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THE UNIVERSITY OF ALBERTA

THE USE OF GROUTING FOR SEEPAGE CONTROL THROUGH FOUNDATIONS OF DAMS ON BEDROCK - A REVIEW

by

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A REPORT

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CHAPTER 1

INTRODUCTION

The control of seepage through dam foundations is a feature of paramount importance in any dam project. The control measures most often taken, either jointly or separately, are:

1) cutoffs
2) upstream impervious blankets
3) horizontal drains
4) relief wells

The first two measures are an attempt, which can be more or less successful, to reduce the rate of seepage through the foundations, whereas the last two are designed to prevent high uplift pressures under the dam, and besides, to prevent piping through the foundations. The later two measures may even cause an increase in the total seepage loss. The figures below illustrate the above mentioned solutions:

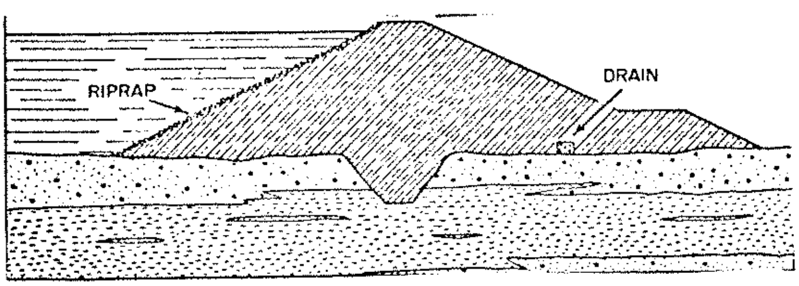


Fig.1.1
CUTOFF
(after Wahlstrom)

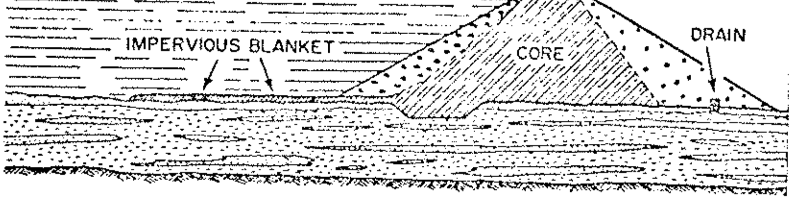


Fig.1.2
UPSTREAM IMPERVIOUS
BLANKET
(after Wahlstrom)

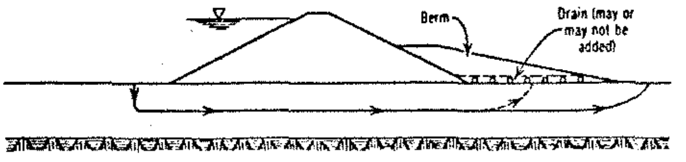
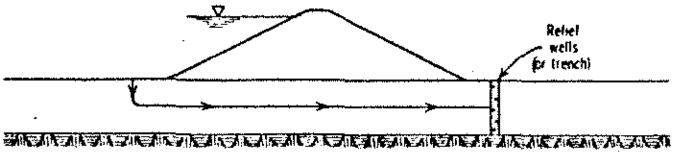


Fig. 1.3
HORIZONTAL DRAIN
(after Sherard)

Fig.1.4
RELIEF WELLS
(After Sherard)



The first procedure listed, cutoff construction can be effective, depending on the type of cutoff used. Cutoffs can be either partial or complete. The types more often used are:

- excavated trench, filled with impervious soil
- sheet pile wall
- concrete trench
- slurry trench
- grout curtain

When the dam foundation is on bedrock, the first four types are either difficult to construct or too expensive, or both. For example, a sheet pile wall generally cannot be driven into bedrock and trenches would probably have to be excavated using explosives. This would be too expensive and may be dangerous, due to cracking induced by the excavation method. The effective- ness of horizontal blankets in the reduction of seepage is doubt- full and dependent on the silting of the reservoir area.

Therefore, when the dam foundation is on bedrock the only suitable seepage reduction method left is grouting.

Since grouting is, from a practical point of wiew, the only method suitable for seepage reduction beneath a dam in a rock foundation, this method has been concentrated on in this report, to evaluate its reliabilty and suitability.

Some 50 references were consulted for this literature review, from which 25 are specifically mentioned in this report.

The format of the report is as follows: Chapter 2 is a brief discussion on a controversial subject; i.e. "the need for grouting a determined dam foundation. Chapters 3,4 and 6 are descriptions of various methods of grouting and of common grouts. Chapter 5 is a presentation of grouting procedures, with special emphasis on the differences between N.American practice and European practice. Chapter 7 is a review of the existing theories and experimental analysis. Chapter 8 is an attempt to summarize and harmonize all the recommendations collected in the literature about grout design. Chapter 9 deals with methods to check the grout curtain effectiveness. Chapter 10 is a short presentation of case histories, which the writer found well documented and interesting due to their singularities and overall importance.

CHAPTER II

THE NEED FOR GROUTING DAM FOUNDATIONS

2.1 General

The appraisal of the need for grouting rock foundations is frequently a controversial issue. Sometimes, outstanding engineers with vast experience in dam building have widely different opinions about whether or not a grout curtain should be constructed for a specific dam. Usually several factors must be taken into account such as: height of the dam (depth of contained reservoir), geology of the foundation, type of structure(earth, gravity or arch dam), the allowable water losses, cost of the operation and others, individual to each project.

In general one could say that when the water in the reservoir is valuable and the foundations are very pervious, grouting definitely should be required. In the opposite case, where there is a large surplus of water and the bedrock is relatively impervious there may be no reason for grouting. In the intermediate cases there may be no definite conclusion, so the choice whether or not to grout is an individual decision (ref.16).

A generally accepted opinion is that grouting will only have some value if the permeability of the rock decreases with depth. The evaluation of the permeability is usually done through water pressure tests, which is more closely examined in sections 2.2 & 2.3.

2.2 Usual Criteria for Appraisal of the Need for Grouting

2.2.1 The Water Pressure Test (or Lugeon Test)

This test consists simply in taking measurements of the amounts of water injected into a given borehole at several depth intervals. The Lugeon test is carried out with a standard pressure of 10 kg/cm^2 the length of each interval tested generally being about 5m. The result is expressed in litres of water absorved /minute/meter. This unit is usually called lugeon unit (Ref. 2).

The following sketch (fig. 2.1) illustrates the device and physical execution of the test.

It is generally accepted (Ref.2) that 1 lugeon corresponds to a permeability of $k = 1{,}3 \times 10^{-5}$ cm/s. This value was calculated from correlations established with results of water tests in granular material.

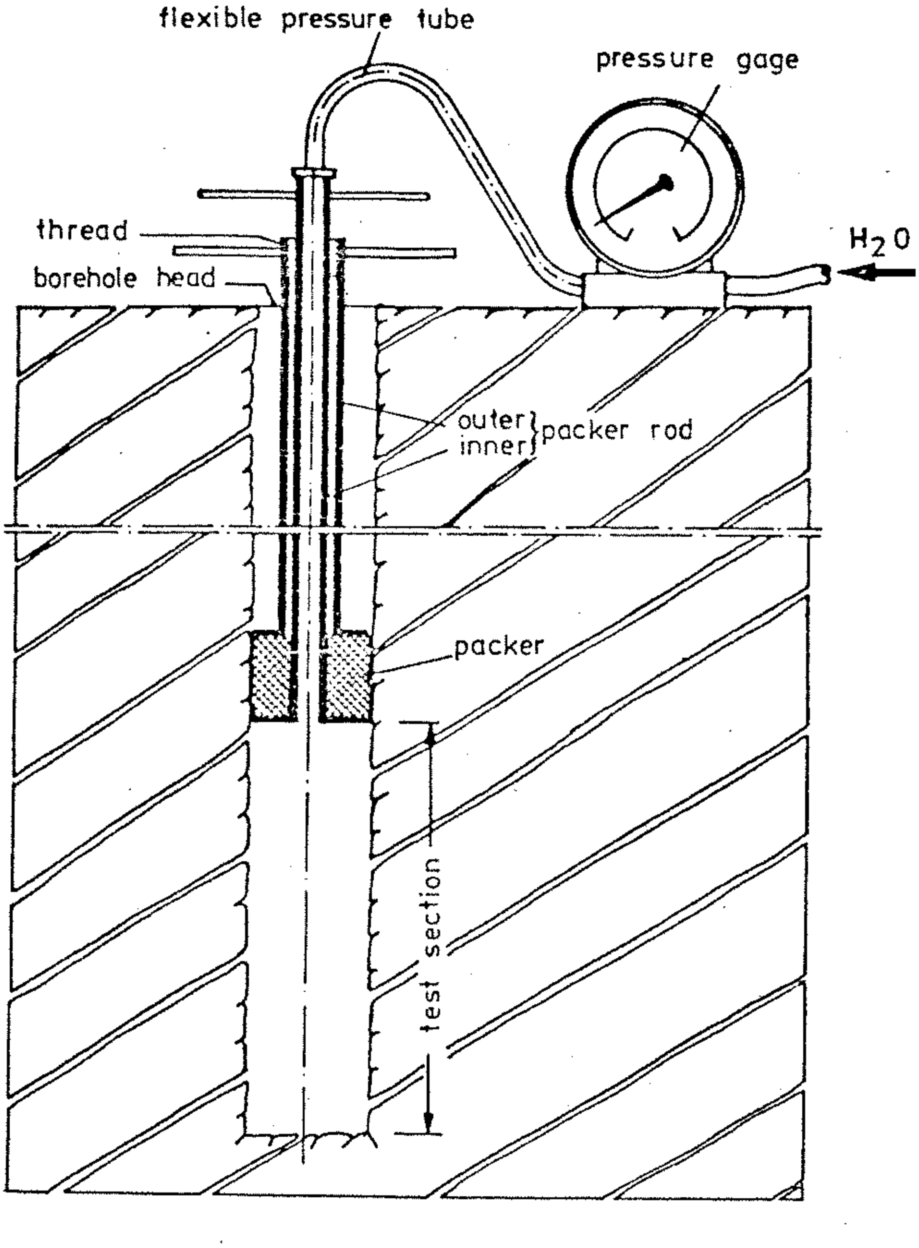


Fig. 2.1 Arrangement of the water pressure test using a single packer

(After Rissler)

2.2.2 Criteria Based on the Water Pressure Test

(a) Lugeon's Criteria

These criteria state that the foundation is sufficiently impervious if the water loss does not exceed 1 litre per meter, under a pressure of 10 kg/cm^2 maintained for 10 minutes and therefore grouting is not necessary. If the dam is lower than 30m, a water loss of 3 lugeon units could be tolerated(Ref.2).

(b) English Practice

According to Lancaster Jones (Ref. 6) acceptable water losses have varied in England from 1 gallon to 0,14 gallon in a 10 ft stage in 10 min at 100 psi. These values are equivalent to 0,22 and 0,03 UL.

(c) Rissler (11) briefly mentions in his thesis that in the USSR a similar method is used. There, the test results are reduced to a unit pressure.

2.3 Criticism of the Existing Criteria

Since the evaluation of the need for grouting has been established based on results of water pressure tests, these criteria have been subject to several criticisms. Some of the notable criticisms are:

(a) Casagrande (Ref.3) mentions some cases of dams where the water pressure tests indicated the need for grouting and the grout curtains proved unnecessary or even useless.

(b) Nonveiller (Ref.10) argues that water pressure tests often provide an overestimation of the permeability of the rock, due to several factors, such as:

- leak around the packer.
- hydrostatic pressures impose new stresses and cause deformation of the surrounding rock with resulting opening of fissures, increasing the actual permeability.
- after the construction of the dam, the seepage pattern may cause some compression of the foundation, thus reducing its permeability.

(c) De Mello & da Cruz (Ref. 4) consider that the type of dam should be taken into account. In this way, they argue, the seepage under a thin arch dam would be much greater than the seepage under a homogeneous earth dam, due to the fact that it would have a shorter seepage path.

They further present some keen considerations about the width of the grout curtain. They consider for example, that for an earth dam having a base width of 275m a grout curtain 4m wide and a permeability 1/4 of the ungrouted rock the improvement would be equivalent to an increase in the seepage path of 20m, what is indeed a very small improvement. In other words, they recommended the use of wider grout zones.

(d) Sinclair, in this PhD thesis (Ref. 17) presents some very interesting considerations on this subject. He appropriately states that the water pressure tests don't yield reliable measurements of the groutability of the rock, since a determined "water take" can be due either to a small number of large fissures or to a great number of narrow fissures.

Fine fissures would allow the flow of water but wouldn't allow grout to penetrate into them. It must be noted however that Sinclair's study refers only to cement grouts.

Sinclair further notices that in cases of open fractures, it is a common occurrence the pump run at full capacity with no pressure developed. This happens when the pump does not have a large enough capacity. Fig. 2.2 illustrates the variation of the permeability with fracture width.

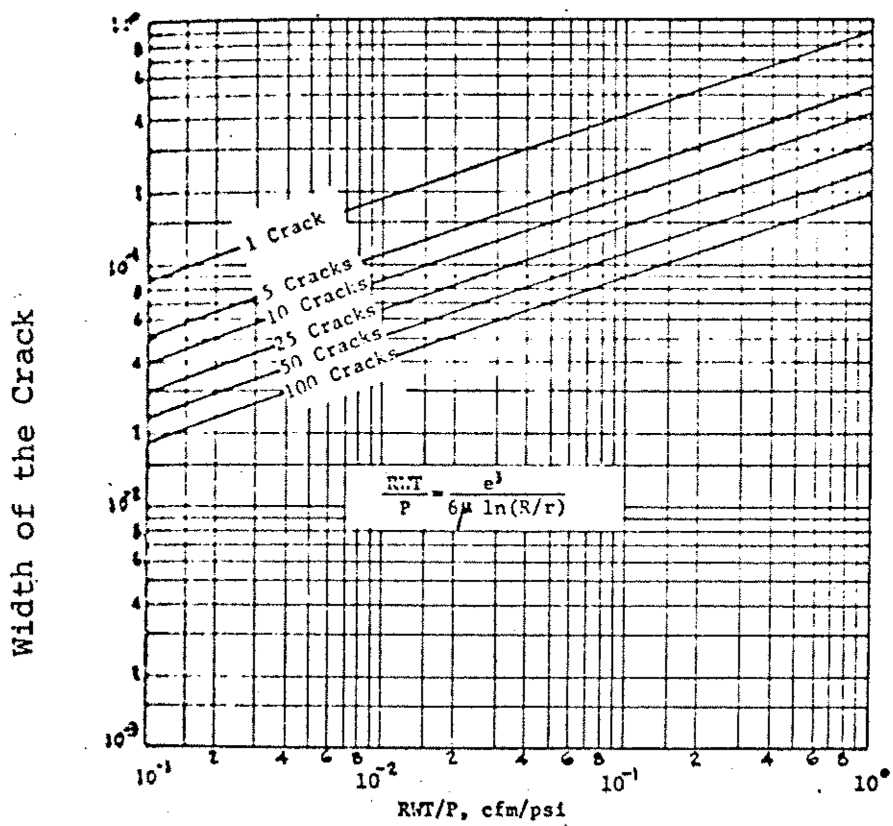


Fig.2.2 Variation of the permeability with fracture width.

(after Sinclair)

2.4 Suggested Criteria for Evaluating the Necessity of a Grout Certain

Some researchers have proposed changes in the current grouting criteria, in view of the criticisms mentioned in section 2.3.

The contributions of De Mello(4) & da Cruz, Nonveiller (10), Sinclair (17) and Rissler (11) particularly reflect on this subject. Their suggestions are presented below in a summarized form.

De Mello & da Cruz (4) propose the following criteria:

(a) No treatment should be necessary for dams of the narrowest "impervious" base widths (approx. 20% of the height if the foundation rock yields coefficients smaller tham 0,3 lit/min/m/atm for dams under 30m; 0,2 for dams between 30 and 100m and 0,1 for dams higher than 100m.

(b) For dams with wider impervious base width the above limiting coefficientes may be increased in proportion to the increase of the base width up to a maximum coefficient of 1,0 lit/min/ m atm.

(c) In all cases, foundation rock with coefficients higher than 1,0 must be carefully considered with regard to requiring both a drainage system and a sealing system.

(d) For intermediate cases either an effetive drainage system or an effetive sealing system may be selected.

Nonveiller (10) proposes the following changes in the water pressure test procedures:

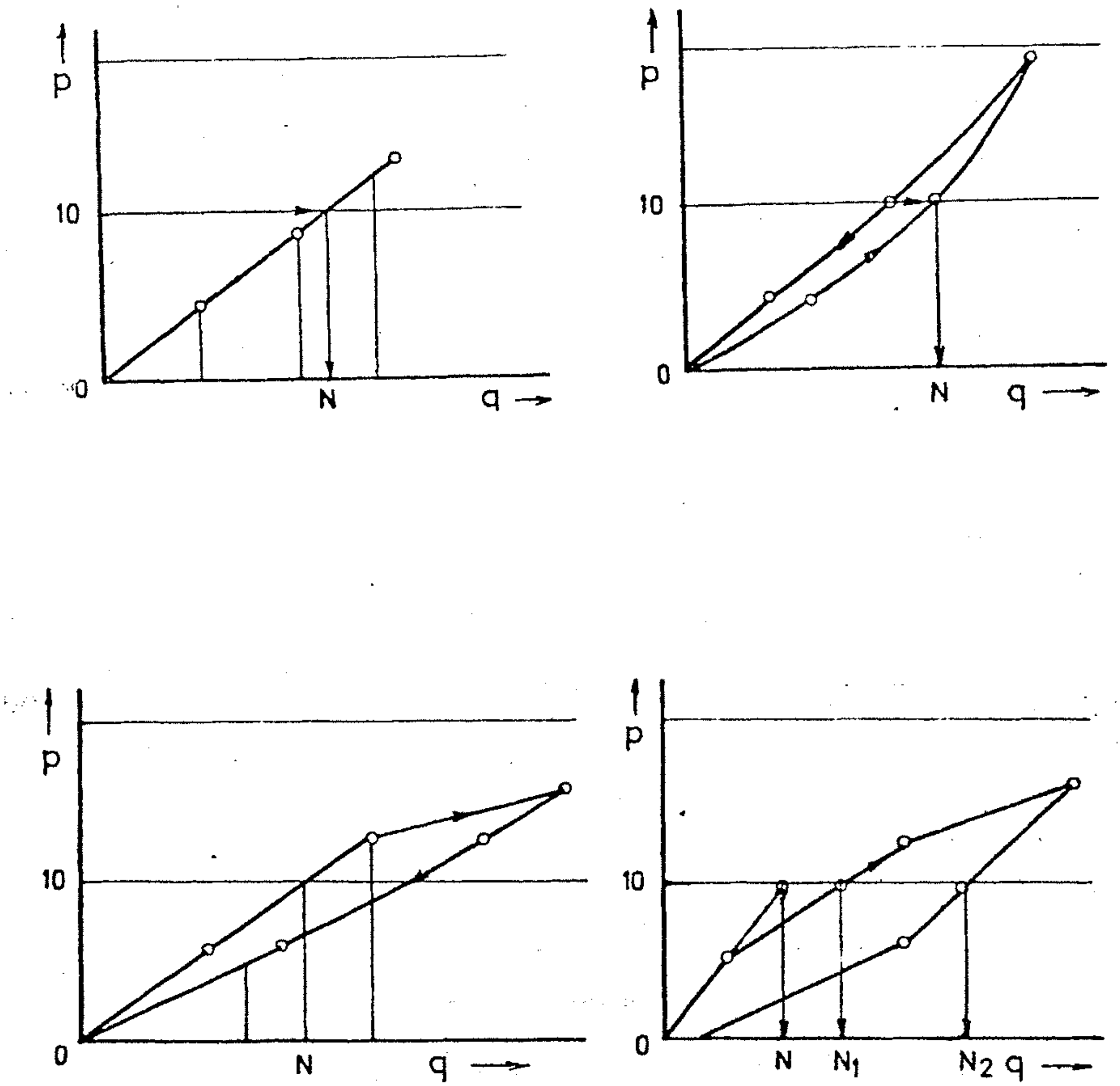
- The percolation should be measured in any borehole stage for three different pressure levels, the lowest pressure being not greater than the hydrostatic head to the depth of the packer.
- The permeability should be computed from the percolation corresponding to the lowest pressure level and not from the percolation at 10 kp/cm^2 pressure, as is usually done.

Nonveiller illustrates his proposal with a table and graphs, showing the differences in the estimated value of permeability using the usual criteria and his criteria. The mentioned table and graphs are presented below (Fig. 2.3).

TABLE 2.1

Checkhole nº	9	17
1- Rate of flow computed from water pressure tests in all stages lit/min	630	65
2- Detto, actually measured lit/min	43	8,3
3- Ratio of computed to measured flow	12	8

Estimated x computed flow of water under dams



Percolation vs pressure diagrams. (A) laminar flow homogeneous material, (B) turbulent flow, (C) opening rock structure under pressure (D) erosion of fissure filling

Fig. 2.3 (After Nonveiller)

Nonveiller, therefore proposes the evaluation of the need for grouting to be done using corrected values of permeability.

Sinclair (17) proposes measures to offset the short-comings of the water pressure tests, consistently with his criticisms. His suggestions are:

- use of pumps of greater capacity so as to make possible the determination of the water (or grout) take for high permeability rocks.
- The substitution of the water pressure tests by grout injection tests, so as to make possible the distinction between groutable and non-groutable fissures. According to Sinclair the water pressure test would consist of two short consecutive periods of equal length grout injection. The average rate of grout slurry absorption for the first period is designated as Grout Injection Index. This parameter relates only to the groutable cracks, unlike the water injection index. The ratio between the rates of injection for the second and first period is termed Ratio of Injection Rate (RIR). Systems of narrow fractures will have - low RIR and low groutability and inversely.

It is the writer's opinion that Sinclair's work is more directed towards an evaluation of cement groutability than towards of the need for grouting from the point of view of permeability. Indeed, if cement grouting is not possible there are other grouts which could be suitable.

Rissler (11) presents in his thesis work a very careful treatment of the subject. He proposes an experimental-theoretical method for determining the anisotropic permeability of fissured rock from the results of the water pressure tests, when the spacial orientation, number of fissures and apertures and relative roughnesses of the fissures are known. In some circunstances a good estimate of these factors can be made as in the case of tunnel excavation, which permits the visual inspection of the systems of joints.

According to Rissler's conclusions the anisotropic permeability resulting from a system of fissures can be calculated using the formula:

$$K = \frac{g}{12\ B\,\nu} \sum_{1}^{n} \left\{ \frac{2(\,a\ i\,)^{3}}{1+\ 8.8\ (Ki/D_{Hi})^{\ 1.5}} \begin{bmatrix} 1-n_x n_x & -\ n_x n_y & -\ n_x n_z \\ 1-n_y n_x & 1-\ n_y n_y & -\ n_y n_z \\ 1-n_z n_x & n_z n_z & 1-\ n_z n_z \end{bmatrix} \right\}$$

n_x, n_y and n_z are the components of the unit vector, normal to the joint planes. The meaning of the other symbols can be found in Fig. 2,4, bellow:

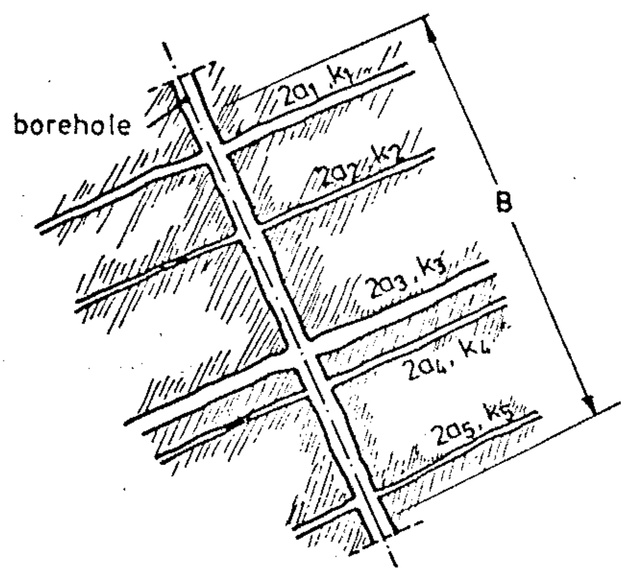


Fig.2.4 Example of a joint set consisting of 5 joints of various apertures, roughnesses and spacings

(After Rissler)

If several sets of joints exist the total permeability of the rock mass can be determined by superimposition of the permeability tensors of each set of joints. Rissler doesn't propose actually grouting criteria, but an acurate method for evaluating the convenience of it.

2.5 Considerations Regarding Safety of Dams as Related to Grouting

This issue was approached by Casagrande, in his Rankine lecture(Ref. 3). He raised doubts about the effectiveness of single line grout curtains for control of seepage. According to Casagrande's opinion grouting should not solely be relied upon for safety of the dam, a drainage system being an obligatory component of the design.

Walker states that "the hazard associated with seepage at dam sites has little if any relation to volume of water loss".

De Mello & da Cruz(Ref. 4) endorse Walker's statement in their 1959 paper,but in his Rankine Lecture De Mello (Ref. 5) points out that "in a rock with open joints hindering infiltration is a definite a-fortiori pre-requisite", meaning that grouting has to be done in this case. He further stresses that a drainage system is obligatory whereas grouting can be a permissible complement in some cases.

Sabarly (Ref.10) shows that drainage systems can become clogged and this could be a very dangerous situation. In a hypothetical situation like this, grouting would then become very important since it would reduce the percolation amount.

Schmidt (Ref. 14) relates the case of Hales Bar Dam, where the fissures in limestone progressively widened under the action of flowing water.

According to he writer's opinion, the safety of a dam relies to a certain extent on the effectiveness of a grout curtain in the circunstances described below:

1- When the drainage system cannot handle the total amount of seepage which would occur without grouting (due to inadequate proportioning, clogging of drains, increase of permeability, etc).

2- When the foundation rock is either erodible or soluble.

CHAPTER III

TYPES OF GROUTING - A GENERAL VIEW

3.1 General

From a survey of the literature several basic types of grouting were identified, according to the individual purpose as listed below:

- foundation grouting
- anchor grouting
- grouting for reclamation of civil engineering works
- grouting for correction of settlements

Foundation grouting in its turn, can be subdivided into two types, according to the foundation terrain: grouting of soils and grouting of rocks.

Grouting of fissured rock can be divided into 5 classes according to the Corps of Engineers (21) and into 2 classes, according to the usual classification (2, 16,23).

3.2 The Corps of Engineering Classification

According to the Corps' classification there are 5 classes of rock foundation grouting, as described below:

- Curtain grouting: This is the drilling and grouting of one or more parallel lines of holes in a foundation or along reservoir rim for the purpose of creating a barrier of cutoff against excessive seepage.
- Consolidation grouting is an operation performed over an areal grid pattern in plan and to a

relatively shallow depth into the foundation, for the purpose of "consolidating" a mass of highly fractured or otherwise defective rock.

- Contact grouting is the injection of a grout slurry at the contact of a structure with a vertical or nearly vertical rock surface for the purpose of sealing any water passages that may exist due to shrinkage.
- "Dental Treatment" is the operation of cleaning a rough surface and filling with mortar, grout or concrete. Localized pockets of weathered rock, potholes, faults or other foundation flaws that extend below the general foundation surface.
- "Slush Grouting" is the filling of surface irregularities and open fractures in a rock foundation with a sanded grout or mortar on which earth fill is to be placed.

3.3 The"Usual"Classification

According to the more common dam designer's terminology there are two basic types of rock foundation grouting, ie. curtain grouting and consolidation grouting. The other 3 types mentioned by the Corps' classification are considered complementary civil engineering works.

In this report only curtain grouting will be dealt with.

CHAPTER IV

GROUTING METHODS

4.1 General

As an introduction to this chapter we shall first of all define clearly the distinction between grouting methods, grouting procedures and grouting design. Grouting design is defined as determination in advance of construction of:

- geometrical characteristics of the grout curtain such as depth of the grout holes, configuration of the curtain, number of lines, inclination of the holes, spacing between holes, initial guides on grout mixes, pressures and when to grout.

Grouting procedures are defined as general field measures, which can be changed according to the conveniences and requirement by the Field Engineer, or by the Inspector. The procedures refer to drilling operations, choice of grouting pressure and grout mixes during the conduction of the works.

Grouting methods are defined as one of the two standardized ways of conducting the grouting operations for a determined job, i.e. stage grouting and packer grouting.

In stage grouting the hole is drilled and grouted in stages of depth from the top, with the grout being inserted through a pipe nipple which is fixed into the rock at the top of the hole. The hole is usually drilled in two or more increments of depth. After the first stage hole is grouted completely, the hole is redrilled and extended down to the second stage. The new, greater lenght of hole is now grouted under a higher pressure. If a 3rd stage is used, the process is repeated.

Cambefort (2) recomends the lenght of each stage not to exceed 10m. Sometimes, stage grouting has been employed for full depth treatments of considerable lenghts. However, as Cambefort stresses this is not advisable because of the likelyhood of sedimentation in such a long columms.

In packer grouting the holes are drilled at once to the full proposed depth. Subsequentely a "packer" which is a mechanical device for sealing off the hole at any elevation is inserted and sealed against the walls of the hole. After the packer is in place grout is injected. Grouting of the hole is completed by gradually raising the packer in sucessive stages and grouting at successively lower pressures. If a double packer is used, grouting could be done either from the bottom of the hole up, or from the top down.

The figures below illustrate the physical execution of the mentioned methods.

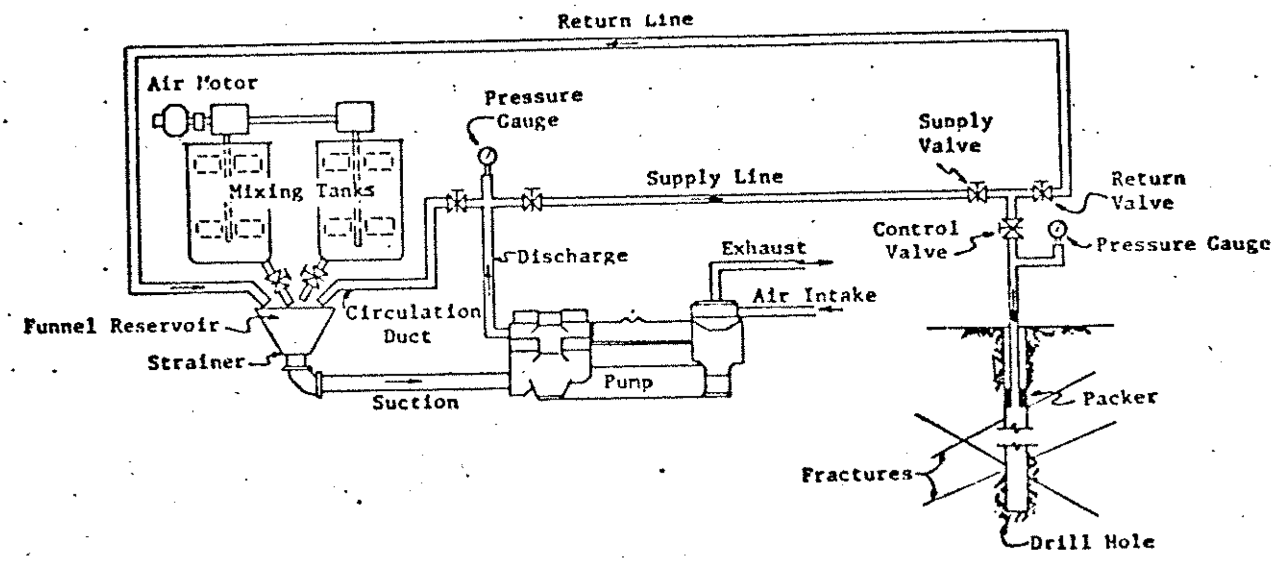


Fig. 4.1

Typical arrangement of a packer grouting installation (After Sinclair)

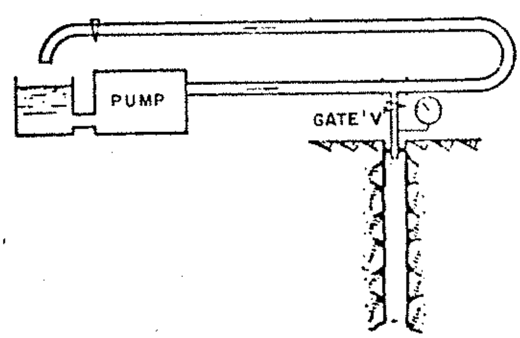


Fig. 4.2

Stage grouting

(After Sabarly)

required and less pipe connections are needed.

2- Packer grouting allows the use of high pressures without the attendant risk of surface breakouts.

3- Packer grouting"on the way down" enables very localized treatment of any seam encountered during drilling. This is accomplished by setting the packer just above the "leaky" zone and injecting with the desired mix. Double packer grouting can also be used to treat "leaky" zones even if the entire hole is open. Stage grouting is less versatile in these respects.

Sometimes in extremely fractured rock it is not possible to use packer grouting, since the grout can find its way upward, and the packer can be become grouted in the hole.

CHAPTER V

TYPES OF GROUT

5.1 General

A survey of the literature on grouting revealed that the bibliography on this important subject is extensive. Indeed there are works by Cambefort (2), Task Comitees of Cement Grouting (19) and Chemical Grouting (20) and tens of other references. However, it is the writer's opinion that the only one who approaches the problem of curtain grouting in bedrock in a systematized and comprehensive way is Cambefort (2). Therefore, this chapter will be a brief summary of Cambefort's work on types of grout. Cambefort divides grouts in 3 basic types:

(a) Cement grouts

(b) Chemical grouts

(c) Asphalt grouts

Cement grouts can be further subdivided into stable and unstable grouts.

There is still a fourth type, which he calls aerated grout. It consists of an air emulsion of any one of the other 3 types.

In the following sections each one of the afore mentioned types are discussed and also the criteria that should be used for the choice of a particular grout type.

5.2 Relevant Properties of Grouts

The more relevant properties of mortars of injection are, namely:

- viscosity
- consistency, including thixotropy and rigidity
- particle sizes
- bleeding or segregation
- setting time

These properties can be measured by specific tests whose description is beyond the scope of this work.

There is no such thing as standard properties for grouts. Indeed, the requirements which have to be satisfied by a grout vary widely with the characteristics of the foundation to be treated and with its purpose.

The destinction between stable and unstable grouts is made on the basis of the "bleeding" properties of the grouts as it will be seen in sections 5.3 & 5.4.

5.3 Cement Grouts

5.3.1 Unstable Grouts

The prototype of the unstable grout is the common cement - water mortar, provided the mortar is dissolved enough. It is "unstable" because the cement grains are large, so as to display appreciable sedimentation before setting of the cement.

Other kinds of unstable grouts are the mixtures of water, cement and a filler, such as sand, sawdust, flour,etc. The properties of the grout may vary widely according to the choice of the filler.

In a general way, unstable mortars have more than enough strength to withstand the stresses originated from the water pressure.

5.3.2 Stable Grouts

Stable grouts are suspensions in water of grains small enough so as not to display any bleeding or sedimentation during the injection. The simple addition of a colloidal clay to an unstable mortar renders it stable, since the colloidal particles prevent the sedimentation of the larger ones.

Cambefort (2) divides the stable grouts into three main classes: suspensions of cement with high final strenght clay suspensions and suspensions of sand-clay-cement. Each of these classes comprehends several grouts, as listed below:

(a) Suspensions of cement with high final strength

- cement bentonite
- cement sodium silicate
- activated cement mortars
- cement - ashes
- cement - rock flour
- cement - sawdust

(b) Clay suspensions

- treated clay
- clay - oil
- clay gels

(c) Suspensions of clay-sand-cement

- clay - cement
- clay - cement - sand

Each one of the above mentioned grouts has its peculiar composition and characteristics. Sometimes the difference between two of them is just a matter of grain size distribution of the clay, or of a particular admixture.

It would be pointless to describe here all the above grouts and their properties (See Ref. 2 for detailed description).

In section 5.7 there are descriptions of the use of a few grouts, with provide an illustration of what can be achieved by the choice of an appropriate mortar.

5.4 Chemical Grouts

Cambefort (2) distinguishes basically between 3 classes of chemical grouts:

- hard gels sodium silicate and lignochromiun
- plastic gels of sodium silicate and deffloculated bentonite
- organic resins

Chemical grouts are generally liquids with about the same viscosity as water, and therefore with great capability for penetrating fine granulated soil and fine fissures in rock. Some of these grouts set with the consecutive injection of the grout and the reactive and others set over a requisite time period.

The use of chemicals in injection of fissured rock is not common mainly because of their high cost and also because the common cement grouts are generaly satisfactory for most jobs. Indeed, in a survey of the literature the writer could not find any case of rock impermeabilization with chemical grouts.

5.5 Asphalt Grouts

These grouts according to Cambefort's opinion would be the ideal injection mortars if they were cheaper and easier to place in the construction site.

There are basically two types of asphalt grouts:

- Solid asphalt which must be heated in order to be injected.
- Asphalt emulsions, which can be injected without heating. The rupture of the emulsion is brought about by the addition of a specific chemical.

There are at least two patented asphalt grouting procedures: Shellperm and Soletanche (2).

In order to decrease the cost of these grouts,in some circunstances a filler such as sand, or any other granular material can be added.

5.6 Aerated Grouts

These consist of any one of the aforementioned grouts with emulsionated gas, in such a fashion as to form a foam. In some circunstances this expedient can afford substantial savings due to a smaller grout consumption. This solution is particularly interesting for injection of cement grouts, since it improves some properties of this mortar. For instance, the bleeding decreases, the rigidity increases and the penetrability increases remarkably.

These grouts can be prepared by the addition of a specific chemical. For example, alumininium powder added to cement grout gives origin to hydrogen bubbles.

5.7 The Choice of an Appropriate Grout for a Specific Case

In the process of choosing an appropriate grout for a given job several factors have to be taken into account:

- properties of the grout, such as viscosity, thixotropy, setting time.
- relative price of the grouts - this a very important factor indeed. Cambefort presents a comparative table of prices for French conditions, as reproduced below:

TABLE 5.1 Comparative prices of grouts

Grout	Relative price
Water cement suspensions	4,2
Clay - cement	1
Joosten Gel	10,7
Hard gels Soletanche	6.3 - 11.4
Gels of Lignochromium	6.5 - 8
Plastic gels silicate-aluminate	2 - 4
Deffloculated bentonite	1,8
Deffloculated clay	11
Clay - oil	20 - 35
AM 9	50 - 130

Grout	Relative Price
Resorcin - formol	10 - 40
Asphalt - silicate emulsions	6
Asphalt - resorcin emulsions	25

- Nature and conditions of the rock foundations
- Depth and equilibrium state of the grounelwater

The nature of the problem doesn't allow the establishment of definite rules, but there are some general guides which can be regarded as good first indications as summarized below:

- for finely fissured rocks, the common cement grout is the treatment to start with. Grout consistencies start with 1/10 cement/water relation in volume and thicken progressively up to 1/4 or even 1/2.

- The grouting of fissures in porous rocks is difficult since the porous walls of the fissures absorb water from the grout, rendering it ungroutable. Is is necessary to inject a sodium silicate gel before injecting cement grout. The sodium silicate gel obturates the pores of the porous rock, allowing for a later common cement - grout injection.

- Open fissures can be treated with a clay - cement grout, since there will be considerable cement savings. Sometimes it is convenient to use grouts with short setting time, so as to reduce the grout losses. Another expedient is the addition of sawdust to the grout, in such a way as to rapidly increase its viscosity, so preventing the grout from travelling long distances away from the injection hole. The figures below illustrate these concepts:

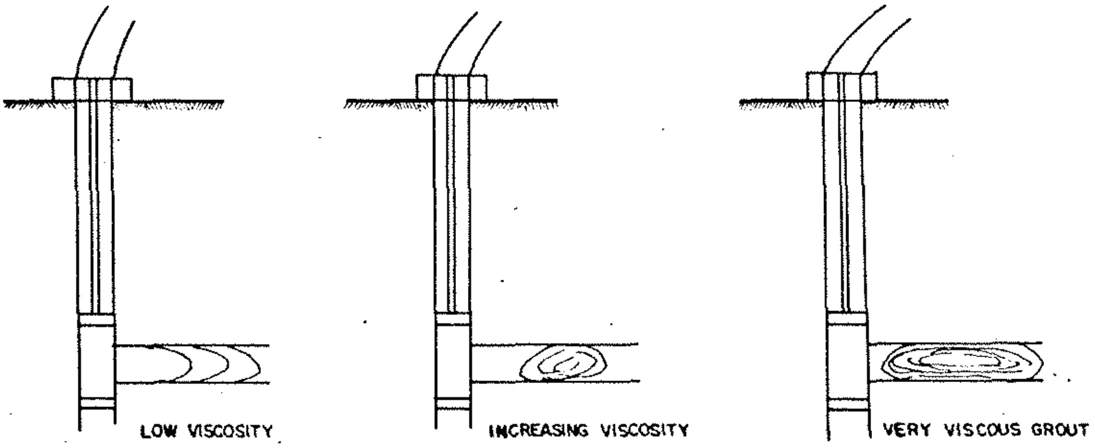


Fig.5.1 -Injection of a mixture grout-sawdust

When the fissures are important zones of water flow as is often the case in limestone terrain, common grouts would be washed away. The solution in this case would be to construct an inverted filter in the cavity and after the filter is in place to inject hot asphalt wich would coagulate immediately in contact with the water. The fig. 5.2 below, illustrates these procedures:

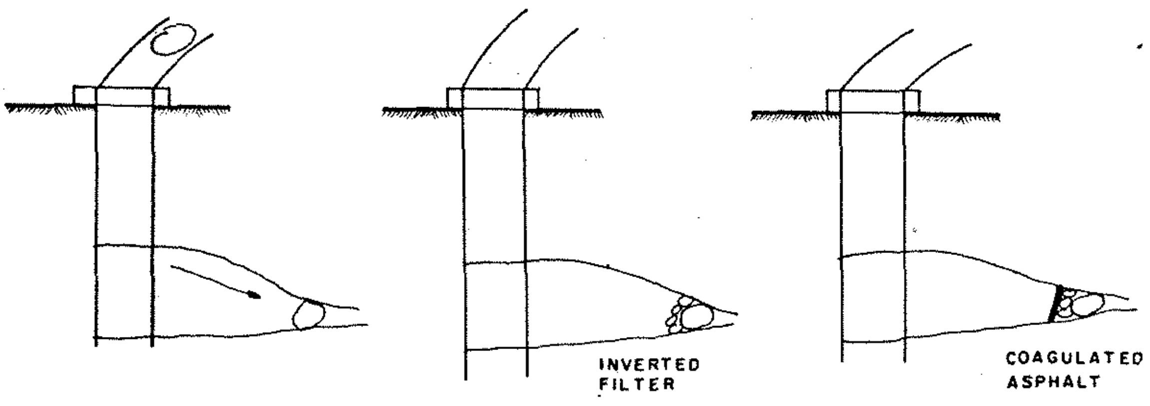


Fig.5.2 - Suggestion for grouting of fissures in limestone

When there are both wide fissures and fine fissures the large cavities would be obturated first using proper grouts and the fine fissures would have to be grouted using a more diluted grout.

Finally, we should remind that the injection of fine fissures with hot asphalt could yield good initial results but it would not be a final solution, since the water pressure would slowly expel the asphalt from the cracks. An asphalt injection combined with a cement grouting would be a lasting solution.

CHAPTER VI

GROUTING PROCEDURES

6.1 General

As mentioned in section 4.1 Grouting Procedures refer to drilling operations, choice of grouting pressure and grout mixes during the conduction of the works.

There are two main grouting practices: the European (or French) Practice and the North American Practice. The methods and design of grouting are not too different between them, but the Procedures are widely different. Several authors have already discussed the conflicting features of this subject, but there is no definitive conclusion so far. Therefore, we will briefly present both practices and the applicable criticisms.

6.2 Drilling

Drilling can be done either by percussion or rotary techniques. There is a controversy about the advantages and disavantages of each method, as summarized below:

(a) Advantages of percussion drilling-lower cost

(b) Disavantages of percussions holes - greater possibility of clogging of the entrance of the fissures by the drill cuttings - deviations.

- Roughness of the walls of the hole.

(c) Rotary drilling doens't have such disavantages but is has a much higher price per meter.

It must be stressed that if packer grouting is to be used, rotary drilling is preferable, since it provides a better seating for the packers.

It is a generally agreed opinion that the diameter of the borehole is irrelevant. In most cases the diameter of the hole does not have any influence on groutability or "grout take" and therefore the smaller the diameter, the greater the cost savings. In North American terminology there are 3 standard sizes of holes:

EX (Ø = 1 1/2"); AX (Ø = 1 7/8") and BX (Ø = 2 3/8").

6.3 Grouting Pressure

This is definitely the most controversial issue between the European and North American Practices.

- Both practices accord that grouting of upper layers of the bedrock should be conducted at lower pressures, and
- the grouting pressures should be a function of the weight of the overlyng rock.

Regarding other factors, there are considerable disagreements, as will be discussed in the sections following:

(a) North American Practice

The traditional North American Practice is to recommend injection pressures not to exceed 1 psi/ft of depth. These pressures would be such as to insure against uplift on the basis of the weight of the overburden, neglecting the uplift resistence of the rock mass. This criterion is recognized as conservative, however, and is reserved only for weak rock, horizontally jointed rocks or to the upper layers of the foundation.

In 1962 the "Task Comitee for Cement Grouting" (19) presented a suggested graph for alowable grouting pressures as a function of the rock type. Substantially higher pressures are recommended in this work. This graph is presented in fig. 6.1 .

North American Engineers generally object to the occurrence of either uplift or fracturing caused by grouting. They argue that hydraulic fracturing would open new fractures which could be dangerous and that if these fractures were filled with grout this would be a waste of grout. The pressure which could give rise to hydraulic fracturing was computed by Morgenstern & Vaugham (8). This subject will be treated more closely in section 7.

An important factor that must be considered in this discussion is that the penetrability of the grout depends on the relative dimensions of the cement grains and the fissure apertures; in North America, generally finely ground cement is used and therefore the necessary pressure needed for achieving a certain penetration of the grout is lower than in Europe, where coarser cements are used.

Some North American researchers found that the shear strenghts of a newly grouted rock is less than that of a intact rock. It is the writer's opinion that the loss of strenght is irrelevant in most practical cases, since the strenght of fractured rock is good enough from the point of view of bearing capacity.

In North American Practice the refusal is viewed as the situation where there is very litle grout absorption under the maximum alowable pressure.

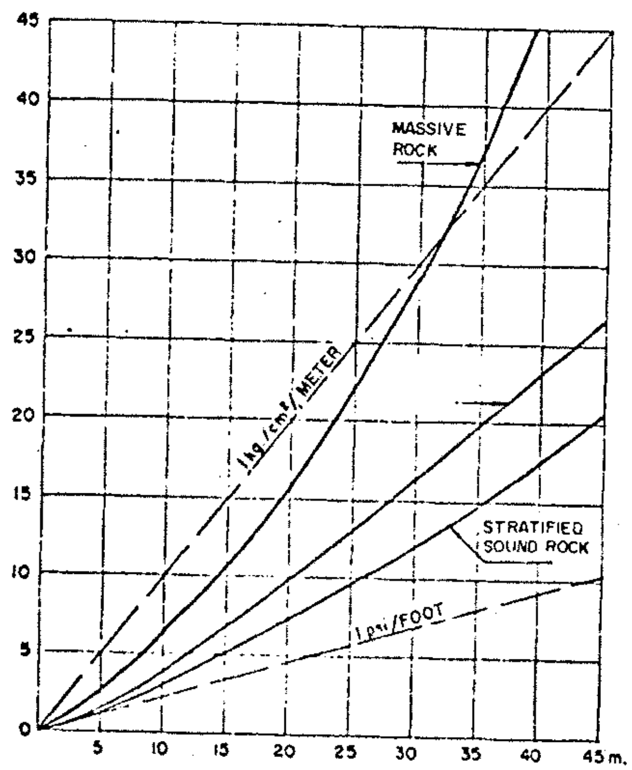


Fig. 6.1 - Grouting pressure x depth
Comparative graph
(After Sabarly)

(b) European Practice

Traditional European Practice recommends the use of pressures up to 5 psi/ft of depth. Cambefort (2) in his comprehensive book on grouting argues that the only way to grout certain fissures is through the use of high pressures, in such a fashion as to provoke the elastic deformations of the rock mass, with the consequent increase of the opening of the fissure. When the application of the pressure ceases, the grout is compressed by the rock, forming a watertight joint, as sketched below:

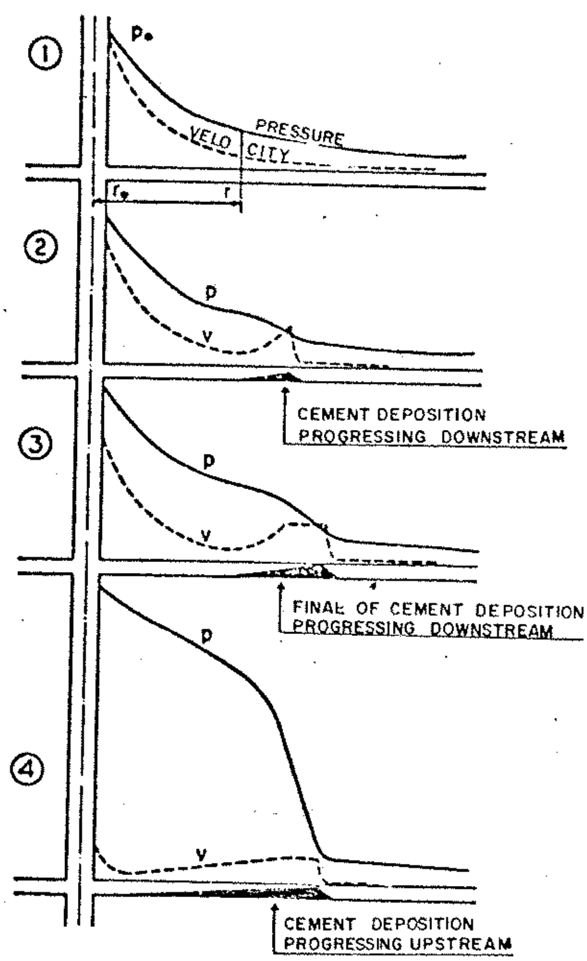


Fig. 6.2 - Formation of a joint under high grouting pressure

(After Cambefort)

Cambefort further states that in some instances the hydraulic fracturing of the rock mass could be even desirable. Indeed, when the state of stress in the rock mass is such as to allow the formation of vertical cracks, one can take advantage of this situation and try to form a continuous vertical grout curtain. However, this a very particular situation. In the more general cases, Cambefort regards cracking as undesirable but unavoidable since use of lower pressures would conduct to unsatisfactory grouting. He views the phenomenon of cracking as not being so dangerous as depicted by the N.A. engineers.

Cambefort's views are generally supported by other European authors, like Sabarly (10), Nonveiller (10), Zaruba (25) and others.

In European Practice the refusal is understood as the situation where the grout take is very small under a pressure a little higher than the pressure needed to grout, ie, to open the existing fissures and not as to give rise to uplift. One could think that this concept would lead to exaggerated grout takes, but this is not so because the viscosity and groutability of the mortar can be controlled so as not to allow waste.

6.4 Grout Mixes

In chapter 5 the problem of choice of the grout has been approached and the great number of available grouts has been mentioned. In this chapter some brief comments on the usual procedures of cement grouting will be made. It is beyond the scope of this report to go into details on such an extensive subject.

(a) North American Practice

The tradicional North American (Corps of Engineers) Practice of cement grouting is to use a maximum water/cement ratio of 3 or 4 and a ratio of 1:1 is the most common. Sometimes the use of fillers or admixtures is considered depending on the conditions of the foundation to be injected. The grout consistences are not changed very often during a job in the N.A. practice.

In a survey of the North American literature the writer failed to find any recommendation about the grout consistency as a function of the water pressure test results.

(b) European Practice

Cambefort and Zaruba present some very interesting observations based on their experience in Europe.

Zaruba (25) suggests that grouting of closed joints should be done with a thin mixture (W/C = 8/1) and for larger cracks near the drill hole a thicker mixture (W/C=4/1 to 1/1) is used. Wide cracks should be filled with a thick mixture (1:4 eg) at the beginning and completed with a thin mixture (1:8 to 1:10). Zaruba (2) suggests that when cracks are narrower than 0,2 mm clay or chemical grouts should be used. As already discussed this is not Cambefort's opinion.

Cambefort (2) suggests that a approximate relationship between the results of lugeon tests and grout mixtures can be established for a given refusal pressure of 50 kg/cm^2 (very high pressure). This relationship is summarized in the table below, for unstable grouts.

TABLE 6.1 Suggestions for grout mixes

Water absorption lugeon units	Grout mix C/W at the beginning of the Injection	Final Grout Mix
1 - 2	1/8	1/4
2 - 5	1/8 1/4	1/2
5 -10	1/4	1/2 1/1
10	1/2 or stable mortar	

These are only rough guides, of course.

CHAPTER VII

THEORETICAL AND EXPERIMENTAL STUDIES ON GROUTING

7.1 General

As an introduction it can be said that grouting presents problems practically impossible to tackle analytically. Indeed, the heterogeneity of rock foundations is such as to render useless any mathematical or physical model. The degree of accuracy in any feature of grouting design is quite distant from that usually achieved in Civil Engineering in the sense that in most fields of civil, it is possible to predict the results of a determined design procedure with a certain reliability. In grouting the reliability of these predictions is doubtful at best. In some situations the efficiency of the treatment is zero, in others it is satisfactory. Nevertheless there are some specific points in this subject where important qualitative conclusions can be drawn from simplified models. These particular points will be briefly reviewed in section 7.2.

7.2 Theoretical Approach to Grouting

The few points in this subject which are amenable to any theoretical treatment are:

(a) Injection of a newtonian fluid into a fissure

An equation relating pressure, quantity of flow and depth of penetration of grout into a horizontal single fissure has been derived by W.J. Baker (2).

$$P = \frac{6 \nu Q}{\pi e^3} \ln \frac{R}{r_o}$$

where:

ν = viscosity coefficient

e = width of the fissure

Q = quantity of flow at a determined point

R = distance from the grout hole

r_o = radius of the grout hole

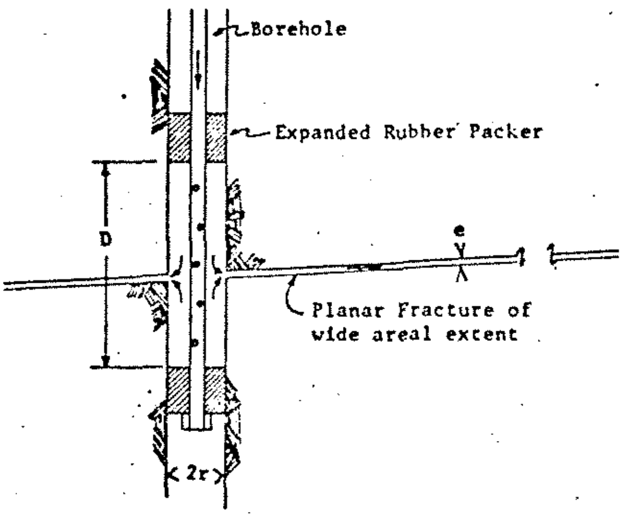


Fig.7.1 - Injection of a fissure (After Sinclair)

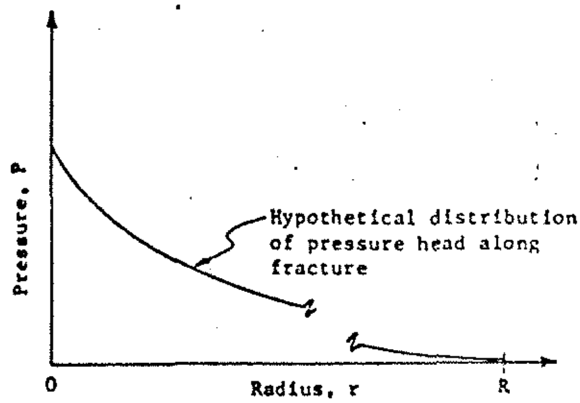


Fig. 7.2 - Distribution of pressure along the fissure (After Sinclair)

From the equation above, the pressure at the beginning of the injection is strongly dependent on the quantity of grout injected for a given fissure and a given grout mix.

(b) Influence of pressure on the opening of fissures

Cambefort and Sabarly studied the effect of pressure on the opening of a single fissure (2). Cambefort derived the simplified formula below:

$$W = \frac{2}{\pi} F \frac{1 - \left(\frac{1}{m}\right)^2}{rE} = 1{,}5 \frac{p\ r}{E}$$

where:

W = deformation or opening of the fissure

p = average pressure

r = radius of action of the pressure to be considered

E = elasticity modulus

Sabarly(13) derived a similar formula, as below.

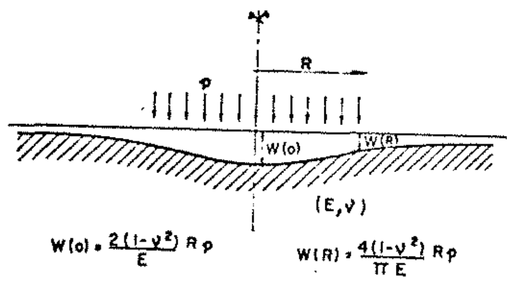


Fig.7.3 - Fissure grouting After Sabarly

(c) Determination of the allowable grouting pressures

Morgenstern and Vaughan (1963) (9) developed a theoretical criterion for the determination of the allowable grouting pressures, so at not to instigate hydraulic fracturing. They derived the formulas below:

for $K < 1$

$$P_e = \frac{(\gamma h - \gamma w \ hw)(1+K)}{2} - \frac{(\gamma h - \gamma w \ hw)(1-K)}{2 \sin \emptyset} + c' \cot \emptyset'$$

for $K > 1$

$$P_e = \frac{(\gamma h - \gamma w \ hw)(1+k)}{2} - \frac{(\gamma h - \gamma w \ hw)(1-K)}{2 \sin \emptyset'} + c' \cot \emptyset'$$

where

$\emptyset$ = angle of shearing resistance

c' = cohesion intercept

γ = bulk density of the material above the level under consideration h = heigth of the material above the level under consideration

hw = piezometer level of the groundwater above the level under consideration

$$k = \frac{\sigma_h}{\sigma_v}$$

P_e = allowable pressure

This study finds its best application in the case of injection in soils and in porous or weak rocks.

(d) From a comparative point of view Cambefort (2) and Sabarly (13) studied the relationship between the number and width of fissures with the permeability. This study is illustrated in Fig. 2.2.

(e) From a macroscopic point of view, Sinclair (17) determined some equations relating the results of water pressure tests with "grout take". This subject is studied more closely in Chapter 8, Section 8.4.

(f) Again from the macroscopic point of view Nonveiller (10) presents curves relating amount of "grout take" and permeability with hole spacing. However he does not mention the range of application of these curves, since they are based on statistical studies (see Fig. 8.12).

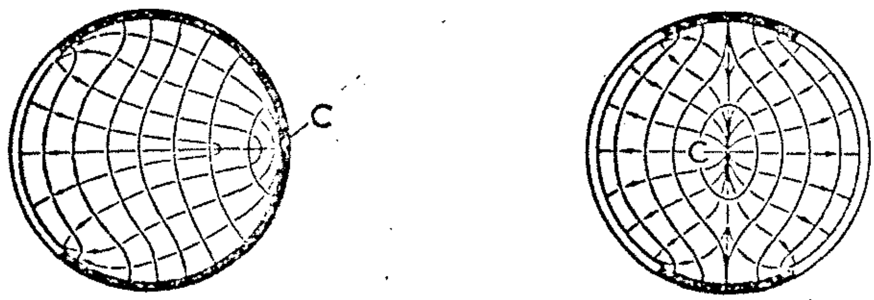
7.3 Experiments on Grouting

7.3.1 Laboratory Experiments

The writer could find only 2 references with respect to laboratory experiments on grouting, in his survey of the literature.

The earlier experiments were conducted by Bernatzik (2) and refer simply to the problem of grout circulation inside the boreholes. He carried out experiments injecting grout into porous pipes with impermeable spots and studied the configuration of the cement deposits inside the core.

The illustrations below reproduce some of his findings:



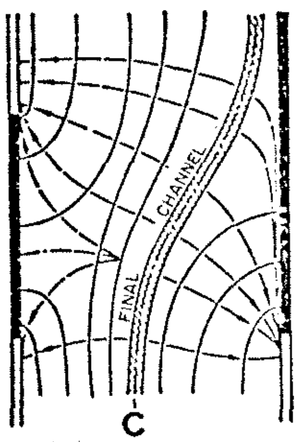


Fig. 7.4

Equipotentials and flowlines in a borehole during injection (After Cambefort)

The later experiments were performed by the US. Corps of Engineers in Vicksburg (23). They consisted of grouting tests in an artificial opening in such a way as to induce planar flow of the grout. The sketch below illustrates the device used for the tests.

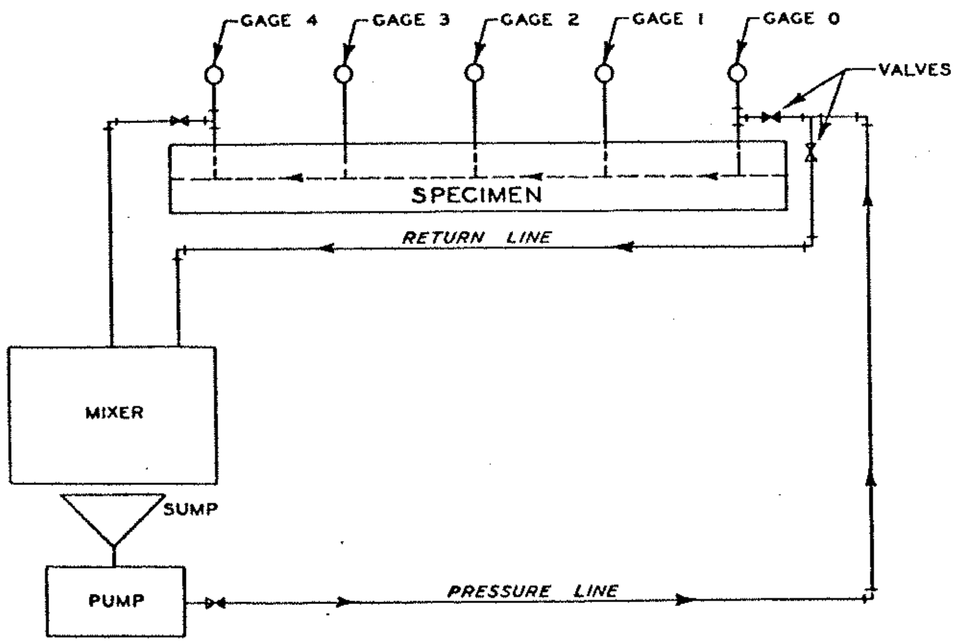


Fig.7.5

Device used by the U.S. Corps of Engineers for grouting tests

The test results were not satisfactory as it was never possible to completely fill the fissures due to bleeding of the grout. Therefore, there was never bonding between the grout and the upper wall of the fissure.

Cambefort(2) comments on these experiments, considering that probably the unsatisfactory results of the experiments were due to failure in reproducing the field conditions, ie, using too rigid a device. Besides, the tests results were conducted in a planar flow fashion, which is not the actual grout flow pattern.

7.3.2 Field Experiments

Field experiments consist usually in full scale or sometimes small scale tests with possibly extraction of cores. Cambefort relates 4 cases of such field experiments. In some circunstances excavation of tunnels has been done in order to observe the results of the injection works.

Benko relates in detail the way a large scale grouting test was performed at Portage Montain Dam site in B.C. (see section 10.4.).

CHAPTER VIII

GROUTING DESIGN

8.1 General

As discussed in Section 4.1 Grouting Design is understood as the determination of the geometrical characteristics of the grout curtain, the establishment of initial guides on the procedures, methods of grouting and when to grout. An approximate prediction of the grout takes would also be desirable for tender documents.

As some items in this chapter have already been treated in early chapters, we will only refer to the respective section, if no further considerations are judged necessary.

8.2 Required Input Data for Grouting Design

The input data usually necessary for designing a grout curtain are:

(a) Detailed geological survey of the damsite, including information about types of rock, ground water, faults, fissures, foldings.

(b) Permeability of the bedrock. These data are generally determined from water pressure tests (or lugeon tests) as described in section 2.2.

(c) Overall design of the dam.

(d) Availability and cost of grouts and grouting equipment.

(e) Availability and cost of drilling.

(f) Information about the state of the stress in the bedrock.

(g) A study of similar case histories.

Information about a, b and c are usually available prior to the bid; d and e would have to be estimated by comparison with other jobs. Information f has not been usually available but more recently the trend is to attempt to determine the state of stress of the bedrock in situ.

It shoud be added that it would be desirable to carry out some additional tests, as listed below, prior to the beginning of the grouting operations.

- laboratory tests on all the suitable constituents
- grouting tests at the dam site, for determining allowable pressure, grout hole spacing, grout mix to be used effect of admixtures, setting time of the grout and estimation of the total "grout takes".
- drilling tests, to determinate the suitability of a determined drilling method and diameter of hole.
- grouting tests as proposed by Sinclair (see section 2.4) would provide useful information on the cement groutability of the rocks mass.

8.3 Geometrical Chracteristics of the Grout Curtain

The geometrical characteristics of the grout curtain which would have to ne predicted are namely:

(a) Plan view arrangement of the grout holes

(b) Depth of the grout holes

(c) Number of grout lines

(d) Inclination of the grout holes

(e) Grout hole spacing

(f) Drilling diameter

These features will be discussed separataly below:

(a) The grout holes should be located in such a way as to constitute a continuous watertight barrier together with the dam or together with the upstream impervious membrane, as sketched below.

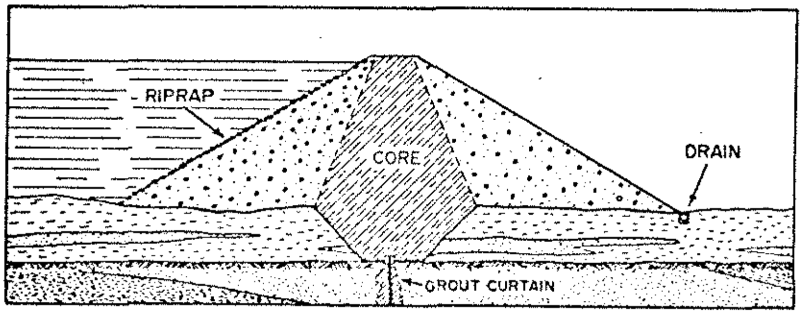


Fig. 8.1
Position of the Grout Curtain
(After Wahlstrom)

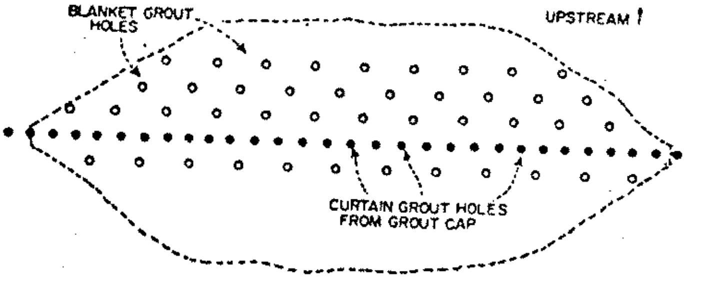


Fig.8.2
Plan View of the Grout Curtain
(After Wahlstrom)

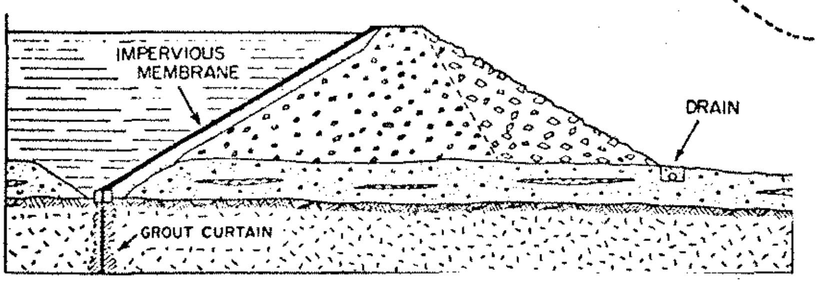


Fig.8.3 - Grout curtain under dams with upstream membranes

When the dam is a homogeneous earth embankment or a concrete gravity one, the grout curtain should be located close to the heel of the embankment, in such a way as to leave enough space downstream for the relief wells.

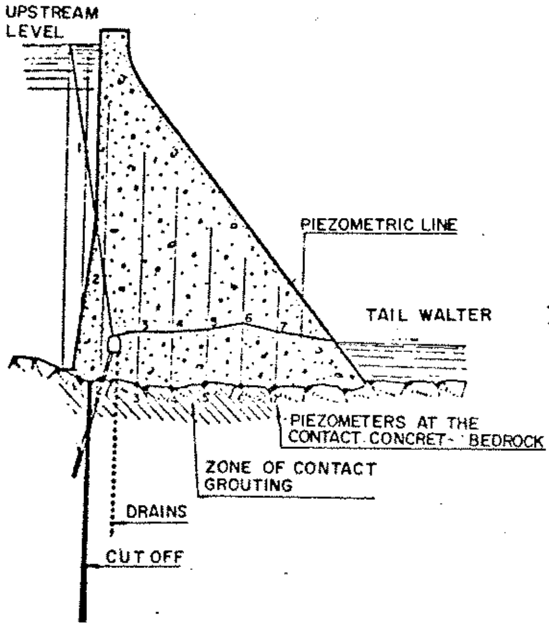


Fig. 8.4
Position of the under a concrete gravity dam (After Sabarly)

When it is a thin arch dam, the deformation of the structure must be taken into account. Indeed, by the time of the first filling tension cracks may occur at the heel of the dam, is such a way as to render the grout curtain useless. Sabarly (13) proposes a design to offset this problem.

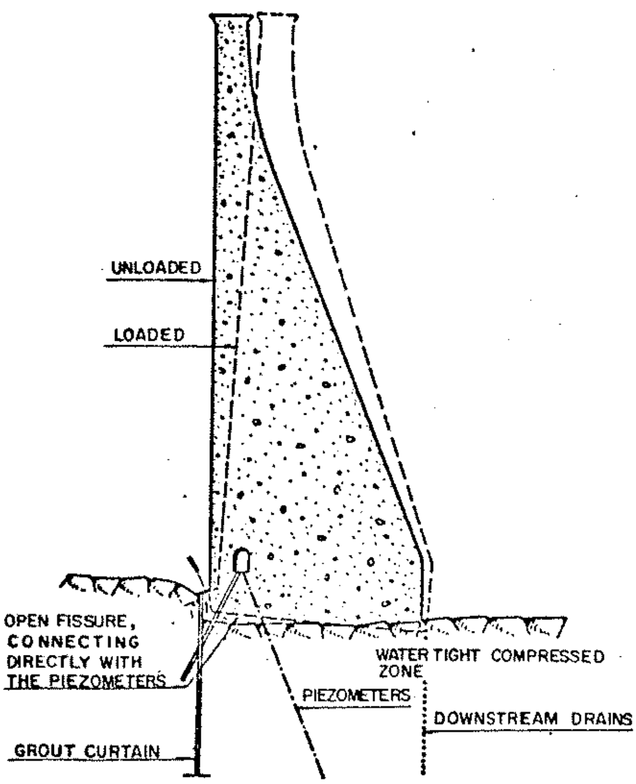


Fig. 8.5
Cracks under an arch dam
(After Sabarly)

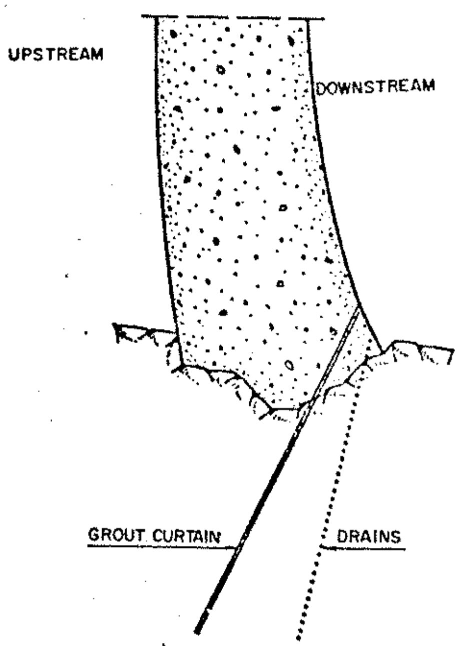


Fig. 8.6
Design proposed by Sabarly

(b) The depth of the grout curtain is controlled by the geological conditions at the dam site. A usually utilized criterion is to grout to a depth where the permeability is 1-2 lugeon units.

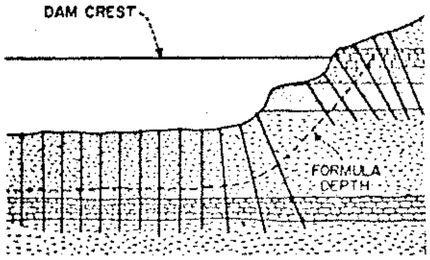
Wahlstron (24) suggests that in the absence of geological controls, the formula $D = (h/3 + 50)$ ft could be used (D = Depth; H = height of the dam).

In cases where the permeability does not decrease with depth, the grout curtain is likely to be useless.

When there is a decrease in permeability characterized by the occurrence of distinct strata, the grout holes should penetrate into the impervious layers at least 5m. See fig. 8,7 below for illustration.

Fig. 8.7

(After Wahlstrom)



Suggested configuration for grout holes when distinct strata occur

(c) The determination of the number of grout lines required is a somewhat nebulous matter, depending or which design rules are regarded. The Corps of Engineers recommends that when the dam is lower than 66m a single line grout curtain is isually satisfactory. Otherwise, multiple line grouting shoud be used.

Casagrande (3) in his Rankine lecture, raises doubts about the efficiency of single line grout curtains; Sherard at al (16) state that no rules can be given for the circunstances under which it is desirable to use more than one grout line; Swiger (18) strongly recommends the use of multiple line grout curtains, based on the argument that probably grout cannot be forced into the smaller cracks and joints until be the larger openings are grouted and that if cutoff is narrow, even small openings may pass considerable amouts of water.

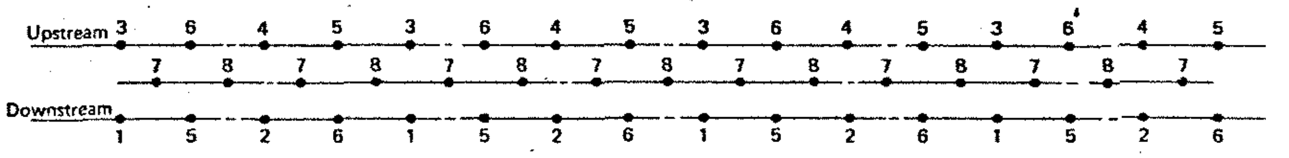


Fig.8.8 Plan – multirow grouted cutoff. Holes drilled and grouted in sequence as indicated by numbering. ●6 Holes may be inclined from vertical as required to intersect joints systems most effectively.

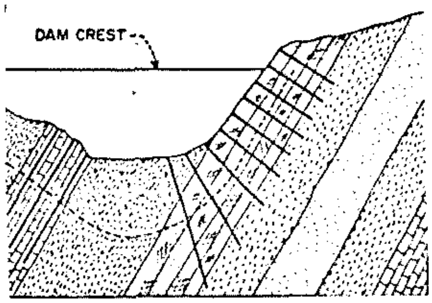
(After Swiger)

De Mello(4) stresses the importance of the width of the grouted zone, and therefore in an oblique way he favors multiple line grouting.

From a scan of the literature the writer concluded that multiple line grouting is particularly desirable when open fissures are being injected. The procedure of first grouting the outer rows and subsequently the inner ones provides a barrier preventing the grout from travelling long distances away from the grout hole, thus reducing the waste of grout. At Portage moutain dam (1) these considerations were used regarding grouting of horizontally bedded weak shales and sandstones. The results are reported to be very satisfactory (see section 10.4). In the case of massive rock, however, no indication was found opposing single line grouting.

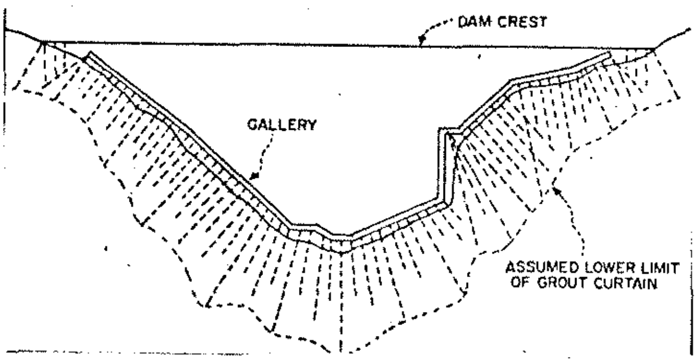
(d) The inclination of the grout holes, according to most grouting experts is not critical (Ref 2,13,19,20). The Corps of Engineers mentions the desirability of the grout holes to intercept perpendicularly the principal joints in the rock. Sherard et al.(16) point out that depending on the nature of the rock, inclined grout curtains could be desirable, since the same length of grout curtains will cross more of the potential leakage cracks than in the case of a vertical curtain. The sketch below illustrates this hypothesis.

Fig.8.9
Inclined Grout Curtain
(After Wahlstrom)



When the permeability decreases markedly with depth grout holes could be drilled in a fan - wise fashion, as sketched below:

Fig.8.10
(After Wahlstrom)



Fan - Wise grout curtain

This, however, is irrelevant for wide valleys.

(e) In the determination of grout hole spacing one has to take into account two conflicting situations:

- the more fissured the rock mass, the further apart can be the holes, because large and numerous fissures allow the grout to flow more easily.

- on the other hand, the further apart the holes, the greater the amount of grout that has to be injected to achieve a continuous curtain.

Therefore, a compromise of the prices of drilling and grout has to be sought. If we further take into account the influence of the grouting pressure and consistency the problem becomes even more complicated. One can say that there is no single theoretical solution for this problem. What is usually done is to conduct field tests for determining the optimum borehole spacing, or to use the experience acquired in other jobs.

When the grout curtain consists of only a single row, the test is performed by grouting 2 adjacent boreholes, and to grout a control hole in between. According to the results of the grouting of the control borehole, it can be deduced whether the spacing has to be increased or decreased.

In the case of a double row grout curtain the test consists of grouting 3 holes in a triangular configuration, and after that, grouting a 4th hole in the center. For 3 or more rows similar tests would have to be conducted.

The spacing between rows is determined in the same fashion.

A good example of the determination of the grout hole spacing is the case of Portage Moutain Dam, where a 5 row grout curtain was constructed.

In several cases, however, the spacing is chosen on the basis of past experience, drilling is then done with a much greater spacing and if found necessary, the spacing is reduced using sucessively the split- spacing technique, ie, grouting a secondary hole in between each two primary holes until the grout take in the higher order holes display a remarkable decrease.

This technique (ie, split - spacing) is presented in Fig. 8.11 below):

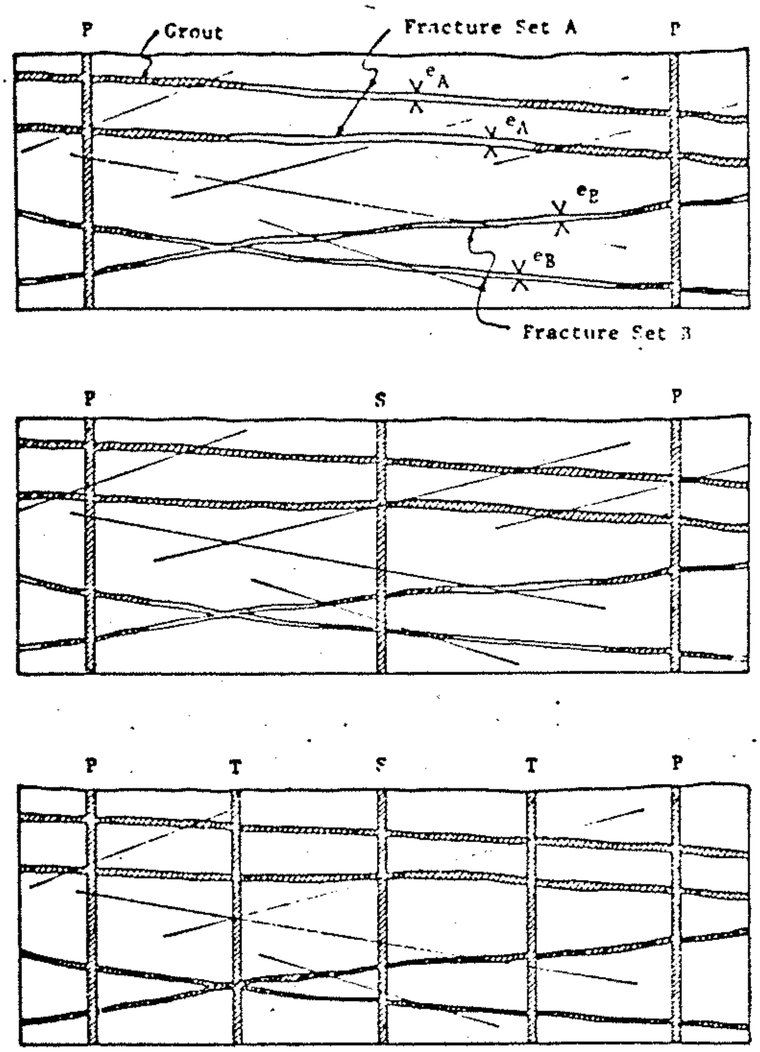


Fig. 8.11
Split-Spacing
Technnique

(After Sinclair)

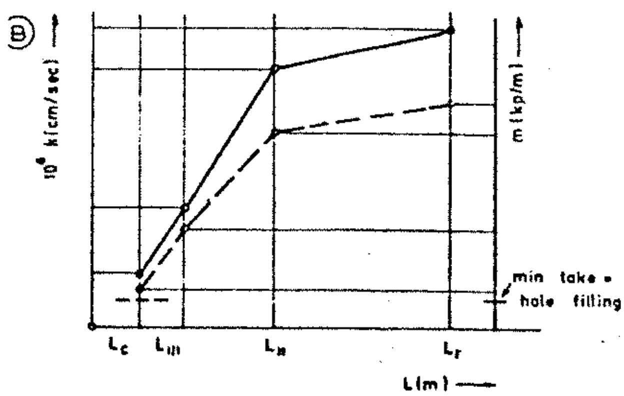
P - Primary Hole
S - Secondary Hole
T - Tertiary Hole

$e_A > e_B$

Nonveiller (10) presents a relationship between "grout take" and permeability, versus spacing for an average rock, as below:

Fig. 8.12

(After Nonveiller)



Relationship between grout take and hole spacing according to Nonveiller

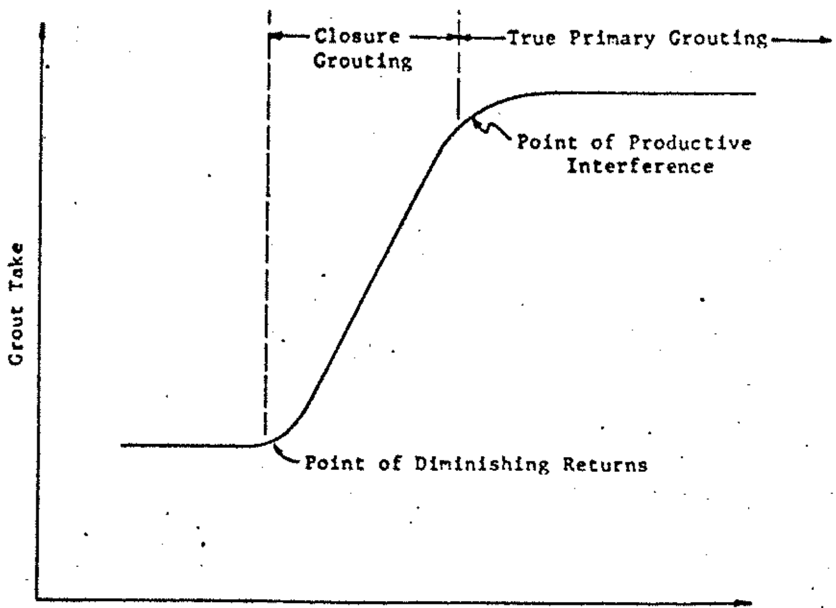
According to the Corps of Engineers (21) the final spacing may be as close as 10 or even 5 ft. However, it is the writer's opinion that this is a consequence of the low pressures used by the Corps. Most authors (1,2,4,20,7,9,14) generally discuss final spacings of the order of 4 - 5 m.

Sinclair (17) presents some useful conclusions on this subject. He proposes the determination of a so-called point of productive interference. This is illustrated in the graph below:

Fig.8.13

(After Sinclair)

Grout take x Hole Spacing



Once the point of productive interference is reached one or two split spacings will be enough for the closure of the grout curtain.

(f) Drill hole diameter

As mentioned in section 6.2 the drill hole diameter is usually irrelevant and so the smallest suitable diameter is usually chosen. However, in some cases of deep holes in a sucession of soft-hard inclined layers of deposits deviation of the drill may occur and in this case extra grouting might be required. Therefore, one has to be aware of this case, since small diameter drill bits are more susceptible to deviations.

8.4 Guides on Grouting Pressures, Grout Mixes and Method to be Followed

Regarding grouting pressures, there is really nothing to be added to what has been said in section 6.3, except that the control of foundation displacement should be done during the conduction of the job.

Regarding choice of grout mixes, the only thing to be added to section 6.4 is that lab and field grouting tests should be conducted for correct planning of the grouting operations.

As to which method is utilized, whenever possible packer grouting should be used. However, the possibility of using packers only can be assessed after field tests.

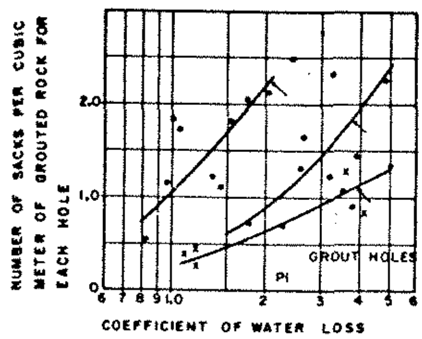
8.5 Estimation of Grout Take

The prediction of grout take as a function of tests previously conducted has been attempted for a long time by some researchers, such as De Mello & da Cruz, Cambefort, Nonveiller, Molina and more recently, Sinclair.

De Mello & da Cruz suggest that plots as shown below can be used for evaluation of grout take as a function of water pressure test results.

Fig. 8.14

Suggested Relationship Between Water Take and Grout Take



NUMBER OF SACKS GROUTED PER HOLE AND PER CUBIC METER OF ROCK VS. COEFFICIENT OF WATER-LOSS

Cambefort (2) states that there is no general relationship between water takes and grout absorption, but there are only general trends, and the possibility of establishing correlations for some sites. Cambefort's opinion is endorsed by Benko's views on the results of grouting at Portage Moutain Dam.

Nonveiller (10) presents a statistical treatment of the relationship between hole spacing and grout takes, as mentioned in section 8.3.

Molina (8) suggested grouting test in isolated test holes as a method of determining cement consumption in grout curtains consisting of two lines. He studied the relationship between the final hole spacing S, a reduction coefficient, r, defined as the ratio of the grout take for the curtain. In short he proposes estimating the potential grout absorption by observing the grout take in tests, and applying the appropriate reduction coefficient to calculate the average grout take.

The most compreensive approach to this subject however is that by Sinclair (17) (1972). Based on a very thorough study and on sharp observations, he proposes for a single line grout curtain the following equations:

$$\log GT/P = 0{,}66 + 0{,}63 \log RWT/P$$

where

GT = grout take

P = injection pressure

RWT = rate of water take

He further proposes the estimation of grout takes using grout tests results. The following equation was found.

$$\log GT/p = 0{,}87 \log RGT/P + 0{,}11/RTR + 0{,}02\ TPT$$

Where

GT = grout take

P = injection pressure

RGT= rate of grout take

RTR= rate of injection rate as defined in section 2.3

TPT= test period time

CHAPTER IX

APPRAISAL OF THE EFFECTIVENESS OF A GROUT CURTAIN

9.1 General

This subject has been approached by several authors such as De Mello & da Cruz (1959), Casagrande (1961), Lancaster-Jones, Nonveiller and Cambefort.

Modern authors including all the above mentioned agree that the success of a grouting operation should not be evaluated in terms of the amount of grout consumption, although most of the recent papers on the subject still refer to very large grout takes in a cheerful manner. This stand, however, has been recognized as nonpertinent, as the only available indications of effectiveness of a grout curtain are those ones based on observation of an actual decrease of permeability of the rock foundation. In section 9.2 the more common methods for such an assessment are briefly described.

Some authors, like Lancaster - Jones (6) for example, pay attention on the seepage reduction due to a hypothetical reduction of the permeability. In some ways, it is a good manner of showing what can be achieved with grouting. The table below illustrates his computations:

TABLE 9.1 - Effect of Reduced Permeability in Curtain

Thickness m	Ratio k_2/k_1	% reduction in flow at contact
5	0,1	38
5	0,02	77
5	0,01	87
5	0,002	97
10	0,1	54
10	0,02	86 1/2
10	0,01	93
10	0,002	98 1/2

De Mello & Da Cruz take the same approach, stressing on the importance of the width of the grout curtain.

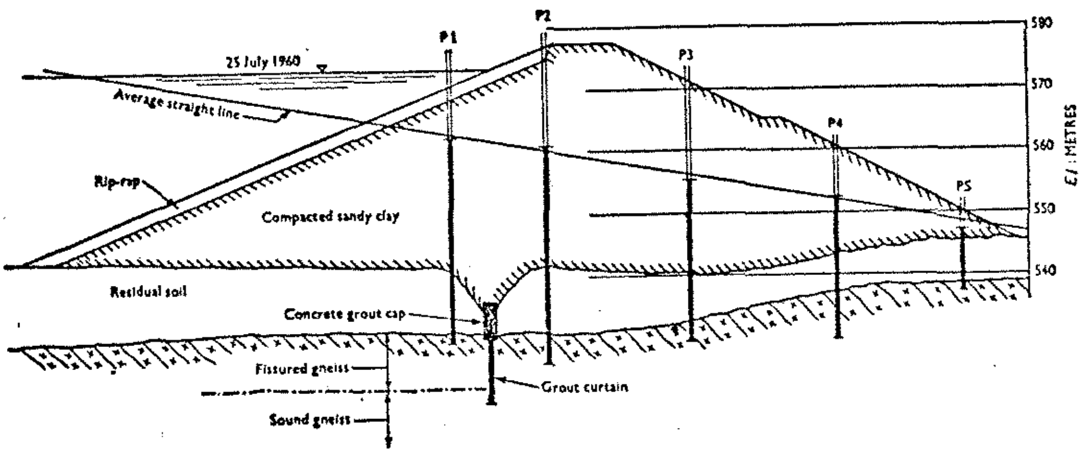
Benko presents some considerations about ways of checking the effectiveness of grouting at Portage Moutain Dam. He describes the situation of recovered cores as a proof of the effectiveness of grouting. It should be pointed out that according to the terminology used in this work, this is a proof of a grouting job well conducted, but it is not a proof of the effectiveness of the grout curtain.

9.2 Methods for Evaluation of the Performance of Grout Curtains

There are only 3 ways of evaluating the performance of a grout curtain.

(a) By an observed reduction of the amount of leakage. This observation can be done based on the measurement of the discharges of relief wells, boils or springs downstream of the dam. This kind of evaluation,however, is only possible when the grout curtain is injected after the first filling of the reservoir. In section 10.2 a good example of this type of procedure is presented.

(b) By observation of the piezometric pressures in the bedrock upstream and downstream of the grout curtain. This is indeed the most widely used method. When there is a sharp drop on the piezometric line in the neighborhood of the grout curtain, this indicates a good efficiency; if the piezometric line is continuous, we can state that the curtain is practically useless. Casagrande presents a good example of a useless grout curtain as illustrated below:



Piezometer observations in pervious rock underlying earth dam

Fig.9.1 (After Casagrande)

(c) This method should not be solely relied on but used in conjunction with (a) or (b) above: the comparison of water pressure test results prior and after the injection. In this case, one would have to assume that the permeability is inequivocally related to the results of water pressure tests, which is not allways true. Nevertheless, as it is the only evaluation that can be done prior to the first filling of the reservoir it is frequentely used.

CHAPTER X

CASE HISTORIES

10.1 General

In this chapter 3 cases histories are presented emphasizing the more pertinent points in the writer's opinion. Two of the cases refer to karst foundations, and the third one refers to the case of Portage Montain Dam, where a very thorough grouting program was conducted. This last dam is founded on interbedded shales and sandstones.

All the 3 cases are described in a summarized form and if further details are required it is suggested that the reader refer to the individual published case histories listed in the references.

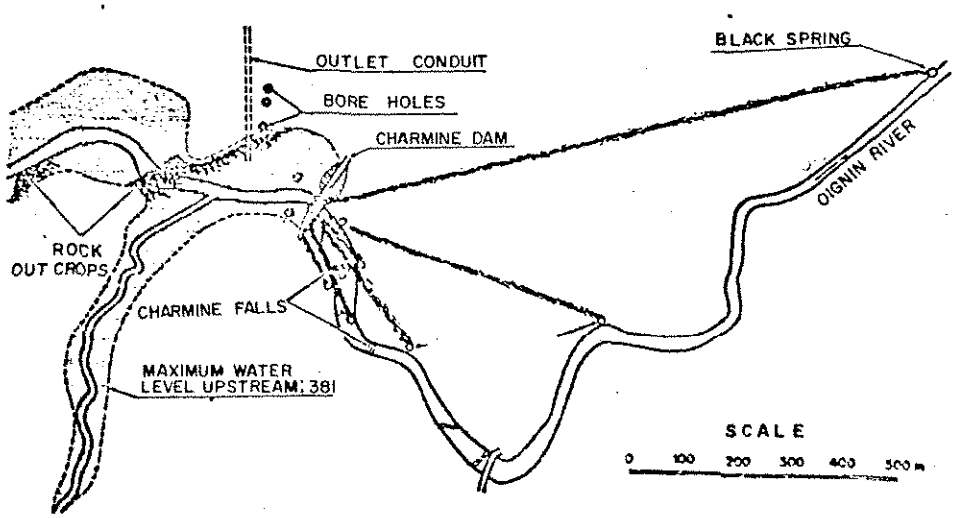
10.2 Charmine Dam, France

This case was related by Riviére & Lescail (12). It deals with a 17m high concrete gravity dam in France on the Oignin River. The geology at the dam site is straightforward. It consists of very fissured, cavernous limestone.

During the construction period, when the diversion tunnel was under construction, the circulation of water in the fissures of the rock mass was detected. There were some springs downstream of the dam site, prior to the construction of the dam, as can be seen in fig. 10.1 below.

Fig.10.1
(After Cambefort)

A Plan View of Charmine - Dam Site



After due consideration, it was agreed that there would be 2 possible treatments:

- Impermeabilization of the total reservoir area
- Attempt to grout the fissures with a deep grout curtain

The first solution proved economically unfeasible and so the second was attempted. During the first attempts to grout the fissures it was found that after some reduction in the leaks, grouting material was appearing in the springs downstream, indicating that it was being eroded and carried away. Meanwhile, the construction of the dam was continuing and after the filling of the reservoir the leaks were of such a magnitude that all injected material was washed away. It was decided therefore to construct an inverted filter, injecting initially coarse material and gradually finer and finer grains until a relatively watertight barrier was formed. This method was very successful as can be seen by the results presented in the table below:

Dates	Water level in the reservoir	Leaks (l/s)	
		Black Spring	Falls
May 15 th, 1950	372,50	220	260
Aug 22nd, 1950	372,50	115	85
Oct 13th, 1950	372,50	55	45
Oct 30th, 1950	375,00	33	10

TABLE 10.1 - Reduction of Seepage under Charmine Dam

The process of constructing an inverted filter can be schematically illustrated as below:

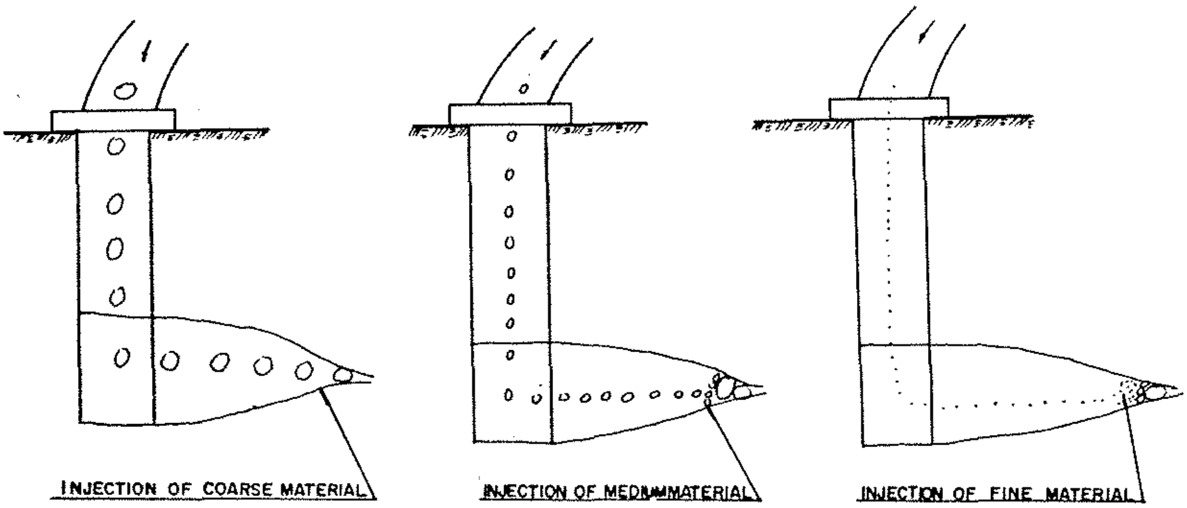
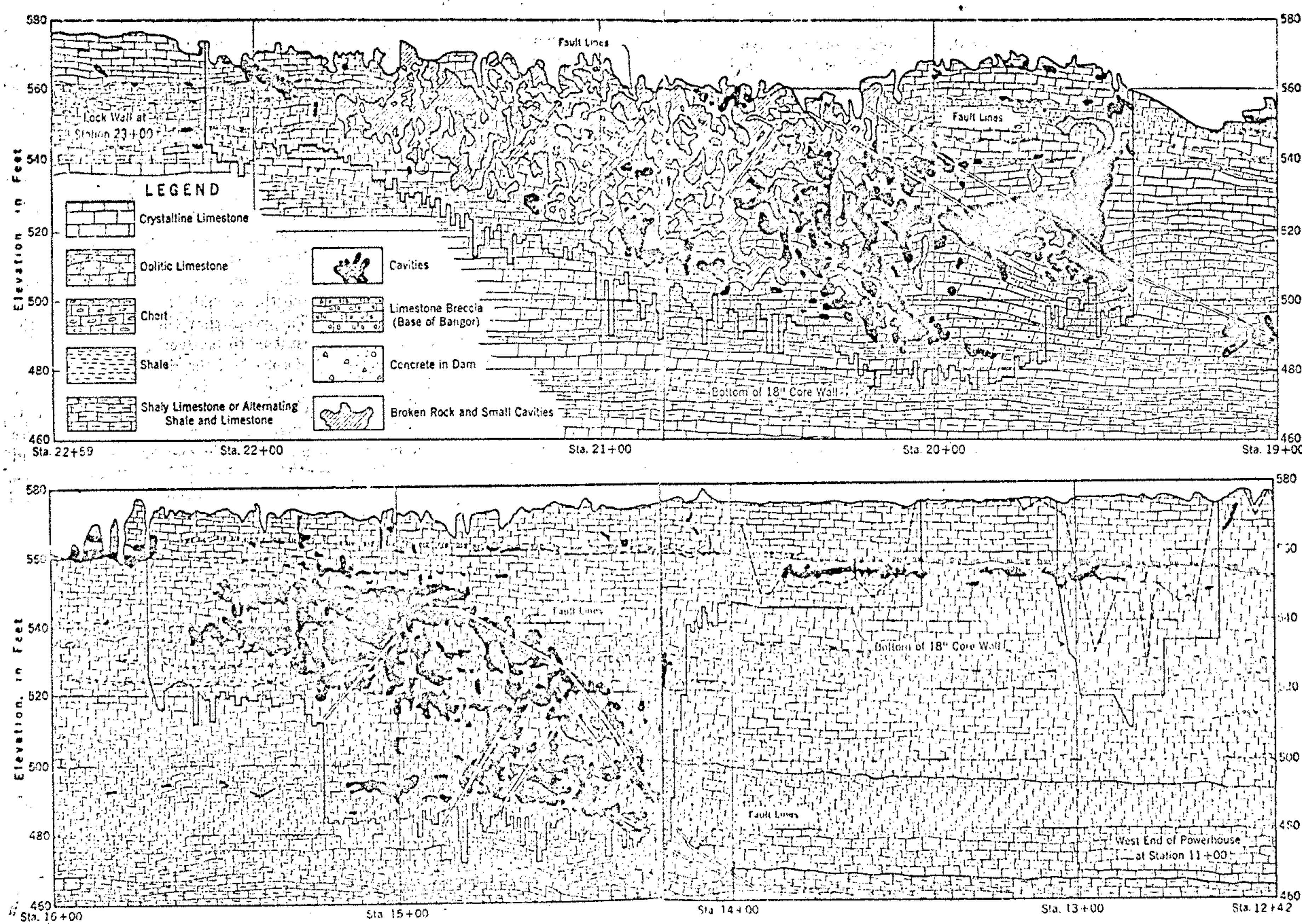


Fig. 10.2 - Construction of an inverted filter for grouting fissures under Charmine Dam

10.3 Hales Bar Dam, USA

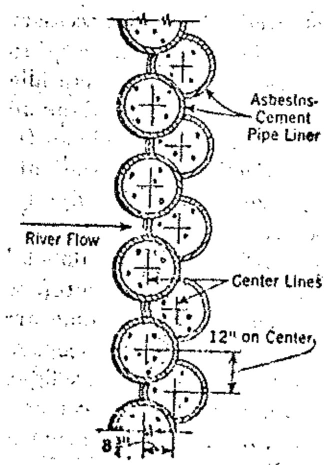
This case was described by Schmidt (14) and it relates to a concrete gravity dam 20m high, founded on cavernous limestone, on the Tennessee River. It was built in the period 1905-1913, and in 1939 it produced a leak of approximately 50 m^3/s. The local geology is illustrated in fig. 10.3. It was apparent the formation of caves in the limestone below the ground water level, then tougth to be unlikely. After several unsuccessful attempts to stop the leakage through usual grouting methods the Tennessee Valley Authority finally solved the problem by constructing what may be called a "cast-in-place" diaphragm wall. Indeed, the grout holes were 18" in diameter



CROSS SECTION THROUGH THE FOUNDAT[illegible] [illegible]NDER HALES BAR DAM, FACING UPSTREAM

Fig.10.3 - Cross Section Through the Foundation under Hales Bar Dam, Facing Upstream

and the centers were 12" spaced, in the fashion sketched below:



Configuration of the grout holes at Hales Bar Dam

Fig.10.4

Quick setting cement had to be used in order to prevent the grout from being carried away. An injection of asphalt upstream of this diaphragm further decreased permeability. The final reduction of leakage was estimated to be around 75%.

This is an excellent example of the difficulties involved in grouting under flowing water condictions.

10.4 Portage Mountain Dam, British Columbia

This case was presented by Benko (1) and refers to a 182m high dam in B.C. Canada, on the Peace River. The geology at the dam site is relatively straightforward. Beds of sedimentary rocks of the cretaceous age strike approximately parallel to the axis of the dam, and dip downstream at angles ranging form 3º to 10º. The figure below illustrates the local geology.

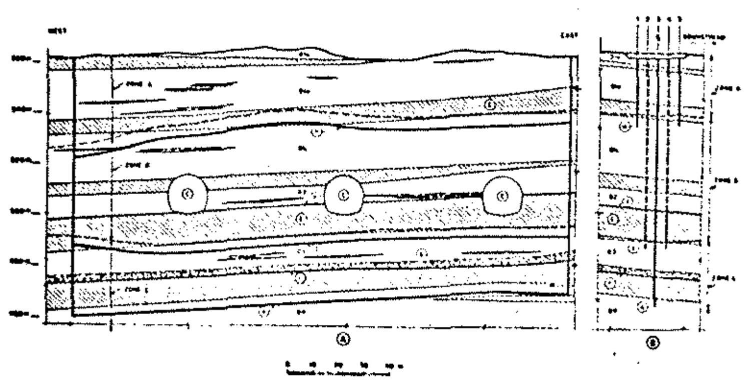


Fig.10.5

Geology at Portage Mountain Dam Site

Sections of Grout Curtain — Section 1.

(A) Section on centreline.
(B) Typical cross-section.
(C) Diversion tunnels.
(D1 to D4) Sandstone of the Dunlevy Formation, beds 1 to 4 (U-upper, L-lower
(E) Shale.
(F) Coal.
(G) Conglomerate.
(H) Bottom of grout holes in Lines 1 and 5.
(I) Bottom of grout holes in Lines 2 and 4.
(K) Bottom of grout holes in Line 3.

Water pressures tests revealed the existance of 3 major zones of permeability: A, B and C (see figure). The permeability clearly decreases with depth, from a maximum of 100 U.L. at the upper layers to a minimum of 1-2- UL at zone C. A final permeability 1 UL was set as the goal to be achieved subsequent to grouting.

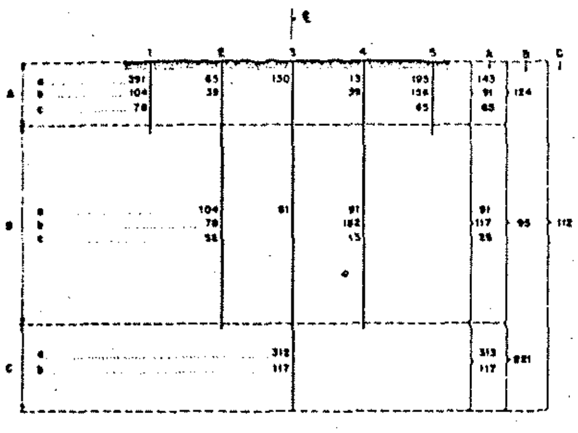
The determination of the grout mix to be used was done through an extensive testing program, which included testing of several types of cements and several admixtures. Grouting pressures were established according to the European Practice, ie, 1,7 kg/cm^2 plus 0,23 kg/cm^2 per m depth to packer.

The control of foundation uplift was conducted using a network of surfaces markers and automatic instruments. In some cases, these observations determined the reduction of the grouting pressures. The tentative design of the grout curtain took into consideration the geology and estimated permeability conditions, the maximum hydrostatic, assumed distances of travel of thin grout mixtures, estimated allowable grouting pressures, and the design of the diversion tunnels.

The almost horizontal stratification of the rock indicated a condition which would tend to promote uncontrolled lateral spreading of grout along seams parallel to bedding planes unless grout flow in the foundation was properly confined. Therefore, it was decided that the design should be generally based on grouting through a multiple line system of holes, the outer lines being grouted first to provide confining barriers.

An arrangement of grout holes was prepared in such a fashion that zone A would be grouted through 5 lines of shallow holes penetrating about 4-5 m into zone B, Zone B through 3 lines of intermediate depth holes penetrating about 9m into Zone C, and Zone C through one line of deep holes terminating in relatively impervious strata. The spacing of the holes and lines was tentatively set at 4,5m and the use of split-spacing technique and packer grouting was prescribed.

The sketch below illustrates a typical section of the grout curtain:



Multi row grout curtain, at Portage Mountain Dam

Fig.10.6

Average grout takes — kg per m of hole.

(1-5) Grout hole lines 1 to 5.
(A) Averages for holes in zones.
(B) Averages for zones.
(C) Overall average.
(*a*) Average for primary holes.
(*b*) Average for secondary holes.
(*c*) Average for tertiary holes.

It was found during the installation that the hole spacing could be increased. Indeed, final grouting, was completed with holes spaced at 4,5m on line 1,2,4 and 5 in zone A, 9,0m on line 3 in zone A, and 9,0m on all lines in zones B & C.

The reduction of permeability was checked by conducting water pressures tests prior and after grouting. The results were very good, as can be seen in the typical records of a test presented below.

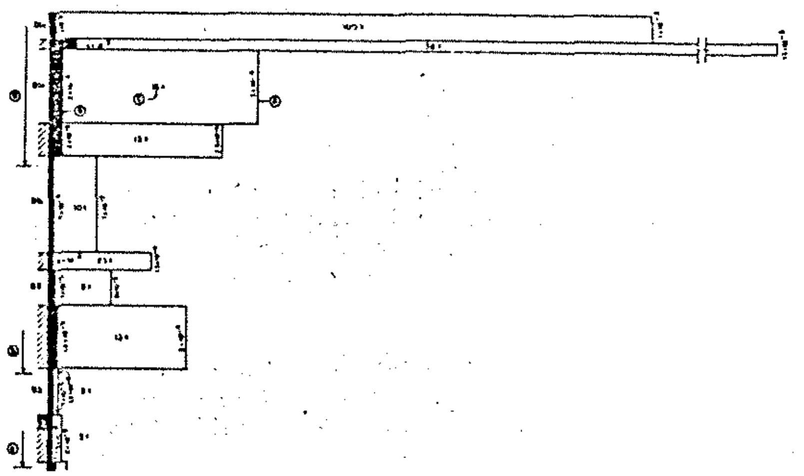


Fig.10.7

Secondary permeability of the foundation before and after grouting, cm/sec.

(A) Before grouting.
(B) After grouting.
(C) Reduction factor.
(D1-D3) Geology shown as on figure 1.
(*a*) Bottom of grout holes in Lines 1 and 5.
(*b*) Bottom of grout holes in Lines 2 and 4.
(*c*) Bottom of grout holes in Line 3.

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