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GROUTING OF ROCK MASSES

Invited key note lecture by

Giovanni Lombardi Dr Eng. PhD h.c.

(with additional comments)

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Grouting of Rock Masses

Giovanni Lombardi[1]

Abstract

The grouting of rock used to be a quite empirical constructional technique aimed to improve somewhat the physical properties of the masses, which are actually part of the civil engineering project.
Different "grouting schools" or "ways of thinking" did develop and a considerable number of recipes were dictated by "great old men" and enforced later on by regulations of any kind.
Only in the last decades the grouting process started to be studied in a more scientific way and a number of well-founded results are now on hand.
Nevertheless, improvements are still possible and even required so to be put into practice in order to optimise the grouting process in adapting it better to the actual rock conditions as well as to the real objectives of the project.

Introduction

The theme I was asked to deal with was quite simply "Rock grouting". It covers a very wide field of theoretical and practical aspects so that some limitations had to be considered.
In addition, a certain number of concepts are understood differently by the various authors who treat this subject. A definition of a few fundamental notions are thus likely to be necessary in order to avoid at least some of the usual misunderstandings.
These differences are due to the fact that during the last two centuries different "school of grouting" did develop and drifted apart each from another mainly due to historical reasons.

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1 Giovanni Lombardi, Dr Eng. PhD.h.c., President Lombardi Engineering Ltd., Via R. Simen 19, 6648 Minusio (Switzerland), tel. +41-91-744 60 30, fax +41-91-743 97 37, info@lombardi.ch, www.lombardi.ch

In spite of the more scientific recent achievements, it appears, that research, studies and new improvements of theory and practice are still necessary and desirable to better fit the needs of the construction industry in the fields of foundation, rock mass stabilisation, tunnelling and construction of water retaining structures.

Due to personal experience, emphasis will be given to consolidations of foundations and to grout curtains for dams. For time and space limitations, the question of the optimal orientation of the boreholes in relation to the geological features, nor the drilling and grouting equipments will be dealt with in the following presentation.

The theme announced should thus be cut down and changed into "Cement grouting of jointed sound rock masses for dam foundations", which concept will be defined in more details later on.

The jointed sound rock mass

The first delimitation will be obviously against granular or fine soils, which present a high percentage of voids and thus a high deformability. They require the use of special techniques different from the ones we will deal with hereafter.
But, also highly weathered rock masses like for example hydro-thermally altered granite completely or partially turned to sand, or decomposed dykes, or completely crushed zones in faults as well as highly porous rock lay outside of the field of our discussion.
Karstic phenomena in sound rock require equally a special treatment as for example a filling of the cavities with mortar, concrete or other matters before the very grouting of the zone can take place.
On the other hand absolute massive unjointed rock masses cannot, nor need to be grouted.

In between these limits there are a great number of real cases of rocks, which are sound in the matrix - that means that they possess an appreciable ultimate compressive strength as well as noticeable deformation moduli - but are subdivided by strata, joints, cracks, fissures, faults or by any kind of discontinuities.
Indeed, these rock masses are more like a pile of blocks, which do fit perfectly each to another; their interfaces presenting properties of friction, cohesion and viscosity. Occasionally, some spotwise continuity of the rock across the joints can be observed.

A special kind of discontinuities needs a particular mention, which may be described as "potential joints". They are in fact weakness surfaces in layered or schistous rocks, which may open so easily that one doesn't know whether they were open from the beginning or whether they just did open during the grouting process. "Hydrofracturing" is then called in, of which we will talk about later on.

In the aforesaid conditions of a jointed, sound rock the net of discontinuity surfaces plays the most important role in the grouting process, or is even the only aspect to be seriously taken into account. So one should rather talk of "grouting the discontinui-

ties " than the rock. This explains also the fact that quite similar techniques can be used to repair cracks in concrete blocks (Turcotte 1994).
The "opening" of the joints, the nature of their surfaces, their stiffness, and frequency, their continuity as well as the interconnection between them represent a system - that means a complex of elements - which the grouting process must correctly take into account.

The RQD (Rock Quality Designation) (Deere 1968) was a first quite original and useful attempt to quantify the properties of the fissured rock masses, which depend on said discontinuities.
The empirical relationship between deformability and RQD is shown in **Figure 1**.

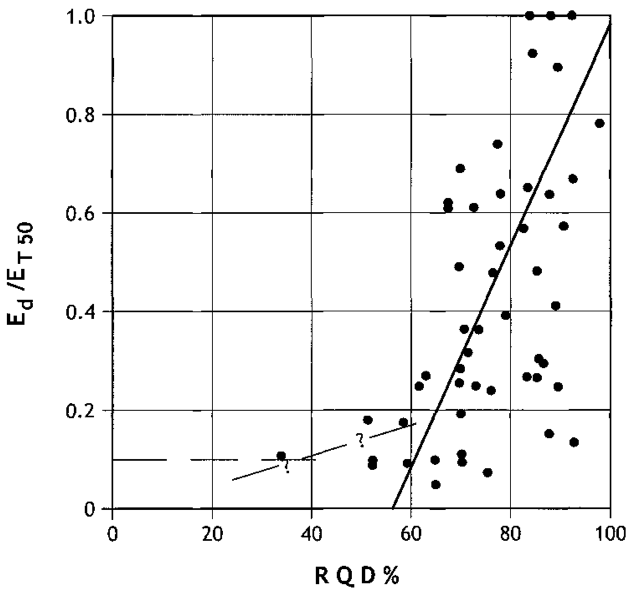

Figure 1: Relationship between RQD and a deformability factor defined as the ratio: modulus of deformability E_d to modulus of elasticity $E_{T\,50}$ (Coon and Merritt 1970).

The FES-Rock mass model

To correctly understand the grouting process, a model for the rock mass is thus necessary (e.g. Hässler 1992).
In recent times a number of such models were set up, but the so-called FES (Fissured, Elastic. Saturated) rock mass model appears to be still very useful (Lombardi, 1989, 1992)[2] to solve problems in different fields of rock mechanics (settlement, grouting, tunnelling, etc.).
For a simple uniaxial compression of a sample of dry rock loaded perpendicularly to the joints the relationship shown by **Figure 2** holds quite convincing.

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2 The FES Model was first set up and developed to solve the problems of the settlements occurred in 1978 at the Zeuzier arch dam in Switzerland due to the draining effect of an exploratory adit.

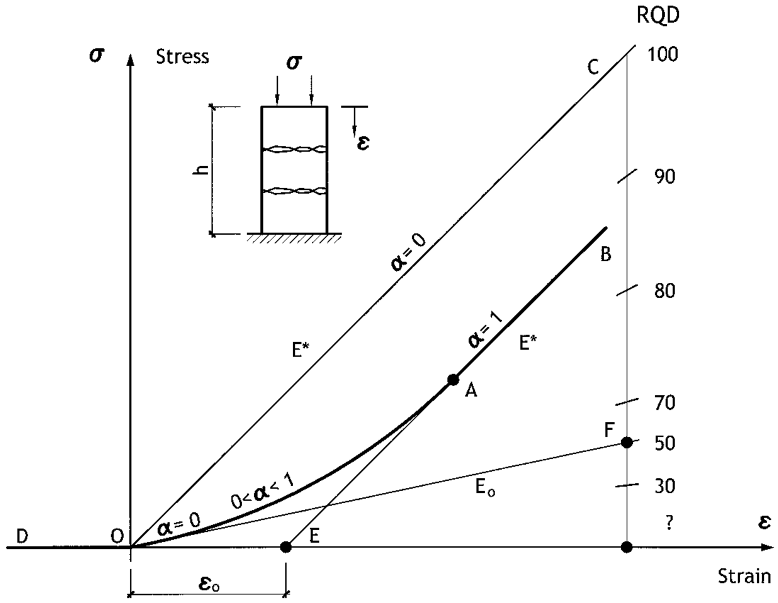


Figure 2: Strain stress relation for a dry fissured rock sample loaded perpendicularly to the joints.

OC : Elastic deformation of the matrix with $E^* = \dfrac{E \cdot (1-\varsigma)}{1-2\varsigma^2-\varsigma}$

OD : No tension (joint opening)

OF : Initial compression modulus E_0 for RQD = 50. (Figure 1)

OA : Actual progressive closing of the joints (α= closing rate)

AB : Possible elastic deformation after complete closure of the joints.

ε_0 : Total closing of the joints in contact (or possible joint aperture until loosing contact) referred to the height of the sample (h).

If an interstitial water pressure is considered, the FES model can be completed as shown by **Figure 3**. A point P on the graph corresponds to a given total stress, to a certain interstitial water pressure and to a degree of opening of the joints.
This simple representation considers an uniaxial homogeneous loading case, but, meanwhile the method has been developed to a complete 3-D Finite Elements Model (including the desired number of joint systems of any orientation and properties) apt to solve all the problems, which may be of interest in Civil Engineering. However, there is no need for our considerations to enter in more details.

Grouting matters

A number of matters can be used to consolidate jointed rock masses.
Among them: cements, with or without adjunction of ashes, fillers and silica fumes, resins, silicates and at least theoretically many other chemicals may be mentioned.
However, in the following only cementitious grouts will be discussed because they represent the most popular and economical way to improve the properties of the rock masses.
Different types of cement can be considered, which may differ one from another mainly in matter of granulometry and chemical resistance. Also the addition of dif-

ferent kind of chemical admixtures can be taken into account to improve the properties of the slurry or of the hardened grout.
To mention is finally the possible combination of different products in the same project.

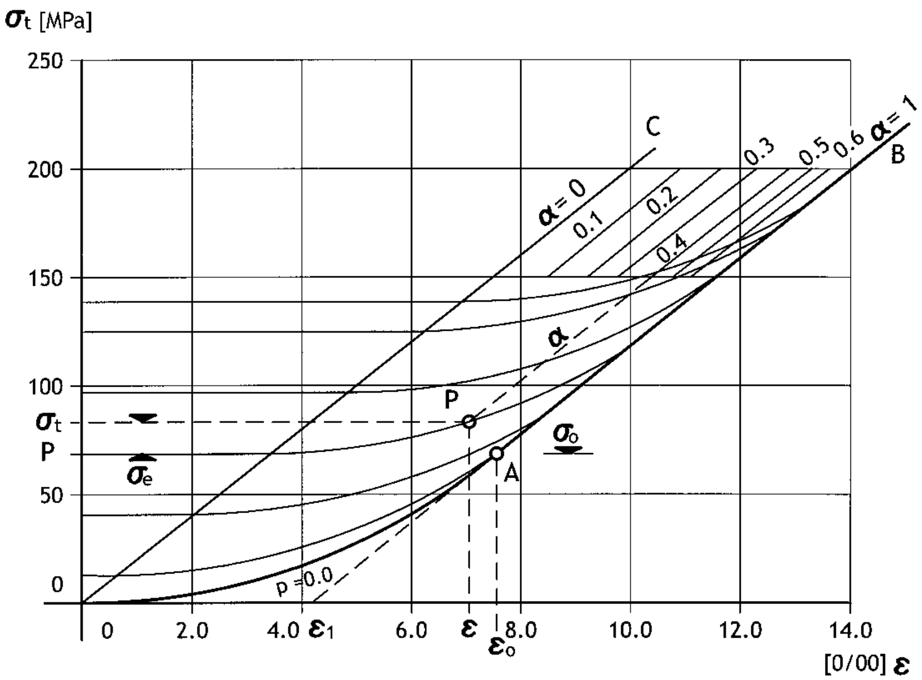


Figure 3: Example of a FES model for a given rock mass.

ε=strain; σ_t=total stress; *p*=neutral water pressure; α=degree of closure (α=1 fissures completely closed, α=0 fissures completely open):
A (ε_0, σ_0)= point of total closure at nil water pressure; σ_e=effective stress; *P*(ε, σ_t, *p,* σ_e, *a*) means: general strain, total stress, neutral pressure, effective stress and closing ratio at point *P*.

Cement grout

Definition of the water-cement ratio

Essentially the cement grout, or slurry, is a mixture of cement and water.
It is a well established rule in the concrete technology to define the ratio W/C that is "water to cement" by weight (e.g. W/C=0.5: means 0.5 kg water added to 1.0 kg cement), thus considering the cement as the base of the mix. Strange enough, in the field of grouting the habits are quite unstable. They refer to this ratio, but also to its inverse, from time to time they use the weight but also the volume of the components.
It is felt that a conformity with the concrete technology should be enforced by any mean to avoid additional confusions and that only the W/C ratio by weight should be used, as shown in **Figure 4** (Deere 1982, Houlsby 1982).
In the following only said water/cement ratio will apply.

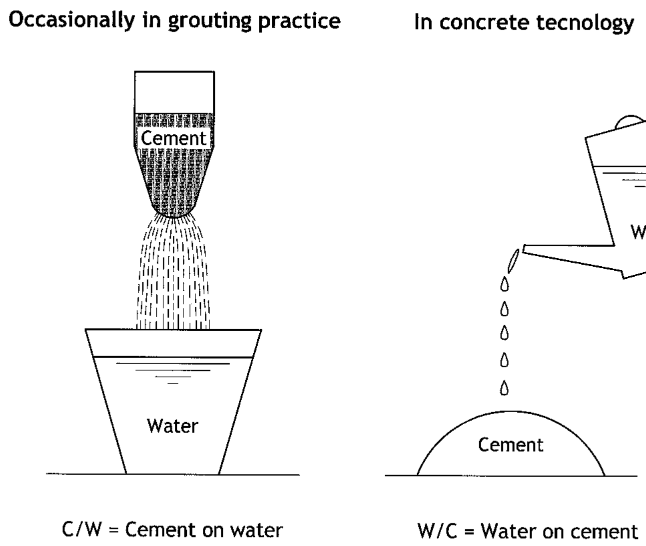


Figure 4: Definition of the water to cement ratio (W/C).

The role of water

The discussion about the type of slurry to use: "thick or thin", is quite old (Deere, Lombardi 1985).
Fundamentally the observed discrepancies in opinion are due to the various roles the water is supposed to play during the grouting process:

- The first role of the water is to hydrate the cement, and there is obviously no discussion on the necessity of this first amount of water (let say W/C=0.3 to 0.4);
- A second role, achieved by an additional quantity of water, is to produce a fluid mix with reduced cohesion and viscosity so that the slurry can propagate into the joints of the rock mass. This function can however be taken over, at least partially, by some admixtures like plasticizers. An additional of, let say, W/C=0.3 is nevertheless necessary and quite normal (depending on the Blaine Value).
  The scope of this additional water amount is, in fact, to avoid the direct contact between the single cement particles and thus the building up of an internal friction in the slurry. This friction would cause an exponentially increasing pressure along the joint so to stop immediately the progression of the slurry (Lombardi 1985).
- A third amount of water in the mix is intended to compensate for possible water losses during the grouting process due, for example, to water absorption by a dry rock. It is not possible to estimate the quantity of water required for this function. However, this risk does not exist below the ground water table and can be eliminated above it in saturating adequately the rock mass just before the grouting starts.
- An additional role attributed to water is that of a "joint opener" to facilitate the penetration of the slurry in thin cracks. But, this role can again be taken over by the existing ground water or the saturation water, which will automatically be pressed into the joints in front of the grout itself, without having to be included in the slurry and thus without affecting its properties.

When using a normal Portland cement a total water-cement ratio of the order of 0.6 to 0.7 is a practical minimum but also a quite adequate value in a great majority of cases. See **Table 1**. For micro-fine cements however a higher W/C ratio is required, e.g. up to 1.0 or 1.2 (Bremen 1997).

Dam	W/C	Fluidifier
Paute (Ecuador) upper part	0.6	Intraplast 1.4%
Alicurá (Argentina)	0.67	Intraplast 1.2%
El Cajón (Honduras)	0.7	Bentonite 0.2%
Clyde dam (New Zealand) 2nd part	0.6	Intraplast ~ 1%
El Chocón (Argentina) repairs	1.0	Bentonite 0.5%
Sir (Turkey)	0.7	Puzolanic cement, Mistra 1%
	or 1.0	Bentonite 1.2%
Katze (Lesotho)	0.59	Cement + ash. Conplast 1.5%
Pichi Picún Leufu (Argentina)	0.7	various
Potrerillos (Argentina)	0.7	Rheobuild/Viscocrete 0.7-0.8%
Aït Hamou (Morocco)	1.0	Bentonite 2%
Casecnan lower tunnel (Philippines)	0.63	Intraplast 1%

Table 1: Few examples of unique thick mixes used in a number of important dams and tunnels (sometimes: mainly used mix).

In spite of these considerations, there is still a historical concept often referred to, which considers the water as being mainly a vehicle for transporting the cement grains and introducing them into the joints of the rock mass (also for washing them in).
It is also often believed that the penetration of the grout will be the higher, the more water is added. However, a difference must be made between useless penetration of water and desired penetration of cement grains into the joints.

Thin to thick mixes

Said historical concept led to develop the recipe of progressive thickening of the grout mixes. A number of quite personal rules were developed. They are of the kind: "you inject 200 litres of a stained water of the type W/C=20, then 200 litres of coloured water of the type W/C=10, then again 200 litres of a dirty water with a ratio W/C=5, and so on, until you possibly reach, by chance, a real grout mix of the order of more or less W/C=1".
The basic idea is to try to start the grouting with something like water of very low yield stress and to increase its cohesion step by step using liquids of higher and higher viscosity until a thick mix is possibly injected in the joints.
In the background of these recipes there is the mentioned idea that thinner mixes will enter into finer cracks better than thicker can do.

The main difficulty with this, let say "classical", procedure is firstly that one doesn't know from the beginning when the process will end and thus which mix will be the last one and consequently which will be the quality of the works carried out.

In fact any stage will be grouted with a mix of different quality.
Additionally, the mixing of the different slurries in the joints is quite erratic and absolutely unpredictable.
So, the writer could observe, for example, in an adit excavated in an already "grouted" rock mass, that the lower parts of the vertical joints were filled with a white flour formed by loose hydrated cement grains, while their upper parts were simply void. Only "coloured water" had been used to grout the rock.

As proved many times, the final strength and the chemical resistance (durability) of the mix harden in the joints decreases sharply with its water content, that is with the excess of water introduced in the slurry (Houslby 1982).
Also it is not true that the new thicker mix will wash away the thinner ones and replace them.
As shown by **Figure 5**, the velocity of penetration of the grout into a joint is not uniform. It is higher in the centre and reduced, even to nil, along the joint walls. Consequently, at the end of the grouting process the walls of the joints will be covered by weak, more or less white matter corresponding to the thin mixes, while the core of the joint will possibly be filled by a stronger grey cement body, resulting from the hardening of a thicker mix.

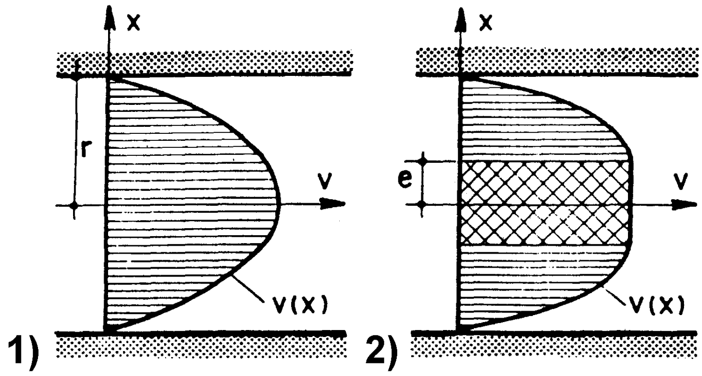


Figure 5: Velocity distribution of the grout in a joint.

1) Newtonian fluid (e.g. water);
2) Binghamian body (e.g. cement slurry)
   2.e = stiff kernel.

The non-uniform distribution of velocity makes that the former slurry sticks on the walls and the following one progresses further in the centre.

This layering of the grout may be often observed in boreholes cores drilled in a grouted rock mass.
Unfortunately, the mechanical and chemical properties of this "sandwich" are dictated by the outer weaker white layers, corresponding to the thinnest mix used. Obviously, some additional mixing of the different slurries may also take place, so the situation is not always that clear. (Also the presence of ground water or saturation water may produce such a layering of the grout set.)

In matter of penetration of the grout into fine joints, one may observe the **Figure 6** and see that there is an illusion to believe that thinner mixes will enter the joints better than the thicker do. In fact, the diameter of the grains and the clusters they may form in relation to the aperture of the joints are determinant, not the amount of water, which will have to flow around the grains and to continue its way in entering the joints. There is thus something like an inverse "filter criterium" (to the one used for fill dams) to be taken into account.

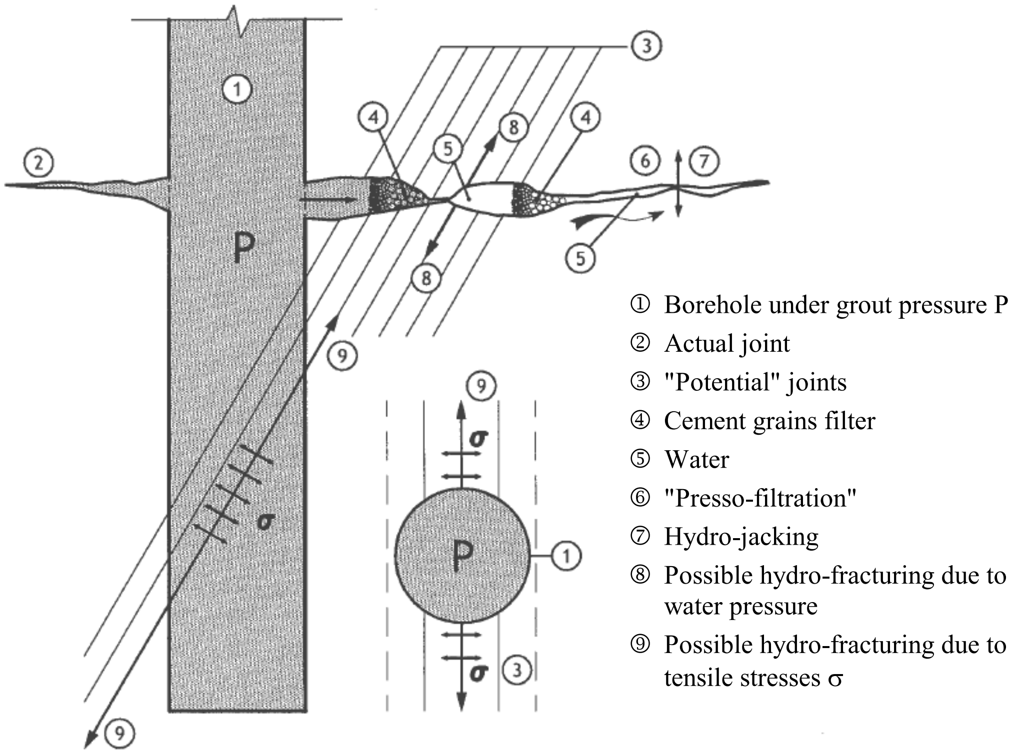


Figure 6: Penetration of a thin grout under pressure in a joint.

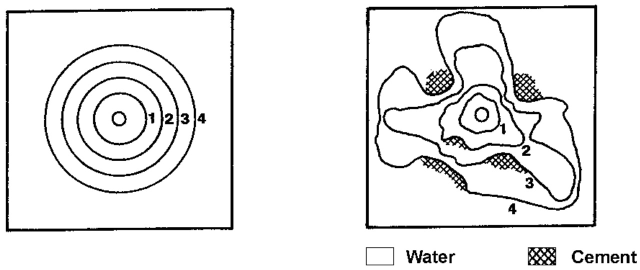
The theory of "presso-filtration" developed decades ago in order to explain this process and to try to find some assumed advantages in it, is apparently outdated.

In fact, to ease the penetration of the grout from the borehole into the joints there are a number of possibilities available as:

- adding a plasticizer to the grout to avoid the forming of clusters of cement grains due mainly to electrical forces;
- using finer cement (if economically justified);
- using higher grouting pressures, and
- counting, at least to some extent, on the pressurized water in front of a thick mix to open the joints and thus the way to the cement grains.

In conclusion, there are a number of good reasons, to abandon the so-called classical cumbersome technique of "thin to thick slurries series" and to go somewhat more modern grouting ways.

One of the main drawbacks with thin mixes is that they are not stable. Their progression in the joints is unpredictable (**Figure 7**). Also they will shrink more at setting than thicker mixes do.



Stable mix

Isotropic progression of the grout in four steps

Unstable mix

Irregular progression in four steps due to the water separation from the slurry

Figure 7: Experimental grouting tests in a joint of constant thickness (by Deere).

Stable mixes

The reasons to use thick mixes, called "stable mixes" (with limited bleeding, e.g. less than 5% in two hours) are numerous and related to the final properties of the grout mix after setting in the rock cracks, that is the quality of the grouting works carried out.
The main advantages over thin mixes are:

- complete filling of voids and joints by cement avoiding bubbles due to an excess of water;
- high mechanical strength (like in the concrete technology);
- reduced shrinkage potential so to avoid, or at least to limit, micro-cracks in the hardened grout;
- good or better bound to the rock surfaces (due also partially to the higher grouting pressures required);
- high resistance against chemical leaching out (Houlsby 1982);
- predictability of the grouting process.

Due to the good results obtained, at many dam sites, for the grout set in the cracks, it appears that stable grouts should be preferred. In order to simplify the process only one type of slurry should be used for any single project: the most adequate one, or the "best" one, which by definition is unique[3].

3 At the Paute Dam (Ecuador) the lower part of the grout curtain was grouted following the classical rules with thin to thick mixes. In the upper part only thick mixes were used. The grout cement taken per metre borehole was about the same; the amount of water uselessly injected in the lower part of the grout curtain was obviously much higher, leading to a weaker grout when set.

Properties of the slurry

To judge a slurry, two groups of properties must be considered, which are obviously not independent each from another.
The first group refers to the fresh grout, which is indeed a "suspension" of grains in water and is supposed to follow the Bingham's body law, while water is a Newtonian body (**Figure 8**).

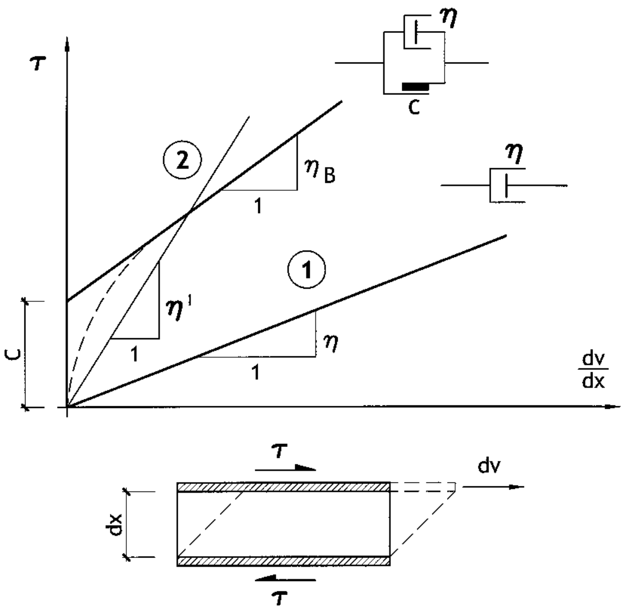


Figure 8: Rheology of a cement slurry vs water.

τ = shear stress, $\frac{dv}{dx}$ = shear velocity

① Newtonian body (viscosity only): (water) $\tau = \eta \cdot \frac{dv}{dx}$

② Binghamian body (cohesion and viscosity):

(approximation for grout mix) $\tau = c + \eta_B \cdot \frac{dv}{dx}$

- - - - real stable mix; c = cohesion (yield point)
η=dynamic viscosity; η_B=plastic viscosity; η^1=apparent viscosity

The first group includes mainly
- density;
- bleeding;
- viscosity;
- cohesion (yield point);
- set time.

The second group of properties concerns essentially the hardened grout and refers to
- mechanical strength,
- resistance to chemical agents, and to
- permeability.

The rheological properties of the fresh grout, which are mostly of interest and even decisive for the actual grouting process, can be influenced, modified or even determined in using a number of admixtures available today on the market.

Just to recall an already quite old example, **Figure 9** shows the influence the addition of water, bentonite or of a chemical admixture may have both on the final mechanical strength and on the apparent viscosity of the fresh slurry, or better said, on the flow time from a Marsh Funnel.
Starting from a W/C ratio of 0.67 (Point A) the addition of the Intraplast plasticizer results in a sharp decrease of the flow time but also of the mechanical strength.

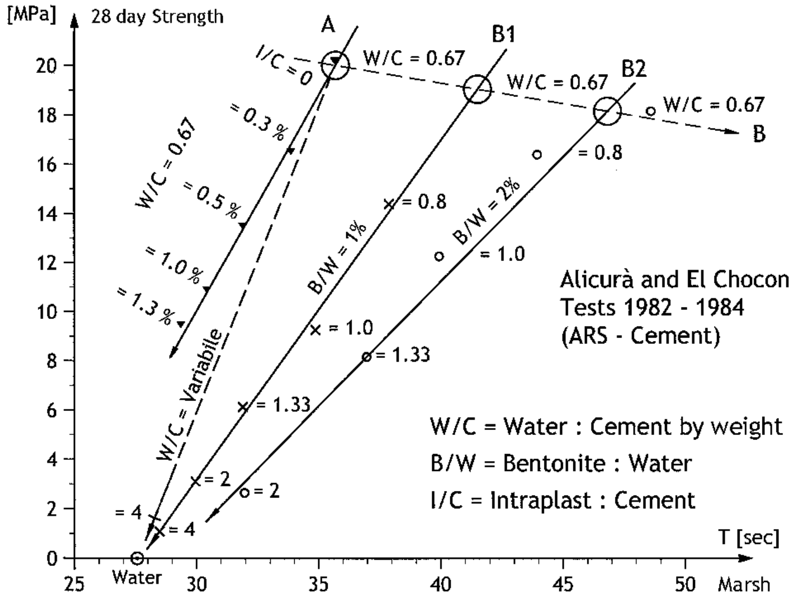


Figure 9: 28-day strength vs Marsh Funnel flow time (Am).

When using bentonite (B/W=1%) and starting again from a W/C ratio of 0.67 (Point B_1) the increase of this ratio has a similar effect tending to the viscosity and the "strength" of pure water, which is nil.
If a 2% B/W ratio is used, the reduction of the strength is quite the same but the flow time remains higher.

Consequently, the effect of added bentonite is just the contrary of that of lubrification it is often believed to be (Bremen 1997). Indeed, adding bentonite to a cement slurry is an indirect way to have the excess of water sought out of the mix[4].
For a given required final strength of the hardened grout, a much greater reduction of viscosity is obtained by the addition of an adequate plasticizer than of bentonite.
In the same way, a higher final strength is obtained for a given required Marsh flow-time.

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4 However, it may be the intention of the designer to get a cheap mix just apt to fill voids in the rock mass, while mechanical and chemical weaknesses of the result are not main concerns (Bremen 2001).

In conclusion, the required properties of the fresh mix to allow an easy penetration, which are mainly: cohesion, viscosity and set time can be obtained using adequate additives without jeopardising the final properties of the slurry and thus the quality of the grouting works carried out, as it happens when the W/C ratio is increased.

Obviously, as mentioned here-above, a minimum water content of the slurry is always required to avoid the building-up of an internal friction. An optimal balance between the two groups of properties must thus be set up.

The objectives of grouting

Main aspects

The principle of grouting is to fill the open voids existing in a rock mass in introducing, by pressure through boreholes, a certain amount of a "liquid" matter, in fact a suspension, that will harden later on. The properties of the grouted rock complex should be modified in the desired way.

The main expected improvements are well known. They are:
- reducing the permeability of the rock mass;
- reducing its deformability, and
- increasing its strength especially against shearing forces.

The relative importance of these three objectives of the grouting works depends obviously on the type of construction dealt with.

However, there are also other aspects to be considered like:
- the feasibility;
- the expansion of the rock mass produced by the grouting process;
- the durability of the expected beneficious effects;
- the economics of the treatment including the costs and the time required to carry it out.

Limitations to grouting

There are a number of conditions and circumstances, which may prevent the grouting process, except some special measures are taken.
So flowing water may require the adjunction of some anti-wash additives. Also too low temperatures may be of harm.
A very high sensibility of existing buildings or structures (e.g. underground steel lined pressure shafts or a system of drains) may make grouting work inadvisable. Considerations of different kinds related to the environment may not permit the use of certain types of grout.
The nearby presence of springs used for drinking water may preclude the use of any kind of grout and might impose a temporary freezing of the groundwater instead of a grouting of the rock mass for example during the time period, an underground construction is carried out.

Expansion of the rock mass

Another aspect of the grouting is sometimes ignored. This is the expansion inevitably suffered by any rock mass when grouted, as grouting is done by definition in using pressures, which will and must open the joints. This expansion may produce a heaving of the ground, which is often limited by some specification, but is in fact a prerequisite for any grouting (**Figure 10**). The problem is then to limit the heave - should it be of harm - but in no way to avoid it completely.

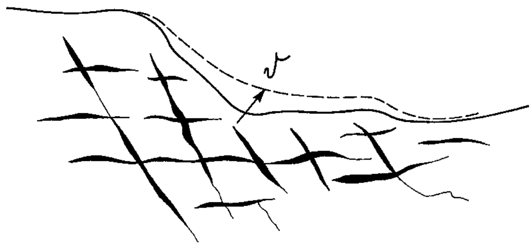


Figure 10: Unavoidable heave of the ground surface above a grouted zone.

There is however a further aspect of this expansion process, which is seldom taken into consideration. In some cases a successful grouting is not feasible because such an expansion is not possible. As an extreme example let us consider the following situation (**Figure 11**), which may be encountered in some fractured zones in a massive very strong rock.

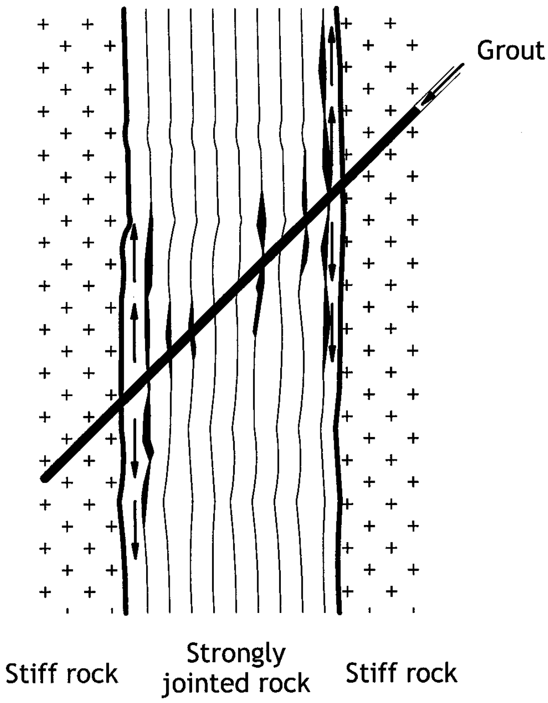


Figure 11: Due to the restraint by the stiff rock only few of the fine joints can be grouted. → grout penetration

A heavily jointed rock zone may be strongly fitted in between two massive blocks. Suppose a frequency of 10 thin or flat joints per meter. In using a certain cement grain size, the joints should be opened by 0.3 mm each to be grouted and correctly filled with cement. This would result in an expansion of the rock mass of 3/1000 in the corresponding direction. If the modulus of deformability of this mass is 20 GPa the grouting would lead to a final transversal compressive stress in the rock mass of 60 MPa to achieve an uniform opening of all the joints, as well as an even higher grouting pressure.
As such an extremely high pressure is practically not feasible, only few joints out of the entire series can be opened and thus grouted. If a grouting pressure as high as 6 MPa is used only 1 joint out of 10 might theoretically be grouted.

The real situation is even worse because the joints are not absolutely identical, so one of them will open first and wider than necessary, while the others will tend to close and can no longer be grouted (Lugeon 1933).
Consequently, the objectives of the grout process can not be reached and a factual limitation to grouting must be accounted for.
To solve the problem finer cement or chemical products might possibly be used instead of normal cement.

Durability of the treatment

It was observed that some grout curtains for dams had practically disappeared after a number of decades (Houlsby 1982). The too thin grout mixes used were washed out by leaking water. By the way, this fact could be considered, in some cases, as a proof that the curtains were not necessary at all!

The attack of the set cement grout requires two conditions:
- a weak material, weakly bounded to the joint walls, but also
- the possibility for the water to seep along the grouted joints;

this last condition is fulfilled when grouting pressures lower than the final water pressure were used or when the grout could shrink enough to open a way for the water along the surface of the joints, or if voids left by bled water from the grout did exist.

Obviously, this phenomenon may not always be of concern, for example if only a temporary tightening of rock masses is required during the construction of permanent structures. In many cases however, the necessity of repeating the grouting works after a number of years may have heavy consequences on the economy of the project. These aspects should be adequately considered by the designer of the grouting works, while it is recognized that the problem is not easy to be quantified and that often only an engineering judgement and experience help to solve it.

Water pressure tests

It is customary, since almost one century to use water pressure tests to evaluate the permeability of a rock mass before and after its treatment by grouting. Among them the Lugeon test is well known and widely used (Lugeon 1933). Obviously, it furnishes only a quite rough criteria but it is, at same time, a quite simple and useful testing procedure.

However, some too simplistic assumptions regarding these tests as well as old grouting habits justify some scepticism. For example, a rule of thumb was in use, which stated that a grout curtain is necessary where the Lugeon values is higher that a certain limit; this regardless of the kind of dam, its height and the type of foundation which are concerned nor the depth below ground where the test is carried out.

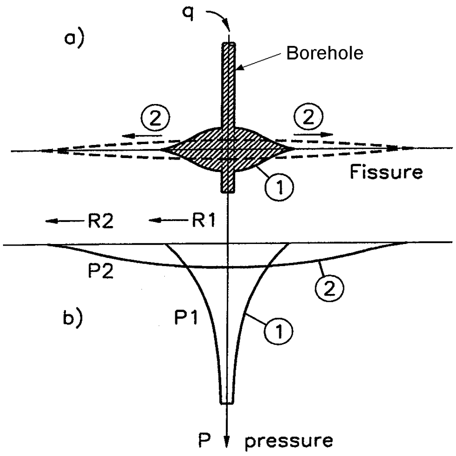
Another quite misleading interpretation of these tests is the temptation to set-up a statistical relationship between the Lugeon values and the volume of take to be expected at grouting stage.
Indeed, the flow rate of water is put in relation with the grout volume disregarding also the fact that water is a Newtonian body, while the cement slurry is a suspension of grains of a certain size that follows, at least approximately, the Binghamian body laws.
A high frequency of fine joints may give the same Lugeon value as an unique wide crack. In this last case the grout take can be very high, while in the first one no cement at all can enter the fine joints.

In such theoretical exercises on statistical correlations, the real conditions of the grouting process are often ignored.
In fact, the experience shows that water pressure tests may, at the best, give an approximate indication of the reduction of permeability obtained by a grouting work, but are practically useless to define the grout take to be expected, nor to give a correct indication even on the groutability itself of the rock mass.
Indeed the degree of groutability can only be defined by grouting tests.
It is thus felt that water tests carried out at every grouting stage do simply represent a systematic waste of money, without any beneficial effect on the grouting process itself. Even more they may have an unfavourable effect because they may cause damages in re-opening already grouted joints.

Hydro-jacking and hydro-fracturing

Two concepts, which are often imprecisely understood, are "hydro-jacking" and "hydro-fracturing". This confusion may lead to wrong decisions on the grouting site.
It is believed that hydro-jacking should refer to the opening by the grout of pre-existing joints in the rock mass, while hydro-fracturing describes the forming of new cracks due to an increased pressure of water or slurry.

In fact, hydro-fracturing is a relatively seldom event during normal civil engineering grouting works. Not every one of the observed drops in the pressure during grouting can be interpreted as a hydro-fractioning event. Indeed, pre-existing joints may also open suddenly. According to **Figure 12**, a kind of elastic instability takes place.
In the current practice, true hydro-fracturing is mainly related to the already mentioned "potential joints" like weak inter-stratigraphic or beddings planes.
The opening of such planes is due to the tensile stresses induced in the compact rock by the nearby pressure of grout or of water.



a) sudden extension of the open stretch of a fissure at grouting
b) spreading out of pressure

To a) and b):

① before
② after
hydro-jacking

q flow rate of the grout
p pressure
R Reach from the borehole

Figure 12: Hydro-jacking or claquage as a kind of "elastic instability" or bifurcation.

This happens more frequently when the borehole is parallel to the planes of weakness, while hydro-jacking do happen more or less in the same way regardless of the angle between the borehole and the joint (see Figure 6).
Combinations of hydro-jacking and of hydro-fracturing are also possible as shown on the same figure. However, the risk is clearly higher when water or a thin mix is pressed into the fine "potential" joints where a thick mix can hardly penetrate.

The main question is however whether a hydro-fracturing is always of harm or not, while the hydro-jacking is fundamentally the expression of an effective grouting, which does more than just fill - at no pressure - opens voids in the rock mass.

If a strong slurry with good bounding properties to the rock is used, a hydro-fracturing is seldom harmful from the technical point of view; except it produces an excessive heave at shallow depth. But, the possible useless waste of grout and thus the related additional costs due to important hydro-fracturing events causes unfavourable economical consequences.
The same can obviously be said of any excessive hydro-jacking of existing joints.

The often used French term of "claquage" may apply to both cases and refers mainly to the sudden opening of a new way for the grout, which will concentrate along certain surfaces as interpreted by Figure 12.

Hydro-jacking and hydro-fracturing are both related to splitting forces across the joint considered.
These forces are obviously the integral of the pressures acting on any single elements of the joint surface. They are thus function of the pressure applied in the gout hole, but also of the extension of the surface submitted to pressure. It can be assumed, as an acceptable approximation, that said surface is in some way related to the volume of grout already pressed in, obviously only as long it has not yet set.

The penetration of grout

The process of the penetration of grout under pressure into the rock joints depends on a number of factors, the first one being the geometry of the joint walls, that means their shape and variations in their opening, their extension and the interconnections between them. Very complicated cases can be and were analysed from a theoretical point of view. Nevertheless, the most important aspects of the grouting process can be studied in a simple but quite reliable way on the basis of very simple models, like open flat joints of constant thickness. **Figure 13** (Lombardi 1985) shows the theoretical model, which gives the relationship between grouting pressure, opening of the joints, cohesion (or yield point) of the slurry and maximal reach of the grout.
Accordingly, the higher the pressure, the wider the joint and the lower the cohesion, the greater will obviously be the reach. More refined geometrical models may represent the reality somewhat better, but cannot change significantly this relationship. Additionally, the reality will always be different from any model chosen and is also different from spot to spot of the same rock mass.

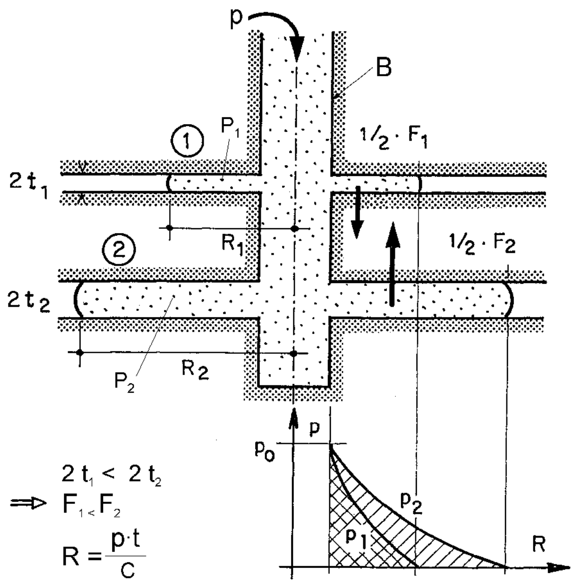


Figure 13: Pressure distribution and forces in a two-joint system, during grouting.

B pressurised borehole. ① Joint closes; ② Joints opens;
p = pressure; c = cohesion; 2t = opening; R = reach;
F_1, F_2 = splitting forces.

The opening of any single joint - no matter how such opening may be defined - is different, so that the grout will penetrate more easily in some of them and more hardly in others, as already said.
From the practical point of view this means, roughly speaking, that any grouting stage will fill mostly, or at least to a greater distance, only the main, wider, not already grouted joints, while the thinner ones will have to be grouted later on. This leads to classical grouting procedures in steps with various successive series of split-spacing boreholes and possibly with increased grouting pressures from series to series.
The possible closing of the thinner joints due to the expansion of the main joints may just be recalled at this place.

Also the relationship between grain size and joint opening has to be taken again into account. Due to this fact, a differentiation must be done not only between wider and finer joints, but also between different spots of the same joint, which obviously does not show a constant opening along its surface. In the thinnest part of a joint only water will enter, while the cement paste will stay in the wider spots, and follow preferential "channels", along the joint (**Figure 14**). The beneficial effect of hydro-jacking is that the joint will open and the cement can then enter any spot of the joint, increasing thus the proportion of the surface, which will be really bounded.

The mentioned relationship between, joint opening, cohesion of the slurry, grouting pressure and reach does apply obviously only to stable mixes, where no excess water exists, which may separate from cement.

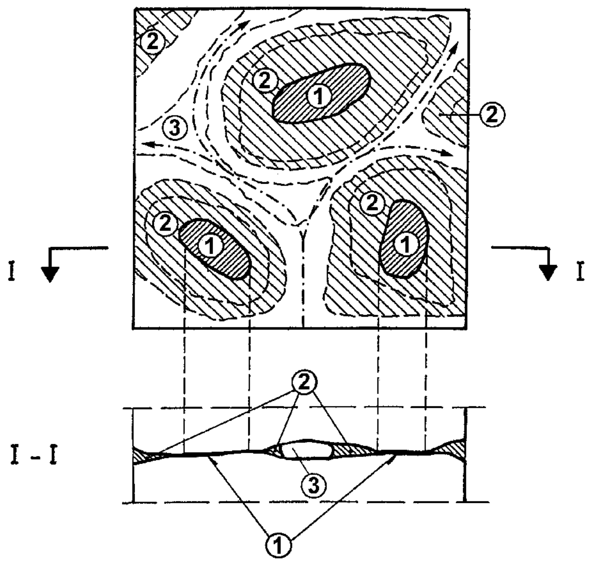


Figure 14: Zones around contact areas ungrouted or poorly grouted because the local aperture is too small in relation to the grain size of the cement used.

① contact areas; ② ungrouted or poorly grouted zones;
③ well grouted zones; –·–·–·→ main flow-lines of the grout.

As already shown in Figure 13 the pressure of the grout will decrease from the borehole away, so that the extension of the pressurised surface will be limited and the average pressure acting on it can be estimated to be about only one third of the grouting pressures applied.
The fear of heaving the rock surface is thus strongly reduced - at least by three times - respect to the usual formulation, which limits the grouting pressure to the simple weight of the overburden disregarding thus the mentioned fast decrease of the pressure with the distance from the borehole. This pressure drop is sharper and thus the reach smaller the higher the cohesion of the slurry is. Consequently, at shallow depths higher pressures as usually assumed may be applied.
The requirement to limit the heave, that is to reduce the pressure and to increase the cohesion, is just contradictory with the requirement of a good penetration that means high grouting pressures as well as low viscosity and cohesion of the slurry. Therefore these limits should be placed as high as possible.

The grouting intensity

The grouting pressures should be as high as possible to increase the reach of the grout, but low enough to avoid hydro-fracturing.
A way out of this dilemma appears to be shown by the concept of "grouting intensity". It is known by experience that limiting only the grouting pressure and the grout take is not sufficient to avoid or even only to reduce the risk of hydro-jacking and hydro-fracturing and thus a possible waste of grout.

The El Chocón dam in Argentina required important rehabilitation works because of the internal erosion of the clay core, which had taken place (Aisiks E. 1991 and Vardé O. 1991). It was necessary to consolidate the rock below and laterally of the core, but also the contact surface between rock and core, as well as the core itself.
The works had to be carried out at almost full reservoir, as a drawdown was not possible. The risks involved in these works were high. A hydro-fracturing of the rock near the dam foundation or of the core itself had to be absolutely avoided.

Based on the consideration presented here-above the notion of "grouting intensity" was introduced, about twenty years ago to solve said problem, and developed in the following years.
This parameter is simply the product of the grouting pressure by the volume taken any time the grouting process is stopped and the grout flow rate is nil.
As it may be seen on **Figure 15**, this intensity represents a rough estimate of the Energy pressed into the rock mass. If the processes were absolutely elastic - this means linear - and when the internal energy losses due to the viscosity of the grout, the slippages of the grout along the walls as well as to the inelastic deformations of the rock mass are disregarded, the intensity would be the double of the elastic energy introduced by the pumps and theoretically accumulated in the rock mass.
In the real grouting process the conditions are significantly more complex, but nevertheless the intensity, as previously defined, proved to be a quite useful tool to manage the grouting process.

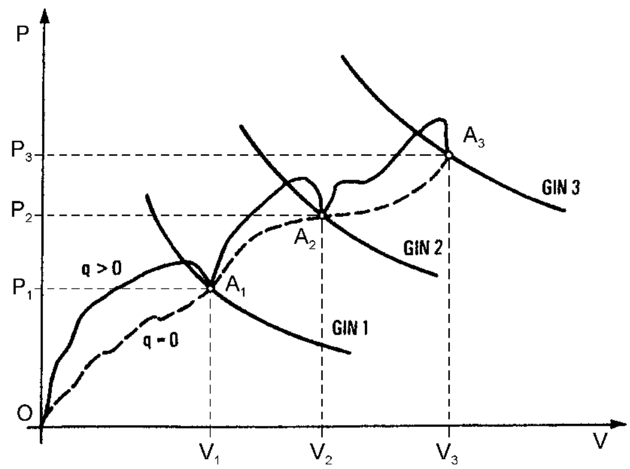


Figure 15: The grouting process can be stopped at any final pressure required or by reaching any required GIN value (there is no such situation as "refusal" by the rock). The successive values of GIN, represented by the rectangles $O{\cdot}P_i{\cdot}A_i{\cdot}V_i{\cdot}O$ are an approximation of the energy $E = \int_O^V p \cdot dV$ injected.

——— actual grout path;
--------- successive points of equilibrium at nil flow rate q.

The GIN value is evaluated at a nil grout flow-rate; that is at pumps stopped.
During the grouting process however, head losses have to be overcome, so that the manometric pressure will be higher than the final one required by the GIN rule. This overpressure may be of the order of 10 to 20% of the final one.

The essential definition of the GIN value is shown in **Table 2**.

Starting for joints of constant aperture, from

$$R = \frac{p \cdot e}{2 \cdot c} \qquad \text{(reach achieved) and}$$

$$V = \pi R^2 \cdot e \qquad \text{(volume taken)}$$

and defining

$$GIN = p \cdot V$$

one obtains for real joints

$$\boxed{GIN = p \cdot V = 2\pi \cdot n \cdot k_p \cdot k_v \cdot c \cdot R^3} \text{ (intensity independent of joint opening)}$$

where

GIN	=	Grouting Intensity Number
c	=	cohesion (yield point) of the slurry
R	=	average reach of the grout
p	=	final grouting pressure
V	=	grout take (per m borehole)
e	=	opening of the theoretical flat joints
n	=	number of main joints (per m borehole)
k_p	=	coefficient to consider pressure losses due to the rugosity of the rock walls of the joint and the variation of the aperture
k_v	=	coefficient to consider volume increase due to undulations as well as to variations of the opening of the joints

so that

$$R \cong \sqrt[3]{GIN}$$

or

$$R = R_t \cdot \sqrt[3]{GIN / GIN_t}$$

R_t and GIN_t being test values.

Table 2: The GIN concept.

The GIN-principle

The most immediate application of this concept consists of taking into account, during the grouting process, the intensity value reached at any instant. This leads to the definition of a so-called "Grouting Intensity Number" or GIN-value, which is used to limit the grouting process accordingly to **Figure 16**.
The main result is to avoid the simultaneous occurrence at high pressures and high grout takes, which combinations lead to the dangerous zone for hydro-fracturing or hydro-jacking indicated in the figure.
Consequently three limits are to be taken into account in designing a grouting work:
– the maximum pressure;
– the maximum take or grout volume, and
– the maximum intensity.

The maximum pressure must be in someway related to the water pressure to be expected at that spot during the future life of the structure. A ratio of 2 to 3 in respect of this water pressure appears reasonable.
The volume limit should not be seen as an absolute boundary, but more likely as an indication of the necessity to take a decision, which could be:
- continue the grouting;
- stop the grouting definitively;
- stop the grouting for a time period and restart it later on;
- abandon the borehole and drill another nearby;
- add, for example, an anti-wash product to the mix, or
- take any other adequate measure.

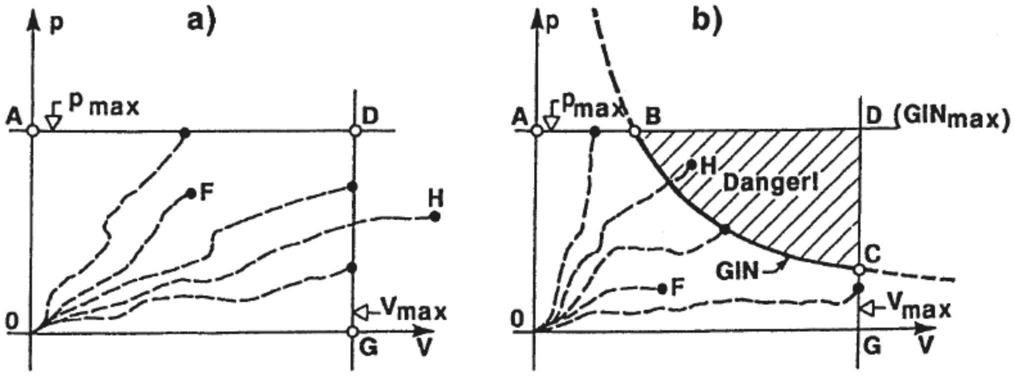


Figure 16: The limitations of the grouting process.

a) traditional method: limit ADG — b) GIN-method: limit ABCG
p_{max}=maximum pressure; — V_{max}=maximum take;
GIN=limiting curve ($p \cdot V$= const.);

In the upper corner danger of hydro-fracturing exists. Grouting paths like F and H are not allowed. The traditional method is a special case of the GIN method when GIN$\geq p_{max} \cdot V_{max}$.

The GIN-value is mainly an indicator of the average distance reached by the grout. It depends essentially on the requirements of the project; the GIN-value itself being approximately proportional to the third power of the reach. These relationships may be seen also on **Figure 17**.

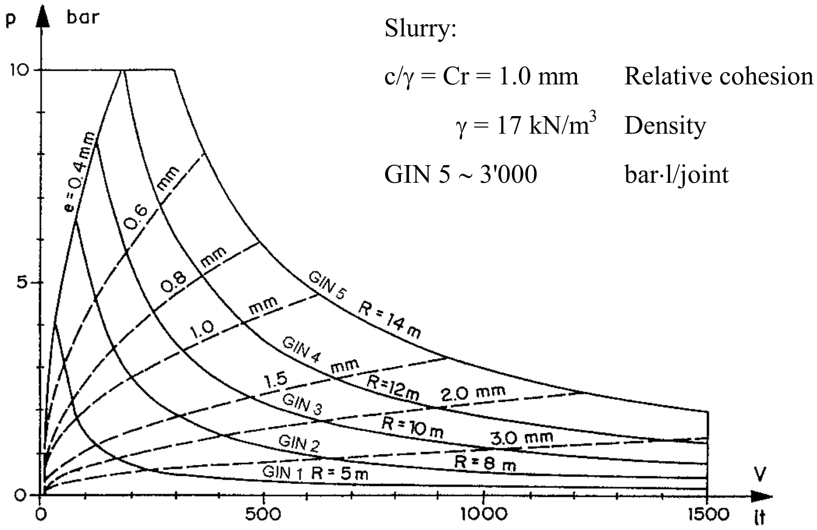


Figure 17: Example of grouting of a single joint.

Relationship between: p=pressure; V=volume taken; e=opening of the joint; R=reach of the grout. In this example it was considered that a thinner than 0.4 mm joint is not groutable with the cement used.

It is interesting to notice that any reach corresponds to a GIN value independently on the actual opening of the joints. A thinner joint absorbs less grout volume but requires a higher grouting pressure in the inverse proportion.

A limiting factor of the grouting is clearly the risk of excessive hydro-jackings of the joints. **Figure 18** shows that hydro-jacking phenomena do occur at constant intensity when they start. The optimal value of the GIN must be defined on the site by grouting tests in order to adapt it to the local conditions of any homogeneous rock domain.

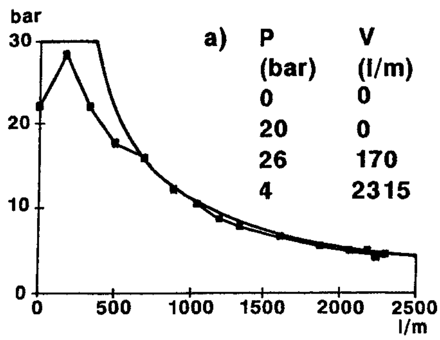


Figure 18: The GIN value is the governing factor of a hydro-jacking event. In this case an extremely high GIN value of 9000 bar·l/m was used.

Figure 19 gives some reference to practical values dictated by the experience gained at many grouting sites, where sound jointed rock was present. They are called "standard" limiting curves or guidelines.

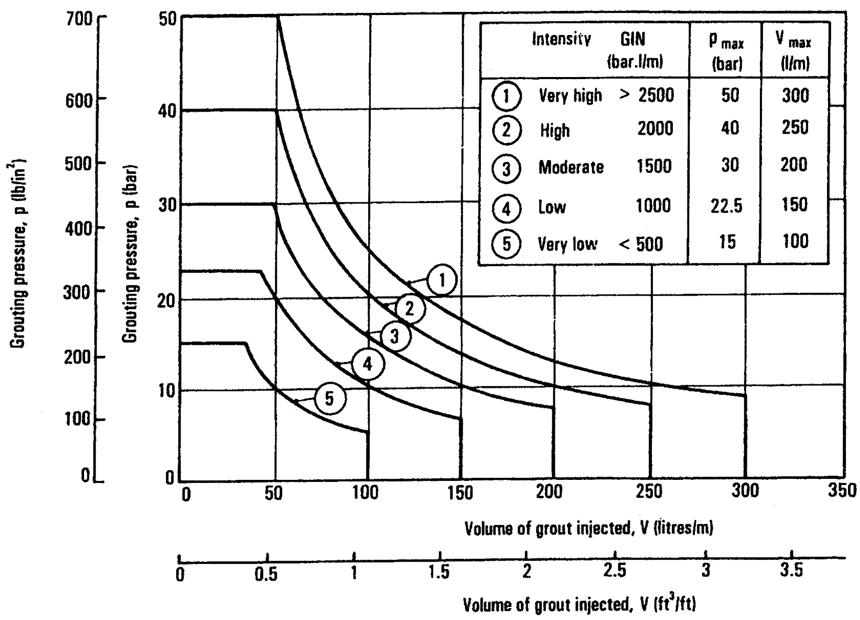


Figure 19: Set of "standard" GIN-limiting curves, which may be helpful as starting guidelines.

The fundamental principle of the GIN-Method is to reduce the grouting pressure in function of the volume of the grout injected. [5]

In using successive series of grouting holes one may observe how in following the GIN principle the take diminish, in average, from one series to the next more or less in the proportion of 2 to 1, while the final pressures increase in the inverse proportion (**Figure 20**); confirming that any grout hole series fills practically only the widest joints not yet filled by former series[6].

If the maximum pressure is reached at very small takes, one may consider the grouting of this stages as tests, which are much more effective than any water test in showing the low groutability of the rock mass.

---

5 The principle was already applied in 1985 in the headrace tunnel of the Ruzizi 2 power plant in Zaïre, in order to limit the losses of grout through wide-open joints. The reduction of the pressure took place at that time stepwise and not continuously as the GIN principle achieves it automatically.

6 Deere and Lombardi (1985) proposed to increase the grouting pressure in the bore-holes of a grouting curtain as follows: primarily holes p, secondarily 1.5 p, tertiary 2.0 p, quaternary 2.5 p. A similar more flexible progression is automatically achieved by the GIN-Method.

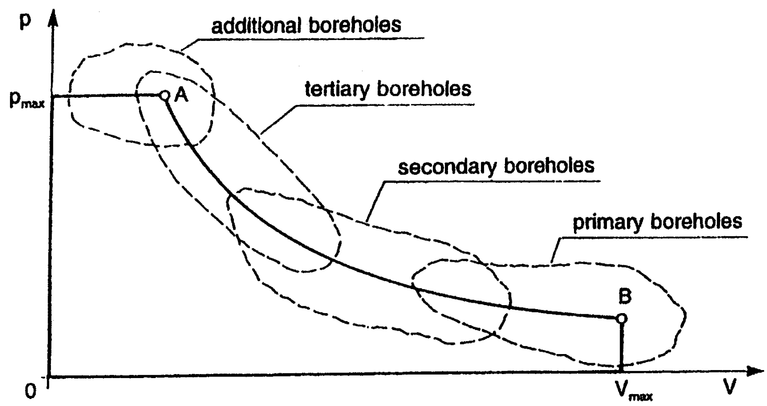


Figure 20: Example of grouting results for a grouting curtain. Final points of the grouting paths of all the borehole grouting stages (typical).

The grouting of any stage of the boreholes of a grout curtain will, as a rule, result in a decreasing penetrability with time as shown by **Figure 21**, where this factor is defined as the ratio of the flow-rate to the grouting pressure.

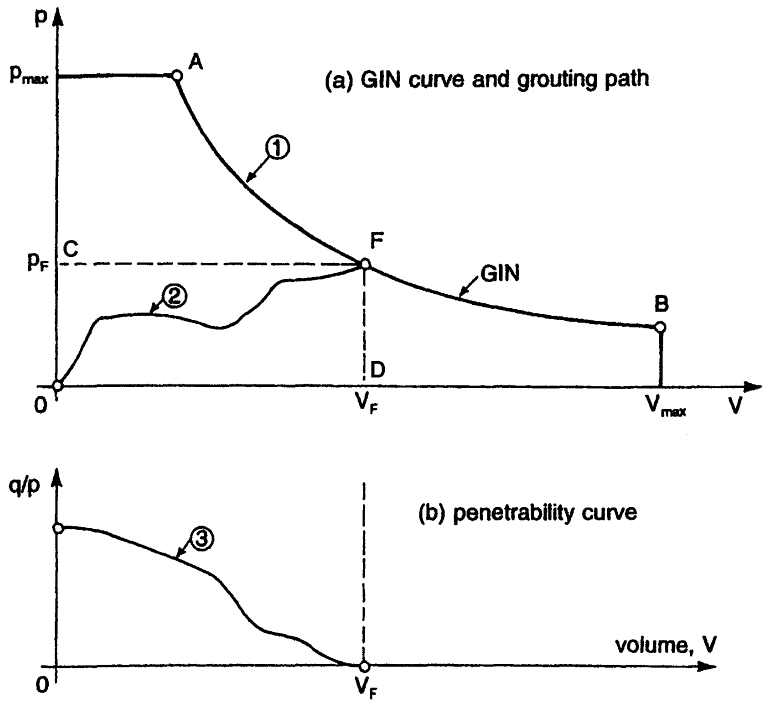


Figure 21: Grouting process of a single borehole stage (typical) where:

1 = limiting curve, pressure versus grout take;
2 = actual grouting path, pressure versus grout take;
3 = penetrability (q/p) versus grout take.
F = final point of the grouting; p_F = final grout pressure; and
V_F = actual grout take.
The rectangle OCFDO is defined as the "grouting intensity"

Monitoring and recording grouting works

Given the present high level of the grouting equipments available, no important grouting activity should any longer be carried out without a corresponding real time monitoring and a complete recording of the main parameters of each grouting stage. The most important parameters to be analysed and plotted are:
- pressure vs time;
- flow rate vs time;
- take vs time;
- pressure vs take;
- penetrability vs take;

the two last being considered the most important ones.

The real time interpretation of the plots of these functions allows to characterise the grouting process and to detect at early time any incipient hydro-jacking or hydro-fracturing and thus to adapt the grouting process to the real conditions of the rock mass.

There are today a number of software applications available in this field, which proved to be very effective.

The GIN-Grouting Method

GIN was originally a simple numerical parameter of the grouting procedure.
However, with the years a number of rules to be followed in grouting fissured sound rock masses were developed and implemented. Finally this complex of rules got the name of "GIN-Method".

The corresponding principles, which are intended to produce a quality grouting while simplifying the grouting process and avoiding excessive hydro-fracturing, are summarised in **Table 3**.

The most important aspects are:
- use an unique recipe for the mix, the most adequate one, defined on the base of extensive laboratory tests, in particular to ensure the durability of the works;
- use a GIN limiting function (with the three parameter p_{max}, v_{max}, GIN), a borehole distance, a length of the stages and a cement grain size that best fit the properties of the rock and the requirements of the project, on the base of field tests and rock-mechanics considerations;
- follow the grouting process on an electronic display and record the data of the grouting process;
- analyse continuously the recorded data in order to optimise the grouting process;
- avoid water pressure tests in the already grouted rock mass.

The main advantages of the method are:
- simplifying the procedure in using an unique slurry and eliminating or at least reducing the wastage of unused mixes;

– reducing or even avoiding the risk of excessive hydro-jacking and possibly of hydro-fracturing in eliminating the combination of high takes at high pressure;
– equalising approximately the reach of the grout at any grouting stage regardless of the rock quality, thus making the process more predictable;
– producing a set of coherent data, which allows to judge the progress of the process as well as the result achieved and thus to optimise it;
– but, first of all the GIN-procedure is a self-adaptive one which compensates for an appreciable part the scatter of the natural conditions met in the rock mass.

Principles of the GIN-Method

- Define exactly the scope of the grouting works.
- Do design, not specify, the grouting process.
- Find out the "best mix" for the project by laboratory tests both from a technical and an economical point of view. Only stable mixes, generally with super-plasticiser, should be used. (Thick stable mixes have been favoured by European grouting experts since various decades.)
- Use only a single mix, "the best possible one", for all grouting stages in order to ensure the quality of the results and to simplify the procedures. This will also reduce wastage of grout.
- Define the parameters of the GIN limiting curve: p_{max}, v_{max} and GIN-value = $p \cdot V$, taking into consideration all determinant geological and rock mechanics parameters as well as the scope of the works and the related economy of the project.
- Confirm the studies by field tests and check the works carried out by additional test groutings.
- No water pressure tests to be carried out during the grouting works; they are useless and dangerous.
- The split-spacing method for the boreholes is not a new technique, but it is used in the GIN-Method as a self-adaptive and self-regulating procedure.
- The use of stage lengths, increasing with depth below ground, is progressively recognised as a way of speeding up the grouting works and of making some, albeit small, savings.
- Inject water in dry, water-absorbing rock formations above the ground water table, shortly before grouting, as a mean of avoiding a sudden blockage in the grouting process due to water losses from the mix.
- The necessity of a new borehole and its length (e.g. in a grout curtain) is decided on the base of the volume of grout taken by the nearby ones.
- Computer controlled procedures are an obvious pre-requisite for optimal grouting works. Many pieces of information can be obtained from these plots. Very interesting statistical relationship can be set up. Unfortunately, at several projects the graphs were plotted without trying to draw the due conclusions.

Table 3: The generally accepted principles of the "GIN-Method".

The basic idea of the GIN to reduce the grouting pressure in function of the take is not that exotic as it may appear at a first glance.

Indeed, if one considers not just the grouting pressure itself, but the so-called "normalised pressure"[7], that is the dimensionless ratio between said pressure and the cohesion of the slurry an interesting relationship can be set up as shown by **Figure 22**. Common to the traditional method and the GIN-Method is the intention to avoid uselessly high takes of grout. In the first case the limitation is obtained in increasing the cohesion from the thinner to the thicker mixes (in function of the volume already taken).
In the case of the GIN-Method said objective is achieved by a constant cohesion in reducing the target pressure in function of the volume taken.

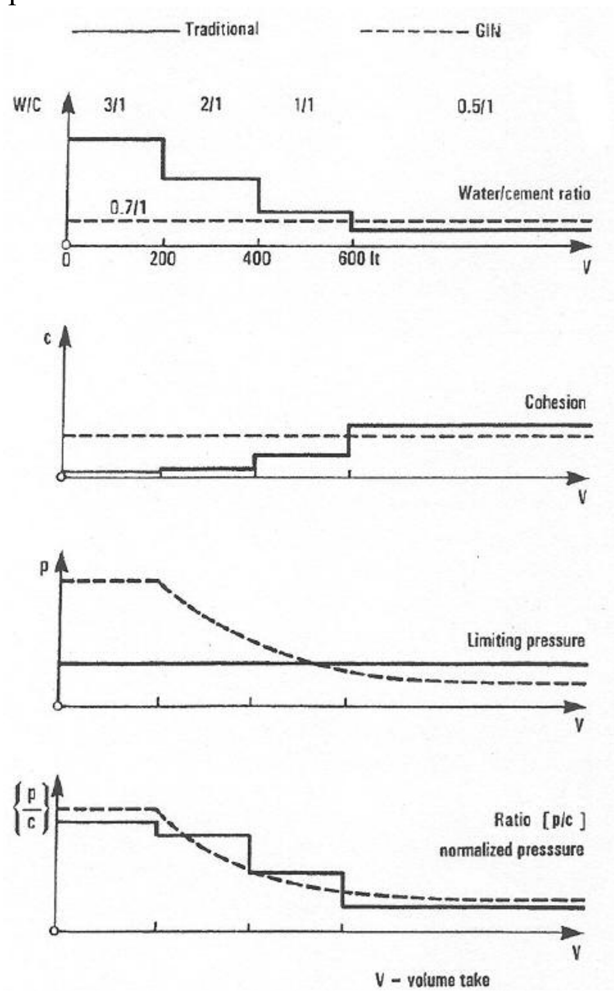


Figure 22: Comparison, by way of an example, of the traditional and the GIN grouting methods. The "normalised pressure" $p_n = p/c$ is decreasing with increasing take, both for the traditional and the GIN method.

7 For stable mixes the value of the normalised pressure may usually be comprised between 10^4 and $5 \cdot 10^6$. This value tends to infinite for thin mixes or water.

In both cases the result is obtained in reducing stepwise or progressively the normalised pressure; the fundamental difference lies in the quality of the results obtained and in the predictability of the process.
Additionally, the GIN-Method allows higher maximum pressures and higher maximum takes at no risk.

Finally, the GIN-Method can be seen as being a simple way to achieve an excellent grouting by avoiding low strength and easy to leach out mixes, while ensuring a complete filling of the voids and a homogeneous treatment of the rock mass (as far this is physically possible). Additionally, the risk of hydro-fracturing and of excessive heave of the ground are practically eliminated.

An example of application

The recently very successful rock groutings below the plinth of various Concrete Faced Rockfill Dams may be mentioned as an example of the implementation of the GIN-Method.
A sketch of consolidation and tightening of the rock below the plinth is given in **Figure 23**.

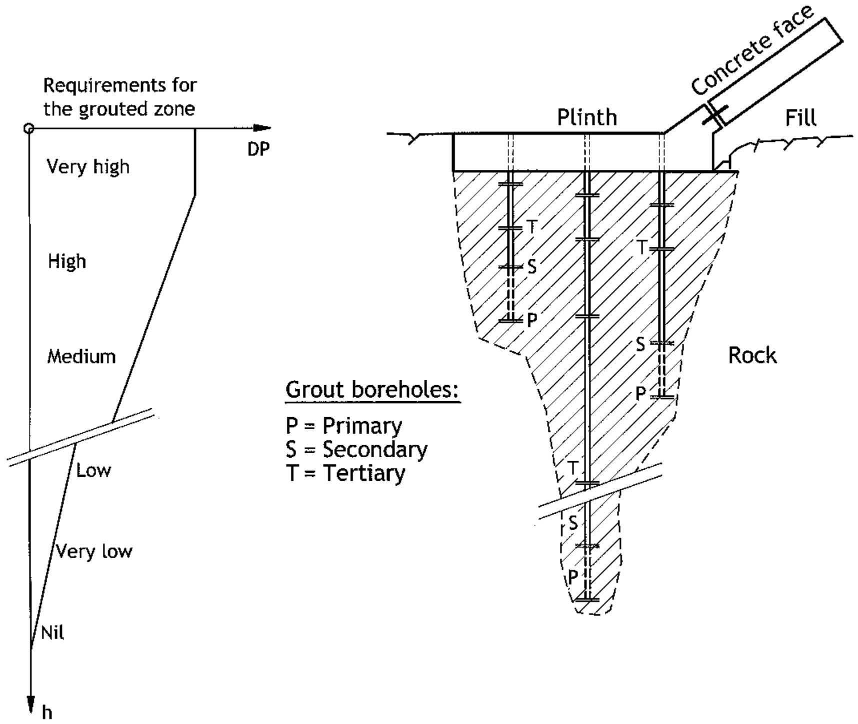


Figure 23: Requirements to and extension of the grouted zone (consolidation and curtain) under the plinth of a CFRFD.

DP = differential water pressure between upstream and downstream of the grouted zone.
Staggered grouting stages of increasing length with depth are designed.

Indeed, there is no reason to make a distinction between consolidation and tightening; there is simply a zone where the rock properties need be improved for any reason what so ever. The extension of the zone must be defined at design stage on the base of geotechnical and hydraulic considerations.
Said figure shows also in a schematic way the overall requirements for the project. Immediately below the plinth the improvement required is the most important because the hydraulic gradient is quite high (e.g. of the order of 15).
At lower elevation the gradient is diminishing. So the requirements do. At the very tip of the grout curtain the differential pressure from upstream to downstream is nil and thus also the requirements for the grouted zone.
This diminution of the treatment with depth is achieved by a number of design features as

- reduction with depth of the grouting rows from three to two, and finally to one;
- each row of grouting is formed by successive series of primary, secondary, tertiary and where necessary of quaternary holes of different depths;
- the length of the grouting stages is increasing with depth as schematically shown in the figure. This time and cost saving feature ensures that with increasing depth only the wider and wider joints are grouted accordingly to the actual requirements of the design;
- at the contrary the grouting intensity referred to by the GIN-value and measured at the borehole mouth is kept constant. The actual effective pressure increases obviously with the depth of the stage below ground. This increasing pressure is considered apt to compensate, more or less, the increasing total stresses in the rock mass due to the overburden as well as the increasing future water pressures with depth. It has to be considered that the density of the grout is comprised in between that of the water and that of the rock.

An additional comment to be done refers to the length of every borehole. Traditionally the depth of the holes is rigidly staggered so that secondary holes are shorter than primary and again tertiary are intended to be shorter than the secondary ones. In fact, there is no stringent reason to do so. It is felt that the result may be better and obtained at lower costs if the decision to drill any hole and to define its depth is taken in function of the absorption of grout of the nearby holes, as shown in **Figure 24**. The limit of take or the "critical take" must be carefully defined. One logical way is to set it equal to the theoretical take at the maximum pressure for a GIN reduced to half the normal value.
Obviously, a preliminary overall investigation through longer holes is necessary.

Another aspect to be considered from the point of view of the economy of the project is the possibility to approach the best possible balance between the drilling costs and the cost of the quantities of slurry injected to reach the same result.
The question may also be raised whether the distance between the boreholes must be kept absolutely constant or not. It is felt that, for instance, the tertiary holes could be placed somewhat nearer to the primary than to the secondary ones. However, this is a marginal aspect of design.

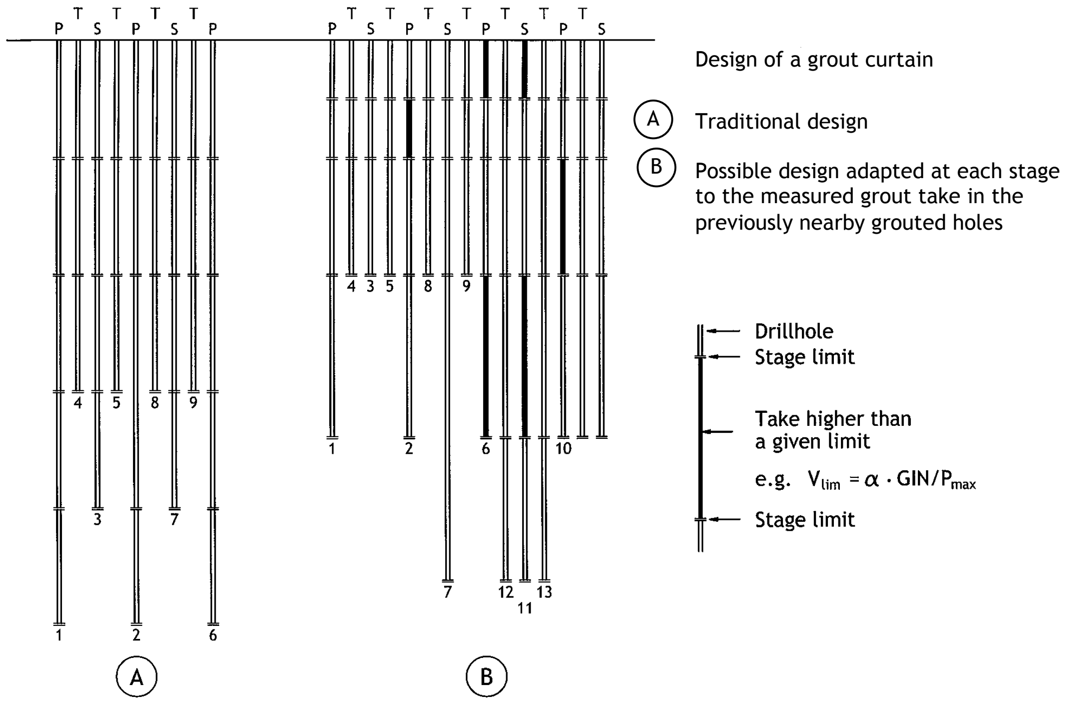


Figure 24: Design of a grout curtain.

In summary, the optimisation of the distance between the boreholes as well as of the length of the single grouting stages, but also the grouting pressures to be used, may provide interesting possibilities of saving.
Unfortunately, a number of these possibilities of optimising the design of a grouting project are often overlooked or not accepted because old-time specifications are blindly enforced.

Flow rate of grout[8]

An important question frequently addressed to, is how to manage the flow rate of grout during the process and even more how to reduce it in nearing the GIN limit so to finally reach the target intensity.
Depending on the actual conditions, there are different manners to proceed. However, the following one appears to be the most convenient. It is represented on **Figure 25**.
Being GIN the target intensity, two new limits at, let say, 0.95 GIN and at 1.05 GIN as shown in the figure should be defined.
The selected basic grout flow rate "q", is kept as constant as possible all along the grouting process until the 0.95 GIN limit is reached. The flow is then reduced stepwise to 2/3 q_0, 1/3 q_0 and zero at crossing each of the three limits.
A similar rule can be used in respect to the maximum pressure, while for the maximum volume the process can be stopped at one time.

[8] Section added after the presentation at the Conference

Obviously the number of successive limits and thus the number of steps in the pressure reduction, as well as the width of the fringe defined in said way may be adapted to the local conditions.
The basic flow rate depends of course on the equipment available, but has to be checked in any case by grout tests in function of the scope of the grouting works. As a rule however a medium or moderate flow rate is preferred to a very high one.

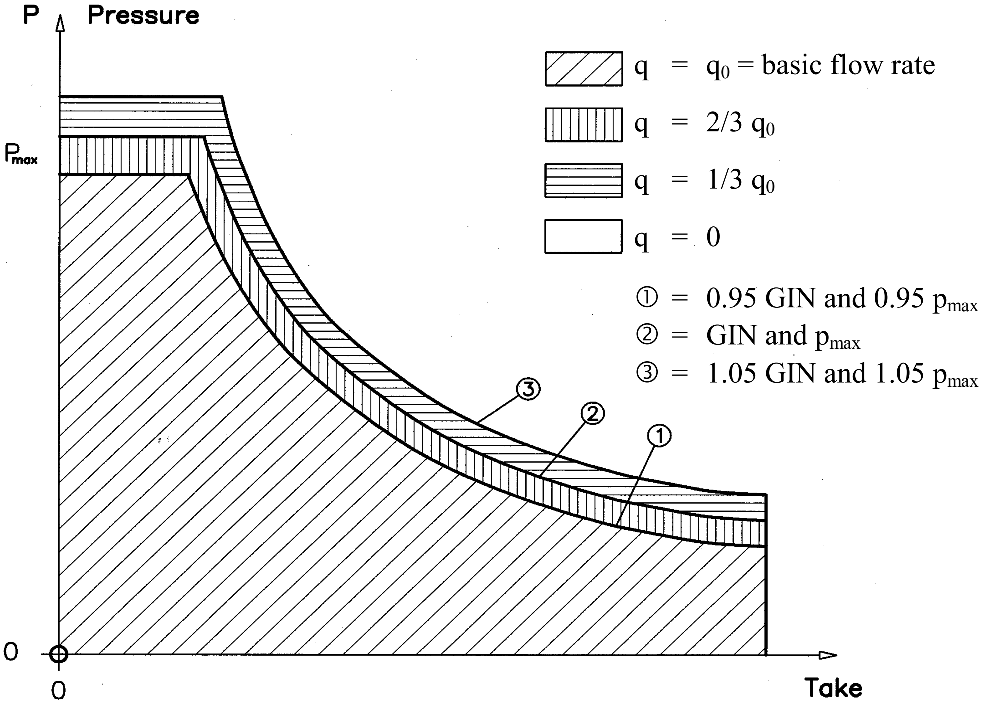


Figure 25: Management of the flow rate of grout (typical).
GIN = Target grouting intensity
q_0 = basic rate
$q = q_0$ if $p \cdot V \leq 0.95$ GIN
$q = 2/3\ q_0$ if 0.95 GIN $\leq p \cdot V \leq$ GIN
$q = 1/3\ q_0$ if GIN $\leq p \cdot V \leq 1.05$ GIN
$q = 0$ if $p \cdot V \geq 1.05$ GIN

Conclusions

The technique of rock grouting is an already old one. Unfortunately, the results of the treatment carried out can not be observed directly and thus are difficult to be exactly assessed.
With the decades, this situation led to a number of historically founded beliefs based on single experiences but seldom cross-checked. So, while each one of them can be true under given conditions, more general correlations were missing.
The heterogeneity and anisotropy of the rock mass and consequently the great scatter of its properties, do not really favour the definition of clear general concepts.

In this paper a few simple notions and ideas were presented, which any grouting project should refer to.

The suggested grouting method takes these concepts into account, it is based on a critical examination of the current habits and tries to replace subjective beliefs by physically founded notions. In no way, it does represent the final stage of the technique of rock grouting and future developments are to be expected.
Meanwhile, it is however felt that the GIN-Method represents a clear step in the right direction of better optimised and even of more economical grouting processes.
The GIN-Method has obviously to be solely intended as a design support, which does not replace a sound analysis of each specific project. In particular the parameters to be used must result from this analysis as well as from laboratory and field tests.

It is however well known that the implementation of new ideas and working methods is always slow because there is worldwide a number of people who prefer to stay comfortably with the old rules enforced by specifications taken over, without any change, from former tender documents. They thus decide not to follow new ways of thinking and to skip the necessary efforts to understand new developments.

Main errors to be avoided[9]

The main points of the GIN Method are summarised in Tables A to H.
According to Table H (or 3 of the main paper), the GIN Method is based on 12 rules.
There are thus at least 12 manners to make a wrong use of it!
Some of the most usual mistakes are listed hereafter.

1st mistake
Decide on grouting works just to comply with some old tradition, not with the real necessities of the project.

2nd mistake
Specify from the beginning a GIN Number and forget to confirm it by grouting tests. The selected value may not fit the actual geo-mechanical conditions and hydro-fracturing may happen too frequently, or the reach may result insufficient.

3rd mistake
Specify the distance between the boreholes and forget to confirm it by grouting tests. The classical split-spacing method is not wrong in itself and helps a lot, but may not represent always the optimum.

4th mistake
Shift to a second mix with higher cohesion and viscosity just to comply in a formalistic way with some specification on grouting pressures.

[9] Comments included in the oral presentation.

The scope of grouting is to inject a certain volume of grout and to reach a certain distance not to arrive at a given pressure at any grouting stage.

5th mistake
Specify rigidly a single, double or triple row curtain all the depth down, while the requirements to the curtain are decreasing at depth; so the number of rows should do.

6th mistake
Specific a fix stage length, while the requirements are not constant all the depth down.

7th mistake
Believe that a relationship can exist between water pressure tests and groutability and continue to carry out useless Lugeon tests at every grouting stage.

8th mistake
Continue to pump grout in, after a hydro-fracturing did show up on the display or the GIN number has been reached. Doing so, you certainly will confirm again that hydro-fracturing is governed by that intensity, but you just waste money.

9th mistake
Specify from the beginning the lengths of the boreholes for a grout curtain and stick to them without taking into account the actual rock conditions, borehole by borehole and stage by stage.
In fact it is not a drama if at the end of the day tertiary holes will be deeper than the secondary and even the primary, provided they are required by the local rock conditions.

10th mistake
Record the data, but analyse them later on in your cosy office when you don't have nothing else to do.
The recorded data must steer the grouting process and must thus be looked at immediately on the site. Practical simple rules must in any case be set up, so to be easily followed by the operator. Nevertheless, unexpected situations may occur, which require some thinking and decisions.

11th mistake
Think (and possibly write also in technical papers) that the GIN method implies high grouting pressures, which may increase the risk of hydro-fracturing, while it is your responsibility to choose the grouting intensity and the maximum pressure the rock mass may resist at no or at reduced risk.

12th mistake
Forgot to adapt the grouting intensity (GIN), in increasing or decreasing it, if the conditions of the rock mass do not correspond to the expected ones, or at the contrary change continuously the GIN value so that nobody, including yourself, can under-

stand what kind of job was really carried out, because an interpretation of the data recorded is no longer possible.

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Additional notes

To avoid frequent misunderstandings, one should consider the following in respect to the paper "Grouting of rock masses". The Tables A to H summarise for clarity the main points presented in the paper.

1. Actually, one should talk of "grouting the discontinuities" rather than "grouting the rock".
2. The notion of "best mix" used, e.g. on points 3 and 4 of the following Table H (or Table 3 of the paper), refers obviously to the "best mix found and defined in the frame of the project dealt with under given circumstances", and has obviously not to be understood in a general absolute sense.
3. In matter of "potential joints" it must also be noticed that the risk of hydro-fracturing depends, at least to some extend, also from their direction to the bore-hole (Refers also to Figure 6).
4. To Table G (or Table 2 of the main paper), notice should be taken that the GIN is not a simple arbitrary number but a function of various factors. In particular, the reach is related to $\sqrt[3]{GIN}$.
   Lower GIN value means thus reduced distance between the boreholes, that is more boreholes and additional costs.
5. To Figure 16 one may additionally remark that the GIN Curve has firstly to be understood as a "warning line" against excessive hydro-fracturing and hydro-

jacking. The target of a grouting process is to introduce in the rock mass a certain amount of grout not to reach a given pressure.

6. Figure 17 refers to the contact surface "core to rock" at the El Cochón dam (Argentina).
7. To Figure 18. Indeed the hydro-fracturing is governed by the GIN value not that much by the grouting pressure (which of course is necessary to produce the grouting intensity). It can be assumed that a high enough grouting intensity always exists to cause a hydro-fracturing. This value may however lay outside of any practical or reasonable range as this was the case for Figure 18.
8. To Figure 19. Notice must be taken that the "standard GIN Curves" do in no way mention any *maximum* nor *minimum* limit for the intensity or the pressure to be used at a given project. It is the responsibility of the engineer to define the values to be used in any single case.
9. To Table H (or Table 3 of the main paper). The previous saturation of the discontinuities to be grouted is ever since usual in the grouting of contraction joints of concrete dams.

TABLE A

The role of water
1. Hydrate the cement 2. Produce a fluid mix (avoid internal friction) 3. Compensate for water losses from the mix 4. Open the joints in front of the grout mix

TABLE B (Table 1 of the main paper)

Dam (Examples of unique mix)	**W/C**	**Fluidifier**
Paute (Ecuador) upper part	0.6	Intraplast 1.4%
Alicurá (Argentina)	0.67	Intraplast 1.2%
El Cajón (Honduras)	0.7	Bentonite 0.2%
Clyde dam (New Zealand) 2nd part	0.6	Intraplast ~ 1%
El Chocón (Argentina) repairs	1.0	Bentonite 0.5%
Sir (Turkey)	0.7 or 1.0	Puzolanic cement, Mistra 1% Bentonite 1.2%
Katze (Lesotho)	0.59	Cement + ash. Conplast 1.5%
Pichi Picún Leufu (Argentina)	0.7	various
Potrerillos (Argentina)	0.7	Rheobuild/Viscocrete 0.7-0.8%
Aït Hamou (Morocco)	1.0	Bentonite 2%
Casecnan lower tunnel (Philippines)	0.63	Intraplast 1%

TABLE C

To ease the penetration of grout

1. Add plasticizer
2. Use finer cement
3. Use higher pressures
4. Count on water in front of the mix

No reason any longer for "thin to thick" mixes

TABLE D

Advantages of stable mixes and high pressures

- Complete filling of joint; no water bubbles
- High mechanical strength
- Reduced shrinkage
- Better bound to rock
- High chemical resistance
- Predictability of the grouting process

TABLE E

Properties of the mix

Fresh mix
- Density
- Bleeding
- Decantation
- Viscosity
- Cohesion (yield point)
- Set time

Set mix
- Shrinkage
- Mechanical strength
- Chemical resistance
- Permeability

TABLE F

Expected improvements of the rock mass by grouting

- Reducing the permeability
- Reducing the deformability
- Increasing the strength

Aspects to be considered

- Feasibility
- Expansion due to grouting
- Durability of the beneficious effects
- Economics of the treatments

TABLE G (Table 2 of the main paper)

The GIN concept

For joints of constant aperture, from

$R = \frac{p \cdot e}{2 \cdot c}$ reach and $V = \pi R^2 \cdot e$ (volume taken)

and defining $GIN = p \cdot V$ one obtains for real joints

$$\boxed{GIN = p \cdot V = 2\pi \cdot n \cdot k_p \cdot k_v \cdot c \cdot R^3}$$

(the grouting intensity is independent of the joint opening)

where

GIN	=	Grouting Intensity Number
c	=	cohesion of the slurry
R	=	average reach
p	=	final grouting pressure
V	=	grout take (per m borehole)
e	=	opening of the theoretical flat joints
n	=	number of main joints (per m borehole)
k_p	=	coefficient for pressure losses
k_v	=	coefficient for additional volume

so that

$R \approx \sqrt[3]{GIN}$ and $R = R_t \cdot \sqrt[3]{GIN / GIN_t}$

R_t and GIN_t being test values.

TABLE H (Table 3 of the main paper)

Principle of the GIN Method

1. Define the scope of grouting
2. Design the grouting process
3. Find out the best mix
4. Use a single mix (the best)
5. Define the GIN limits
6. Confirm by tests
7. No water pressure tests
8. Split-spacing as self-adaptive process
9. Variable stage length
10. Previous saturation of dry rocks
11. New boreholes steered by grout take
12. Real-time grouting control

Notice

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