right) \cdot t^3}{\left(12\mu \cdot L\right)}$$

$$\Delta p_f = \gamma H \cdot 1.5 \tan^2\left(45 - \frac{\varphi}{2}\right) \cdot \tan\varphi \cdot 1.40$$

Bingham flow

$$\tau = -\mu_B \frac{du}{dz} + \tau_o \ [\mathrm{N/m^2}]$$

$$v^{\mathrm{Bingham}} = \frac{\left(p_{S/R} - p_{f,r}\right) \cdot t^2}{\left(12\mu L\right)} \cdot \left(1 - \frac{4}{3} \cdot e_o + \frac{e_o^4}{3}\right) \ [\mathrm{m/s}]$$

$$q^{\mathrm{Bingham}} = \frac{b \cdot \left(p_{S/R} - p_{f,r}\right) \cdot t^3}{\left(12\mu L\right)} \cdot \left(1 - \frac{4}{3} \cdot e_o + \frac{e_o^4}{3}\right) \ [\mathrm{m^3/s}]$$

$$e_o = \frac{2\tau_o L}{t \Delta p}$$

Figure 5.8 Grout flow along soil/bedrock interface for Newtonian and Bingham fluids. The Bingham effect is defined according to Håkansson (1993).

The factors that may slow down or inhibit further advance of grout under unconfined circumstances are,

- reduced injection pressure and flow rate;
- abating grout pressure as the distance between the advancing grout front and its source increases;
- increasing depth of soil cover (increasing γH_S) and/or growing front resistance;
- increasing viscosity and thickening of the grout due to consolidation related to loss of water to a draining environment (in French known as '*décantation*');
- thickening consistency due to thixotropic effects and enhanced Bingham behavior, where τ_o is a function of time ;
- increasing shear strength due to hardening of the grout.

Note that the effects of consolidation, thixotropy and hardening are phenomena, which are strongly dependent on 'the running time' defined as the covered distance L_S divided by the velocity v of grout flow.

Again, grout spread under ***unconfined*** *conditions* is not directly related to either injection pressure or to the mechanical properties of the sub-ground. For this reason alone, ***pressure cannot constitute*** *a* ***relevant stop criterion.*** Hence, focusing of grout treatment to the intended areas, and prevention of undesired spread of grout to the environment must also in this case be controlled by ***limiting the grout take per round.***

5.3 Comments regarding hydraulic fracturing and other current grouting philosophies

As mentioned previously, grouting engineers involved in dam construction often argue that high pressure grouting might be harmful because of the formation of new cracks induced by the associated displacements.

5.3.1 The GIN – Grouting method

The grouting procedure generally known as the 'GIN method' is, for example, claimed by its authors to be based on permeation grouting and on obviating hydraulic fracturing, (Lombardi & Deere (1973). The stop criteria used are threefold:

a) Maximum pressure, b) maximum grout take and c) the so called GIN value, the latter being the product of pressure and injected volume of grout per metre bore hole at zero flow. (GIN = Grouting Intensity Number). The GIN value is supposedly constant for similar types of rock.

Objections have been raised against the GIN method for not considering many of the relevant factors involved but accounting for these is not within the scope of this presentation.
However, departing from the basic concepts of this report, the main criticism with regard to the GIN principle would be that the grout takes according to GIN only relate to voids and open cracks despite the fact that grouting pressures of about 3 times those due to overburden are allowed. Yet, theory predicts and practice shows that hydraulic fracturing may readily be taking place under such pressure conditions.
(For instance, if the grouting depth is 30 m below ground level, the allowable injection pressure according to the GIN method may be in the order of $3 \cdot 30 \cdot 26.5 \approx 2400$ kN/m^2 = 24 bar in terms of total stress. This implies an overpressure of about 24–8 = 16 bar in excess of the vertical in situ stress σ_v, i.e. a condition very likely

to provoke hydraulic fracture. Furthermore, when fracturing in vertical planes is considered, the widening of present open cracks and related jacking effects are even more likely to occur, as the pore water pressure in such cracks is limited to current ground water heads).

In addition, the effects of strain, deformation and stiffness of the rock mass are disregarded in the GIN method, when defining the pressure limit for avoiding hydraulic fracturing. In the authors' opinion, the absence of deformation analysis strongly invalidates the results of this kind of estimate.

Another weakness in the GIN approach is inadequate consideration of the relationship between frequency and width of cracks on one hand and permeability, Lugeon tests or GIN values on the other. (Confer e.g. Appendix in this report.)

5.3.2 Ewert (1996b) versus Lombardi (1985)

Ewert (1996b), Parts I & II, has forwarded serious, and in the author's view, well-founded objections to the GIN method. Again, it is not within the scope of this report to account for this critique. However, in the context of hydraulic fracturing issues, it is of interest to note the following:

In Part II of Ewert (1996b), Ewert clearly demonstrates that in four dam-related records of grouting work performed according to the GIN method, a major portion of the grouting stages were *actually hydraulic fracture events*. Ewert also emphasises the *general difficulty of avoiding hydraulic fracturing* in grouting.

5.3.3 Recommended hydraulic fracturing principles in accordance with this report

However, in Ewert (1996 a,b), Ewert also adheres to the notion that hydraulic fracturing is an unwelcome phenomenon in dam grouting. Admittedly, grouting for dams may be different from many other grouting applications in that the hydraulic heads often are extremely high in the service stage.

Yet, one of my comments on this account to those who strongly advise against high pressure grouting is that *temporarily applied pressure alone* does *not generate permanent change* in a ground formation. The lasting effect, in fact, is more likely to be a function of the *amount of grout actually injected*.
For instance, if only 100 litres of grout is forced into fissured rock at a pressure of say 70 bar and at a moderate rate, the permanent distortion will be quite insignificant compared to the effect of injecting 1000 litres at say 15 bar or less. Hence, the permanent change and possible damage in a rock mass is more related to the

amount of grout injected than to whatever pressure has been applied in placing the grout. (Or, speaking metaphorically, the damage caused by an electric spark is related to the energy released by the spark rather than to the difference in voltage.)

Shape of grout-stone intrusions

Further, when speculating on the implications for permeability of the thin fracture induced tapering grout-stone layers, more attention should be paid to the elastic response of the rock mass. The elastic character of the rock mass allows a high degree of adaptation to the intrusions of hardened grout.
Moreover, although the deformations generated by the fluid grout during the grouting operation may be significant, they are nevertheless likely to contract and deform considerably on account of consolidation due to the forceful elastic rebound of the rock mass. In fact, the grout-stone intrusions themselves are *basically shaped* by the nature of the elastic reaction of the ground. Furthermore, the rebound effect promotes grout-stone quality by drainage of surplus water in grouts with high water/cement ratios to the surroundings.

In addition, the pressure build-up (the pre-stress) inevitably linked with the formation of these intrusions tends to close other cracks and/or consolidate loose material in cracks filled with soil.

It has been stressed earlier in Section 5.1 under the heading “Configuration of action plane” that the grout-stone intrusions are likely to be of rather uniform thickness and, in principle, not ‘dendritical’. In this context it may be of interest to consider the shape of intrusions of magma into the earth crust of the kind shown in Figure 5.9 constituting the result of ‘grouting’ activity performed by Mother Earth into its lithosphere of hardened rock.

As is evident from the forgoing sections, I do *not*, for a number of reasons, generally subscribe to the notion of the highly deleterious nature of *hydraulic fracturing phenomena,* at least for moderate hydraulic heads.

One *important reason* for this position is that, in the experience of the author, closer studies of case records of intended penetration grouting usually reveal evidence of a high frequency of hydraulic fracture events. This has readily been possible to observe in pressure/flow logs and in excavations of ground previously subjected to grouting treatment. Of the numerous successful grouting operations investigated or controlled by the author, none of them could actually be classified as genuine permeation grouting. This, as previously pointed out, is largely due to the fact that the legitimate aim of the operator to attain target volumes of injected grout, more often than not, compels him to raise the pressure to levels leading to

Figure 5.9 Typical intrusion of magma in gneiss of igneous origin that was once located deep down in the crust of the earth. Grout-stone intrusions observed in excavations in grouted rock usually have an identical appearance.

opening of the medium by hydraulic fracturing. The review by Ewert (1996b) and the Graad & Hedlund (1996) thesis effectively corroborate this experience.

In conclusion, if most successful grouting projects – like the GIN dam case records – to a major extent actually represent hydraulic fracture events, then hydraulic fracturing cannot be that harmful – provided the spread of grout is kept within specified limits.

*In the opinion of the author, few cases of **intended** low pressure permeation grouting would be successful without **unintended** hydraulic fracturing.*

5.4 Case records

5.4.1 Grouting of a formation of stiff pleistocene clay below Dry Dock No II, Gdynia, Poland

The first of the following case records does, admittedly, not deal with grouting in a rock material. However, as already stated in Sections 2 and 3.2, the progression of grout, although being influenced by the structure and stratification of a formation, is *in principle* not very different in formations of densely fissured igneous rocks, sedimentary rocks and stiff soils such as e.g. tertiary clays. This applies of course only when hydraulic fracturing is at stake. The difference in response to the jacking effect of injection pressure relates primarily to the ability to deform, and is therefore governed by pressure and stiffness characteristics. When ground formations, which are similar in structure and *relative* stiffness, are subjected to the same grouting treatment and the same grouting pressures, the grout consumption will in principle reflect the difference in the mean elastic moduli of the different formations as stated in Section 5.1 above. The impact of varying stiffness of rock formations is demonstrated by the examples 2B and 2C in the Appendix.

5.4.1.1 Description of the construction site

The 70 m by 380 m (= 25 600 m^2) large Dry Dock No II in Gdynia was designed and constructed as a 'turn key project' in the years 1973 → 1976 by Skanska AB (then AB Skånska Cementgjuteriet). In plan, about half of the dry dock was situated onshore involving deep excavation down to about 13 m below the ground surface. The offshore part of the dry dock was completed in reclaimed land. Sections through the finished dry dock and through the deep onshore excavation are shown in the Figures 5.10 and 5.11.

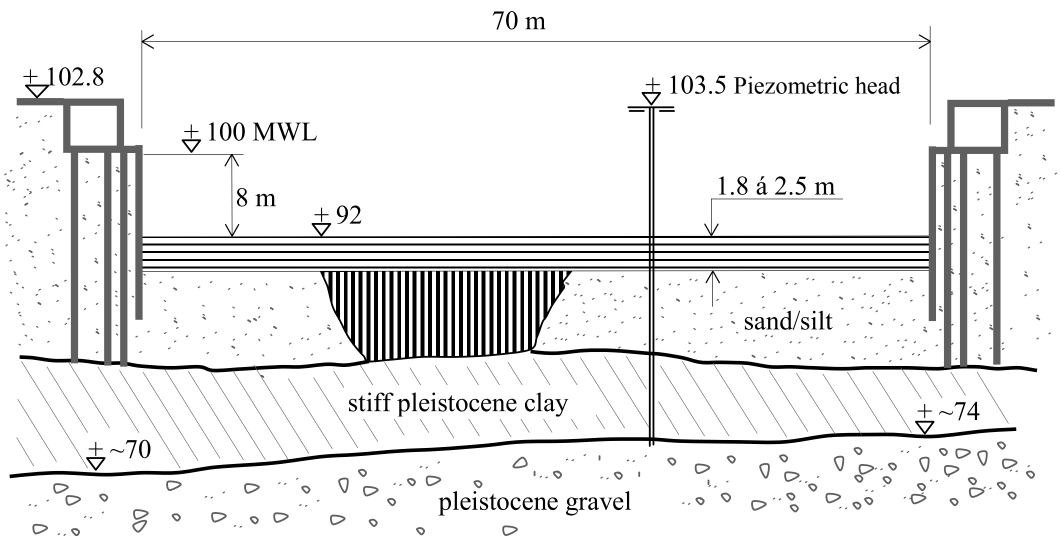


Figure 5.10 Section through Dry Dock No II, Gdynia, Poland.

5.4.1.2 Ground conditions

The bottom slab of the dry dock was founded on wooden piles in varved sandy and silty sediments extending some 8 m to 10 m below the slab. The silty sands are underlain by a formation of stiff pleistocene clay, which varies between 4 m and 10 m in depth. Below the stiff clay, there is an extensive deposit of extremely permeable pleistocene gravel to considerable depth, constituting an artesian aquifer with a piezometric height exceeding the mean sea level by some 3.0 to 3.5 m.

5.4.1.3 Description of the dry dock design

The dry dock being designed as a drained structure (as opposed to a gravity type of dry dock) it was mandatory to control and minimise ground water flow as well as preventing pressure build-up under the dock bottom slab. The potentially low transmissivity of the stiff pleistocene clay was therefore a crucial requirement for the design principle and for a successful realisation of the dry dock.

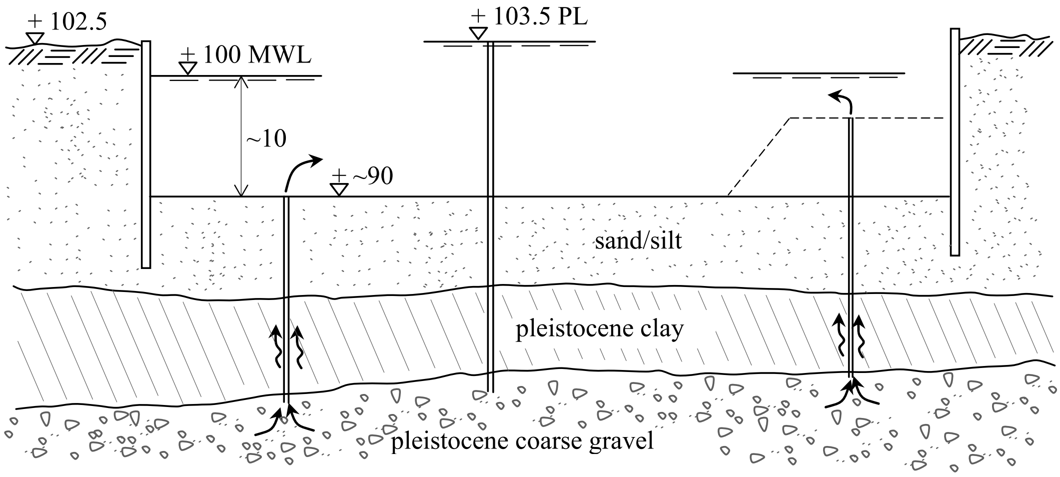


Figure 5.11 Section through the on-shore excavation for the dry dock.

5.4.1.4 Leakage problems during construction.

However, already at an excavation depth of about 4 m below ground level and onwards, serious leakage problems manifested themselves. In one instance, at about 4 m above the target bottom level of the excavation, a local leakage flow of 300 m^3/hour was registered.

The sources of leakage were primarily found to be associated with recent and old bore holes through the stiff clay from soil exploration as well as with obsolete water supply wells, of which some were not properly documented. Hence, apart from the recent bore holes related to the ongoing project, the documentation of the locations of old bore holes and pump wells were either uncertain or virtually unknown - i.e. the range of uncertainty sometimes being 20 to 30 m.

5.4.1.5 Remedial measures

In order to cope with the leakage problems, which jeopardised successful execution of the drained dry dock design, closing of the clay formation by grouting was decided upon.

After some preliminary but rather unsuccessful trials by the subcontractor based on *permeation grouting*, a new extensive but detailed grouting program was devised by the design department of the main contractor. The revised program was now based on the *principles of hydraulic fracturing* involving:

a) Sufficiently high injection pressures - i.e. no limiting pressure criteria.

b) Fixed volumes of grout per stage - i.e. the grout was allowed to stiffen between the grouting rounds.

c) Short stage TaM grouting was applied in order to promote the incidence of horizontal hydraulic fracturing.

d) In each bore hole, 8 grouting stages were executed in principle as indicated in Figure 5.12.

A maximum of 1 m^3 of grout was injected at each stage (i.e. 8 m^3/bore hole).

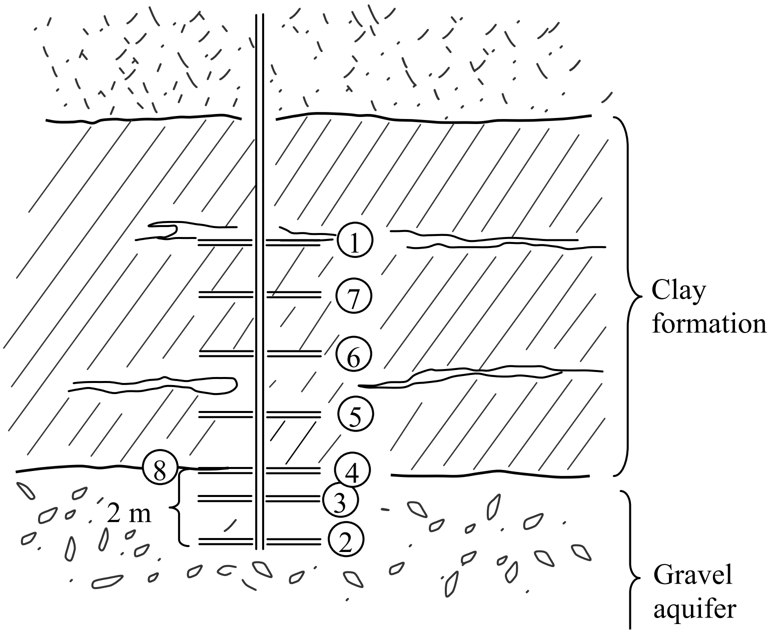


Figure 5.12 Levels of short stage TaM grouting.

The comparatively high amount of grout per stage, as compared to what might be recommendable in rock grouting, was partly due to the lower *E*-modulus of the stiff clay and partly to the fact that environmental damage from long-range grout propagation was not a crucial issue in the current case.

The adoption of the grouting procedure outlined above was based on the conviction that the *probability of encountering and closing distant leakage paths* would be greatly enhanced by the long range spread possible by hydraulic fracturing.

In practice this program proved to be a highly successful approach, as it was often observed how leakage flows were reduced or stopped even when grouting at other more or less distant locations was going on. In one instance, for example, a major leakage was totally blocked, while injection was carried out in a point some 40 meters away.

The varying confidence in respect of the locations of potential but *not yet identified* leakage paths, which was a particularly high risk factor in the off shore area, was dealt with as follows:

For each suspected – but uncertain - point of potential leakage, three injection holes were drilled at the corners of an imaginary equilateral triangle with its centre at the assumed point of leakage and with its sides about two times the radius of 'uncertainty'. Hence, if the range of uncertainty was estimated at e.g. 10 m, then the distance between the injection holes would be 20 m. In this way, it was deemed that at least one of the injection points would not be too far from the source of leakage.

In places, where the degree of uncertainty was less than 4 m, only one injection hole was established.

Altogether some 70 injection holes were made with the object of closing the stiff clay formation below the dock bottom. The total amounts of grout consumed for this purpose in the *onshore area* were about 500 m^3 of cement/bentonite grout and some 35 m^3 of chemical grout ('stabilodur').

The grouting intensity in terms of grout volume per m^2 of dock bottom area thus corresponded to about $535/13000 = 0.04\ m^3/m^2 = 4$ cm.

In the offshore area the average grouting intensity was considerably less.

5.4.1.6 Results from grouting of the stiff clay formation below the dock bottom

The outcome of the grouting work proved fully successful. The excavation for the dry dock could be completed to the required levels, while specified requirements regarding leakage water into the excavation were met.

Figure 5.13 shows an example of how the excess artesian overpressure was gradually reduced in the vicinity of the mentioned heavy leakage of 300 m^3/hour during the time that the grouting work progressed in the south-western corner of the dock. Curve (1) represents the artesian overpressure gradient *before* grouting commenced, whereas Curve (5) depicts the corresponding gradient when grouting in the surroundings was completed. The difference between the two curves reflects the radical reduction of transmissivity through the clay layer achieved by means of the grouting treatment. Hence, as can be deduced from curve No 5, virtually all of the artesian pressure drop is located within the clay formation.

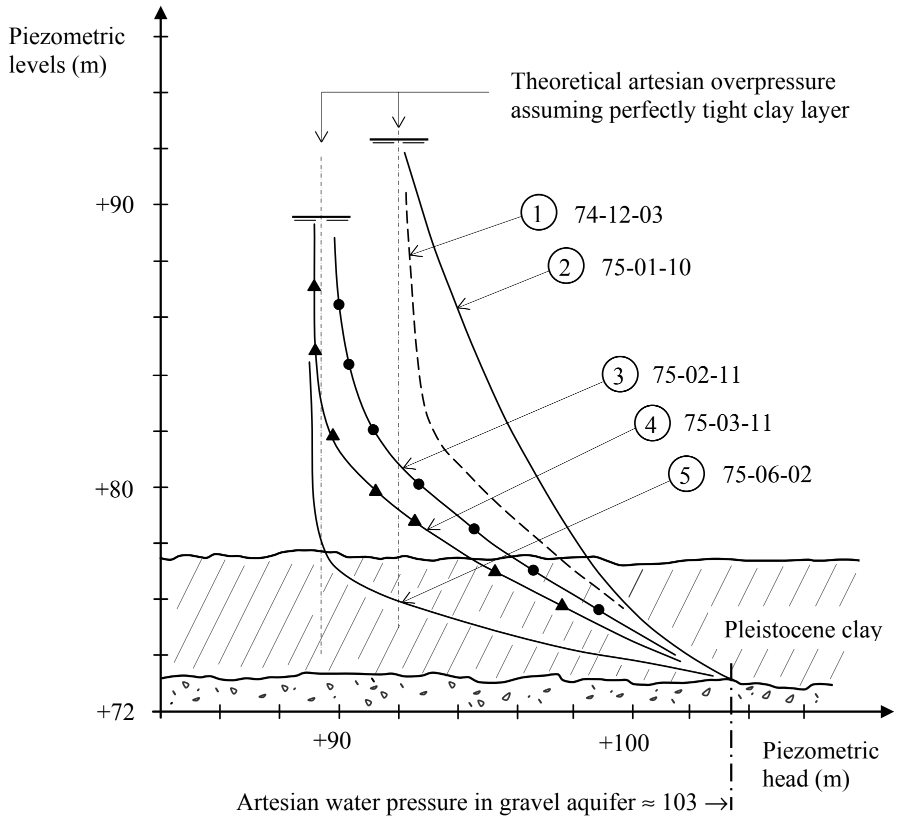


Figure 5.13 Curves showing the development of abating artesian overpressure *as grouting of the pleistocene clay in the south-western part of the Gdynia dry dock progressed. The difference in basic piezometric head in Curves 1 to 2 on one hand, and 3 to 5 on the other, relates to increasing excavation depth during January 1975.*
Curve (1) Prior to incidence of leakage at exploration bore hole No 2.
Curve (2) Leakage at bore hole No 2 under control but sealing of clay layers in the surrounding area is ongoing.
Curve (3) Leakage at bore hole No 2 sealed. (75-02-11).
Curve (4) Grouting outside western sheet pile wall is ongoing.
Curve (5) Leakage at deep well Dj 407 sealed. Grouting of western sheet pile wall completed. (75-06-02).

5.4.1.7 Conclusions

The very positive results from the hydraulic fracture grouting thus entailed that the dry dock could be finalised according to the intended design - i.e. with drainage under the dock bottom slab.

The *cost* of *realising the hydraulic fracturing program* proved to be only about 1/10th of the originally estimated cost of acquiring closure of the gravel layers just below the stiff clay by permeation grouting.

5.4.2 Tunnelling for sewage pipe line in Alexandria

In connection with tunnelling for the West Zone Collector System, Phase I, (1985-1988) in Alexandria, Egypt, a grouting program similar to the one in Gdynia was successfully implemented over a section near the West Treatment Plant. The tunnel in this area went through a formation of mixed layers of cemented sands and sandstones of varying induration. Contractor: MacLEAN GROVE & Company Incorporated, Greenwich, U.S.A. Project Engineers: WWCG, U.S.A in association with WARITH/ECG. Detailed design, (construction drawings): SKANSKA Engineering Dpt, Gothenburg.

5.4.3 Grouting trials at Västra Station, Malmö, Sweden

The importance of grouting to the City Tunnel Project in Malmö was understood early in the planning of the project and grouting trials were conducted in the years 1999 - 2002.

In the following, only the first trial at Västra Station (*1998-1999)* is dealt with.

The grouting trial at Västra Station formed part of the early investigations in connection with the City Tunnel Project in Malmö. The site was located at the old Västra Station, which is located North East of Malmö Central Station. The ground conditions on the site are typical of the Malmö C area, with 8 to 10 m of soil layers consisting of fill overlying clayey till. The limestone encountered at this location is reported to be composed of only Bryozoan Limestone, with the Copenhagen Limestone allegedly being absent. (Cf Section 2)

The hydro-geological conditions encountered are typical of the area, with a high permeability zone in the upper part of the limestone. Below this there is a zone of relatively low permeability but with a distinct flow zone at a level of approximately -22 m.

Two rings of grouting holes were installed to a depth of 36 m, each ring being 8 m in diameter. In one ring the tube-á-manchette (TaM) method was used, and in the other the method named End of Casing (EOC) was applied.

The grout mainly used for the trial was a mix having a w/c ratio of 0.8 with Injektering 30 (30 µm cement) and an HPM additive of 0.54 % by weight of cement. A hole spacing of 1.5 m was used for both TAM and EOC rings, with the grouting executed in a primary, secondary and tertiary sequence. The injection stop criteria were set both on maximum volume and maximum pressure. However, it may be noted that the pressure stop criteria were set at rather low values of between 1.0 and 2.0 MN/m^2 - depending on the depth of the stage in the limestone.

Some of the conclusions from the trials were as follows:

- The EOC and TAM methods appeared to be equally effective but EOC appeared to cause more heave.
- An overall permeability reduction by a factor of approximately 5 was achieved. When the initial permeability was low, less reduction of permeability was attained. The permeability reduction was not uniform - the transmissivity being substantially reduced in some zones - whereas in other zones there was no measurable effect.
- The absorption of grout was generally remarkably low as it was often limited by the low pre-scribed pressure criteria. Pre-set target grout takes were thus frequently not met.
- The number of repetitive passes at each grouting level appears to have been insufficient. This applies in particular to the identified local zones of higher permeability, which accounted for a major part of the total transmissivity.

5.4.3.1 Conclusions

- The reduction of transmissivity in the Västra Station grouting tests proved to be insufficient and were smaller than anticipated.
- This insufficiency was, to a major extent, believed to be related to the mentioned failure to attain target grout takes due to the low grouting pressures applied.
- The most important conclusion to be drawn from these trials was therefore, in the opinion of the author, that *volume stop criteria* should be given *priority over pressure stop criteria*.
- The little number of repetitive grouting passes in pervious layers was a negative factor.

Other key aspects of this project in relation to the subsequent trials at Bagers Plats were considered to be:

- The grouting intensity applied to the soil/bed-rock interface was considerably less than that applied in Zone I at Bagers Plats. This may be an additional factor explaining the difference between the results of the two trials.

5.4.4 Grouting trial at Bagers Plats, Malmö, Sweden

The unsatisfactory outcome of the Västra Station trials with regard to the closure of the rock formation formed an important background for decisions, which in due course led to the more comprehensive grouting trials at Bagers Plats (2001-2002).

These grouting trials, including the excavation of a test pit within the treated area, were located some 250 metres west of Malmö Central station. The grouting operations were undertaken by Bachy - Soletanche between October 2001 and January 2002. Sheet pile walls and excavation were carried out by Per Aarslef AS. The equipment used for the trial included modern computer controlled grout pumps allowing the grout injections to be accurately limited to specified criteria and continuous monitoring of grouting pressures for later analysis

The ground conditions on the site correspond to those described previously in Section 2. The detailed geological features and the general arrangement of injection pipes are shown on Figures 5.14 & 5.15.

In the trials pre-and post grouting pumping tests were carried out in order to measure the effect on the hydraulic conductivity of the grout injections. Also, a number of different grouting procedures were tested including injections performed both at lower pressures (max < 1.5 MPa) and at higher pressures (max < 7 MPa). The particulars and objectives of the different phases in the trial are summarised in Table 5.1.

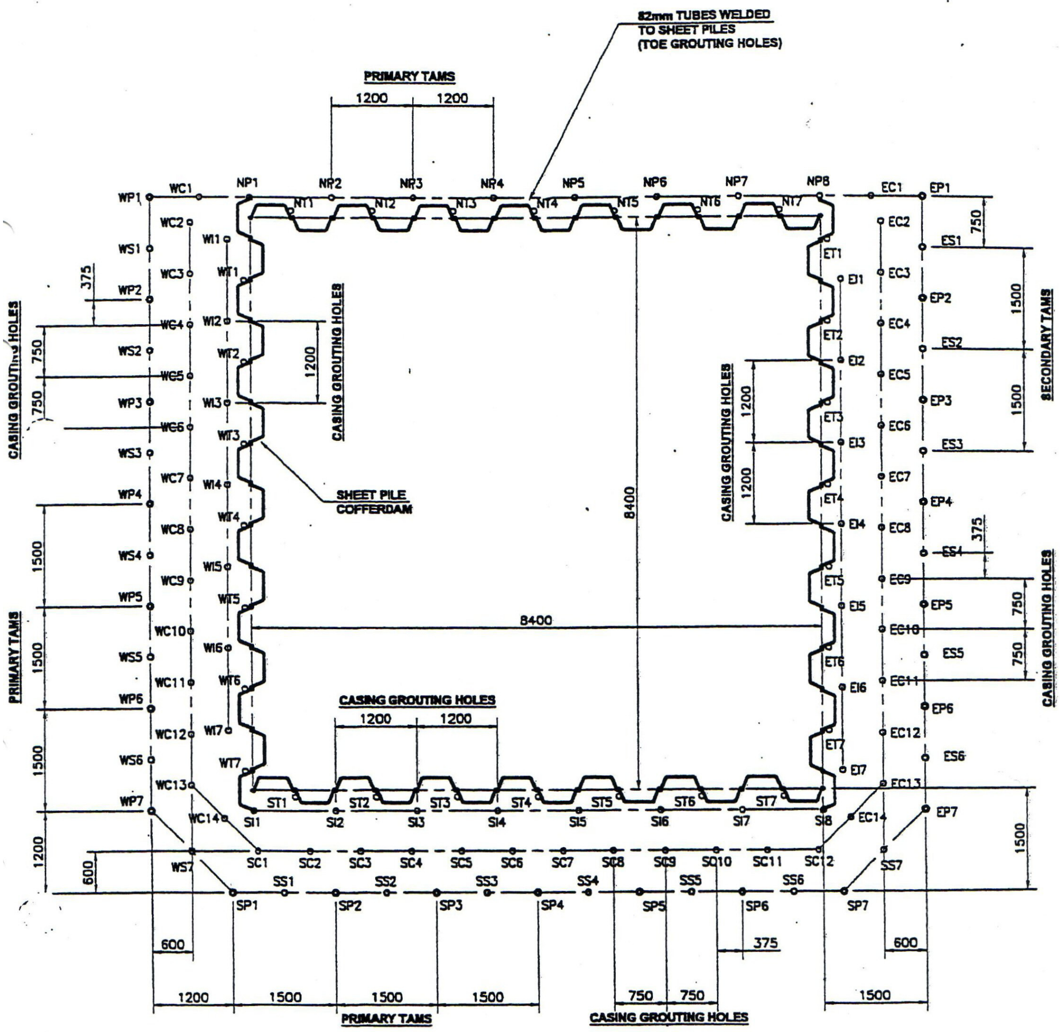


Figure 5.14 Plan of test area and grouting arrangements.

Table 5.1 Summary of data for the different injection phases. Total grout consumption = 82.7 m^3.

Designation	**Level treated [m]**	**Strata treated**	**Objective / Purpose (initially formulated in the grouting program)**	**Technique - Amount of grout injected [m^3]**
Sheet pile toe grouting Grout type 8A	-8 to -10	Uppermost lime-stone (Zone1)	Reducing permeability due to fracturing by sheet piles being driven into limestone	Openhole – via casings welded to sheetpiles ΔV_1= 12.3 m^3
Interface TAM Grout type 4A	-5.8 to -10.8	Lowermost soils, lime-stone / soil interface & uppermost limestone	To reduce the perme-ability of a potentially permeable zone within Zone I (upper part) close to the sheet piles & to improve rock confinement	Tube – à – Manchette ΔV_2= 25.3 m^3
Interface EOC Grout type 8A	-5.8 to -9.8	Lowermost soils, lime-stone / soil interface & uppermost limestone	To reduce the perme-ability at the interface between the main grouted curtain and the sheetpiles & to im-prove rock confine-ment	End of Casing ΔV_3= 16.2 m^3
Bagged TAM Zone I Grout type 4A	-10.8 to -13.8	Copenhagen /Bryozoan Limestone (lower Zone 1)	To produce a “grouted curtain” cutoff.	Bagged Tube - à - Manchette ΔV_4= 16.9 m^3
High Pressure Zone II Grout type 4E	-13.8 to -20.8	Bryozoan Limestone (lower Zone 2)	To produce a “ grouted curtain” cutoff, using high pressure direc-tional hydro-fracture at large (3m) hole spac-ings.	Bagged Tube - à - Manchette ΔV_5= 12.0 m^3

The grout materials that were used at the Bagers Plats trials were selected after a testing programme carried out at the Royal Institute of Technology (KTH), Stockholm. This programme also incorporated results from research on cement-based grouts previously carried out at KTH.

The following mixes were selected:

Table 5.2 Selected grout compositions.

No	Mix Type	w/c	HPM [l/m^3]	Microsilica [l/m^3]
4	Injektering 30 (Cementa)	0.8	21.7	63.3
4E	Injektering 30 (Cementa)	0.6	17.6	123.0
8	'Industricement' av Portland (SH)	1.0	17.4	107.3

Note: HPM is a superplasticiser based on melamin/nepthalene

5.4.4.1 Grouting data

The data resulting from the various tests shown in Table 5.1 were very comprehensive and will not be accounted for at length in this context. In the following, therefore, the discussion is limited to the volumes of grout injected in the different operational phases and to the probable migration and the final destination of the grout.

The total volume of injected grout added up to 82.7 m^3. The right column in Table 5.1 shows how this total is distributed over the different phases of the grouting operation.

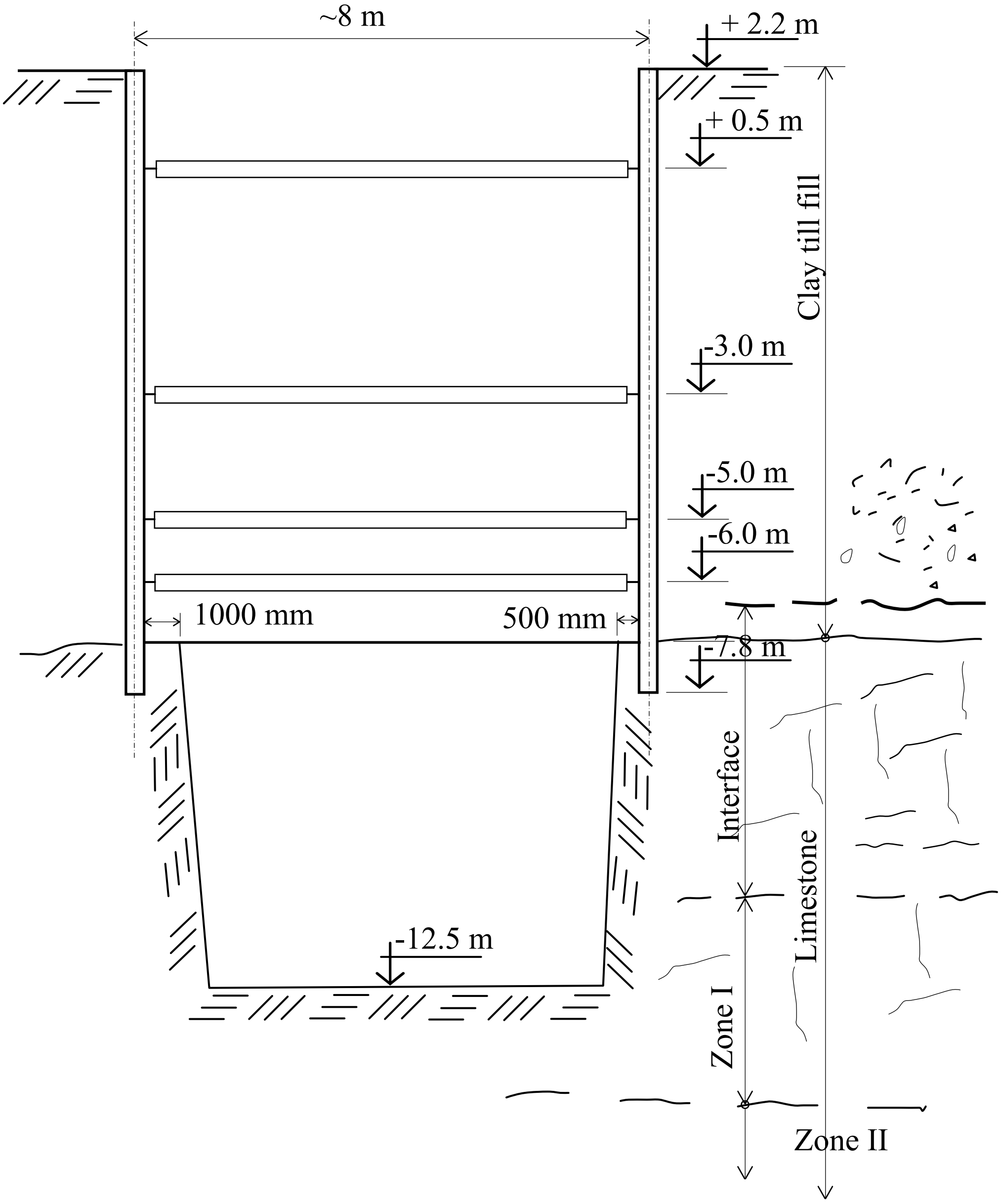


Figure 5.15 Section through the excavated test pit.

Generally the following pressure limits were applied:

Claquage pressure: 5.0 MN/m^2 (In the High pressure phase 7.0 MN/m^2)

Max. injection pressure: 1.5 MN/m^2

The maximum overburden vertical loads in terms of total stresses ($\boldsymbol{\sigma}_v$) are given in Table 5.3. Denoting the claquage pressure as $\boldsymbol{p}_c$ and the injection pressure as $\boldsymbol{p}_i$, the ratios of grouting pressures to overburden stresses in the different phases were:

Table 5.3 Ratios of grouting pressure to overburden vertical stress. Number of grout stops at 1.25 MN/m² as a percentage of all first injection events.

	σ_v [MN/m^2]	**Ratios of** p_c/σ_v	p_i/σ_v	**Grout stop** [%]
Sheet pile toe grouting phase:	0.27	18.5	5.5	100
Interface TaM phase:	≈ 0.29	17.2	5.2	34
Interface EOC phase:	≈ 0.27	18.5	5.5	97
Bagged TaM - Zone I phase:	≈ 0.37	13.5	4.0	40
High pressure - Zone II phase:	≈ 0.56	12.5	12.5	
Mean percentage of grout stop events (1st injection)				33

With regard to hydraulic fracturing it is thus essential to observe that the claquage pressures in the first four phases were about 12 to 18 times greater than the overburden stresses, implying conditions that must be regarded as clear evidence of claquage fracturing. (Cf Section 5.1.1).

Furthermore, the maximum injection pressure limit of 1.5 MN/m^2 exceeds the overburden stress by factors of about 4 to 6, i.e. also indicating proneness to the incidence of hydraulic fracturing phenomena.

The percentages of grout volume stops at a mean pressure of 1.25 MN/m^2 (12.5 bar) in each of the different grouting phases are shown in the Table 5.3. Grout volume stop of this magnitude in tight rock formations is almost certain evidence of hydraulic fracturing – a condition, which however does not preclude the possibility of pressure stop events also being hydraulic fractures.

5.4.4.2 Results

The dominating response of the limestone formation to the applied grouting pressures must have been hydraulic fracturing. (Confer Section 3.)

1) The toe grouting phase

In this phase about 60 % of the injected volume of 12.3 m^3 resurfaced at ground level already for grout pressures of about 0.2 MN/m^2. As the total void volume from displacement by driving the sheet piles into the limestone is estimated to be in the order of 1 m^3, the balance of the grout volume has ended up far outside the intended area of treatment. With reference to Section 5.2, dealing with the 'Response of sub-ground to Hydraulic Fracturing under 'unconfined conditions' it is likely that a considerable portion of the remaining 40 % of grout is to be found in the nearby soil/rock interface.

2) and 3) The TaM and EOC interface grouting in the upper Zone I

In these phases 25.3+16.2 = 41.5 m^3 were injected. The upper Zone I being located close to the bed-rock surface and considering the high ratio of grouting pressure to the over-burden stress, the grouting conditions must be considered as having been clearly unconfined. In the EOC phase as many as 97 % of the first injections were either grout stops or resurgence phenomena at injection pressures of less than 0.31 MN/m^2. The corresponding value for the TaM interface phase was 34 %. Evidently, a large volume of the injected grout was lost to the soil/rock interface and by resurgence along sheet piles to the ground surface.

4) TaM grouting in the lower Zone I

In total 16.9 m^3 of grout was consumed - in this phase without resurgence to the ground surface. However, the claquage and injection pressures were still about 13 and 4 times the overburden stresses respectively - i.e. indicating unconfined grouting conditions with high proneness to grout migrating into the soil/rock interface. Grout stops were recorded in 40 % of the stages in the first injection round.

5) High pressure grouting in the Bryozoan limestone - Zone II

In the high pressure hydraulic fracturing phase some 12 m^3 of grout were consumed. Owing to the confinement generated by the previous grouting operations it is difficult to assess - without deformation analysis - how much of the grout that may have surfaced at the soil/rock interface.

Subsequent to the grouting operations, the area within the sheet piling was excavated down to level –12.5 allowing visual inspection of the walls of the pit from level – 7 down to level –12.5. See Figure 5.15.

5.4.4.3 Conclusions from the Bagers Plats Trials

1) Pre - and post pumping tests indicated a 90 % reduction of transmissivity - i.e. a markedly better result than at Västra Station.

2) Inspection of the exposed walls in the excavation pit - i.e. down to about 5.5 m below the rock surface - revealed the following:

a) Apart from presence of grout in a few isolated cavities - caused by flushing of bore holes close to the sheet pile wall - no indications of grout having permeated the pores of the soil - or natural voids and cavities in the rock material - could be detected. No evidence of permeation grouting as defined in Sections 3 and 4 was observed.

b) The presence of grout only occurred as a few planar 3 to 5 mm thick seams of hardened grout (grout-stone). A rough appraisal of the volume of grout-stone material encountered within the volume open to inspection would amount to some 1 m^3 corresponding to 0.3 % of the relevant excavated volume. Assuming for instance that the area subject to treatment extends 10 m *on average* outside of the sheet pile confinement, then this percentage would correspond to a total grout volume of *only* about 13 m^3.

The whereabouts of the remaining 70 m^3 of grout consumed may be subject to speculation but indicates the mentioned likely loss of large volumes of grout to the soil/rock interface due to unconfined grouting conditions.

3) With regard to vertical or horizontal fracturing, the following may be concluded. Assuming as in 2b) above that the grout spread is 10 m outside of the sheet pile enclosure and that only horizontal fracturing were to take place, then a grout volume of 82.7 m^3 would correspond to a heave of the ground surface of some 100 mm. However, instead a slight settlement was recorded in the initial phases of the grouting operations. In the final phases, a slight heave of a few mm:s was measured. The fact that the heave only amounted to a fraction of the said 100 mm:s indicates that hydraulic fracturing took place predominantly in vertical planes. Measuring horizontal deformations would have given valuable information on this issue but unfortunately horizontal displacement was not monitored at the Bagers Plats trials.

4) The mechanism generating the closure of the formation in the Bagers Plats trials is to be ascribed mainly to hydraulic fracturing phenomena.

6 Final remarks

6.1 Objectives of hydraulic fracturing and hydraulic crack expansion (jacking)

The effects of ***hydraulic-fracturing*** as defined in Sections 3.2 and 5 on sub-ground of rock may be summarized as follows:

a) Expansion and filling of the most accessible cracks in each round. If the rock mass around the injection hole is locally intact (i.e. without fissures), fracturing initiated at the bore-hole will open connections to existing fissure systems provided the claquage pressure is sufficiently high. Once the grout in some measure has spread into the crack that is most available to penetration, a growing jacking or splitting force in an 'action plane' controls from then on the further progression and spread of grout.

b) Extensive grout spread (i.e. long range action) enhancing access to discrete permeable 'groutable' structures and leakage paths, which are **otherwise beyond reach** from the point of injection.

c) Closure of other not 'groutable' finer cracks and fissure systems as a result of horizontal stress build-up. Associated compaction and consolidation of soil in cracks originating from local weathering of rock material.

Item b) above emphasizes the fact that hydraulic fracturing must *not* be thought of as a technique, which in any way excludes permeation or impregnation wherever that be possible.

6.2 Stop criteria - <u>confined</u> conditions

The table in Figure 4.2 indicates that grouting by permeation of stable cement suspension grouts in soils is not feasible unless d_{10} exceeds ≈ 0.2 mm, corresponding to coarse sands or gravel – i.e. in soils, where at least 90 % of the grains are bigger than 0.2 mm.

Therefore, in tight formations of soil or rock material, where the innate permeability does not satisfy the criteria in Figure 4.2, very little can effectively be achieved in the form of sealing effect using cement based grouts without *resorting to hydraulic-fracturing*.

Owing to the fact that hydraulic fracturing by nature entails rather long-range action it is, for reasons of economy and environmental impact, of utmost importance to limit the grout volume to be injected in each step or round. This applies in particular to rock formations, where even high permeability and transmissivity may correspond to small penetrable crack/void volumes. (Cf. Appendix).

In fact, *limiting the grout volume in each pass is the only criterion*, *by which it is possible to confine the grouting treatment in statistical sense to a desired target area.* For instance, as may be concluded from section 5.1.5, the injection of large volumes, even at *low pressure* and *flow rate*, does not in any way guarantee a favorable spatial distribution of the injected grout. (As mentioned there, the numerical analysis indicates that, if a defined volume of grout is injected at low pressure under confined conditions, the grout will in principle migrate further than if the same amount of grout had been injected at a higher pressure.)

However, the implication of the above is that when a specified total amount of grout is to be injected for an adequate treatment, an increased number of injection rounds have to be executed in order to compensate – in some measure - for the reduced volumes per round.

Traditionally, grouting engineers often prescribe limitation of injection pressure with the pronounced good intent of controlling heave of the ground and damage to the environment. However, the fact is that locally and temporarily applied high pressure as such, deep down in a bore hole, is not likely to have much effect at all in terms of lift or other damage at the ground surface. Instead, the *decisive* factor, generating heave of the ground surface and related damage, is *the amount of grout actually forced into a formation*, and the manner in which the grout has been injected.

Hence, in grouting work, pressure as such is not a measure of either the result in terms of closure or of the risk of damage to the environment. The relevant stop criterion, especially when grouting according to the principle of hydraulic fracturing, must therefore be the *injected volume of grout per round*.

6.3 Stop criteria - <u>unconfined</u> conditions

When the maximum volume of grout that can possibly be accommodated in a formation at a certain pressure has been injected in the course of a grouting stage, the risk of resurgence of grout at the bedrock surface is imminent. As illustrated in Figures 5.6, 5.7 and 5.8, grout spread and grout consumption are from then on largely *independent* of the mechanical properties of the ground and evidently no longer in *direct* proportion to injection pressure.

The grouting strategy must therefore be adapted to the ambient conditions on the working site, such as for instance to the depth of the point of injection, to the nature and thickness of the soil cover overlying bedrock, to distance from damageable buildings, to sensitive environmental conditions etc.

Pressure stop criteria are, particularly under unconfined conditions, likely to lead to enormous *waste* and *uncontrolled spread of grout*.

Therefore, the statistical allocation of the grout to the intended areas of treatment, as well as prevention of undesired waste and spread of grout to the environment, must also in this case be controlled by *limiting the volumes of grout injected per stage or round*.

6.4 Pressure as a general stop criterion?

As mentioned in Section 4, permeability of rock is often based on core drilling as well as on water loss tests. The drilling methods for retrieving core samples usually necessitate flushing of the drill bit by water, whereby *fine* particles and *grains* from *soil filled crevices* become poorly represented in the boring logs. Because of this, there is a notorious tendency to overestimate the void volume and the permeability of rock - a fact, which in turn is likely to result in overestimating the expected grout take.

Also, water loss tests (at 10 bar) - often reflecting flow under *hydro-fracture* conditions - may lead to overrating the permeability - particularly of grouts based on cement.

Yet, forcing more grout into a formation than can be accommodated in the available volume of pressure expanded cracks and voids serves no good purpose, and inevitably results in long range grout migration outside the zone intended for treatment. The use of stop criteria *related to pressure*, as often done in current practice, may therefore entail enormous waste of grout and obvious but unnecessary risks of damage to environment.

In the author's opinion, pressure related stop criteria are clearly responsible for many *notable* case records of *unsuccessful grouting* work and of *unintentional*, undesired spread of grout into the surroundings.

But as grouting work is usually remunerated by the m^3 of injected grout, contractors in the trade are not very receptive to this argumentation.

6.5 Additional comments

- It is often maintained that hydraulic fracturing should be avoided because of the risk of heave of the ground surface. Yet, in reality, the general tendency is instead that hydraulic fracture primarily manifests itself in vertical or sub-vertical planes and therefore - especially in the beginning – tends to generate *horizontal* displacements, which are usually not even monitored in current practice. This applies of course especially when the horizontal in situ stresses are significantly lower than the corresponding vertical overburden stresses, which is often the case in the upper (near surface) parts of a rock formation. Normally, heave tends to occur in the final stages of a grouting program when, as a result of stress build-up, the horizontal stresses exceed those of the over-burden. Confer also the discussion in Section 5.3.

- The fact that high pressure grouting tends to open up existing cracks many times over their initial width entails that the requirements on stability and pene-trability of grouts are not likely to be the same for hydraulic fracture grouting as for low pressure permeation grouting.
  In a Master of Science Thesis at Lund University by Graad & Hedlund (1996), the best results in respect of grout absorption were actually reported for a rapid cement grout, the stability of which was foreseen and expected to be *less fa-vourable* for the objectives of the grouting tests. Otherwise, micro-cement grouts were mainly used. The grouting operations were in these tests almost exclusively documented as a series of hydraulic fracture events.

Appendix - Exemplification of deformation analysis

Introductory comments

In the domain of civil engineering, there exist of course an infinite number of grouting scenarios, to many of which widely different grouting strategies may have to be applied. The following examples of grouting strategy are therefore not, by any standards, meant to cover all conceivable grouting situations. The intent is to illustrate the importance of ground deformation to grout propagation and spread, as well as how structure mechanical phenomena can be accounted for in grouting procedures.

Owing to the fact that the erratic characteristics of sub-ground cannot in general be documented in sufficient detail, grouting work is by nature a stochastic issue. Because of this, sealing of sub-ground by grouting can seldom be realised by *one single* injection in some vital point. Instead, in order to achieve sufficient closure, injections have to be repeated two, three or more times in the vicinity of every location to be treated.

However, especially the first injection round is, to a greater extent than the subsequent rounds, likely to fill initially open larger cracks and voids. In addition, adjacent cracks parallel to the current action plane may decrease significantly in width due to horizontal pressure build-up, a phenomenon obviously affecting the effective E-modulus applicable to the ensuing injections.

The pressure required to attain the pre-set volumes of grout per round is therefore likely to increase considerably in the following injection steps, while the calculated grout takes designed to attain the target spread tend to decrease.

In the case of hydraulic fracture grouting, the volume of grout forced into the ground must be estimated on the basis of the pressure applied, as well as on the desired spread of grout with regard to the zone intended for treatment. Due attention should therefore be paid to increasing stiffness of the sub-ground as grouting operations progress.

These circumstances are taken into account in the examples given below.

Ground conditions are rarely possible to define with any higher degree of accuracy, and some of the necessary assumptions made in the following may seem arbitrary. Nevertheless, in the opinion of the author, the exercise demonstrates important aspects and phenomena in grouting, many of which are not generally considered in current practise.

In the subsequent analyses it is important to note that the pressure denoted as p_o is an *overpressure* i.e.

- when grouting according to the principle of permeation, p_o signifies the part of the injection pressure required to maintain penetration flow or
- in the case of hydraulic fracturing, the portion of the injection pressure required to induce stresses *in excess of the prevailing states of in situ stress*. Hence, the active injection pressure as defined in Section 5.1.2 in the report is

  $p_i = \sigma_{\text{in situ}}(r_o) + p_o$

In respect of tension around the injection hole $\sigma_{\text{in situ}}(r_o) = K_o \cdot \Sigma\gamma \cdot \Delta z$

In terms of the gauge pressure p_G, p_o is defined in Section 5.1.2 as

$$\boldsymbol{p_o = p_G + \gamma \cdot H_G - p_{\text{in situ}} - \sigma_{\text{in situ}}(r_o) \approx p_G + \gamma \cdot H_G - (K_o + 1) \cdot \Sigma\gamma \cdot \Delta z}$$

As dissipation of excess pore water pressures is not likely to happen within the duration of one injection round, computations are made in terms of *total stress.* Effective stress analysis in the sense it is normally used in soil mechanics presumes significant water transport through the formation in short time and cannot be depended upon. In rock without fissures pore water dissipation is not a relevant issue.

1. Example 1

1.0 Basic data of the scenario:

The formation to be treated is geologically characterised as a sheared zone of folded sedimentary rock consisting of large, finely fissured blocs or plinths of tight rock material, separated by inclining faults of *high permeability* as shown in Figure A.1.

The nature of the formation is presumed to have been identified by geological survey, core drilling, pump or hydro-pressure tests (e.g. Lugeon tests). The following data have been recorded:

- Mean permeability has been estimated at $\boldsymbol{k = 10^{-3}}$ **m/s**. (Darcy's coefficient).
- The average E-modulus of the rock material has been determined in laboratory tests to be about $\boldsymbol{E = 2500}$ **MN/m²**.
- The maximum tensile strength of intact (not fissured) rock material has been found to be $\boldsymbol{f_t = 1.5}$ **MN/m²** in lab tests.

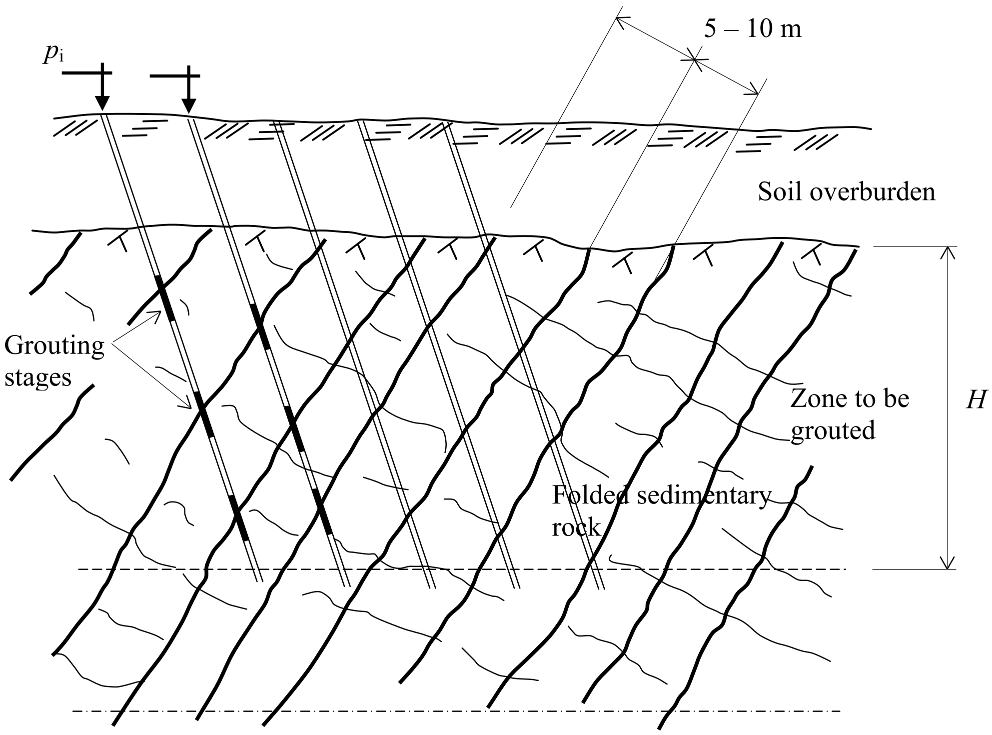


Figure A.1 Analysed rock formation in Case 1.

It is important to realise that the results of the following analyses should be regarded as rough assessments. This follows, among other things, already from the fact that the precise structure, the mechanical properties and the crack patterns of a rock formation are hardly ever adequately known. Nevertheless, in the opinion of the author it is better to perform reasonable assessments of the issues involved than making no analysis of any kind.

For the sake of comparison and simplicity, and in order to be able to compare the relative consumption of grout in the different scenarios, the perpendicular distance between bore holes (a) is taken to be constant i.e. ***a* = 5.0 m**, *although this may not* be the spacing used in reality. For instance, grouting by permeation would in general require much closer spacing of drill holes than when grouting by hydraulic fracturing. (However, with regard to the 'porosity' of the total volume to be sealed, the bore hole spacing is in principle immaterial from a grout consumption point of view.)

Furthermore, for the purpose of being able to compare the *total* volumes of grout take, the length L and the depth H of the zone to be grouted are assumed to be

$\boldsymbol{L \cdot H}$ **= 100·30 = 3000** m^2 in all of the examples.

The spatial location and orientation of dominant permeable faults or cracks have been identified and found to occur at intervals of 5 to 10 metres in Example (1). The width of the target zone intended to be treated is about 5 m on either side of the row of drill holes i.e. a total width of the grouted zone of $\boldsymbol{D_S}$ **=10 m**. The total volume of the treated zone is thus $L \cdot H \cdot D_S$ = 30 000 m^3.

Considerations prior to adopting a grouting strategy:

Departing from a mean permeability of $k = 10^{-3}$ m/s and a distance between pervious features of 5 to 10 metres, a rough assessment of the potential 'groutable' void volume may be made, see also Table A3 where all data are summarised.

Applying the diagram in Figure 4.3 or equation (4.1a), the assumed crack distribution (n_1 = 1/10 = 0.1 m^{-1}, and n_2 = 1/5 = 0.2 m^{-1}) and a permeability coefficient (k = 10^{-3} m/s) would correspond to the occurrence of <u>one</u> crack with a width t_1 = 2.6 mm every 10 m (or one crack with t_2 = 2.0 mm every 5 m). Presuming permeability to be 2– dimensional, (i.e. all *dominant pervious cracks* are parallel to *one* plane) the equivalent *mean void ratios* in the rock mass are then e_1 = 0.026 % and e_2 = 0.040 % respectively.

If the cracks are oriented in two mutually perpendicular planes, the permeability may be regarded as 3–dimensional, the formation being conductive in three directions. If this is the case, then the void ratios according to Figure 4.3 must be adjusted by adding the void ratios related to the permeability in two directions. For example, assume that the coefficient of permeability in the horizontal direction is $k = 10^{-3}$ m/s, and that the spacing between the cracks is 5 m. Then, if the corresponding values in the vertical direction are $k = 10^{-4}$ m/s and 0.5 m (n =2 m^{-1}) respectively, the *total void ratio* according to Figure 4.3 will be 0.040+0.085 = 0.125 %.

With the width D_S of the target zone to be treated = 2 · 5 = 10 m, the void volume per meter bore hole would be $a \cdot D \cdot e = 5.0 \cdot 10 \cdot e = 50 \cdot e$. For e_1= 0.026 % and for e_2 = 0.040 % the maximum volumes of grout that can be accommodated in the voids of the treated rock volume by genuine penetration are only 13 and 20 litres per m^3 respectively. In the following, the greater volume (20 litres per m^3) is applied – i.e. a void ratio = 0.04%.

1. 1 Case 1A - Penetration grouting strategy

Motivation of grouting strategy:

Provided the location, orientation and frequency of the pervious faults in the rock formation are reasonably well known, grouting by permeation may be a viable strategy in view of the width of the few cracks and the large distance between them. Having located the permeable layers by water pressure tests, the packers should be placed so as to attain optimum permeation of these layers.

Procedure:

As the injection pressure, when grouting by the principle of permeation, should not exceed the in situ stresses considerably, the volume of grout to be injected must not be markedly greater than what can be accommodated in the voids and cracks of the treated area. Hence the grout takes should not significantly exceed the values corresponding to the void ratios calculated on the basis of permeability.

Under the assumptions made, the largest total volume of grout that can be absorbed within the entire treated zone would then be about $L \cdot H \cdot D_S \cdot e_2$ = $100 \cdot 30 \cdot 10 \cdot 0.040/100 = 12\ m^3$.
(Note: The grout consumption in projects of this size is normally far greater than 12 m^3 indicating that - unless closed cavities having no impact on permeability are present - true penetration grouting is then not likely at stake. Evidently, hydraulic fracturing accounts for the considerably higher grout consumption, to which we are accustomed in current grouting practice.)

With a crack spacing of 5 m only 100/5 = 20 cracks are present, which means that in total 12/20 = 0.600 m^3 crack = 600 litres/crack can be absorbed.

Grouting pressure

In the current case, considering that the maximum tensile strength of the sedimentary rock is 1.5 MN/m^2, an overpressure in excess of 15 bar would fracture even intact rock.

Yet, hydraulic fracturing may readily take place at even *lower overpressure* as a result of pressure-induced, opening and expansion of existing cracks and fissures – 'the jacking effect'.

Therefore, if the principle of *true permeation is to be maintained*, the sustained pressure during the grouting operation may not significantly exceed that of the overburden or the in situ horizontal principal stress, which at a depth of e.g. 20 m only amounts to some 530 kN/m^2 =5.3 bar.

Conclusion: Excluding void volumes not contributing to permeability, the maximum total grout consumption by genuine permeation would be 12 m^3 or 600 l/crack corresponding to the mean void ratio of 0.04 % and a maximum grout pressure of about 0.8 MPa = 8 bar.

1.2 Case 1B - Grouting strategy based on hydraulic fracturing

Motivation for choice of grouting strategy:

For comparison, *deliberate hydraulic fracturing* is applied also to Example 1.

This grouting strategy is particularly favourable, when orientation and spacing of the pervious seams are of a random nature and not very well mapped in advance, which, incidentally, is normally the case.

Furthermore, the transmissivity of sedimentary rock structures is often governed by discrete, random and widely scattered sources of leakage. The prospects of striking such permeable features by low pressure grouting may be very unfavourable with normal and economical drill hole spacing. Even when a drill hole happens to be close to an isolated water conductive channel, connection by low pressure grouting is highly uncertain. Under such conditions, grouting at high overpressure is likely to be more successful.

Procedure:

As in this case, hydraulic fracturing is deliberate, not only the claquage pressure but also the sustained injection pressure must exceed ambient in situ stresses. In cases when for instance a TaM stage turns out to be totally confined by intact rock or in material impenetrable by cement grout, access to adjacent more open crack systems must be established by widening of existing fissures or by new fractures. (Cf Equation 5.1) The claquage pressure should therefore exceed the tensile resistance of the intact rock material i.e.

$$p_{\text{claquage}} \approx \Delta\sigma_{\text{claquage}} > f_{\text{t,rock}} + \sigma_{\text{in situ}}$$

For instance, at a depth of 20 m and with a tensile rock resistance of 1.5 MN/m^2, the minimum claquage pressure in terms of *total stress* may be in the order of [1.5+20·(26.5)/1000] = (1.5+0.530) = 2.03 MN/m^2 = 20.3 bar. However, the sustained ‘post fracture pressure’ during injection is usually much lower but must remain in excess of the in situ stresses, which are usually related somehow to the weight of overburden.

Choosing a grouting pressure p_i of 3 MPa = 30 bar then the overpressure with regard to weight of overburden is $p_o = p_i - \sigma_{in\ situ} \approx 3.0–0.53 \approx 2.5$ MPa.

As the injection pressure is unlimited, grout spread has to be controlled by limiting the amount of grout injected per round or stage. Assuming that grout enters a dominant crack from a TaM or EOC stage, then the spread of grout may be estimated by means of the ratio of E-modulus to overpressure, (i.e. $E/p_o = 2500/2.5 = 1000$). With an intended grout spread diameter of $D_S = 10$ m, the diagram on Figure 5.5b (or Equation 5.9) defines the maximum pressure induced volume of grout per round to 230 litres. If four rounds are taken to be required to cover the total depth of 30 m of the treated zone, the total grout take due to deformation in one crack will be some 4·230 = 920 litres. According to Case 1A, true penetration grouting would absorb 600 litres/crack (at 5 m crack spacing).

Figure 5.5c (or Equation 5.7a) gives with $D_S = 10$ m a maximum growth of crack width of 7.3 mm. This means that the crack width during ongoing injection increases from 2 to 9.3 mm, i.e. in the current case by a factor of about 4.6. The importance of this dramatic widening of the initial crack to grout propagation can hardly be overestimated.

(After the consolidation imposed by the elastic rebound of the rock mass, the width of the grout-stone intrusion is reduced by a measure largely depending on the water/cement ratio of the grout used).

Conclusions: By applying hydraulic fracturing, a total grout volume per crack of 920+600 = 1520 litres is injected already in the first four grouting rounds i.e. an increase of the volume injected by mere penetration by some 150%.

Yet, the impact on the final grout consumption is difficult to define unambiguously, as hydraulic fracturing normally generates horizontal pressure build-up, a phenomenon likely to affect the crack width of adjacent crack systems as grouting goes on.

Moreover, in the current case there is little reason to believe that the resulting closure would end up being better by grouting at low overpressure than at higher pressure.

The risk of grout spreading into the environment is avoided by limiting the grout takes per injection round.

It is important to observe that the effect of *consolidation* on the grout intrusions due to the *elastic rebound* of the rock mass is likely to result in reduced water/cement ratios owing to bleeding *rendering superior grout-stone quality* as compared to genuine penetration grouting.

Hydraulic fracturing is a viable option also in a scenario like the one defined in Case 1, characterised by previously identified, widely spaced pervious faults or large cracks.

2. Example 2

2.0 Basic data of the scenario:

In example 2, the formation subject to grouting treatment constitutes geologically a sheared zone of sedimentary or igneous rock, characterised by a close pattern of cracks of approximately the same width. The mean distance between cracks and fissures has been documented to be about 0.5 m. Discrete features with higher permeability *may or may not exist*. (See Figure A.2)

The nature of the formation has been identified by geological survey, core drilling, pump or hydro-pressure tests (e.g. Lugeon tests) generally indicating a rather uniform permeability. The following data have been recorded:

- Mean permeability has been estimated at $\boldsymbol{k} = \mathbf{10^{-4}}$ **m/s**. (Darcy's coefficient)
- The average E- modulus of the rock material has been determined in laboratory tests to be about $\boldsymbol{E}$ **= 2500 MN/m²**. In Case 2B, the E-modulus of the igneous rock is **30000 MN/m²**.
- The maximum tensile strength of intact (not fissured) sedimentary rock material has been found to be $f_{t,sed} = 1{,}5$ MN/m². The tensile strength of the igneous rock is $f_{t,ign} = 3.0$ MN/m².

As before, in order to be able to compare the relative consumption of grout in the different scenarios, the perpendicular distance between bore holes (a) is taken to be constant $\boldsymbol{a}$ **= 5.0 m**.

Considerations prior to adopting a grouting strategy:

Departing from a mean permeability of $k = 10^{-4}$ m/s and a mean distance between cracks of 0.5 metres ($n = 2$ m^{-1}), a rough assessment of the potential grout take by permeation is made.

Applying the diagram shown in Figure 4.3, the assumed crack distribution and permeability coefficient ($k = 10^{-4}$ m/s) would correspond to the occurrence of two

0.425 mm wide cracks every metre. The equivalent *mean void ratio* in the rock mass is then $e = 0.085$ %.

Now, if the permeability is taken to be equal in all directions, then - according to the **discussion in** Section 1.0 above - the void ratio from Figure 4.3 should be doubled, i.e. $e = 0.17$ %.

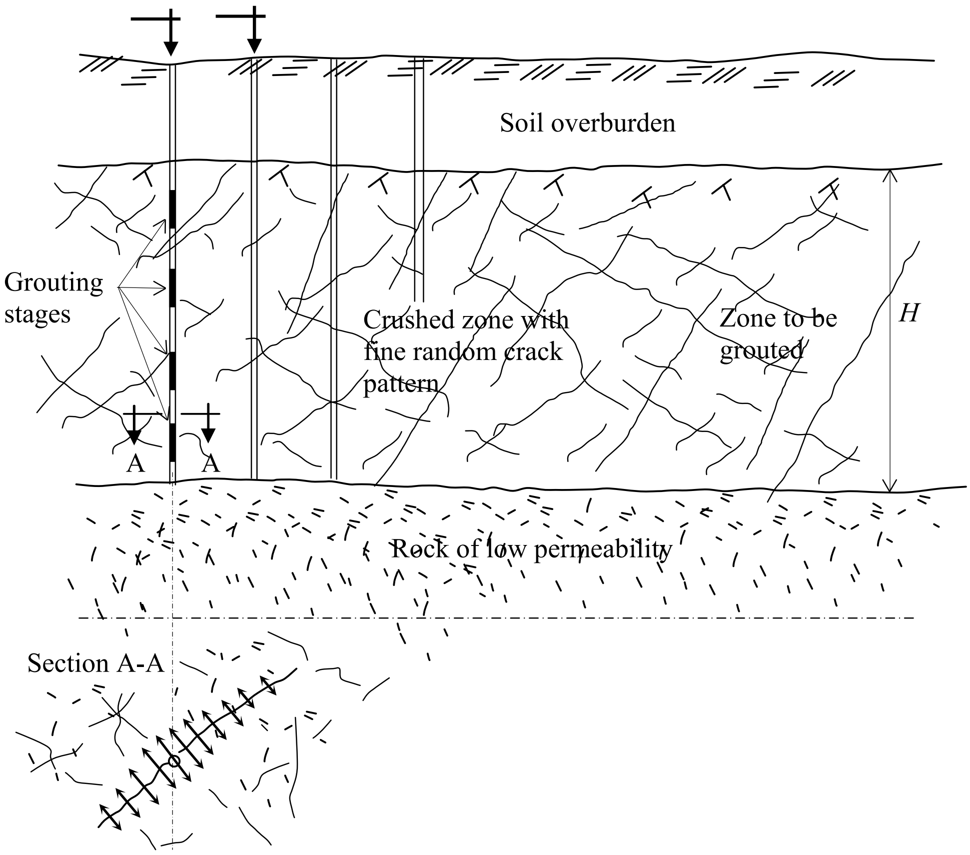


Figure A.2 Analysed rock formation in Case 2. Grouting in each bore hole is presumed to be carried out in 6 stages of 5 m length.

The width D_S of the target zone to be treated is $2{\cdot}5 = 10$ m. For $e = 0.17$ %, the maximum void volume per cubic metre of the treated area is 1.7 litres. For the entire zone to be grouted, the total grout take is $0.0017{\cdot}30000 = 51$ m^3, which incidentally is considerably more than the 12 m^3 of grout absorbed in Example 1, Case 1A.

However, for the rock volume to be grouted from one of the six stages in one bore hole the corresponding grout take is only $51/(20{\cdot}6) = 0.425$ m^3 = 425 litres

2.1 Case 2A - Penetration grouting strategy

Motivation for **not adopting** this grouting strategy:

- Of the many cracks and fissures, some may be readily penetrable by cement based grouts, but very likely a *large portion of the crack system* will be *inaccessible* to such grouts.
- Furthermore, as the orientation and location of the cracks in the rock mass are randomly distributed, permeation grouting focussed on specific pervious features is *not likely to be a viable strategy.*
- If discrete water conductive veins do exist, the odds of striking them or connecting to them are adverse, unless the bore hole spacing is extremely tight.
- Moreover, because of insignificant pressure build-up in the rock mass, low pressure grouting is not likely to result in closure of cracks and fissures that are not penetrable or reached by cement based grout.
- Grout-stone quality will be inferior to that produced by high pressure grouting.

Conclusion: *Genuine low pressure permeation grouting is not likely to be a successful approach in Example 2.*

2.2 Case 2B - Grouting strategy based on hydraulic fracturing – sedimentary rock

Motivation for choice of grouting strategy:

Considering the problems listed above under Section 2.1, grouting by hydraulic fracturing should be a favourable strategy in the current case (Figure A.2). This option is particularly favoured by the fact that the orientation and spacing of the pervious seams are of a random nature and hardly possible to identify à priori. Also, many of the cracks – may be a major part - will simply *not be penetrable* by cement grouts under low pressure.

Additionally, if the transmissivity of the formation is actually dominated by local, widely scattered leakage paths – which in reality is mostly the case – the chances of striking such permeable features by permeation from widely spaced drill holes are unfavourable. Yet, applying hydraulic fracturing makes it possible to penetrate and widen certain cracks by grout under high pressure, thus providing access to adjacent as well as distant cracks and local permeable zones.

Furthermore, as fracturing initially – not least for structure mechanical reasons - tends to occur in vertical or sub-vertical action planes, horizontal pre-stress of the formation takes place entailing widening of the dominant action plane at the expense of the width of adjacent minor cracks that, anyhow, may *not be 'groutable'*.

Assumptions in Cases 2B and 2C:

First injection round: In the first round, 1/3rd of the total void volume is assumed to be filled with grout i.e. $\Delta e_1 = e_o/3 = 0.17/3 = 0.0567$ %. The elastic modulus of the formation is taken to be only 20 % of that of the intact rock due to the presence of cracks.

Second injection round: In the second round, another 1/6th of total void volume is assumed to be saturated with grout i.e. $\Delta e_2 = e_o/6 = 0.17/6 = 0.0283$ %. The elastic modulus is now believed to have increased to 40 % of the modulus of intact rock.

Third injection round: In the third round, an additional 1/9th of total void volume is presumed to be penetrated by grout i.e. $\Delta e_3 = e_o/9 = 0.17/9\% = 0.0189$ %. The E-modulus has now increased to 60 % of the modulus of intact rock.

Hence, after three injection rounds 1/3+1/6+1/9 = 61.1 % of the initial total void volume has been filled with grout. The *logic behind these assumptions* is that an important part of the initial void volume of 100 % is *not penetrable by cement based grout* and that some of it *simply vanishes* because of increasing compressive stresses induced by the grouting work.

The postulations above may seem arbitrary but are in fact necessary for demonstrating important grouting phenomena. The knowledgeable reader may of course rerun the exercise, substituting the values of the different parameters with values of his *own choice and experience*. Grouting work consists in repetitive stages and input data applying to the current site can be gathered as operations go on.

Procedure:

Hydraulic fracturing being intentional, not only the claquage pressure, but also the sustained injection pressure must exceed ambient in situ stresses. Access to adjacent crack systems is generated by opening of existing fissures or by new fractures.

As the permeability of the rock mass is fairly evenly distributed the grouting operation can be carried out according to the EOC method or preferably as longer TaM stages. However, if environmental conditions require special control of the grout spread, *it is recommended that the spatial location of the injections* be defined using *short stage* TaM grouting. Short distance between packers increases the chances of *forming jacking planes at known locations*, which is a condition promoting the reliability of the analysis of grout spread.

1st injection round

The grouting of each bore hole is assumed to be carried out in 6 stages, each being $H/s = h$ =30/6 = 5 m deep. Every grouting stage is therefore considered to relate to a rock volume of aD_Sh = 5·10·5 = 250 m^3.

In the first round 1/3 rd of the initial void volume is presumed permeated by the grout. The corresponding grout volume is then $\Delta V_e(1)$ = 0.0017·250/3 = 0.142 m^3 = 142 litres.

In view of the uniform conditions in Example 2, the first hydraulic fractures are to be expected in vertical or sub-vertical planes. Again, the claquage pressure should exceed the tensile resistance of the intact rock material i.e.

$$p_{\text{i, claquage}} = \Delta\sigma_{\text{claquage}} > f_{\text{t,rock}} + \sigma_{\text{in situ}}(r_o)$$

The minimum pressure at a depth of say 20 m required to balance the total in situ stress is 20·26.5/1000 = 0.53 MPa. The tensile resistance of the sedimentary rock being 1.5 MN/m^2, the claquage pressure p_c must at least be in the order of p_c = 1.50+0.53 = 2.03 MN/m^2 = 20.3 bar. However, once fracturing has taken place, the sustained 'post fracture' pressure $p_{\text{i,flow}}$ required to maintain grout flow at grout volume stop is normally lower.

Assume the post fracture injection pressure at the pre-set volume stop *is found to be* $p_{\text{i,flow}}(1)$ = 1.4 MPa = 14 bar in the first round

$$p_o(1) = p_{\text{i,flow}}(1) - \sigma_{\text{in situ}} = 1.4\text{–}0.53 = 0.87 \text{ MN/m}^2$$

Note: In the following, $p_{\text{i,flow}}$ denotes the final pressure (p_i), at which the pre-set '*volume stop*' criterion is achieved. Yet, $p_{\text{i,flow}}$ as such is *not* a pressure stop criterion.

As the injection pressure is unlimited, the grout spread must be controlled by restriction of the amount of grout injected per round or stage. Assuming that grout enters a dominant crack between the TaM packers, then the spread of grout in the action plane (the jacking plane) may be estimated by means of Figure 5.5b. (If long stage TaM or EOC grouting is applied, Figure 5.4b should be used.)

When estimating the displacement in the hydraulic fracture plane, the effect of cracks on the stiffness of the rock mass as grouting work proceeds should be considered. Because, evidently in the first injection round, compression of initially open cracks must result in considerable reduction of the effective E-modulus as compared to the modulus of the intact rock material. Following the assumptions made above, the E-modulus in the first round is 20 % of that of the intact rock = $0.20 \cdot 2500 = 500$ MN/m^2. The final overpressure p_o at the end of the grouting operation being 0.87 MN/m^2, then $E/p_o \approx 500/0.87 = 575$.

With an intended spread diameter D_S of 10 m, the additional volume of grout consumed per round can be estimated by means of Figure 5.5b (or by the relationship (5.8a) between injected volume and spread given in the diagram):

$$D_S = 1.631 \cdot (E \cdot V/p_o)^{1/3} \text{ i.e.} \qquad (5.8a)$$

$$V = (D_S/1.631)^3 \cdot p_o/E = (10^3/4.339) \cdot 0.87/500 = 0.401 \text{ m}^3 \approx 401 \text{ litres} \qquad (5.9)$$

For $E/\Delta p_o \approx 575$, the grout take in the *first round*, lest grout migrates outside the intended zone of treatment is 401 litres. The total grout take in the first round including permeation is thus $\Delta V = 142+401 = 543$ litres.

As six stages are presumed to be needed for covering the total depth of 30 m of the treated zone, the total grout take for the full height will be about $6 \cdot 543 = 3258$ litres.

For $E/p_o = 575$ and $D_S = 10$ m, Figure 5.5c or the Equ. 5.7a ($\delta_A = 0.728 p_o D_S/E$) gives a maximum crack width growth of 12.7 mm, implying that the total crack width during ongoing injection increases by a factor of about 30 from 0, 425 to 13.1 mm. The *dramatic nature of the widening* of the *initial crack* is rather insensitive to the presumptions made in the analysis, inferring that even strongly modified data would not in principle change this condition. Its importance to grout propagation and outcome of the grouting treatment can hardly be exaggerated.

2 nd injection round

In the *second round*, the fraction of initial void volume likely to be filled by grout is presumed to be 1/6th. Following in principle the computations in round 1, the portion of the grout take constituting permeation will be

$$\Delta V_e(2) = 0.0017 \cdot 250/6 = 0.071 \text{ m}^3 = 71 \text{ litres.}$$

Further, in the second round, the injection pressure p_i must probably be set higher than the value in the previous round of 1.4 MPa = 14 bar, but it is reasonable to assume that also the horizontal in situ pressure increases on account of pressure build-up in the 1st round. When estimating displacement in the hydraulic fracture plane, the effect on the required grouting overpressure p_o of the increasing stresses in the rock mass should be considered as grouting operations proceed.

A tenable approach may then be to monitor the pressure $p_{i,flow}(2)$ in the final phase of the 2nd injection round and then adjust this value with a certain fraction f of $p_o(1) = 0.87$ MPa in the first round as a measure of increased grouting resistance. The new overpressure p_o suitable for evaluating the deformation and the grout take in the second round would then be:

$$p_o(2) = p_{i,flow}(2) - f{\cdot}p_o(1) - \sigma_{in\ situ} \qquad (3)$$

Assuming, for instance that the new grout pressure needed to maintain hydraulic fracture conditions at 'volume stop' is found to be $p_{i,flow} = 2.7 = 27$ bar and that the ratio f is taken to be 0.5 then

$$p_o(2) = 2.7\text{-}0.5{\cdot}0.87\text{-}0.53 \approx 1.77 \text{ MPa} = 17.7 \text{ bar.}$$

In addition, the mean E-modulus of the rock formation has increased to 40 % of the lab test value of 2500 MN/m^2 in the second round i.e. $E = 1000$ MN/m^2.

For $E/p_o(2) = 1000/1.77 = 565$ and an intended spread diameter D_S of 10 m, the grout take per round relevant to displacement can be estimated by Eq. (5.9):

$$V = (D_S^3/4.339)p_o/E = (10^3/4.339){\cdot}1.77/1000 = 0.408 \text{ m}^3 \approx 408 \text{ litres/round}$$

The total grout take in the second round is then

$$\Delta V = 71+408 = 479 \text{ litres.}$$

As 6 stages are required to cover the total depth of 30 m of the treated zone, the total grout take in second four rounds will be 6·479 = 2874 litres.

Figure 5.5b (or the equation 5.1a $\delta_A = 0.728{\cdot}\Delta p_o{\cdot}D_S/E$) gives a corresponding crack growth of some 12.9 mm.

3 rd injection round

In the *third round*, the fraction of the initial void volume likely to be filled by grout is presumed to be 1/9th. Following in principle the computations in rounds 1 and 2, the part of the grout take constituting permeation will be

$$\Delta V_e(3) = 0.0017{\cdot}250/9 = 0.047 \text{ m}^3 = 47 \text{ litres.}$$

The same approach as for the second round is applied, i.e. registering the *over-pressure actually required* at volume stop. Thus if $p_{i,\ flow}(3)$ is found to be 42 bar = 4.2 MPa, and the final value $p_o(2)$ in the previous round being 17.7 bar = 1.77 MPa, the new overpressure in the third round is:

$$p_o(3) = p_{i,flow}(3) - f{\cdot}p_o(2) - \sigma_{in\ situ} = 4.2\text{-}0.5{\cdot}1.77\text{–}0.53 = 2.79 \text{ MPa} = 27.9 \text{ bar.}$$

The value $p_o(3)$ = 27.9 bar is then used for predicting deformation and grout spread.

Also, the mean E-modulus of the rock formation has increased to 60 % of the lab test value of 2500 MN/m^2 in the third round i.e. E = 1500 MN/m^2.

For E/p_o = 1500/2.79 = 538 and an intended spread diameter D_S of 10 m, the grout take per round relevant to displacement is estimated by Equation (5.9):

$$V = (D_S^3/4.339)\cdot p_o\,(3)/E = (10^3/4.339)\cdot 2.79/1500 = 0.429 \text{ m}^3 \approx 428 \text{ litres/round}$$

The total grout take in the third round is then

$$\Delta V = 47+428 = 475 \text{ litres.}$$

As 6 stages are required to cover the total depth of 30 m of the treated zone, the total grout take in the third round will be about 6·475 = 2850 litres.*

(* **Note:** In the example – in order to avoid having to repeat the exercise six times over and over again – the $\sigma_{\text{in situ}}$ applicable to a stage depth at 20 m has been used for all of the six grouting stages. In a real case, of course, the $\sigma_{\text{in situ}}$ valid for each separate level should be used.)

Figure 5.5b (or the equation $\delta = 0.728\cdot p_o(3)\cdot D_S/E$) gives a corresponding crack growth of 13.5 mm.

The total amount of grout injected in the 3 rounds in six stages is then V = 3258+2874+2850 = 8982 litres, of which 6(142+71+47) = 1560 litres have been presumed to be absorbed by permeation of voids and cracks encountered by the hydraulic fracture action planes. The balance 7422 litres constitute the amount of grout consumed by *compression* of cracks and *displacements* in the rock mass.

Now, if grouting is carried out at six stages with a distance between the bore holes of 5.0 m, then the total amount grout injected in all of the 100 m long and 30 m deep zone to be sealed is (100/5)·8.982 ≈ 179.6 m^3. Of this value 31.2 m^3 is due to permeation and the remaining 148.4 m^3 due to *hydraulic fracturing displacement effects*.

(After the consolidation imposed by the elastic rebound of the rock mass, the widths of the grout-stone intrusions are reduced by a measure largely depending on the water/cement ratio of the grout used).

When sufficient horizontal pressure build up has taken place, horizontal hydraulic fractures (lifting) are likely to occur. However, in well documented grouting operations in sedimentary rock in the Malmö area, very little heave was recorded when a volume of more than 100 m^3:s of grout was injected within an area in order of 10 by 10 m^2. Slight heave was only observed in the final stages of the grouting work.

Conclusions:

The overall grout consumption in Case 2B amounts to 179.6 m^3, which is consistent with 0.60 % of the total volume of the treated zone. This is *considerably more* than the 0.17 % constituting a measure of the initial void volume, which has partly been penetrated by grout, and partly vanished by compression of not 'groutable' cracks. The balance represents lateral displacement induced by grouting pressure.

Obviously, the analysis made above only presents rough assessments of the studied parameters. Nevertheless, in the opinion of the author, the exercise offers good insight, based on structural analysis, of what actually takes place in the ground when performing repeated injections of grout under high pressure.

Applying this type of analysis in real grouting jobs will in due course provide *a data base of more accurate input parameters*. The repetitive character of grouting work will even allow establishing input data during an ongoing project.

The study also allows rational estimates of the necessary limitation of grout takes per round, rendering a reasonable control of unwanted spread of grout into the environment.

In cases where such control is immaterial, the analysis prevents the wasting of grout far outside the areas intended be sealed.

2.3 Case 2C - Grouting strategy based on hydraulic fracturing – igneous rock

Case 2B applied to sedimentary rock with an intact E – modulus of 2500 MN/m^2. For comparison the exercise in Case 2B is repeated with the sole difference that the formation consists of igneous rock having greater strength and stiffness characteristics as defined by a tensile stress resistance of 3.0 MN/m^2 and a modulus of elasticity of 30000 MN/m^2. All other parameters in Case 2C are presumed to be identical with those in Case 2B. Hence, definitions, motivations and assumptions are not recapitulated in every detail.

Motivation for choice of grouting strategy: See Case 2B

Assumptions in Case 2C: These are the same as those in Case 2B

First injection round: In the first round, 1/3rd of the total void volume is assumed to be filled with grout i.e. $\Delta e_1 = e_o/3 = 0.17/3 = 0.0567$ %. The elastic modulus of the formation is chosen to be only 20 % of that of the intact rock due to the presence of cracks.

Second injection round: In the second round another 1/6th of total void volume is assumed to be saturated with grout i.e. $\Delta e_2 = e_o/6 = 0.17/6 = 0.0283$ %. The elastic modulus is now believed to have increased to 40 % of the modulus of intact rock.

Third injection round: In the third round an additional 1/9th of the total void volume is assumed to be filled with grout i.e. $\Delta e_3 = e_o/9 = 0.17/9 = 0.0189$ %. The elastic modulus has now increased to be 60 % of the modulus of intact rock.

Hence, after three injection rounds 1/3+1/6+1/9 = 61.1 % of the initial total void volume has been filled with grout. The logic behind these assumptions is partly that some of the initial void volume of 100 % is *not penetrable* by cement based grout and partly because some of it *vanishes due to increased compression* induced by the grouting work.

Procedure:

Hydraulic fracturing being intentional, not only the claquage pressure, but also the sustained injection pressure must exceed ambient in situ stresses. Access to adjacent crack systems is generated by opening of existing fissures or by new fractures.

As the permeability of the rock mass is fairly evenly distributed the grouting operation can be carried out according to the EOC method or preferably as longer TaM stages. However, if environmental conditions require special control of the grout spread, it is recommended that the spatial location of the injections be defined using *short stage* TaM grouting. Short distance between packers increases the prospect of *forming jacking planes at known locations*, which is a condition promoting the reliability of the analysis of grout spread.

1st injection round

As before, the grouting of each bore hole is assumed to be carried out in 6 stages, each being $H/s = 30/6 = 5.0$ m deep. Every grouting stage is therefore considered to relate to a rock volume of $a \cdot D_S \cdot h = 5 \cdot 10 \cdot 5.0 = 250$ m^3.

One third of the initial void volume is presumed permeated by the grout. The corresponding grout volume is then as previously

$\Delta V = 0.0017 \cdot 250/3 = 0.142\ \text{m}^3 = 142$ litres.

In view of the uniform conditions in Example (2), the first hydraulic fractures are to be expected in vertical or sub-vertical planes. The claquage pressure must exceed the tensile resistance of the intact rock material i.e.

$p_{\text{i, claquage}} > f_{\text{t,rock}} + \sigma_{\text{in situ}}$

Again, the minimum pressure at the depth of 20 m required to balance in situ stresses is 20·26.5/1000 = 0.53 MPa = 5.3 bar. The tensile resistance of the igneous rock being 3.0 MN/m^2, the claquage pressure p_c must at least be in the order of p_c= 3.0+0.53 = 3.53 MN/m^2. However, once fracturing has taken place, the sustained 'post fracture' pressure during injection is normally lower.

Assume the post fracture injection pressure at grout volume stop *is registered* as being p = 2.0 MPa = 20 bar.

In the first round $p_o(1) = p_{\text{i,flow}}(1) - \sigma_{\text{in situ}}$= 2.0 – 0.53 = 1.47 MN/m^2.

Following the assumptions made previously, the E-modulus in the first round is 20 % of that of the intact rock = 0.20·30000 = 6000 MN/m^2. With an overpressure $p_o(1)$ at the end of the grouting operation = 1.47 MN/m^2, then $E/p_o \approx 6000/1.47 = 4082$.

With an intended spread diameter of 10 m, the additional volume of grout per round can be estimated by means of the relationship 5.9 (or by Figure 5.5b between injected volume and spread given on the diagram):

$\Delta V = (D_S/1.631)^3 \cdot p_o/E = 10^3/4.339 \cdot 1.47/6000 = 0.0565\text{m}^3 \approx 57$ litres

The grout take in the *first round*, lest grout migrate outside the intended zone of treatment is thus 57 litres. The total grout take in the first round is then

$V = 142+57 = 199$ litres.

As 6 stages are required to cover the total depth of 30 m of the treated zone, the total grout take in the first round will be 6·199 = 1194 litres.

For E/p_o = 4082 and D_S = 10 m, Equation 5.7a gives a maximum growth of the crack width of some 1.8 mm, which means that the crack width during ongoing injection increases from 0.425 to 2.2 mm – i.e. in the current case by an amplification factor of about 5. The *widening* of the *initial crack* is not very sensitive to assumptions in the analysis and its importance to grout propagation and outcome of the grouting treatment is paramount.

2 nd injection round

In the *second round*, the fraction of initial void volume likely to be filled by grout was presumed to be 1/6th. Following in principle the computations in round 1, the part of the grout take constituting permeation is then

= 0.0017·250/6 = 0.071 m^3 = 71 litres.

As before, the increase of rock stiffness with grouting progress is considered.

Using the approach (in Case 2B) of registering the final *overpressure actually required* to achieve the pre-set volume of grout and adjusting this value using the corresponding values in the first round, the new overpressure suitable for evaluating the grout take in the second round is defined. The overpressure $p_o(2)$ in the second round would then be:

$$p_o(2) = p_{i,flow}(2) - f{\cdot}p_o(1) - \sigma_{in\ situ}$$

Assume, for instance that the new grout pressure at volume stop is found to be $p_{i,flow}(2)$ = 3.7 MPa = 37 bar. The actual final value of $p_{i,flow}(1)$ proved to be 20 bar = 2.0 MPa giving a value of $p_o(1)$ = 2.0–0.53 = 1.47 MPa. With the ratio f = 0.5 we get

$$p_o(2) = 3.7–0.5·1.47– 0.53 \approx 2.435 \text{ MPa} = 24.35 \text{ bar}.$$

Also, in the second round the mean E-modulus of the rock formation has increased to 40 % of the lab test value of 30000 MN/m^2 i.e. E = 12000 MN/m^2, whence $E/p_o(2)$ = 12000/2.435 = 4928.

For E/p_o = 4928 and an intended spread diameter D_S of 10 m, the grout take per round relevant to displacement can be estimated by Equation 5.9:

$$V = (D_S/1.631)^3{\cdot}p_o/E = (10^3/4.339)·1/4928 = 0.0468 \text{ m}^3 \approx 47 \text{ litres/round}$$

The total grout take in the second round is thus

$$\Delta V = 71+47 = 118 \text{ litres}.$$

For the 6 stages required to cover the total depth of 30 m of the treated zone, the total grout take in the second round will be about 6· 118 = 708 litres.

Figure 5.5c (or the equation $\delta = 0.728p_o{\cdot}D/E$) gives a corresponding crack growth of some 1.5 mm. This implies an increase of the crack width by a multiple of about (0.43+1.5)/0.43 ≈ 4.5.

3 rd injection round

In the *third round*, the fraction of initial void volume likely to be filled by grout was presumed to be 1/9th. Following in principle the computations in rounds 1 and 2, the part of the grout take constituting permeation will be

$= 0.017 \cdot 250/9 = 0.047\ \text{m}^3 = 47$ litres.

The same approach of registering the *overpressure actually required* to initiate hydraulic fracture is used. The overpressure p_o suitable for evaluating the grout take in the third round is

$$p_o(3) = p_{i,\text{flow}}(3) - f \cdot p_o(2) - \sigma_{\text{in situ}}$$

Assume that the grout pressure at volume stop is now found to be $p_{i,\text{flow}}(3) = 47$ bar = 4.7 MPa. The actual final value of $p_{i,\text{flow}}(2)$ being = 2.435 MPa, the resulting $p_o(2)$ is then = 2.435–0.53 ≈ 1.90 MPa = 19.0 bar. Using 0.5 for the ratio f then

$$p_o(3) = 4.7–0.5 \cdot 1.90–0.53 \approx 3.22\ \text{MPa} = 32.2\ \text{bar}.$$

Also, the mean E-modulus of the rock formation has increased to 60 % of the lab test value of 30000 MN/m^2 in the second round i.e. $E = 18000$ MN/m^2.

For $E/p_o = 18000/2.9 = 5590$ and an intended spread diameter D_S of 10 m, the grout take per round relevant to displacement can be estimated by Equation 5.9:

$$V = (D_S/1.631)^3 \cdot p_o/E = (10^3/4.339) \cdot 3.22/18000 = 0.0412\ \text{m}^3 \approx 41\ \text{litres/round}$$

The total grout take in the third round is thus

$$\Delta V = 47+41 = 88\ \text{litres}.$$

For the 6 stages required to cover the total depth of 30 m of the treated zone, the total grout take in the third round will be about 6·88 = 528 litres.

Figure 5.5c (or the equation $\delta = 0.728 \cdot p_o \cdot D_S/E$) gives a corresponding crack growth of some 1.3 mm, i.e. a crack amplification factor of about (0.43+1.3)/0.43 = 4.0.

The total amount of grout injected in 3 rounds in six stages is V = 1194+708+528 = 2430 litres*, of which as in Case 2B, 1560 litres have been presumed to be absorbed by permeation in cracks encountered by the hydraulic fracture action planes. The balance, 870 litres, represents the amount of grout consumed by compression of cracks and displacements in the rock mass.

(*** Note:** In the example – in order to avoid having to repeat the exercise six times over and over again – the $\sigma_{\text{in situ}}$ applicable to a stage depth at 20 m has been used for all of the six grouting stages. In a real case, of course, the $\sigma_{\text{in situ}}$ valid for each separate level should be used.)

Now, as grouting is carried out at six stages with a distance between the bore holes of 5 m, the total amount of grout injected in all of the 100 m long, 30 m deep and 10 m wide zone to be sealed is V_{total} =(100/5)·2.430 ≈ 48.6 m^3. Of this volume permeation accounts for 31.2 m^3. The balance of 17.4 m^3 is due to *hydraulic displacement effects*.

Synopsis

Obviously, the analysis made above is based on rough assessments of the studied parameters. Nevertheless, the exercise offers insight, based on structural analysis, as to what actually takes place in the ground, when performing repeated injections of grout under high pressure.

The study indicates a difference in total grout consumption due to stiffness between the two formations of sedimentary and the igneous rock of 179.6–48.6 = 131.0 m^3. As the volume of permeated grout has been presumed to be same in the two formations with identical crack pattern, all of the deviation relates to the difference in stiffness between the sedimentary and the igneous rock.

Table A.1 Absorbed grout as percentage of the treated rock volume (LHD_S = 100·30·10 = 30000 m^3).

	Case 2B	**Case 2C**
	Sedimentary rock	Igneous rock
Mean stiffness	E = 2500 MN/m^2	E = 30000 MN/m^2
	[%]	[%]
By penetration (permeation)	0.104	0.104
By hydraulic fracturing	0.495	0.058
Total grout take in %	**0.599**	**0.162***
Initial void ratio of rock	0.170	0.170

* This figure indicates that TaM grouting in the stiff formation should have been carried out at more levels – for instance, instead of on every 5 m of the bore hole, the vertical distance between TaM stages should preferably have been chosen to be 4 m or less.

The total grout consumption in terms of a percentage of the volume of the entire treated zone is shown in Table A.1. A notable observation to be made here is that, while injecting an amount of grout corresponding to about 0.6 % of the volume of the treated zone would be *justified in the soft sedimentary rock*, the same treatment in the *igneous rock* formation would be *totally out of proportion*. It would, namely, entail massive waste of grout (≈ 131 m^3) and *uncontrolled spread* of grout into the environment entailing possible damage.

Effect of increased grout volume

It may be of interest to study the following issue:

What would be the response in respect of *grout spread* if, instead of injecting a total of 179.6 m^3 and 48.6 m^3 in Cases 2B and 2C respectively, an amount of 300 m^3 grout had actually been injected in both cases. As has been emphasised above, as well as in the main report, injecting excess quantities of grout entails, apart from waste of material, a high probability of ***unconfined grouting response*** with a correspondingly greater risk of uncontrolled spread far into the environment.

This would imply that the volumes of grout to be accommodated by displacement would increase:
- in Case 2B by a factor of (300–31.2)/(179.6–31.2) = 1.811
- in Case 2C by a factor of (300–31.2)/(48.6–31.2) = 15.45

Using the relationships between grout take per round and grout spread, the implications would be as shown in Table A.2 assuming confined conditions.

Table A.2

Case 2B: (injection round)	Volume/round ΔV [m^3]	E/p_o	Grout spread D_S= $1.631(EV/p_o)^{1/3}$ [m]	Displacement δ_A= $=0.728 \cdot D_S p_o/E$ [mm]
1 st:	1.811·0.401=0.726	575	12.2	15.4
2 nd:	1.811·0.408=0.739	565	12.2	15.7
3 rd:	1.811·0.428=0.775	538	12.2	16.5
Case 2C:				
1 st:	15.45·0.057=0.881	4082	25.0	4.5
2 nd:	15.45·0.047=0.726	4928	24.9	3.7
3 rd:	15.45·0.041=0.633	5590	24.9	3.2

Conclusion: If the *same amount of grout* (300 m^3) were to be injected in both of the studied rock formations under confined conditions, the spread of grout in the igneous rock having an *E*-modulus of E =30000 MN/m^2 would in principle be about *two times* the spread in the sedimentary rock with E = 2500 MN/m^2.

If, instead, EOC grouting had been applied, the corresponding difference in spread would be about *three times* that in the sedimentary formation.

The widening of cracks due to hydraulic fracturing would in the stiffer rock be only about 20 to 30 % of the corresponding value in the sedimentary rock. Yet, the injected grout volume would correspond to 300/30000 = 1.0 % of the zone intended for treatment. Comparing this grouting intensity with the given void volume of 0.17 %, representing the extreme maximum grout take if genuine permeation grouting were at stake, it is evident that uncontrolled spread of grout into the environment would inexorably take place – in particular as regards the stiff igneous rock.

Grouting according to the EOC method compared to TaM

In the exercises above, grout spread and crack widening has been based on concentrated TaM grouting on six distinct levels. (In practice, a closer spacing of grouting stages might preferably have been used). If, instead, continuous long stage EOC grouting had been applied, the values of grout consumption according to Figure 5.4b would be somewhat higher.

Assume for instance that the grouting work in Case 2C were to be carried out at the same grouting pressures in long EOC stages, maintaining the same restriction of 10 m regarding the spread of grout. Then, applying the equations in Figure 5.4 in Section 5, the grout consumption related to hydraulic fracturing effects would nominally increase by a factor of 1.80 – i.e. some 80 % increase.

However, as *in reality,* the EOC long stage grouting pressures would not have been quite the same as the ones applicable to a TaM procedure. The mentioned factor marking the difference between the TaM and EOC grout takes is therefore uncertain.

3. Final conclusions

A summary of the results of the examples is given in Table A3.

1) The grout consumption in the sedimentary rock is in Example 2 about 3.7 times that in the igneous rock for the same void ratio.

2) Applying the TaM method, a somewhat closer spacing between grouting stages than assumed in Case 2C would be recommendable.

3) Application of the analysis demonstrated in Example 2, (Cases 2B and 2C) in practical engineering will soon – i.e. even during an ongoing project – provide

useful experience with regard to the input parameters, thus improving the accuracy of the predictions. *The use of computers will allow the relevant analysis to be performed simultaneously with ongoing grouting operations.*

4) Genuine low pressure permeation grouting could theoretically result in a *maximum* grout take of 0.17 %, representing the initial void ratio. A more realistic number related to the 'groutable' crack volume would probably be 0.10 to 0.14 %, implying that any grout take in excess of the 0.17 % actually indicates the likely incidence of hydraulic fracturing events. This reasoning disregards local random closed cavities having no relevance to the transmissivity of the rock.

5) Cases 2A, 2B and 2C all relate to a permeability in all directions of $\boldsymbol{k} = 10^{-4}$m/s. Although some of the assumptions regarding basic parameters may be varied, it is *important to note* that the analysis indicates that - *even when applying hydraulic fracturing* - the amount of grout that can be accommodated in a crack system corresponding to this permeability ranges between only 0.16 and 0.60 %. The percentage is subject to the rigidity of the formation, the lower value applying to stiff igneous rock.

This implies that forcing, for instance, 1 % of grout (a number often encountered in practice) into any rock mass having a permeability of 10^{-4}m/s almost *inevitably leads to uncontrolled spread of grout into the environment*, and that in particular in stiff rocks of igneous origin.

Again, this conclusion presumes the absence of cavities that do not contribute to the conductivity or transmissivity of water.

Table A3. Summary of Examples

For all examples the volume of the grouted zone is the same:

Length * Height * Width = LHD = 100*30*10 = 30 000 m^3

The distance between the bore holes is also presumed to be the same a = 5 m

Variable / Example	**1A**	**1B**	**2B**	**2C**
Injection Method	Permeation	Fracturing	Fracturing	Fracturing
Rock type	Sediment. 2D	Sediment. 2D	Sediment. 3D	Igneous 3D
Permeability, k [m/s]	10^{-3}	10^{-3}	10^{-4}	10^{-4}
Modulus of Elasticity, E [MPa]	2500	2500	2500	30000
Tensile strength, f_t [MPa]	1.5	1.5	1.5	3
Crack distance, h/N [m]	5	5	0.5	0.5
Crack distribution n = N/h [1/m]	0.2	0.2	2.0	2.0
Mean crack width t [mm] from Eq. 4.1a	**2.00**	**2.00**	**0.425**	**0.425**
Mean void ratio in 2 dimensions, e_{2D} = t/a	**0.00040**	**0.00040**	*0.00085*	*0.00085*
Mean void ratio in 3 dimensions, e_{3D}	*0.00125*	*0.00125*	**0.00170**	**0.00170**
Crack volume per hole and m, v_{cr} = eaD [m^3/m]	0.020	0.020	0.085	0.085
Total crack volume / hole, $V_{cr} = v_{cr}H$ [m^3]	0.600	0.600	2.550	2.550
Total crack volume, ΣV = eLHD [m^3]	**12**	**12**	**51**	**51**
First round, Part of total volume, i	1	1	0.333	0.333
Number of stages, s	1	4	6	6
Volume per stage and hole, $V_1 = v_{cr} \cdot iH/s$ [m^3]	0.600	0.150	0.142	0.142
Chosen grout pressure, p_o [MPa]	0.8	2.5	0.87	1.47
Modulus of Elasticity E_1=E or 0.2E [MPa]		2500	500	6000
Relative stiffness, E_1/p_o		1000	575	4082
Grout volume / stage from Eq. 5.9, ΔV_1 [m^3]		0.230	0.401	0.057
Crack opening from Eq 5.7a, δ_1 [mm]		7.3	12.7	1.8
Grout volume / hole, $s(V_1 + \Delta V_1)$ [m^3]	0.600	1.520	3.258	1.194
2nd round. Part of total volume, i	0	0	0.167	0.167
Volume / stage, $V_2 = v_{cr} \cdot iH/s$ [m^3]			0.071	0.071
Chosen grout pressure, p_o [MPa]			1.77	2.435
Modulus of Elasticity E_2= 0.4E [MPa]			1000	12000
Relative stiffness, E_2/p_o			565	4928
Grout volume / stage from Eq. 5.9, ΔV_2 [m^3]			0.408	0.047
Crack opening from Eq 5.7a, δ_2 [mm]			12.9	1.5
Grout volume / hole, $s(V_2 + \Delta V_2)$ [m^3]	0	0	2.874	0.708
3rd round. Part of total volume, i	0	0	0.111	0.111
Volume / stage, $V_3 = v_{cr} \cdot iH/s$ [m^3]			0.047	0.047
Chosen grout pressure, p_o [MPa]			2.79	3.22
Modulus of Elasticity E_3= 0.6E [MPa]			1500	18000
Relative stiffness, E_3/p_o			538	5590
Grout volume / stage from Eq. 5.9a, ΔV_3 [m^3]			0.428	0.041
Crack opening from Eq 5.7a, δ_3 [mm]			13.5	1.3
Grout volume / hole, $s(V_3 + \Delta V_3)$ [m^3]	0	0	2.850	0.528
Total volume grout /hole [m^3]	0.600	1.520	8.982	2.430
Total volume of grout [m^3]	**12.00**	**30.40**	**179.6**	**48.6**

s = Number of stages per bore hole of height *H*

h = *H/s* = length of grouting stage

a = distance between injection holes

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