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MINISTÈRE DU DÉVELOPPEMENT INDUSTRIEL ET SCIENTIFIQUE

BUREAU DE RECHERCHES GÉOLOGIQUES ET MINIÈRES

SERVICE GÉOLOGIQUE NATIONAL

ROCK HYDRAULICS

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Géotechnique

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ROCK HYDRAULICS

by

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S U M M A R Y

Pages

SUMMARY 1

1 - STATEMENT OF THE PROBLEM 2

2 - LAWS GOVERNING WATER FLOW IN ROCKS 5

2.1 - Fissureless masses 5

2.2 - The Single Fracture 5

2.3 - Hydraulic Conductivity of a Set of Fractures 7

3 - DETERMINATION OF *in situ* HYDRAULICS PARAMETERS 10

3.1 - Systematic structural analysis 10

3.2 - *In situ* hydraulic testing 13

3.2.1 - Introduction 13

3.2.2 - Water tests-theory 14

3.2.3 - New techniques of hydraulic testing 17

3.2.4 - The triple hydraulic probe 21

3.2.5 - The continuous borehole piezopermeameter 24

3.3 - Influence of state of stress 31

3.4 - General discussion on water testing 33

3.4.1 - Criticisms of the water test analysis 33

3.4.2 - Practical procedure 37

3.5 - Conclusion 42

4 - CONTINUOUS OR DISCONTINUOUS MEDIUM 42

5 - THREE-DIMENSIONAL DISTRIBUTION OF THE HYDRAULIC POTENTIAL 43

5.1 - Introduction 43

5.2 - Rock mass with one system of conductive fractures 45

5.3 - Rock mass with two systems of conductive sub-vertical fractures 47

5.4 - Rock mass with three systems of conductive fractures 50

Pages

5.5 - Rock masses with more than three fracture systems....... 56

6 - DRAINAGE IN FISSURED ROCK.................................... 57

6.1 - Introduction.. 57

6.2 - Project of a drainage network............................. 58

6.2.1 - Preliminary survey...................................... 58

6.2.2 - Drainage criterion...................................... 58

6.2.3 - Theoretical calculations................................ 59

6.3 - Drainage of a slope....................................... 63

6.3.1 - Data of the problem..................................... 63

6.3.2 - Theoretical results..................................... 65

6.4 - Conclusion.. 69

7 - PRACTICAL EXAMPLES... 70

7.1 - Water flow in dam abutments............................... 70

7.2 - Flow conditions at the Grand Maison damsite............... 71

7.2.1 - Introduction.. 71

7.2.2 - The damsite and the construction....................... 72

7.2.3 - Structural study.. 72

7.2.4 - Natural flow conditions................................. 76

7.2.5 - Hydraulic characteristics of the site................... 77

7.2.6 - Flow simulation... 84

7.2.7 - Verification of the hypotheses by in situ tests.. 87

7.2.8 - Conclusion.. 87

7.3 - Flow in a fissured rock slope............................. 88

7.4 - Hydraulic problems connected with dam projects............ 90

7.4.1 - Pitan Ridge (Tachien project)........................... 90

7.4.2 - Kraghammer Sattel (Bigge dam)........................... 94

8 - CONCLUSION... 98

ACKNOWLEDGEMENTS... 100

S U M M A R Y

This report gives some new considerations on rock hydraulics with application in the triple field of civil, mining and petroleum engineering. The rock medium is assumed to be a jointed medium with a low conductivity of the rock matrix.

After a brief analysis of the hydraulic characteristics of rock masses and of the laws governing flow in fissures, both continuous and discontinuous, this report examines the different mathematical or physical models (electrical or hydraulic) which make it possible to solve problems of three-dimensional water flow through jointed media with one or more sets of parallel fractures. Several methods are suggested, according to the nature of the rock jointing. The mathematical or physical models are elaborated with the help of the concept of directional hydraulic conductivity directly measured by a new *in situ* technique.

The theory of water-flow through jointed rock has undergone rapid advances in recent years. Computing techniques can now be applied to very large, two and three dimensional problems. However, the application of these methods is completely inadequate if no *in situ* hydraulic parameters are available. Several suggestions are given for the control of groundwater flow and the hydraulic instrumentation.

Several practical examples concerning dam foundation, slope or underground openings, illustrate the methods of solution. In particular, the problem of drainage in rock is examined. A fundamental difference is made between the drainage of the joint network and the drainage of the rock matrix. Practical cases show that the optimum direction of the drain depends essentially on the geometry and orientation of the joints. For a required draining effect, the cost of a drainage system will be notably reduced if the geometry (direction, depth, etc...) is judiciously chosen.

I - STATEMENT OF THE PROBLEM

The engineer is faced in many fields of science with problems connected with the flow of fluids through fissured media. In civil engineering, for example, water often flows through rock masses (foundations, surface or underground structures, natural sites). In hydrogeology, as well as in the field of mining and petroleum engineering, phenomena involving the circulation of fluids through rocks play a very important part.

Research workers have, for a long time, been concentrating on the study of fluid flow through porous media ; even the case of heterogeneous or anisotropic media has been studied. On the other hand, jointed media, particularly fissured rocks, constitute a field that, as yet, is not well known. Basic studies have only been made in recent years, in several countries simultaneously.

The foremost aim of "ROCK HYDRAULICS" is to analyse the hydraulic properties and characteristics of fissured masses of rock, to study flow phenomena and their effects, and lastly, to explain the behaviour of rock masses under the action of groundwater.

It is at present generally admitted that fracturing plays a decisive role in rock hydraulics. In that field, the medium is taken to be anisotropically discontinuous ; the fractures that break up the rock mass give water privileged paths. The word "fracture" is here taken in a wide sense ; it includes all the apertures of the rock mass, whatever their geological origin : stratification and schistosity boundaries, joints, faults, etc... A very simple calculation makes one realize that even a few thin fractures will give the rock mass very high permeability coefficients, in comparison with coefficients for the rock matrix. Therefore, masses of rock where their permeabilities - i.e. flow phenomena - are concerned, exhibit hydraulic anisotropies due to the fact that fractures, through geological and mechanical circumstances at their formation, do not have an arbitrary orientation, but are grouped into one or several sets of plane and parallel fractures.

The very structure of rock masses points to the fracture as the basic element in rock hydraulics. A systematic study was bound to start out with an analysis of phenomena occurring at the scale of the fracture, artificially separated from the mass. The rock mass can, indeed, be taken to be a combination of elementary fractures from the point of view of hydraulics. The problem taken as a whole proves to be most complex, if not insoluble ; it becomes more tractable as soon as one proceeds by steps, that is, by starting with the study of a simple fracture. The first step has therefore been to study the laws governing flow in a simple fissure in laminar and turbulent flow, taking into account all the parameters likely to play a part, among others, roughness, a most important factor, the geometrical shape of the fracture, and the presence of filling material.

These results by proceeding progressively, in other words by extending the laws governing flow to the system of fissures as a whole, led to the determination of the distribution of the hydraulic potential $\phi = Z + P/_{\gamma w}$ of underground water. The hydraulic potential constitutes the essence of rock hydraulics ; indeed, a knowledge of its distribution makes it possible to proceed, even for three-dimensional problems, to calculations on the flow rates and to the mechanical effects of this flow.

It has often been noted that ground-water, both flowing and at rest, has a detrimental effect on the behaviour of rock masses. Underground water affects, in the first place, the stability of the mass ; sometimes, experience has even shown that water can unfortunately have unsuspected consequences and can lead to catastrophes. Thus, when a rock mass is likely to be influenced by groundwater, any stability analysis calls for a preliminary study of the flow network within the mass.

We shall not forget that determination of the hydraulic potential in jointed media remains our main concern. In all previous studies, an effort was made to tackle the problem in three dimensions ; it is only the matter of determining the distribution of the hydraulic potential that proves difficult. The two-dimensional problem can indeed be considered as solved. Apart from the graphical and numerical methods previously suggested (LOUIS, 1967), several types of experimental studies were evolved (LOUIS, WITTKE 1969-70).

It has of course proved necessary, in view of the simplifying hypotheses adopted during computation, to verify theoretical results by measurements on laboratory-scale models (WITTKE and LOUIS, 1968, WITTKE, 1968) as well as on piezometric measurements performed on site (LOUIS, 1972). A comparison between theoretical studies and measurements on the models or *in situ* generally led to favourable conclusions.

In actual practice, a large number of problems in flow through rock masses are three-dimensional. Results obtained by the solution of two-dimensional problems are often devoid of significance. Therefore, the question of the distribution of hydraulic potential must be approached three-dimensionally. A first attempt was given in study (LOUIS, 1967) ; the method unfortunately has only a restricted field of application since it only applies to a very special type of rock fracturing.

This report recalls a few basic results and deals more particularly with the determination, <u>in three dimensions</u>, of the distribution of hydraulic potential in jointed media. In order to get acceptable results, it is of course necessary to introduce <u>representative</u> information into mathematical and physical models ; special stress has therefore been laid on determining hydraulic parameters *in situ*. An account of different methods is given with a few practical examples.

A number of aspects are only briefly touched upon, while others, even more numerous, are not mentioned at all. The entire subject is taken up again in detail, on a perfectly general plane, in one overall study on rock hydraulics (LOUIS, 1974), to be published shortly. Finally, it should also be noted that SHARP's thesis (1970) contributes a number of new and most interesting elements concerning methodology and numerical techniques in the hydraulics of jointed media ; so does the work done by MAINI (1971) dealing more especially with *in situ* measurements.

2 - LAWS GOVERNING WATER FLOW IN ROCKS

2.1 - Fissureless masses

Let us, from the start, eliminate the case of a rock mass having no open fissures. In such a medium, designated as a "rock matrix", laws of flow in porous media, such as Darcy's Law for instance apply. The rock matrix generally exhibits a very low permeability, around 10^{-7} to 10^{-14} cm/s, depending on the nature of the rock. This case is of little interest, however, for on the scale here under consideration (tens or even hundreds of metres), rock masses are always jointed or fissured.

2.2 - The Single Fracture

Let us, first consider the open and unfilled fracture, possibly with bridges of rock. In rock, fractures constitute channels characterized by a high value of relative roughness, k/D_h, (where k is the absolute roughness, and is represented by the height of its asperities, and D_h, the hydraulic diameter, by twice the opening of the fracture). The relative variations in the opening of the fracture are therefore most important, since they cause, during flow, a very high pressure drop coefficient (much higher than that which may be computed from Poiseuille's Law, for instance).

Laws of flow in a single fracture can be expressed as :

Laminar or Steady Flow : $v = k_f \, J_f$ (1)

Turbulent Flow : $v = k'_f \, J_f^{\alpha}$ (2)

In these expressions, v stands for the mean velocity, k_f for the <u>hydraulic conductivity</u> of the fracture, k'_f its <u>turbulent conductivity</u>, J_f for the perpendicular projection of the hydraulic gradient ($\vec{J} = -\vec{\text{grad}}\,\phi$) on the plane of the fracture and, finally, α for the degree of non-linearity ($\alpha = 0.5$ for completely rough turbulent flow).

For fracture flow, the transition from laminar to turbulent takes place at very low values of the Reynolds number* (down to 100 or even 10), decreasing as the relative roughness of the fracture increases.

Within the fracture itself, the transition from laminar flow (α=1) to completely rough turbulent flow (α=0.5) is quite progressive ; the exponent α slowly changes from 1 to 0.5 when the Reynolds number changes, for instance from 100 to 2000.

The hydraulic conductivities defined in relations (1), and (2) are given by the following expressions (fig. 1) :

Laminar Flow :

$$k_f = \frac{\kappa g e^2}{12\ \nu\ C} \tag{3}$$

Turbulent Flow :
(completely rough)

$$k'_f = 4\ \kappa\ \sqrt{g\ e}\ \log \frac{d}{k/D_h} \tag{4}$$

In these expressions, g is the acceleration of gravity, κ is the degree of continuity of the fracture (ratio of the open surface and the total surface of the fracture), e is the mean width of the fracture, ν is the kinematic viscosity of the fluid, and, lastly, C and d are two coefficients which depend on the relative roughness k/D_h of the fissure (according to LOUIS, 1967, $C = 1 + 8.8\ (k/Dh)^{1.5}$, and d = 1.9 for a relative roughness greater than 0.033 ; this, in general, is the case for fractures in rock).

In the case of fractures with filling, the hydraulic conductivity is equal to the permeability of the filling, on condition, of course, that this permeability be definitely higher than that of the rock matrix.

---

* The Reynolds number, defined by the relation $Re = vD_h/\nu$, in fact has a value that is extremely difficult to determine in the case of fissured rocks, since for a given type of flow it can vary enormously from one point to another along the same fracture.

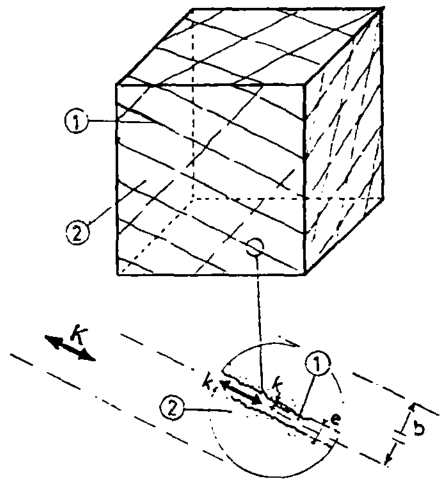

Fig. 1 : Hydraulic Parameters of a rock mass
(1) Fracture of the System Kj,
(2) Rock Matrix,
k_f, Hydraulic conductivity of a fracture,
K, Hydraulic conductivity of the fracture set.

2.3 - Hydraulic Conductivity of a Set of Fractures

As pointed out in Paragraph 1, it is assumed that fracturing in a rock mass is made up of several sets of parallel plane fractures. To characterize the hydraulic properties of such a medium, it will be enough to know hydraulic conductivity K (laminar or turbulent) of each set of fractures. This hydraulic conductivity may be defined, as above, through the relation between the flow velocity V (flow rate in the direction of the fractures, divided by the total cross-section of the mass) and the active hydraulic gradient, as given by the relations :

$$\text{Laminar State : } V = KJ \qquad (5)$$

$$\text{Turbulent Flow : } V = K'J^{\alpha} \qquad (6)$$

The scale of the phenomenon studied is of great importance. In a given volume, individual fractures may, within their own plane, be continuous or discontinous ; these two cases must be studied separately :

a) Set of continuous fractures

Directional hydraulic conductivity of a set of continuous fractures follows directly from the hydraulic conductivity of the individual fractures. It is given (in laminar flow or turbulence) by the expression

$$K = \frac{e}{b} k_f + k_m \tag{7}$$

This relationship can be obtained by dividing the flow rate by the total cross-section of the rock mass. In figure 1, e stands for the mean width of the fractures, b the mean distance between them, k_f their hydraulic conductivity and k_m the permeability of the rock matrix.

In practice, k_m is very often negligible compared with the term $\frac{e}{b} k_f$. On the other hand, if there are no cracks or if they are bounded ($e = 0$ and $k_f = 0$), only the k_m term remains in relationship (7), this case corresponding to that considered in 2.1.

b) Set of discontinuous fractures

A numerical study shows very clearly that a set of continuous fractures, even when they are extremely narrow, has a very large hydraulic conductivity (a single fissure per metre with a width of 0.1 mm corresponds to a conductivity of about 10^{-4} cm/sec ; with a 1 mm width and with the same frequency, the corresponding value is 0.1 cm/sec). These theoretical values are therefore noticeably greater than the ones met with in practice, although most of the time, fissures with a width greater than 1 mm do exist. The low values of the hydraulic conductivities observed in nature can be explained simply by the fact that the fractures, even when of notable width, are of limited extent. Within their own plane, the fractures are therefore discontinuous (figure 2). Within such a medium, flow is evidently anisotropic. The fractures which do not communicate "short-circuit" any flow along their direction. The fractures are at a constant potential and the circulation of water occurs through the rock matrix.

This problem, considered three-dimensionally, has been programmed on a computer to obtain the hydraulic conductivities of such media, whatever their geometric configuration. As a first approximation, it may be assumed that the hydraulic conductivity in the direction of the fissures is given by

$$K = k_m \{1 + \frac{1}{2} (\frac{1}{L-1} - \frac{1}{L})\}* \qquad (8)$$

Whereas the transversal conductivity is equal to k_m, the permeability of the rock matrix. The degree of anisotropy is therefore :

$$1 + \frac{1}{2} (\frac{1}{L-1} - \frac{1}{L}).$$

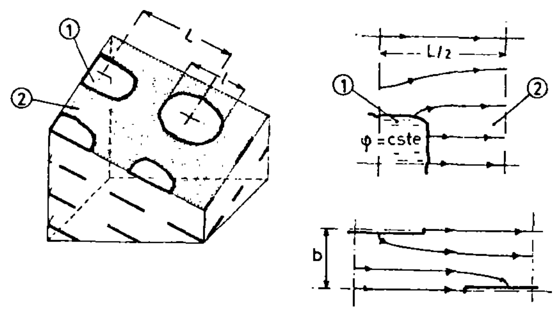

Fig. 2 : Network of Discontinuous Fissures.
(1) Open fissure,
(2) Rock matrix.

It must be noted that in rock masses with discontinuous fissures, the degree of discontinuity $\frac{1}{L}$ ($\frac{1}{L} = \sqrt{2\kappa}$, where κ is the degree of continuity of the surface of the fissure) and the frequency of the fissures b/L are the only important hydraulic parameters. The permeability of the matrix, k_m, occurs merely as a proportionality coefficient in the hydraulic conductivities of the different set of fractures ; its influence only becomes noticeable in the computation of flow rates. The opening and the roughness of the fissures, as well as the geometry of the rock face have no bearing on the problem. In these media, the flows occur partly through the rock matrix and therefore generally remain laminar.

Work has recently been performed, independently by the author and also at the Institut Français du Pétrole, by DUPUY and LEFEBVRE du PREY, (1968) on analogous questions.

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* In fact, the hydraulic conductivity K also depends upon frequency of the fissures $\frac{b}{L}$, and the function $K/k_m = f\ (1/L,\ b/L)$ can only be determined numerically. Relation (8) is acceptable for $3<L/b<6$.

Remarks in this paragraph do not apply to bedding fissures which in general are continuous ; furthermore in this case the hydraulic conductivity is reduced by the presence of filling (deposited by sedimentation, weathering or alteration).

3 - DETERMINATION OF *in situ* HYDRAULICS PARAMETERS

The difficulties generally encountered in the study of practical groundwater flow problems in fissured media have to be underlined. The rock mass has always a very complex geometry in which the discontinuities are spread in an anisotropic and heterogeneous manner.

The use of mathematical or physical models in the study of flow phenomena within fissured rock is only justifiable if one has enough information on the parameters *in situ*. New methods are proposed to measure hydraulic parameters which are characteristic of the rock mass. First a detailed statistical analysis of the fracture system is carried out to establish its geometrical characteristics. The results of this first step are then used for *in situ* measurement of directional permeability in boreholes.

3.1 - Systematic structural analysis

Survey methods of structural geology now make it possible to count and pinpoint the different sets of fissures in a rock mass and also to determine, by statistical means, their orientation, their frequency and their shape. These parameters define the geometry of the medium. This study must be undertaken from a very specific viewpoint, so that the potential water circulation in the mass may be inferred as objectively as possible. It is divided into several distinct phases :

a) A systematic survey from trenches, reconnaissance tunnels, or oriented borehole cores, where each observed structural element is described as thoroughly as possible. The use of the enclosed check list (fig. 3) has proved to be effective for quick compilation of data and ease of computer storage.

Each structural element analysed is described by a line in the check list, corresponding to a punched card for the computer. A fracture is characterized by 17 parameters : (1) = numbering, (2) = location of the survey, (3) = thickness of cover, (4) = geographical position of the surveyed point, (5) = position of the observation plane or axis ; (6) = rock type ; (7) = structural element, (8) = orientation (strike and dip) of the structural element, (9) = continuity, (10) = thickness, (11) = nature of the filling, (12) = extent of free opening, (13) = water flow, (14) = relaxation effects, (15) = spacing between fractures, (16) = continuity of jointing, (17) = friction angle. One column is left for miscellaneous observations.

A large number of structural elements must be surveyed in order to permit a valid statistical treatment - several hundred is a minimum.

b) A statistical treatment : The data is fed into a computer for conventional statistical treatment with representation of the fracture concentrations on a Schmidt diagram. In this treatment, the fractures are given the same weight ; only the number of fractures is considered. However, from the point of view of hydraulics it is certain that all fractures do not act similarly. It therefore was necessary to modify the conventional statistical analysis (based on the bulk density population), in order to obtain a better hydraulic definition of a fissured rock mass by giving some "weight" or importance to fractures appearing as the most likely to channel water. The criteria of importance from a hydraulics point of view consist of the length, continuity and opening of the fractures. Quantitatively a weight P (N) is given to each structural element (N) ; this weight is equal to unity and remains a constant in the conventional statistical treatment. In the new proposed method, it varies and is an intrinsic parameter of each fracture.

Two types of weighting are proposed :

- The first emphasizes the thick and long fractures and is expressed as :

1		2	3	4 Position			5 Pl. Obs.		6 Rock		7	8 Orient.		9	10	11	12	13	14	15	16	17	
p	n		dz	X	Y	L	Str.	Dip	T1	T2	Str.	Str.	Dip	Cont.	Th.	Fill	Op	H_2O	Rel.	Sp.	Join	φ	Observations

6 Type of rock

10		20		30		40	
11		21		31		41	
12		22		32		42	
13		23		33		43	
14		24		34		44	
15		25		35		45	

7 Structure

1	
2	
3	
4	
5	
6	
7	
8	

9 Continuity Long. Rel. :

10 Thickness

1	$< e_1$
	$e_1 =$
2	
	$e_2 =$
3	
	$e_3 =$
4	$> e_3$

13 Water 0 No 1 Yes

14 Relaxation 0 No 1 Yes

15 Spacing (décimètres)

17 Friction

11 Filling

10		20		30		40	
11		21		31		41	
12		22		32		42	
13		23		33		43	

12 Opening 0 No 1 Discontinuous 2 Continuous

16 Joint continuity

0	No
1	In the rock
2	Struc > 20°
3	Struc < 20°
4	Echelon

Fig. 3 - Check List for systematic structural survey

P(N) = 1 + Thickness (N) x Continuity (N) ; the thickness of the parameter is defined on the check list by fixing boundaries between classes. For example, this parameter may vary between 1 and 5 when the thickness of the fractures varies between 0, 1, 5, 10 and 30 millimeters. The term "continuity (N)" is determined directly *in situ* : It is the ratio between the length of the fracture which intersects the tunnel and the total length of the intersection if this fracture were perfectly continuous. (This term then varies between 0 and 1). At the surface it is the ratio between the length of the fracture and a reference length (this reference length can have any value).

- The second selects the hydraulically efficient fractures and is expressed as :

$$P(N) = 1 + \alpha \text{ (water)} + \beta \text{ (free opening)}.$$

The coefficients (water) and (free opening) have been fixed *a priori* through the class boundaries as shown above. Coefficients α and β are used to give to the two scales of (water) and (free opening) an equal importance.

For practical problems conditions P(N) = 0 if free opening is smaller than a given opening, is added to clarify the diagrams by eliminating notably secondary fracturing.

Generally the proposed weighting methods clarify the diagrams enormously by erasing the secondary features of little importance from a hydraulics viewpoint.

3.2 - *In situ* hydraulic testing

3.2.1 - Introduction

It is, then, impossible to try to determine the spatial distribution of all these discontinuities and the hydraulic characteristics of each one. The hydraulic parameters (for example : cross-section of fissures, roughness, filling, degree of separation or of discontinuity of the fractures, etc.), are apparently more difficult to determine *in situ*, if only because of their number (see Chapter 2). Luckily, a technique of *in situ* measurements has been perfected, which makes it possible to determine directly the total effect of all these different parameters through the hydraulic conductivities K of the different sets of fractures. It is therefore not necessary to know the detailed geometry of the fissures. The directional hydraulic conductivity K of each set of fractures is measured separately, as shown in figure 4, with the help of a single or triple hydraulic probe. In the case of a rock mass with three sets of fractures, the direction of drilling in order to test one of the sets will be chosen to be parallel to the direction of the other two joint sets. In the general case, pumping tests must be performed in three different directions. The length of the trial zones of a boring should, in theory, correspond to the length of the corresponding lattices in the mathematical or physical model used to study the medium.

As described before, it is possible to determine the directions and average spacing for the different families of fractures, but it is pointed out that no definite conclusions (for instance, for calculation of hydraulic conductivities) can be made regarding the opening of the fractures. These are always more or less influenced, at the site, by relaxation effects, blasting, etc.. It is thus imperative to use hydraulic techniques to measure *in situ* the specific conductivity of each set of fractures.

3.2.2 - Water tests-theory

Let us assume a medium in which 3 families of fractures F1, F2, F3 with total hydraulic conductivities K1, K2, K3 intersect (fig. 4).

It is possible, by observation, to determine an optimum direction for the borehole, so that this borehole intersects only one family of fractures. To test the system F1, a borehole must be drilled parallel to the intersection of F2, F3. If the medium possessed only one family of fractures, the result would be exactly representative of the conductivity of this family. But when several families intersect, there is during the test a reciprocal influence of one family on the others. That is why it is preferable to carry out tests in several boreholes perpendicular to each family.

A laminar flow in a fracture is a flow under a speed gradient, where

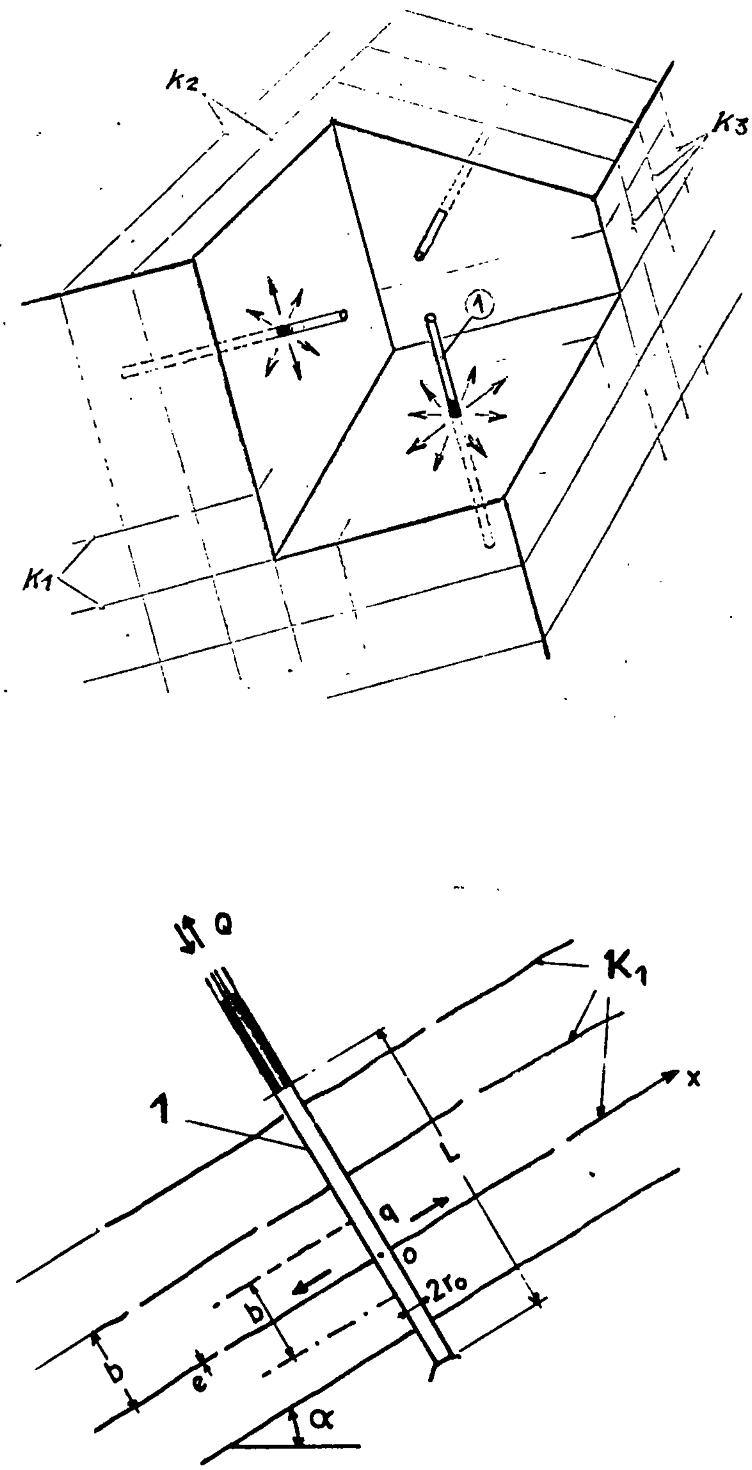


Fig. 4 - Water test in rock mass with three systems of fractures (1) Borehole parallel to K2, K3 for testing the fracture system K1.

$$\vec{v}_f = -\vec{\text{grad}}\,\{k_f.\ (Z + \frac{P}{\gamma w})\} \tag{9}$$

with v_f = the flow velocity in the fracture
κ_f = hydraulic conductivity of the fracture
Z = elevation of the point considered in the fracture
P = water pressure

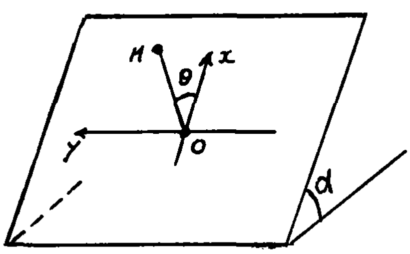
$\phi = Z + P/_{\gamma w}$ is the hydraulic potential at each point.

Because the total hydraulic conductivity of the fracture set is proportional to the conductivity of a single fracture the above equation is also valid for the set of fractures :

$$\vec{V} = -\vec{\text{grad}}\,\{K\ (Z + \frac{P}{\gamma w})\} \tag{10}$$

with V the mean flow velocity
K the total hydraulic conductivity of the fracture set under study.

A pumping in or pumping out test from a borehole creates a flow which can be expressed by the superposition of radial flow and of a uniform flow under gravity (in the plane xoy).



$$\begin{cases} \phi - \phi_o = \dfrac{q/b}{2\pi K_1}\ \text{Log}\ \dfrac{r}{r_o} + (\sin\alpha)\ x \\ \Psi = \dfrac{q/b}{2\pi K}\ \theta - (\sin\alpha) y \end{cases} \tag{11}$$

en M (r,θ)

q is the flow rate in a single fracture
q/b is in fact the linear flow rate (q < o in pumping in
q > o in pumping out)
r_o is the radius of the borehole.

During tests on the section of length L when a flow rate Q is obtained q/b will be replaced by Q/L (Q = total flow rate in the test section).

In the case of a natural flow, the gravity terms in equations (11) must be replaced by other terms taking into account the gradient Jo of the natural flow.

3.2.3 - *New techniques of hydraulic testing*

In situ water tests are made on a large scale. They are therefore more representative than tests carried out on samples in the laboratory, but only in so far as their interpretation is possible. However, often the test cavity is not well defined and the nature of the flows is not well known ; for instance there can be spherical, radial-planar or mixed flows. It is thus impossible with such tests to pretend that a correct interpretation of the results is made, because the relative importance of each flow type is unknown. In an anisotropic medium the problem is even more difficult. Therefore a new testing method had to be devised, in which the elementary permeabilities do not intervene simultaneously.

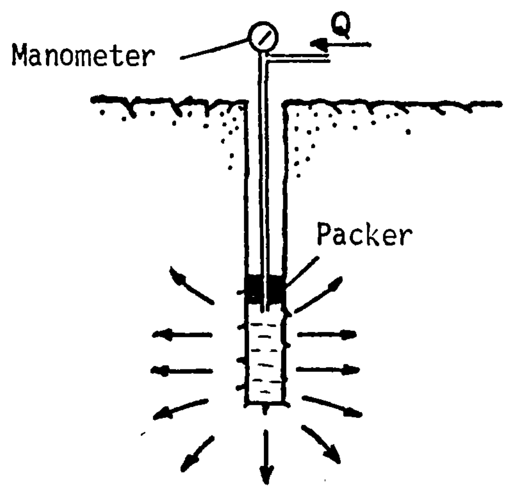


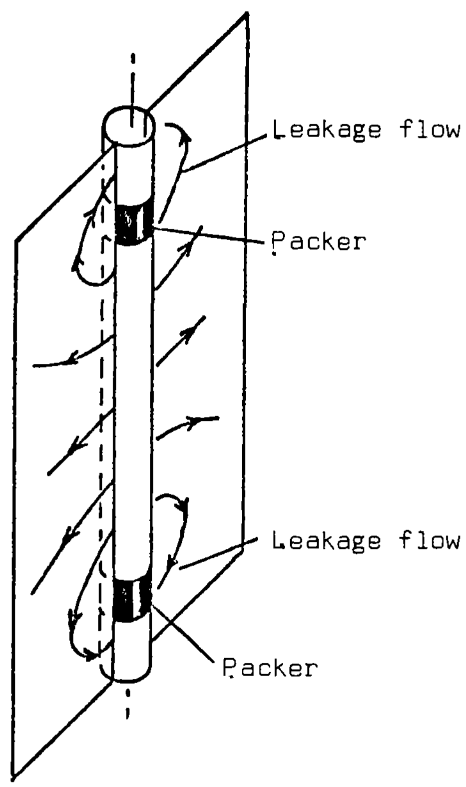
Fig. 5 - Conventional Lugeon test

The technique used is different from the conventional Lugeon test (fig. 5) which only gives a total value of the permeability, without any allowance being made for anisotropy.

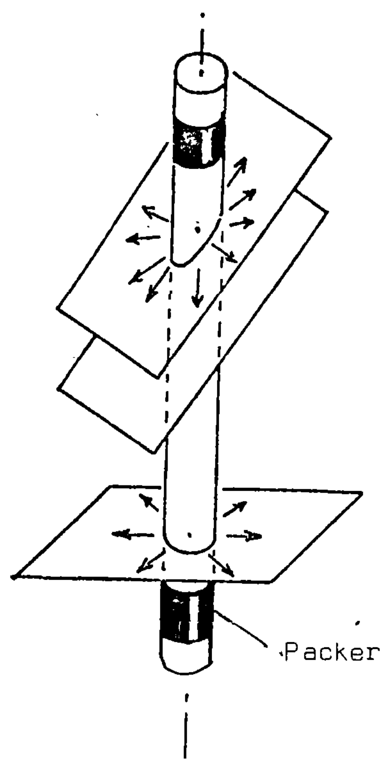
For this kind of testing the flows are indeed either spherical (in the external parts) or cylindrical (in the central part), or they are mixed. Hence, it is impossible to give from such tests a correct interpretation of the results since the relative importance of each kind of flow or, in other words, the contributions of each directional permeability value is unknown. For anisotropic media the directional permeability values are unequal. A precise interpretation of the traditional tests is not possible even for isotropic media, since the relative importance of the cylindrical and spherical flows is not known, and the equations are different for each type of flow.

Moreover, the Lugeon test does not require the borehole to have a specific direction, because it does not take into account the existing fracturing.

It is however essential to take into account the orientation of the fracturing in the interpretation of the test as shown on fig. 6.



Fracture parallel to the borehole



Fracture not parallel to the borehole

Fig. 6 - Influence of the direction of the fractures on the flows during water tests

Finally the conventional Lugeon test is carried out without piezometric control.

In the proposed new method, the test is carried out using a <u>triple hydraulic probe</u>. This eliminates all the defects inherent to the

Lugeon test, that is leakage around the packers, effects at the limits of the injection section, and uncertainty in the injection pressure. The control of piezometry around the borehole is made using a point piezo-permeameter. The new triple hydraulic probe to determine the directional hydraulic conductivities of jointed (or porous) media has been patented by C. LOUIS in France (1972). In the following more details will be given on this new technique.

In order to reduce the inconveniences mentioned in the previous section, it was necessary to develop a testing technique in which, on the one hand the type of flow was unique and completely known and where on the other hand, the elementary permeabilities did not interfere simultaneously. These conditions are all satisfied in the test-scheme of figure 7. The flow in the central section, the dimensions of which are completely known, is perfectly cylindrical with certain conditions, and the influence of the permeability parallel to the axis of the borehole has been eliminated. Such experimental conditions can be achieved by means of a hydraulic triple probe which has three testing sections limited by three or four packers. The probe is mobile i.e. it can operate at every point of a borehole, even if it has large dimensions.

Three packers only can be used (fig. 7) since the last section will be limited on the lower side by the bottom of the borehole. When the flow rate in this section is too great (for example at the upper part of a borehole) a fourth packer could be attached to the probe (fig. 8).

The tests will first be carried out at equal pressures in the three sections. Only the flow in the central section (a typically cylindrical flow) will occur in the calculations. In addition, one could carry out tests at different pressures (with flow possible between the test sections through the medium which is tested), and this would yield as a result the permeability parallel to the axis of the probe. One can obtain a wide variety of applications for the probe by thus manipulating the pressures in the various sections.

The purpose of this chapter is to give a short description of the operational method and the methods of interpretation of the results. To put it simply, piezometric measurements are required in the neighbourhood of the testing zone (at least for precise tests), and boreholes in three directions are recommended for the general case of a three-dimensional

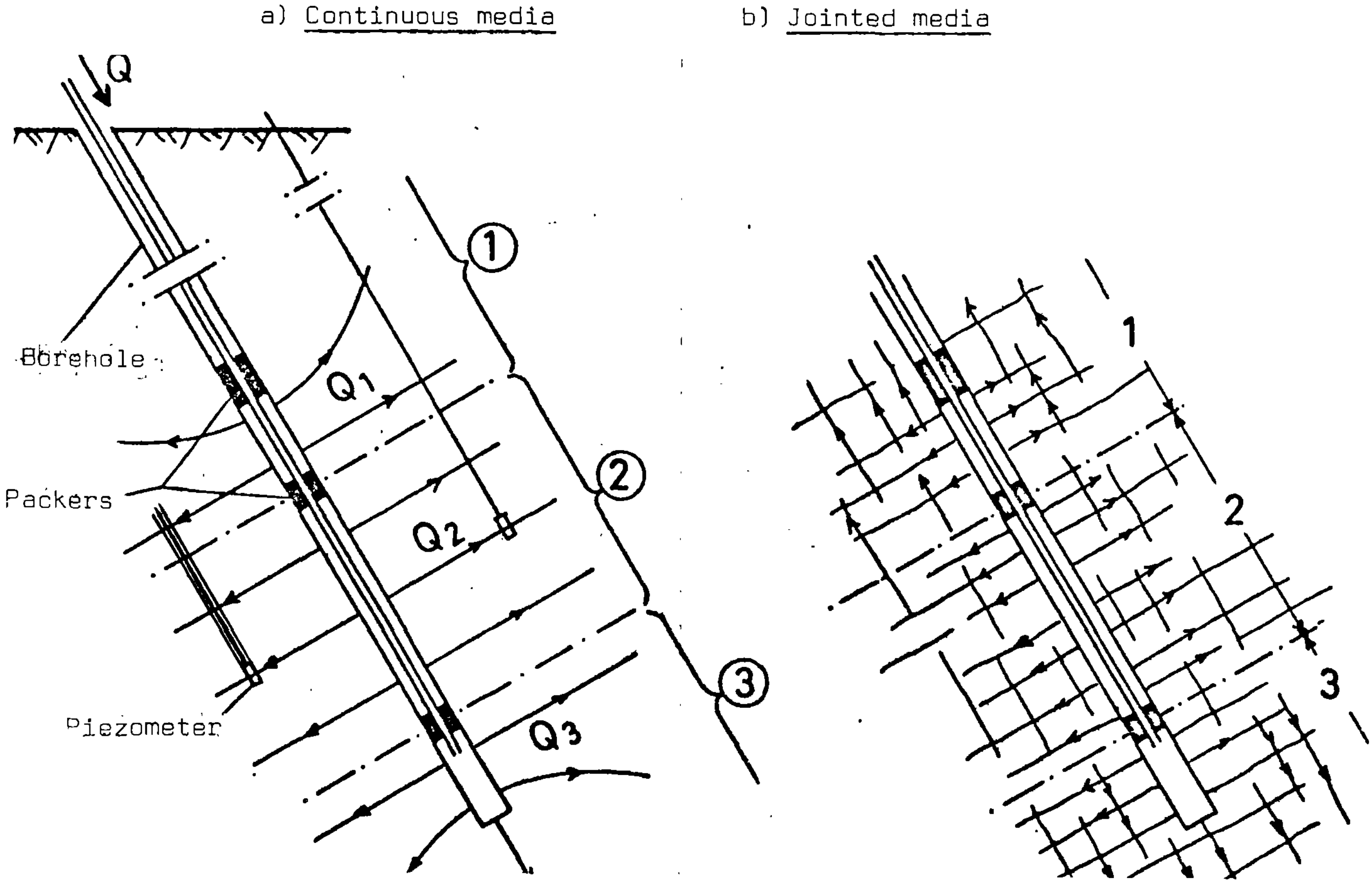


Fig. 7 - Proposed new technique for water tests using the triple hydraulic probe
(1) and (3) Three dimensional flow
(2) cylindical flow

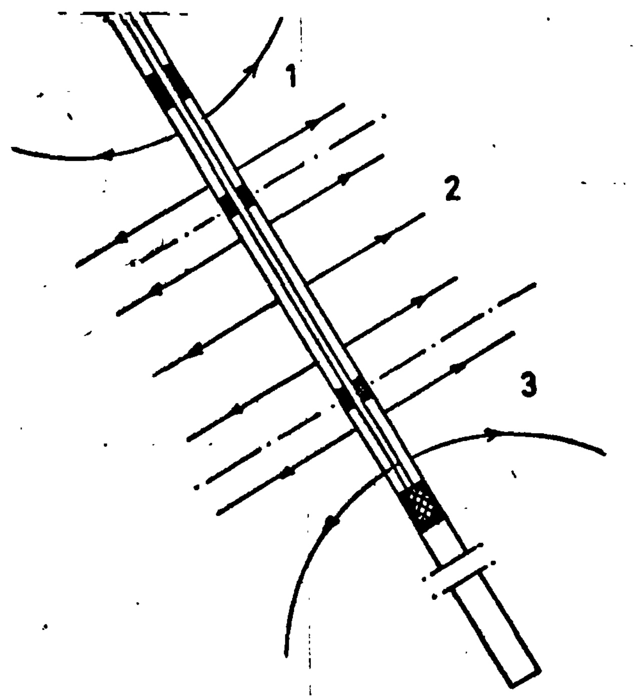


Fig. 8 - Water test arrangement with four packers

problem for a thorough study. However, in special cases, a single testing direction can suffice to determine all the hydraulic characteristics, even for three dimensional problems.

In sedimentary rock or in soil the test directions should coincide with the assumed directions of the principal permeabilites, which are generally parallel and perpendicular to the sediment beds. In jointed rock with, for instance, three systems of parallel joints, the direction of a borehole will be taken parallel to the directions of the two other joints in order to test one of the joint systems.

When there is only one borehole direction available (e.g. vertical), it is always possible to work first with equal pressures and then with different pressures in the three testing sections. In this case, a special technique and interpretation of test results must be used. But it is especially interesting to note that it is possible to determine exactly with one single test (with equal pressures), the horizontal permeability which is the only one appearing in the theory of DUPUIT. It is actually possible to show that the various formulae of DUPUIT which are quite often used nowadays, are correct for the calculation of the flow rates, provided that only the horizontal permeability is used in the calculations. This remains valid for stratified or multi-layered media, with the beds more or less inclined. So this observation contributes considerably to increasing the interest of the method proposed here.

3.2.4 - *The triple hydraulic probe*

The description will be limited to the actual hydraulic probe. The accessory equipment such as surface installations, pumps, flow-meters, meters for registration of the results and piezometers acting in the testing zone will not be discussed. The hydraulic triple probe is schematically illustrated in a longitudinal section in fig. 9. Two models are proposed, one with hydraulic, the other with electric measurements. No dimensions are drawn in the figure because the dimensions of the probe can vary according to each particular problem (the diameter of the borehole can vary from some centimetres to several decimetres). A miniaturised version of the probe is equally possible. To simplify matters, only the system with three packers is considered in the following description (see fig. 9).

a) With hydraulic measurements b) With electric measurements

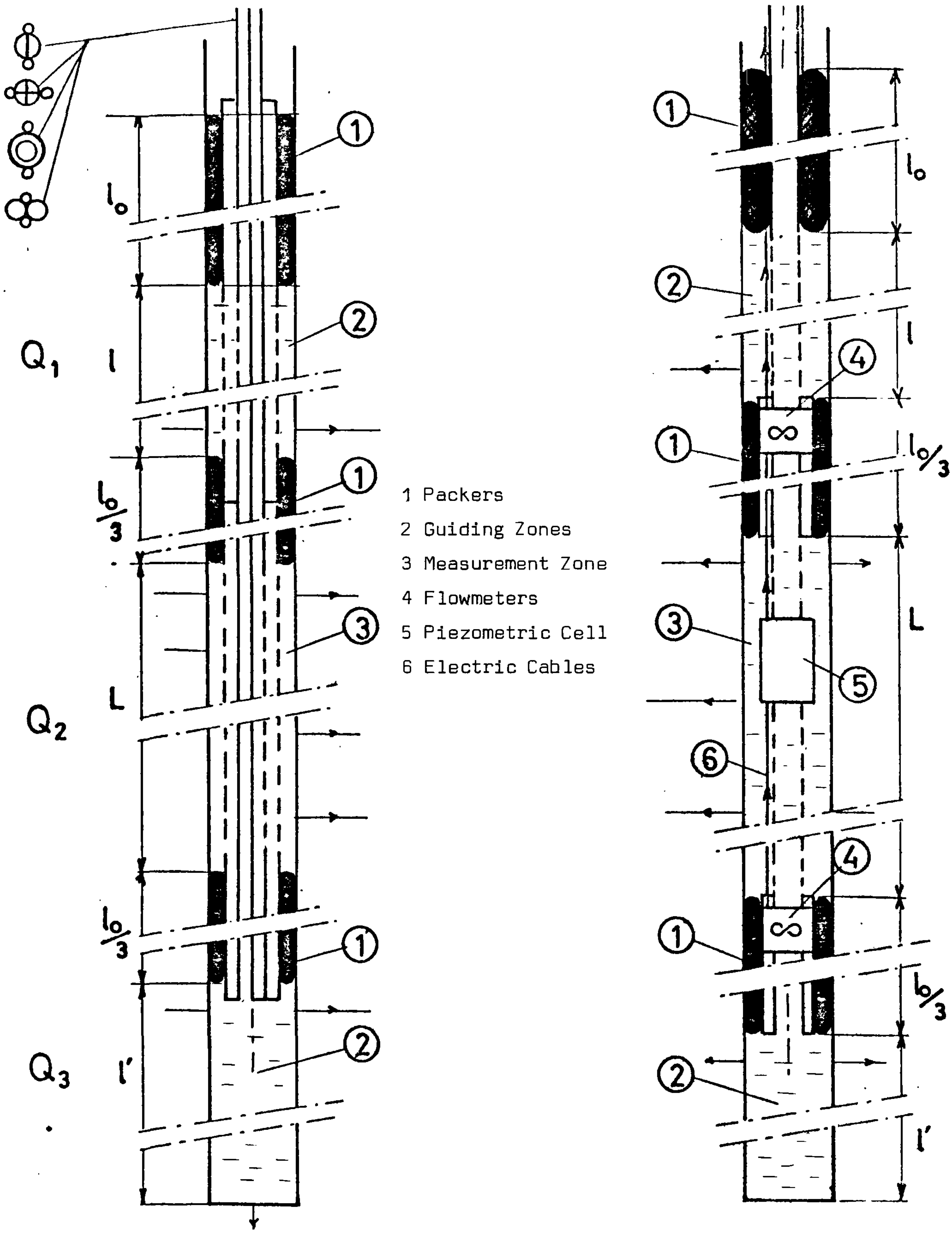


Fig. 9 – Hydraulic triple probe for the determination of the directional hydraulic conductivites of porous or jointed media.

a) Probe with hydraulic measurements

- Double tubing, either coaxial or not (a minimum of two pipes). One of the pipes will serve for the outer sections, the other for the central section. Perforations in the pipes provide the connection between the pipes and the testing zones.

- Three packers (four in the case of fig. 8) preferably of a pneumatic kind which requires the installation of an additional pipe for the compressed air. Other types of packers can also be used.

The discharge ($Q_1 + Q_3$ in one pipe, Q_2 in the other according to the notations of fig. 7) will be measured on the surface and likewise the pressures. However, to account for the head loss in the pipes, the water pressures in the testing sections could be measured by means of attached pipes or pressure gauges (transducers).

b) Probe with electric measurements

- Simple tubing : the probe can thus function with the surface apparatus and the connection pipes used in current methods (with one single pipe).

- Three packers (as before).

- Two flowmeters with electrical transmission. The total discharge is measured on the surface, the flow $Q_2 + Q_3$ by the first flowmeter and finally Q_3 by the second flow-meter.

- A piezometric cell in the central section.

- Electrical connections between the probe and the surface.

Using the triple hydraulic probe with a double water tubing (fig. 9a) it is possible to carry out tests with different water pressures in the external and central sections. On the other hand, the device with the single tubing (fig. 9b) allows only tests with the same pressure in the three sections.

One prototype of the triple hydraulic probe has been manufactured in France and tested on several sites (LOUIS, 1972, 1973). The first case concerns the study of the flow conditions to a large open excavation (diameter 80 m, depth 25 m) in a fissured chalk aquifer in Lille, France (fig. 10a). The second pratical application of this technique was carried out in connection with hydrogeological investigations at the Grand Maison dam-site in the French Alps (fig. 10b). After a first simulation of the flow in the dam abutments, a second important phase of the work was the verification in the field of the effects of the fractured system on the flow conditions. A series of boreholes were carefully placed around the dam-site and pumping tests were performed to verify the results of the theoretical model studies.

The use of such a set-up normally requires some measurements of hydraulic heads during the test in the vicinity of the testing area. For this control the use of a new device "the piezopermeameter" is recommended (fig. 11 or 12). Testing with the triple hydraulic probe with four piezometric measurements makes it possible to determine the three dimensional distribution of the anisotropic permeabilities or hydraulic conductivities.

3.2.5 - The continuous borehole piezopermeameter

This instrument has been designed in conjunction with the triple probe. It can be used alone for a point (finite) piezometric measurement, or in conjunction with a test using the triple probe.

For piezometric monitoring two constraints exist (fig. 13 and 14) :

- The measuring equipment must not disturb the natural flow net under study. However a borehole drilled through an aquifer short-circuits the different layers encountered.

- The information collected should be for a specific point and not integrated due to in-and out-flows of water in the measured zones.

Point values only make it possible to obtain hydraulic gradients.

a) Investigations for the foundation of a large building (Diplodocus, Lille)

b) Measurement of hydraulic conductivities at Grand Maison dam-site (Isère)

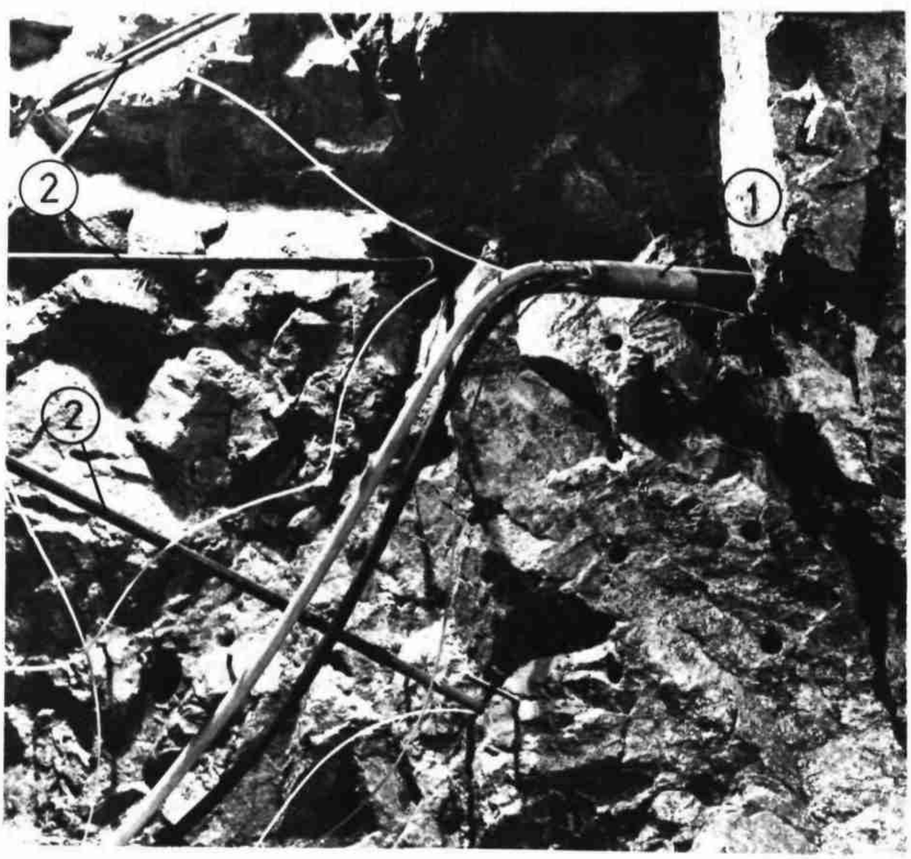


Fig. 10 - Examples of testing with the triple hydraulic probe
(1) Triple hydraulic probe
(2) Lateral piezometers
(3) System to move the probe.

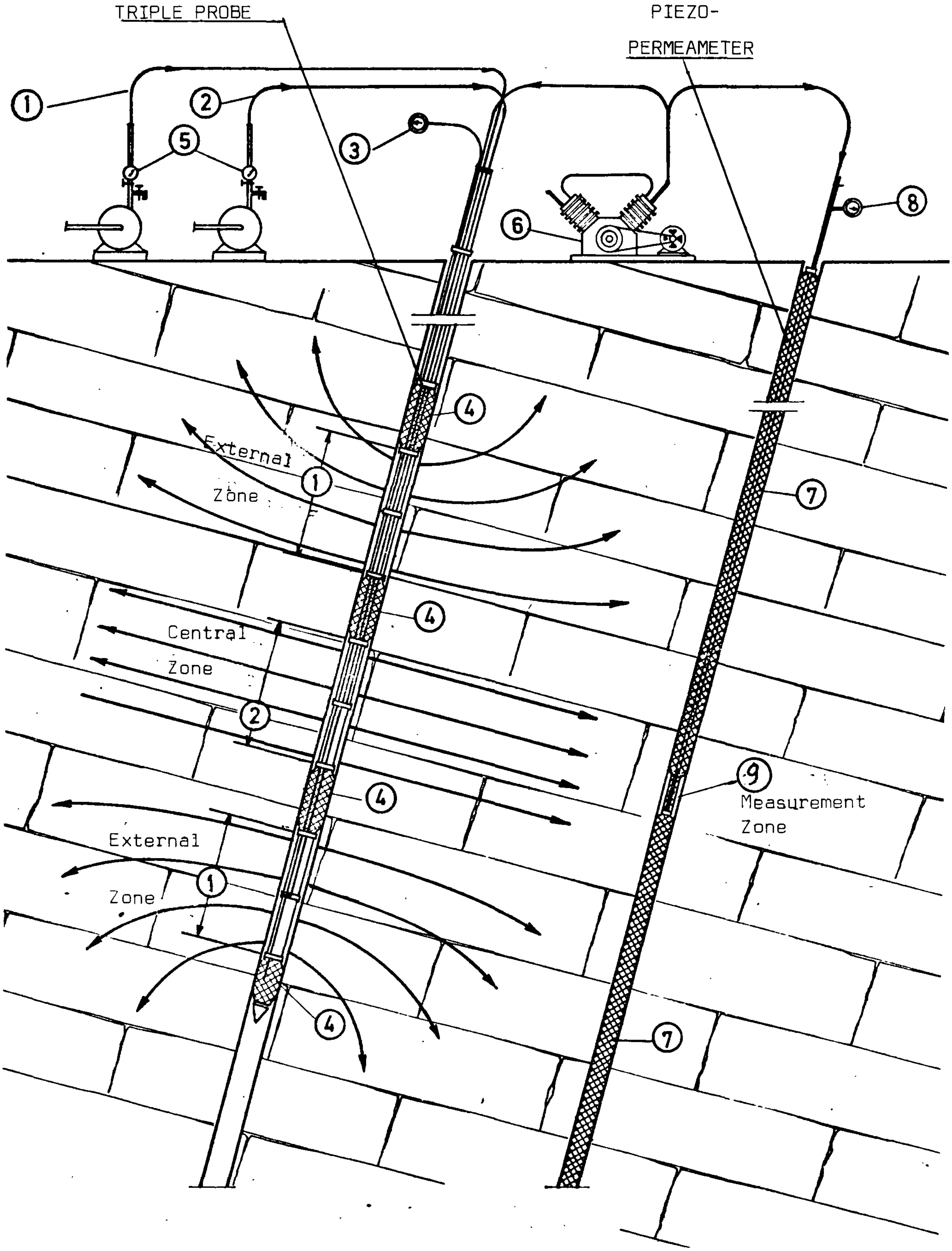


Fig. 11 - Water test by using the triple hydraulic probe and the piezopermeameter

(1). Injection in the external cavities
(2) Injection in the central cavity
(3) Pressure in the central cavity
(4) Packers
(5) Flowmeters
(6) Compressor
(7) Generalised packer
(8) Pressure in the measurement cell 9.

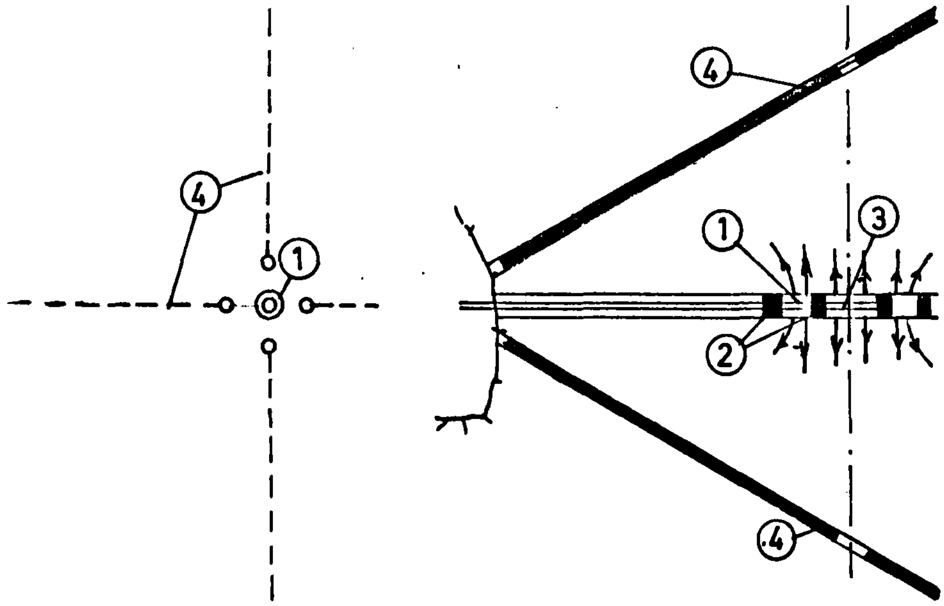


Fig. 12 - Hydraulic test carried out at Grand Maison dam-site (see fig. 10b)
(1) Triple hydraulic probe *(3) Piezopermeameter*
(2) Central injection cavity *(4) Measuring cell*

a) Conventional technique b) New technique

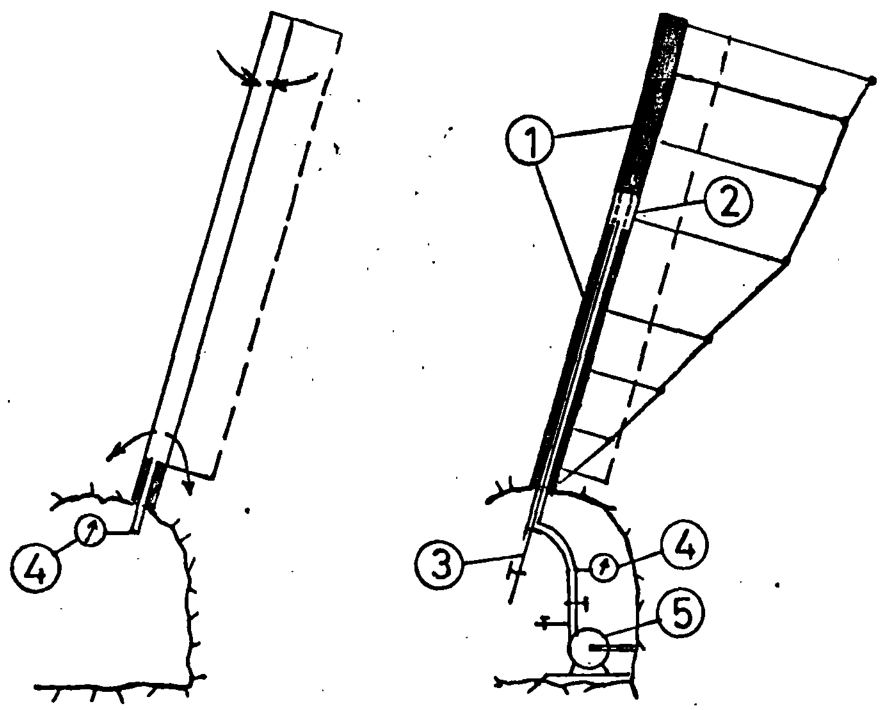
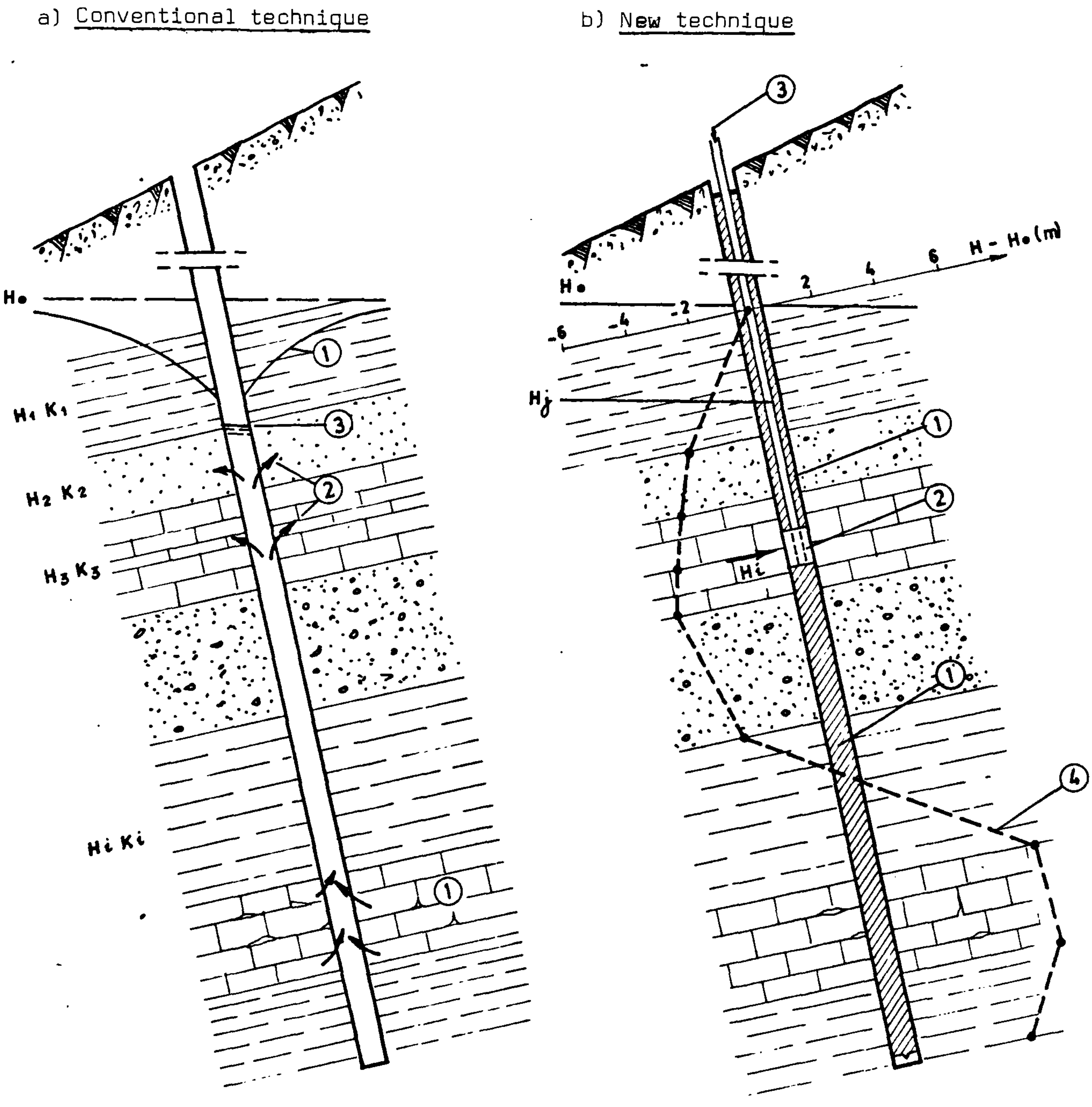


Fig. 13 - Piezometric measurement in gallery
(1) Generalised packer *(4) Pressure gauge*
(2) Measurement cell *(5) Pump*
(3) Bleed valve



(1) Drainage due to the borehole
(2) Flow in layers at low piezometric head
(3) Resulting level

(1) Generalised packer
(2) Measurement cell
(3) Pressure measurement
(4) Piezometric log

Fig. 14 - Piezometric measurements from the surface in multi-layered aquifers

Ho Piezometric head of the free surface

Hi Piezometric head at any point M

Ip = Hi-Ho "Piezometric index"

It is therefore necessary to use a general packer which isolates the screened measurement cell. To facilitate measurements it is better, particularly for media of low permeability, that the volume of water necessary to obtain the measurement be very small. Furthermore, the response time of the system must be rapid. In this method, the measurement cell is saturated by injection of water at a pressure close to that to be measured (air removed by bleed valve). The final pressure, reached after a very long delay can be estimated by two relaxations close to the pressure assumed to exist in the mass. It is the average of the upper and lower values when the dampenings are identical during an increase or a decrease in pressure (fig. 15).

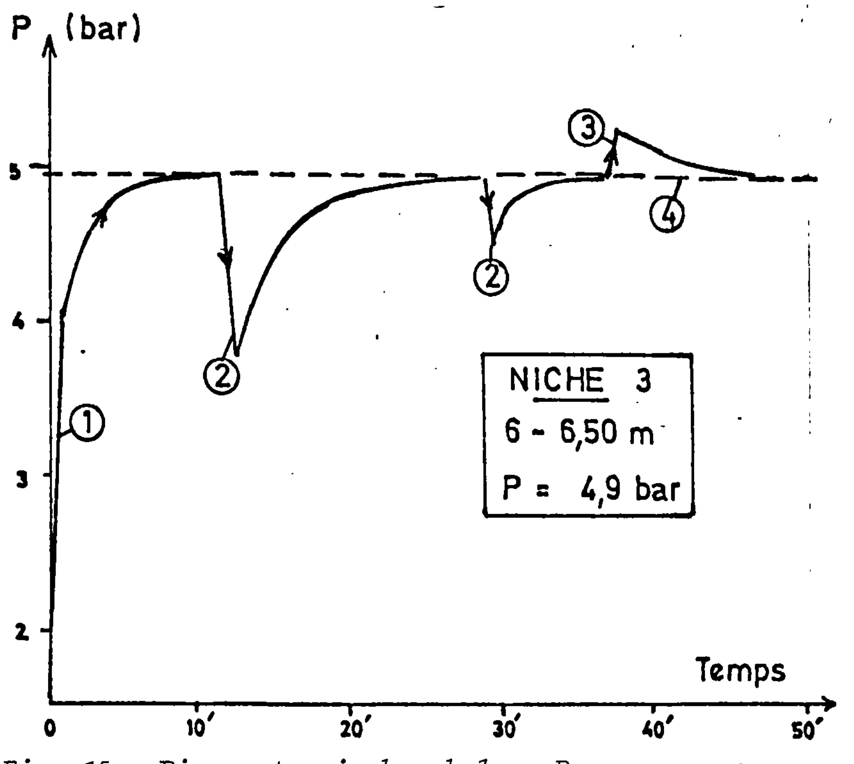


Fig. 15 - Piezometry in borehole - Reponse curve
(1) Air removal and water injection
(2) Air bleed
(3) Injection
(4) Pressure to be measured

It is worth noting furthermore that the rate of variation of the value to be measured and the shape of the dampening curve permit an evaluation of the order of magnitude of permeability within the measured zone.

a) Coupling the screen with the generalised pneumatic packer

b) Introduction of the piezopermeameter into a borehole

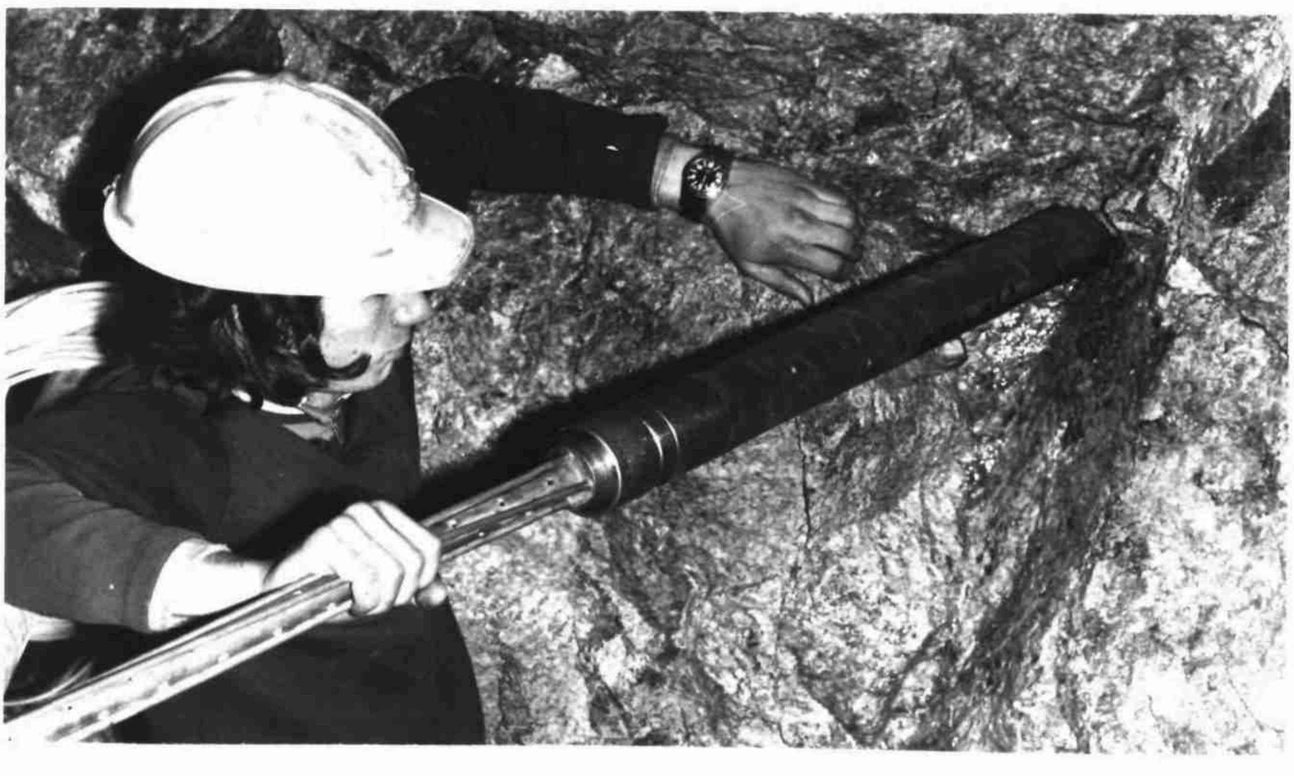


Fig. 16 - Layout of the piezopermeameter set-up

This technique has been tested with success in France by the B.R.G.M. ; some details of the equipment are given on fig. 16.

The piezopermeameter alone is also useful for solving many hydraulic problems, for instance the analysis of water effect on the stability of slopes. A possibility of hydraulic monitoring of a slope is described in fig. 17. First of all, the free surface of the ground water has to be determined by means of a few short boreholes (fig. 17a). Then a piezometric log in a single borehole (perpendiculer to the free surface) determines whether the medium is isotropic (by constant piezometric head in the borehole, fig. 17b), or anisotropic with good natural drainage of the slope by decreasing of the piezometric head (fig. 17c), or with load drainage by increasing of the piezometric head (fig. 17d).

These three cases have similar boundary conditions and free surface by a steady flow. The stability of the slope is, of course, much better in case c than in case d.

The determination of the exact piezometric log gives information on the anisotropy of the medium, the flow-net and finally the stability of the slope.

3.3 - Influence of state of stress

The state of stress has a very important bearing on the hydraulic characteristics of fissured media. Thus a variation of permeability may be observed as a function of depth (self weight of the mass) and under any external influence (e.g. modification of the geometry of the medium).

The influence of stress can be introduced in the expression of hydraulic conductivities or of the permeability tensor $\bar{\bar{K}}\ (x,y,z) = K.\bar{\bar{T}}\ (x,y,z)$ through K, the absolute modulus of permeability. During simulation of flow in a mathematical model, it then becomes possible to modify at will the hydraulic characteristics as a function of the state of stress.

The laws of variation of the parameters which enter in the expression of the hydraulic conductivity of a family of continuous fractures are unknown. They depend on the mechanical behaviour of each type of fracture. Only an experimental approach seems realistic.

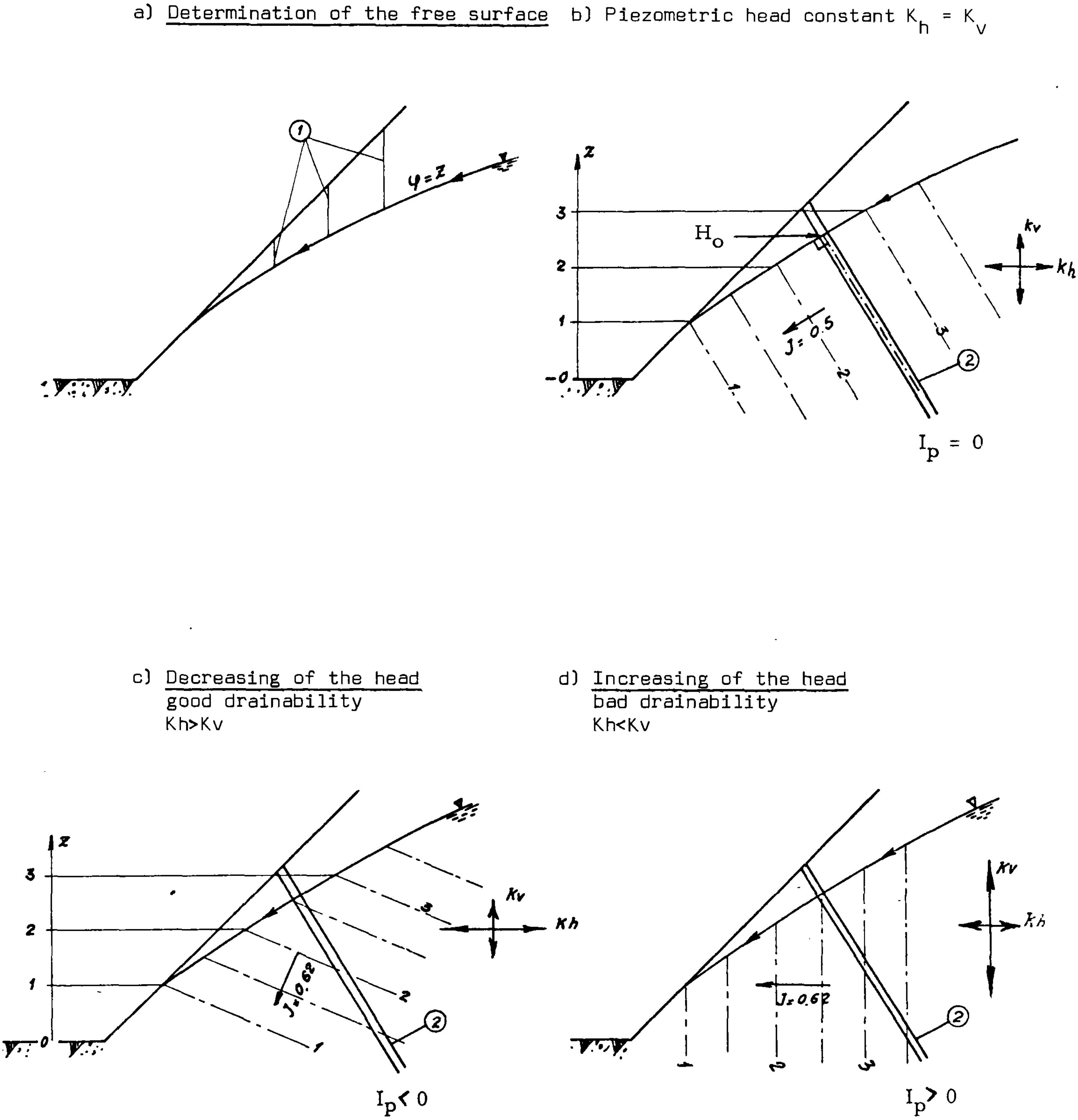


Fig. 17 – Monitoring of the ground water flow conditions in a slope using the continuous piezometer in a borehole perpendicular to the free surface (steady state in a homogenuous medium)
(1) Small boreholes for determination of the free surface
(2) Borehole for the piezometer
Kh, Kv Horizontal and vertical permeabilities
Ip = H-H_o "Piezometric Index"

- *In situ* tests (fig. 18a)

Permeability tests in a borehole at varying depths in homogeneous fissured formations show that the empirical law which described the phenomenon is most often of the type :

$$K = K_o . e^{-\alpha\sigma} \text{ with } \sigma \simeq \gamma . t \qquad (12)$$

where Ko = superficial permeability ; *or reference permeability*
$\gamma . t$ = weight of overlaying formations.

- Laboratory tests

Laboratory tests can be performed on samples crossed by one fissure only (study of permeability under stress), or on cores of smaller size, in order to look at finer jointing (fig. 18). The samples can be studied under any axisymetrical stress field by use of a permeameter with longitudinal flow. A study of numerous samples with different orientations is then necessary.

Exponential laws of variation of K are frequently encountered (LOUIS 1974). These tests give a distribution of permeabilities which are close to reality and can then be used in the simulation by mathematical models.

3.4 - General discussion on water testing

3.4.1 - Criticisms of the water test analysis

The main objections to the water test analysis based on potential theory are as follows :

1) Effect of the radial flow (variation of the velocity in the flow direction).
2) Turbulence effect.
3) Deformation of the medium under joint water pressure during the test.

4) Influence of K_2 or K_3 on the test in K_1 (see fig. 4).

5) Entrance loss.

6) Influence of the time and of possible unsaturated zones.

The influence of the variation of the flow velocity in the flow direction is maximum in completely radial flow (e.g. for $\alpha = 0$, fig. 4). The theoretical correct equation for the potential function is given by :

$$\phi = \frac{Q/L}{2\pi K} \log r - \frac{6}{5}\frac{\bar{v}^2}{2g} + \text{constant} \qquad (13)$$

where $\phi = z + p/\gamma w$ = piezometric head

Q/L = flow rate per length unit

K = hydraulic conductivity of the jointed rock mass

r = radius to the borehole axis

v = mean flow velocity in the fracture perpendicular to the borehole.

The second term, due to the variation of kinetic energy, can be introduced in the elemental equation (11) but in practice this term is often negligible (WITTKE and LOUIS, 1969).

In a water test, turbulence can begin at a very small flow rate because the gradients near the borehole are extremely high. To know if the flow is laminar or turbulent it is, in practice, necessary to plot "flow rate - hydraulic gradient" for many points by increasing flow rate from zero (see next section). The maximum influence of the turbulence is near the borehole and can be taken into account as follows :

$$\phi = \varepsilon \frac{1}{2} \left(\frac{Q/L}{2\pi K'} \right)^2 \frac{1}{r} - \frac{V^2}{2g} + \text{constant} \qquad (14)$$

where $\varepsilon = -1$ in pumping out

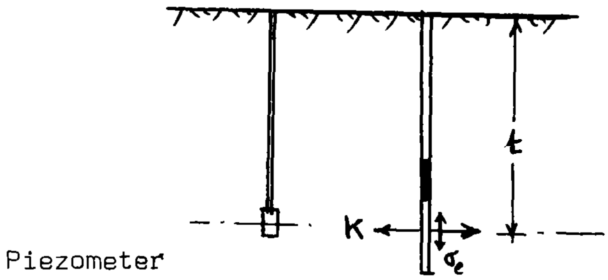
$\varepsilon = +1$ in injecting.

For application of equations (13) and (14) in the water test the variation of kinetic energy $V^2/2g$ between two points 1 and 2 can be given by (for radial flow) :

$$\Delta \left\{\frac{V^2}{2g}\right\}_1^2 = \frac{q^2}{8g\pi} \left\{ \left(\frac{1}{er}\right)^2 \right\}_1^2 \qquad (15)$$

a) In situ Water test

Variation of hydraulic conductivity with the depth



Testing borehole

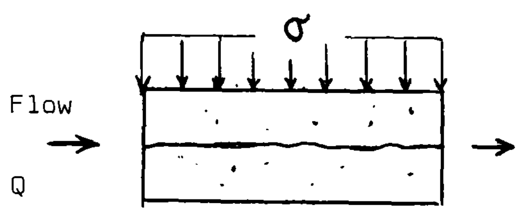
In homogeneous fissured rock

$$\begin{cases} K = K_o e^{-\alpha\sigma_e} \\ \sigma \simeq \gamma t \end{cases}$$

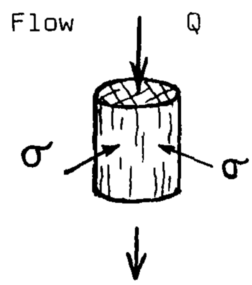
$$\text{Log } K_o - \text{Log } K = At$$

b) Laboratory testing

·On single fracture (25 x 30 cm)



On fissured material



Triaxial flow test

$$K = K_o e^{-\alpha\sigma}$$

Fig. 18 - Investigation on the influence of the state of stress on the hydraulic characteristics (LOUIS, 1974)

where q = flow rate per fracture ≃ Q/N (N number of the fractures in the testing section L),

e = mean opening of the fractures.

In practice the Lugeon test is commonly used. The water pressure in this method is generally extremely high ; this can cause deformation of the medium. In the normal case (fig. 13a) the test length L is intersected by many elemental joints (L>>b). In this case, applying elastic theory, the conductivity of the underformed medium is given by :

$$(K)o = \frac{1}{\left(1 + \frac{\alpha p}{E_m \cdot n}\right)} \quad (K)p \qquad (16)$$

Where p = integrated mean water pressure between the two considered points by the determination of (K)p,

α = coefficient whose magnitude is dependant upon the lateral stresses. α = 1 if lateral stresses are neglected and 0.5 to 0.9 if they are included.

Em = deformation moduli of the rock matrix between two successive joints.

n = $\frac{e}{b}$ = joint porosity of the considered joint system.

(K)p = hydraulic conductivity of the joint system during the test (fig. 19a).

In the extreme case of one joint intersecting the test length (fig. 19b) the equation (16) does not give the correct answer for permeability because the loading zones under water pressure for figure 19b are different to those of figure 19a. This case was considered by SABARLY, 1968.

Finally, the entrance loss in a water test can be expressed as :

$$\Delta\phi = \xi \frac{V^2}{2g} \qquad (17)$$

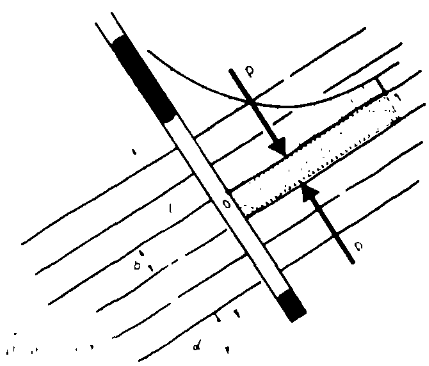
where V is the flow velocity in the joint directly near the borehole.

The coefficient ξ is in the case of a water test approximately 0.5. For 10 m/s flow velocity this loss is roughly 2.5 m head of water. This loss may be allowed for special cases.

3.4.2 - *Practical procedure*

The water test is commonly used in practice. To obtain meaningful results from this test it is necessary, particularly for jointed rock, to observe some fundamental rules.

With a fracture system in the testing area



With a single fracture in the testing area

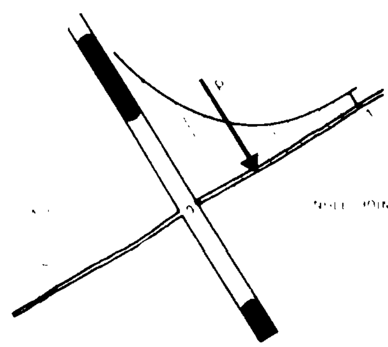


Fig. 19 - Deformation of the medium during the water test

a) The borehole direction

In a rock mass with three joint systems the optimum hole direction for testing one joint system is parallel to the other two systems.

If the extent of jointing is very large and irregular, the medium must be considered as a continuum and a conventional soil mechanics test may be carried out, bearing in mind that the scale of the test must be correspondingly large compared with the size of the rock blocks.

b) Length and diameter of the test section

The length of the water test section is represented by L on fig. 4. It is not possible to give an optimum length for every test. This length depends on the mean spacing b of the joints and on the scale of the studied flow phenomenon (flow through a slope or under a dam etc.). The ideal test length is thus equal to the dimension of the network used by the numerical analysis for the flow, but this condition cannot always be attained in practice.

A test in a gallery or a well ($\phi > 2$ m) is more representative, but more expensive, compared to tests in boreholes. If, for financial reasons, the number of large scale tests is limited, then the first approach is to carry out water tests in boreholes and subsequently to confirm the results by using a gallery or a well. Borehole water tests allow a statistical consideration because of the large number of tests which can be carried out.

c) Test pressure

The interpretation of the water test is rendered difficult by using high pressure, because secondary phenomena influence the test results (turbulence, fissure deformation etc.). Particularly for radial flow, as in a water test, the very high gradient near the borehole causes turbulent flow. In addition the 10 kg/cm^2 pressure used by Lugeon test produces a big deformation of the medium near the borehole. The plotting of "flow rate-hydraulic gradient or water pressure in the borehole" gives

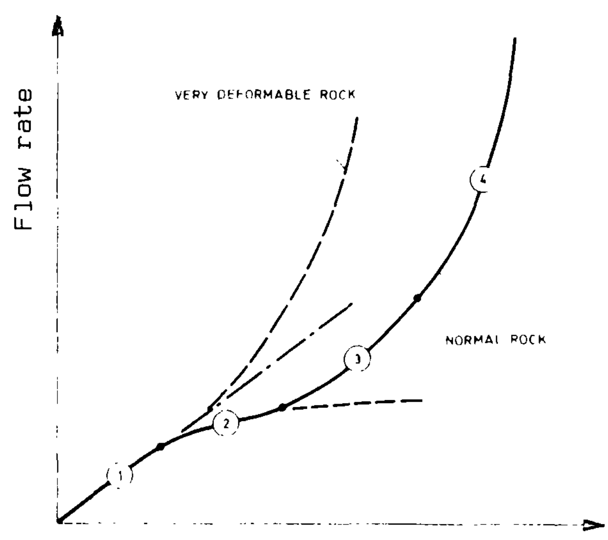


Fig. 20 - Typical results of field water test.
(1) Laminar flow
(2) Turbulence effect
(3) Turbulence offset by fissure expansion
(4) Predominance of fissure expansion effects.

the characteristics curve of the water test. Figs. 20 and 21 show a typical test result in the field. Every principal effect is represented. After a short linear phenomenon (1), turbulence effects are noticed (2). This effect is quickly compensated by the influence of the opening of the joints through the high water pressure (3). After a certain limit (between 4 and 6 kg/cm^2 in practice) the influence of the joint deformation is predominant (4). In very deformable rock the effects as demonstrated by (2) and (3) can disappear.

In practice it is very useful to carry out some tests for the entire range as in fig. 20 so as to get an indication of the inaccuracies involved. The hydraulic conductivity however, is obtained only from zone (1) or, if not possible, then from zone (2). From the turbulent conductivity it is easy to get the laminar conductivity as follows (see above section) :

$$K_{lam} = A(K_{turb})^2 \qquad (18)$$

The shape of the complete curve of fig. 20 gives an idea of the deformability of the medium. Fig. 21 gives some concrete examples.

The amount of pressure which is to be applied during the test must be obtained from fig. 20. This test curve must be obtained initially before a large number of tests are carried out to establish the optimum working pressures for a rock type. It must be remembered that 1 Lugeon corresponds to 1 litre per metre per minute at 10 kg/cm^2 excess pressure but it can also be defined as 1 litre per metre per 10 minutes at 1 kg/cm^2 excess pressure.

d) Measured values

According to the above section the linear flow rate q/b or Q/L and two points on the piezometric line $\phi(r)$ lead to an interpretation of the test. The first point is given by a measurement of the water pressure in the borehole itself ; the second must be chosen, if possible, in the vicinity of the borehole (see fig. 22). For this reason boreholes in the field should be drilled in two's, e.g. : 1 - 2 and 3 - 4 on fig. 22.

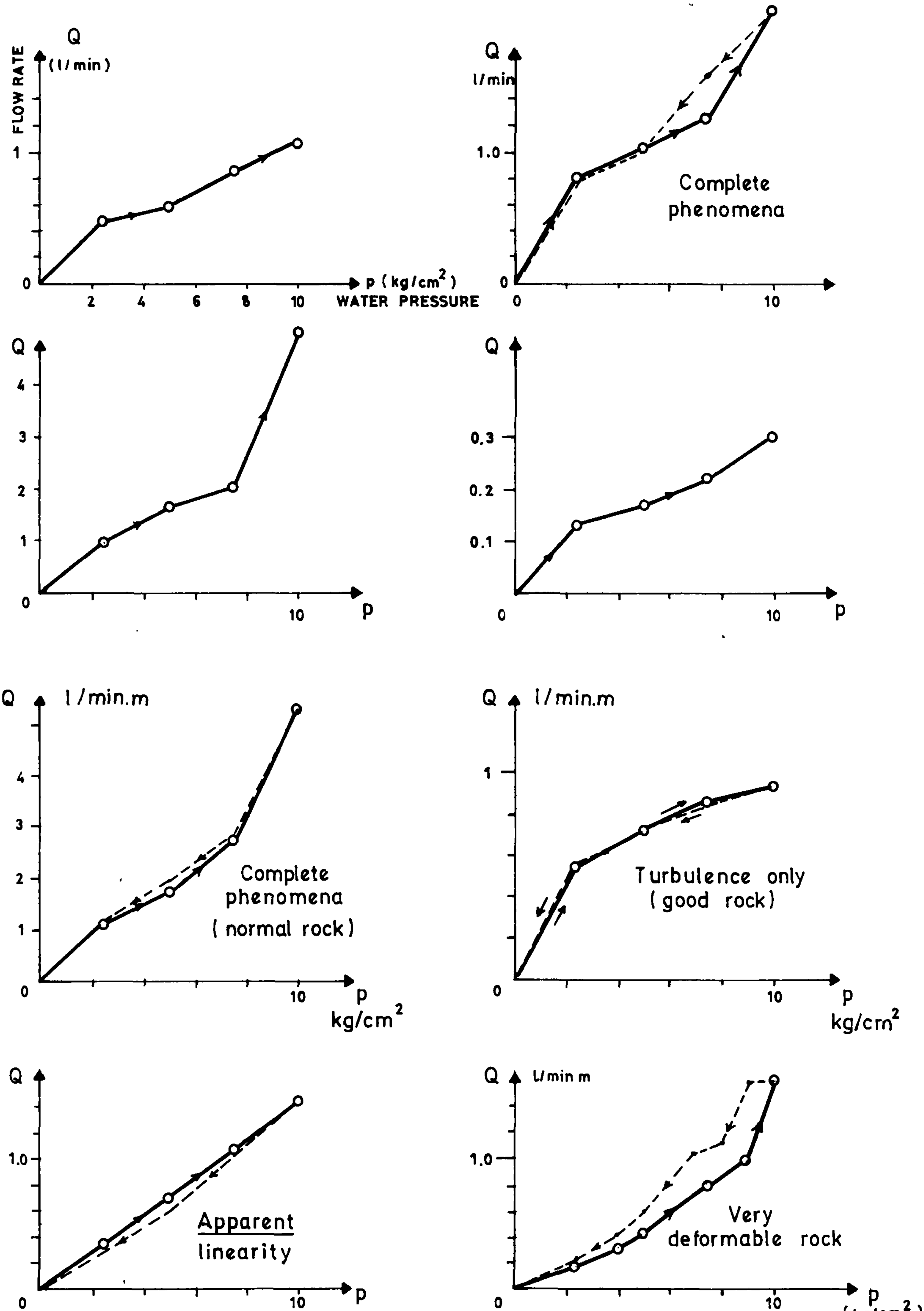


Fig. 21 - Pratical results of in situ water tests

Very often in practice, financial considerations limit the number of boreholes that can be drilled. In such a case the radius of influence R is used as second point (R is the point where initial groundwater conditions do not change). The radius of influence is given by an empirical equation according to SICHARDT (see CASTANY 1963) :

$$R = 3000\ (\phi_{ro} - \phi_R)\sqrt{K} \qquad (29)$$

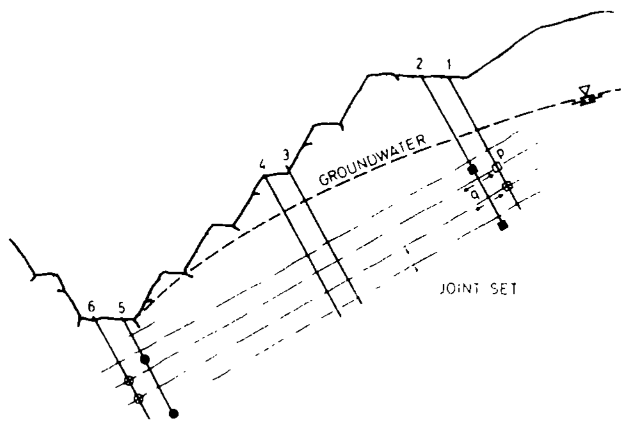


Fig. 22 - Water test procedure
(a) Test in borehole 2, measurement in 1
(b) Test in 1, measurements in 2 etc..

In this way it is always possible to approach the required hydraulic conductivity. Errors in the evaluation of the radius of influence do not affect test results very much. The terms Log $\frac{R}{r_o}$, $\frac{1}{r_o} - \frac{1}{R}$ or $\frac{1}{r_o^2} - \frac{1}{R^2}$ from equations (11, 13, 14) do not depend very much on R because $R >> r_o$. However, the best way to interpret the water test is to measure the piezometric head at two different points.

e) Type of water test

There is an essential difference in results between injecting in or pumping out. A more representative test is to pump out. But this is not always possible if, for example, the depth of the free surface in the borehole below pump is greater than 6 m. At the depths greater than this it is necessary to put the pump in the hole and the cost becomes formidible compared with the usual injection of water from ground level in a small borehole (e.g. NX).

3.5 - Conclusion

To solve most practical problems in rock hydraulics it is always necessary to have information on *in situ* hydraulic parameters. All the hydraulic parameters can be represented by the concept of "directional hydraulic conductivities". These conductivities, which are to be used in mathematical or physical models, can be obtained from the field using the water test.

If the number of the hydraulically principal joint systems is one, two or three, then each joint system can be tested separately. In this way it is easy to describe the anisotropic behaviour of a jointed medium. In interpreting the water test results both the geometry and the regime in which the test is carried out must be considered. The working pressure (e.g. 10 kg/cm^2) in conventional tests is often of sufficient magnitude to place the test conditions out of linear regime.

This method is not applicable if the number of hydraulically principal joint systems is larger than three or if the jointing is irregular. In such cases the permeability tensor of the medium can be obtained with large scale tests, assuming the medium as a continuum.

4 - CONTINUOUS OR DISCONTINUOUS MEDIUM

Before starting on a study of the flow in a fissured medium, it is essential to determine whether the problem is to be considered as being continuous or discontinuous. There is no general rule, and this notion only depends on the relative scale of the phenomenon studied and of the modulus of jointing characterized, for instance, by the mean distance between single fractures. This question of relative scale is outlined in fig. 23, which shows the same hydraulic problem, but for four different media.

It will be correct to consider a fissured medium as being continuous if the dimension of individual blocks is negligible as compared to the phenomenon considered (Case 2, Fig. 23) that is, if one can approximately count, say, 10,000 fissures in any plane section. On the other hand, if the number of fissures is between, for instance, 100 to 1000, the hypothesis of a discontinuous medium is necessary (Case 3) and finally, if in a given section, the number of fissures is less than 10, each fissure will have to be individualized in the mathematical or physical model used (Case 4). The number of fissures given here is subjective ; in fact, the hypothesis to be chosen will have to be very carefully analysed for each given problem.

5 - THREE-DIMENSIONAL DISTRIBUTION OF THE HYDRAULIC POTENTIAL

5.1 - Introduction

In this study, we shall not consider the problem of continuous media. It has been investigated by a number of research workers, either by numerical analysis or by electrical analogy. If a problem in rock hydraulics can, because of very close fissuring, be treated by the methods relevant to continuous porous media (for example, Case 2 fig. 23), we will only give the mathematical method for calculating the anisotropic permeability tensor from the hydraulic characteristics of the different systems of fissures.

A number of research workers have already studied the problem of three-dimensional flow within fissured media : SERAFIM (1965, '68), del CAMPO (1965), ROMM (1966) SNOW (1965, '67). They bring the problem down to a tensorial representation of the hydraulic properties of rock masses, traversed by three mutally-perpendicular systems of parallel fissures. Because of the tensorial notation, it is explicitly admitted that we are dealing with a continuous medium. However, this hypothesis is only very rarely verified in fissured rocks (see Paragraph 4).

In contrast to this approach, the methods developed in the present work, through the concept of "directional hydraulic conductivity", take into account the discontinuous character of the fissured rock masses, their heterogeneity in a given field and the completely arbitrary orientation of the network of fissures. Different methods are suggested to deal with the nature of the fracturation, starting from the simplest case (a rock mass with a single system of conductive fractures) to the complex case of n arbitrarily-oriented sets of fractures.

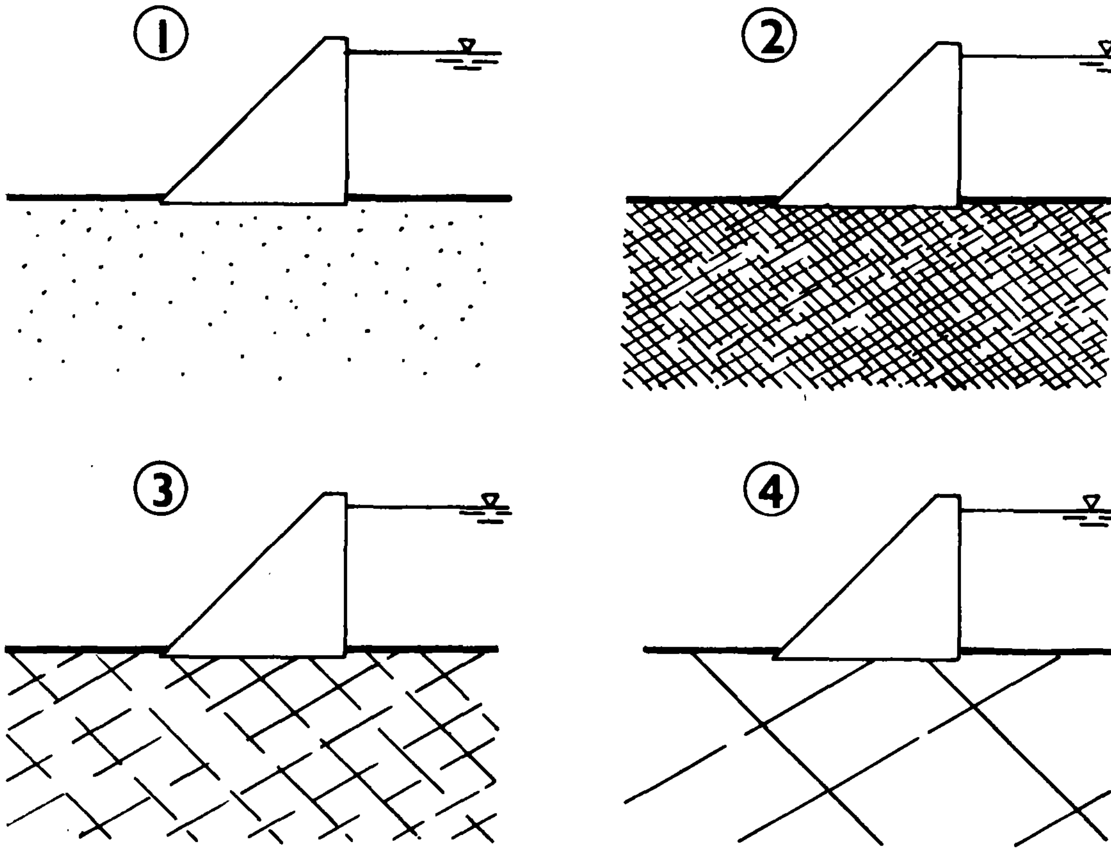


Fig. 23- Continuous or discontinuous medium
Cases 1 and 2 = Continuum
Cases 3 and 4 = Discontinuum.

The methodology we have adopted is based on a fundamental property of the most general types of flow within a fissure. It has been shown that, in the steady state, the flow of water in a fissure, whatever its orientation in space, follows the potential theory when one uses as a velocity potential $k_f\phi = k_f\ (Z + p/\gamma_w)$, which is related to the hydraulic potentiel ϕ. This property also extends to a system of plane parallel fissures when the hydraulic conductivity k_f of a single fissure is replaced by the directional hydraulic conductivity K of the system of fissures.

From this important result, it becomes possible to apply the numerous methods of potential flow theory to each individual fissure or to a series of parallel fissures in a rock mass, as a whole. The problem in space, as a whole, is thus broken down into series of two dimensional problems. In each fissure, or system of parallel fissures, the domain of the hydraulic potential obeys Laplace's equation (harmonic potential). For each individual two-dimensional problem, there are numerous methods of solution : mathematical analysis (conformal mapping), numerical methods (among others, relaxation methods), graphical methods, and also the electric analogy method. As references, we shall quote the works of DACHLER (1936), POLUBARINOVA-KOCHINA (1962), CASTANY (1963), SCHNEEBELI (1966), IRMAY (1968), BEAR (1972), to cite but a few of the most important ones.

It is therefore our intention to find, with the help of mathematical or physical models, a function $\phi(x, y, z)$ which verifies the equation $\Delta\phi=0$ in each individual fissure within a domain D, knowing the directional hydraulic conductivities at the scale of the lattice of the model used, and admitting that function ϕ, or its derivatives taken perpendicular to the boundary, assume given values along the limit of the domain D (flux or potential conditions). Four cases will be considered, depending on the nature of fracturing in the medium.

5.2 - Rock mass with one system of conductive fractures

Fracturing in the rock mass has the following characteristics :

a) A single system K_1 of main conductive fractures (plain or corrugated) and secondary fissures, in the hydraulic point of view of arbitrary orientation (fig. 24).

b) Two systems K_1 and K_2 of main fissures distributed in different domains and intersecting in a limited area. Systems such as K_1 and K_2 are said to be sequent. Secondary fissures may also exist. This case has been considered by LOUIS, 1967-69.

Fissures are said to be secondary if their contribution to the hydraulic potential in the rock mass is very small or nil.

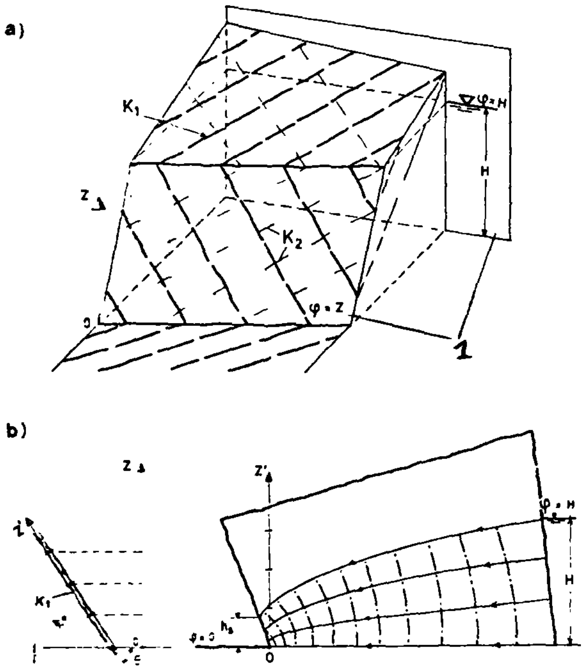


Fig. 24 – Rock slope with network of conductive fissures and flow network in a fissure of the rock mass.
(1) Boundary conditions.

In case (a), the flows in the different fissures of K_1 are quite independent. Each plane of fissuring will therefore be considered individually. In such a plane, the network of streamlines and the equipotential lines can be constructed by the usual methods of two-dimensional potential flow. Figure 24 (b) sketches the streamlines in any single fissure plane K_1 of figure 24 (a). Within the plane of the fissure, axis OZ is an ascending axis directed along the slope. For non-vertical fissures, it is, of course, necessary to multiply values in the vertical direction by sin θ (θ being the slope of fissure K_1 considered in figure 24). One can proceed similarly for all the fissures K_1 of the rock mass. By grouping all the results obtained in three-dimensional space, the total field of the hydraulic potential in the entire rock mass is obtained.

5.3 - Rock mass with two systems of conductive sub-vertical fractures

In this paragraph, we consider a rock mass with two approximately vertical systems of conductive fissures, while the third network, horizontal in this case, is taken to be secondary. Figure 25 sketches the data of this problem.

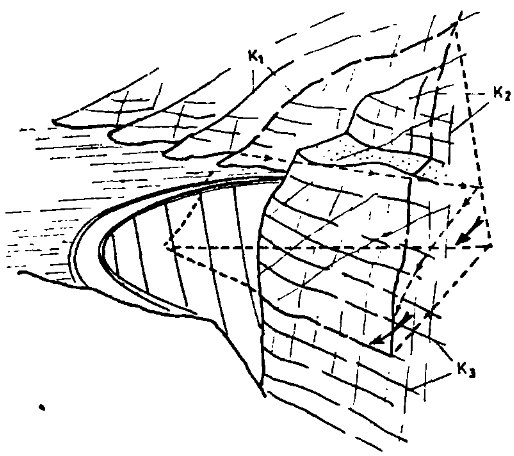


Fig. 25 - Three-dimensional problem in rock hydraulics - Flow in a dam abutment $K_{1,2,3}$: Fissure surfaces.

This case is often met with in practice (e.g. the abutments of the Vouglans Dam). The systems of vertical fissures are, for instance, joints traversing the sedimentary layers, the horizontal network being made up of bedding joints which have become secondary (hydraulically speaking) by a clay filling.

Flows in the vertical networks are no longer independent. The determination of the hydraulic potential distribution must therefore take into account the mutual action of the fissures. The problem can be solved by a numerical method taking into account this reciprocal action ; mathematically this is expressed by the continuity equation obtained by setting the two flows to be equal at the intersection of two individual fissures.

To draw up the model representing the rock mass, it is interesting to consider two cases :

1) When the vertical dimension of the domain under study is negligible as compared to the longitudinal dimension, the problem may then be reduced to one problem in two dimensions, if one takes as unknowns the mean values of the hydraulic potential ϕ_i along the vertical intersections N_i of two individual fissures K_1 and K_2. This is the case when the rock mass is isolated between two impermeable clay-banks (confined flow), or when the mass is not very high and is on an impervious sub-layer (free surface flow, figure 26). Under these conditions, the vertical component of the hydraulic gradient $\vec{J} = \vec{grad}\ \phi$ is negligible, as compared to the horizontal component.

In order to obtain the ϕ_i equations used to solve the problem, it is sufficient to state the equation of continuity for each intersection N_i of fissures K_1 and K_2 expressed by the conservation of flow. This law, known as "Law of Intersections" is given on the algebraic relationship (for the intersection N_i, figure 26) :

$$\sum_j Q_{ij} = 0 \qquad\qquad i = 1, \ldots n \qquad\qquad (19)$$

Q_{ij} is the flow rate in the individual fissure connecting two consecutive intersections N_i and N_j (figure 26). A flow will be taken to be positive if it occurs towards the intersection, negative in the opposite case. The application of this law to all the intersections of fissures K_1 and K_2 in the given domain will yield a system of $\underline{n}$ equations in Q_{ij} equal to the number of intersections, that is, to the number of unknowns ϕ_i.

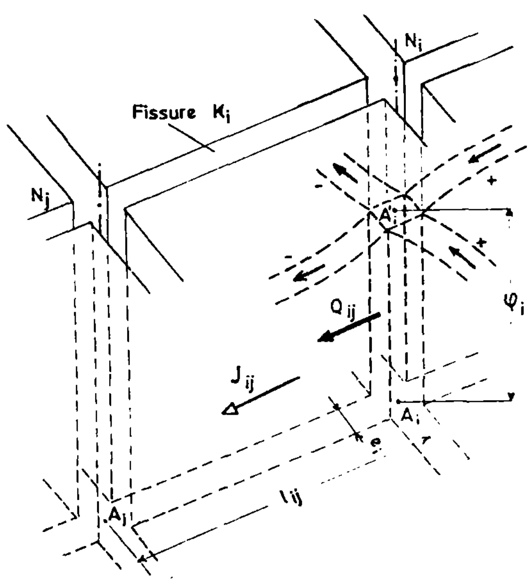


Fig. 26 - Free surface flow in the neighbourhood of the intersection of two vertical fissures.

Since the equations thus obtained are independent, the system is, in theory, solvable. It is sufficient to re-write the equations in terms of ϕ_i, the mean hydraulic potential along the intersection N_i. By taking into account the values of the directional hydraulic conductivities measured *in situ* along a length corresponding to the lattice of the model, the equation (19) becomes

$$\sum_j K_{ij} \frac{\phi_j - \phi_i}{lij} \quad A_i A_i' \; l_{ik} = 0 \tag{20}$$

which finally reduces to :

$$\sum_j K_{ij} \frac{l_{ik}}{l_{ij}} \quad (\phi_j - \phi_i) = 0 \tag{21}$$

The problem can easily be solved on a computer by various iteration methods. Once ϕ_i, the hydraulic potential at a fissure intersection, is known, the problem may be considered as solved. It should be noted that in this case, the free surface flow is known after a single computer run.

2) If the vertical dimension of the given domain is about equal to or greater than the longitudinal dimensions, it becomes necessary to take as unknowns a number of values of the hydraulic potential along the vertical intersection of two fissures K_1 and K_2, this being equivalent to quantifying the flow along the vertical dimension. The method of computation is then very much akin to the method of finite differences, with the vertical lattice network distributed in space. With respect to the previous problem, the number of unknowns is multiplied by the number of horizontal layers considered. The method of solution on a computer is however identical to the previous one.

If the phenomenon under study also exhibits a free surface, it will be necessary to perform successive iterations to determine it, knowing that the final type of flow must satisfy the equation $\phi \geqslant Z$ at all points of the flow area.

One may thus solve a case where water emerges as springs or seepage from a rock slope, for instance. The heights of seepage along the cracks must be determined with the help of an empirical law, this law having been verified by measurements on a model.

Flow phenomena in the neighbourhood of the intersection of two fissures have been studied experimentally. It was found that in the case of the problem sketched in figure 26 and when the gradients are weak (say, less than 0.5), the hydraulic potential along the intersection of two vertical fissures varied but little and could even be taken as constant.

Figure 27 clearly shows the discontinuous character of the flow in fissured media ; along the intersection of fractures (with a ratio of fracture openings equal to 2:1), there occurs an important seepage surface with a resulting ratio in the piezometric heads of 1:4.

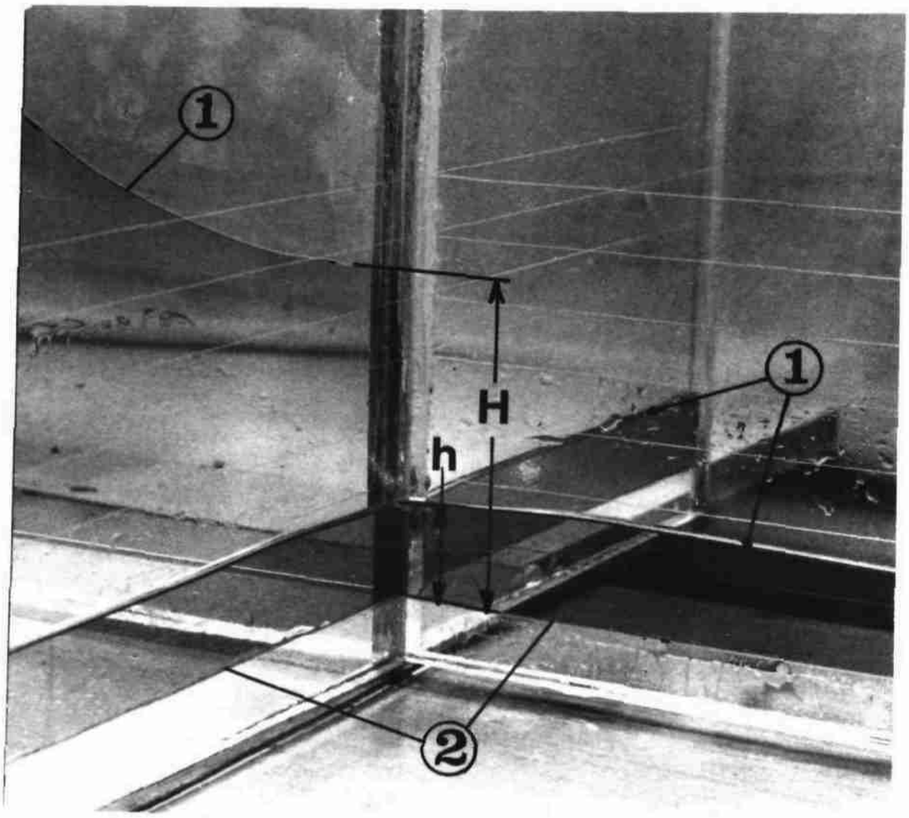


Fig. 20 - Experimental study of the flows in the neighbourhood of the intersection of two fractures
(1) Free surface
(2) Impermeable boundary
H, h : piezometric heights.

5.4 - Rock mass with three systems of conductive fractures

a) Perpendicular fractures

The solving of problems of flow within a medium containing three systems of arbitrarily-oriented fractures, is evidently more complex.

It is noteworthy that in nature the principal directions of fracturation are very often approximately orthogonal. Therefore, the hypothesis of the tri-orthogonality of fractures is usually reasonable. This is why this particular case will be studied in greater detail, after defining the elements allowing for the construction not only of a mathematical model, but also of two physical models, one electric and the other hydraulic.

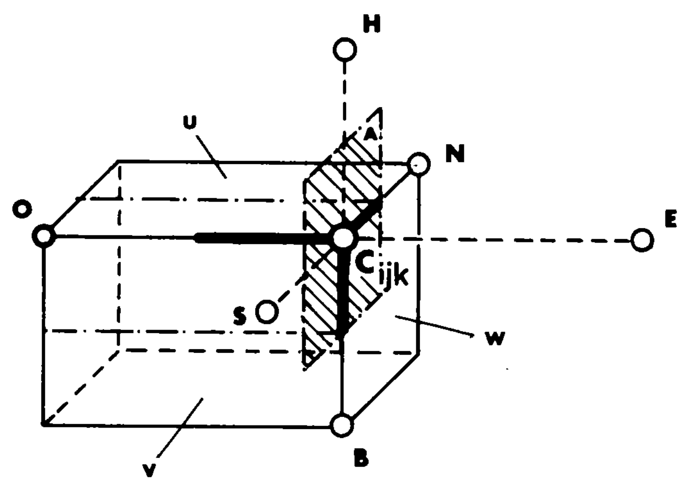


Fig. 28 - Elements of the mathematical tri-orthogonal model.

Let us first consider the numerical method. The mathematical model is built on a tri-perpendicular mesh, as shown in figure 28, which represents an elementary block of rock, and the six nodes (corners of the blocks) adjacent to the central node C, referenced by three indices i, j, k or by a single one, according to the technique of programming. As before, the three directional hydraulic conductivities $K_{u,v,w}$ of the fracture or of the system of fractures are known (see chapter 3). The equation of continuity giving the solution of the problem is still obtained by writing that the sum of flows at node C is zero, which gives

$$q_e + \sum_{O..} (K_u + K_v) M_{oc} (\phi_o - \phi_c) = 0 \qquad (22)$$

Σ for N, S, E, O, H, B nodes, and q_e stands for the external flow injected into (+) or pumped from (-) C. M_{oc} is a coefficient which only depends on the length of the edges of the mesh. In the general case, its value is :

$$M_{oc} = \frac{A}{OC} = \frac{(NC + SC)\ (HC + BC)}{4\ OC} \qquad (23)$$

In the case of cubic mesh, (of parameter a), relationship (22) reduces to :

$$q_e + \sum_{o..} a\,(K_u + K_v)\,(\phi_o - \phi_c) = 0 \qquad (24)$$

Formulated in this manner, the problem can be programmed for computer treatment with no difficulty. As in the preceding cases, the unknowns are the potentials at the nodes ; each equation such as relation (22) thus contains at most seven unknowns.

In the <u>physical models</u> (electrical or hydraulic) it is sufficient to simulate the linear relation (22) by a simple physical phenomenon such as the potential drop in an electrical resistance or the loss of head in a circular pipe in which fluid is flowing.

The <u>linear elements</u> of the three-dimensional meshed network in the electrical or the hydraulic analogue can be obtained by the equivalence among the three fundamental magnitudes (for element OC for instance) : the coefficient of the drop in piezometric head for the fractured medium $(K_u + K_v)\,M_{oc}$, the electrical conductance 1/R + the coefficient of drop in piezometric head of the circular pipe $C_{uv} \cdot M_{oc}$. These equivalences are illustrated in figure 29. The hydraulic conductivity C_{uv} of the circular pipe is obtained from POUSEUILLE's Law. By relating the flow to cross-section A (figure 21), we obtain

$$C_{uv} = \frac{\pi g\, d_{uv}^4}{128\,\nu\,A} \qquad (15)$$

A two-dimensional analogue model with 594 resistances, adjustable from 0 to 10 KΩ for 352 nodes, has been constructed at Imperial College in London by SHARP (1970) ; it is completely automatic, and the results are given through a computer as equipotential lines.

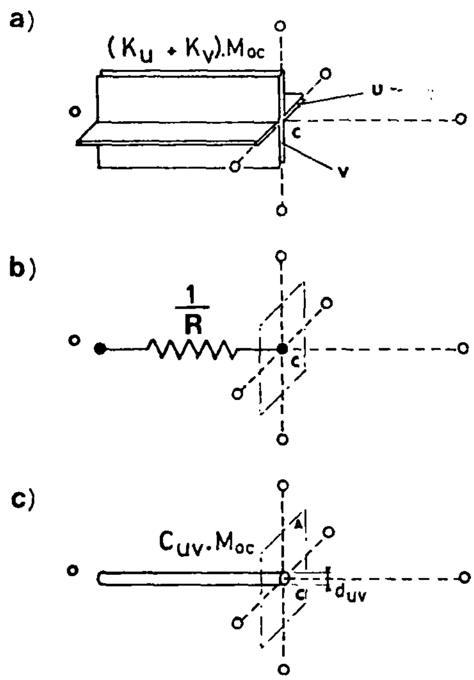


Fig. 21 - Elements of similitude for the three physical models
(a) Elementary fractures
(b) Electrical resistance
(c) Circular pipe.

An element of the meshed network for the hydraulic model is shown in figure 30. The edges of the mesh are the circular pipes and form the basic parallelipiped of the network. The nodes are made out of small perspex prisms (see detail 1, figure 20), and they hold the pipes together. These must be sufficiently rigid to enable the assembly to stand up by itself. It is therefore advisable to use hollow perspex pipes.

Each individual pipe of length l_{uw} represents hydraulically the permeability of the rock mass in the direction parallel to the two fractures U and W, i.e. in the direction of their intersection. The fundamental point in this similitude is that the <u>same ratios</u> as those occurring in nature, concerning hydraulic conductivities parallel to the intersection of the fracture planes, must be retained.

In the hydraulic model, the distribution of the hydraulic potential is determined from measurements of piezometric heights taken, for instance, at each node of the model.

In underground hydraulics, the determination of the free surface constitutes a delicate problem. It can be made fully automatic on mathematical models ; but this considerably increases computation time. On electrical analogue models it is unfortunately manual, and this constitutes a long and tedious step. This same problem is, naturally, quite easily solved on hydraulic models, since the free surface immediately becomes apparent in each case considered.

Utilizing hydraulic models therefore makes it possible to consider a great number of cases without introducing further complications. Thus, for instance, if one wanted to study the effect of a grout curtain or a drainage system, it would be sufficient to make slight modifications (such as the closing or opening of a few pipes) to see immediately the consequences of these modifications.

On the other hand, electrical models have the advantage of allowing for automatic readings of potential at the nodes through electronic systems. This only becomes possible in hydraulic models with the incorporation in the perspex prisms of pressure transducers in place of the water-column piezometric tubes. Such an electrical system would, of course, be quite expensive.

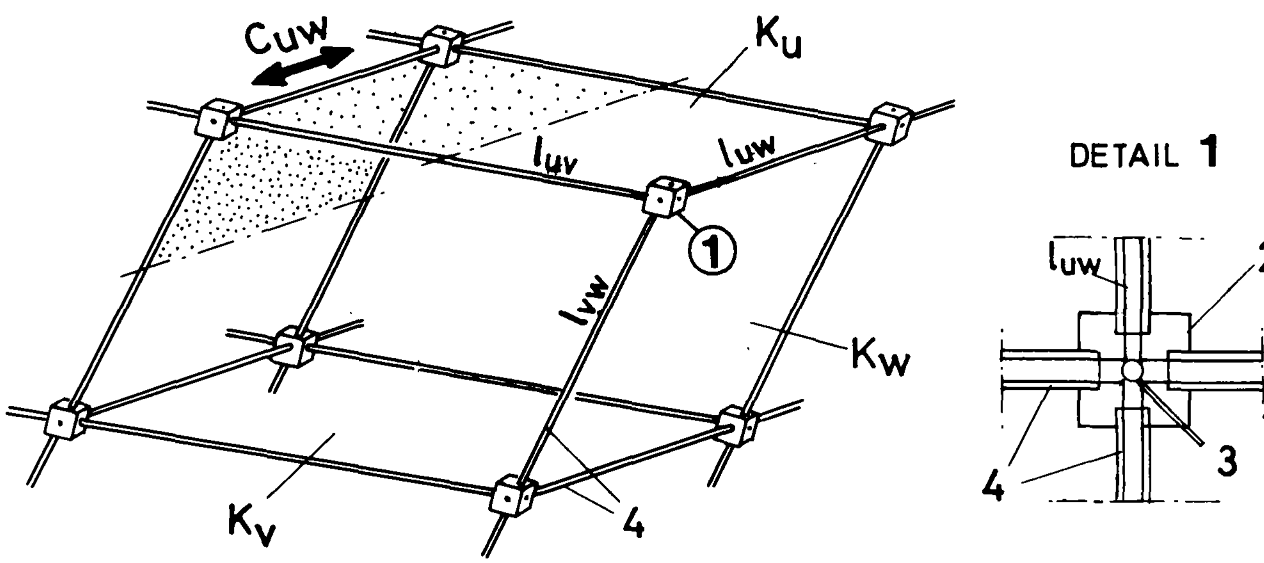


Fig. 30 - Element of hydraulic analogue model
(1) Arrangement of conduits. Detail I : Cross-section parallel to K_u
(2) Perspex prism
(3) Piezometric measurement point
(4) Perspex pipes.

b) Arbitrarily-oriented fissures

The models suggested in the preceding paragraph are made up of line elements : each fracture plane, isotropic or anisotropic, is represented hydraulically by two main permeabilites which must be orthogonal (this being due to the fact that an elementary fracture is a continuous medium in two dimensions). In three dimensions, this type of representation is only possible if the fracture in different systems form a right-angle.

In the case of media with a triple fracturation of arbitrary or random orientation, we are obliged to use models with surface elements. These models are of two types : with triangular elements or with parallelograms or rhombus-shaped elements. In these simulations the finite element technique is recommended.

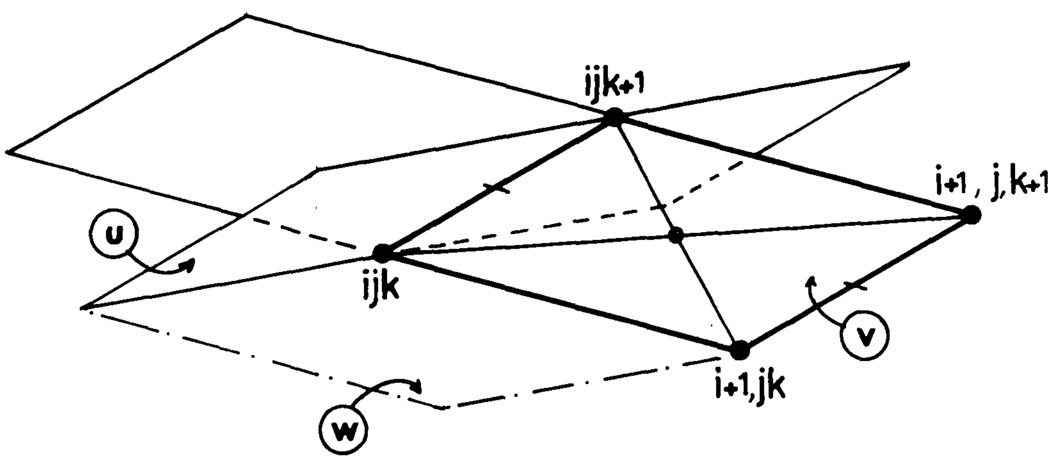


Fig. 31 - Elements of the mathematical model with elementary surfaces

Methodology is therefore relatively simple : as shown in fig. 31, one lays out, in each individual fracture in space, one or more elements (triangle, parallelogram or rhombus) and it will then be sufficient to write, as done previously, that the sum of the flows along an edge or within a mesh is algebraically zero. This type of iteration problem is easily solved with a computer.

5.5 - Rock masses with more than three fracture systems

We now consider the case of a rock mass with n systems of parallel fractures ($n>3$). The study of three-dimensional flow through such a medium is evidently quite complex. One could represent the flows by placing within each elementary fracture a certain number of triangular elements and then applying the techniques of finite elements. This method would require exact knowledge of the geometry of the fracture network, which in the case under consideration is very complex, as more than three systems of fractures are involved and as, furthermore, it would mean a large number of elements.

The method of finite elements therefore proves here to be difficult in practice. It is possible to approach the problem in a different light providing that the medium can be considered as continuous. This assumption may be valid when more than three fracture systems are present. It will then be enough to determine the permeability tensor, and thus reduce the problem to a case of a porous anisotropic medium, for the study of which there are already a great number of techniques. In any case, knowing the principal directions of permeability $K_{1,2,3}$, it will always be possible to solve the problem completely, using the methods suggested in paragraph 5.4 (a). One merely replaces $(K_u + K_v)$ in relation (12) by one of the principal permeabilities $K_{1,2,3}$.

The problem is simply one of determining the permeability tensor. One therefore considers n systems of fractures, shown stereographically in figure 32, of hydraulic conductivities K_i (K_i is measured *in situ* or is determined from relationships (3), (7) or (8)). In a system of perpendicular axes x_i, y_i, z_i associated with the system of fissures K_i (and eventually with the principal directions of permeability in the system K_i) the permeability tensor - with oz_i perpendicular to the system, can be written :

$$\bar{\bar{K}}_i = \begin{vmatrix} k_i' & 0 & 0 \\ 0 & k_i'' & 0 \\ 0 & 0 & k_m \end{vmatrix} \qquad (16)$$

k_i' and k_i'' are the principal permeabilities of the fracture system K_i and k_m is the permeability of the rock matrix. Let P (i) be the transformation matrix

$$x_i,\ y_i,\ z_i \xrightarrow{P\ (i)} X, Y, Z$$

In the system of axes X, Y, Z (figure 32), the total tensor of permeability due to the presence of n systems of fractures K_i is therefore written as (property of linear transformations) :

$$\bar{\bar{K}} = \sum_{i=1}^{n} P(i).\ \bar{\bar{K}}_i.\ P^{-1}(i) \qquad (17)$$

This method is quite general and is applicable whatever the number of fracture systems (1,2,3 ...n), on the express condition that the medium may be considered as continuous (see paragraph 4).

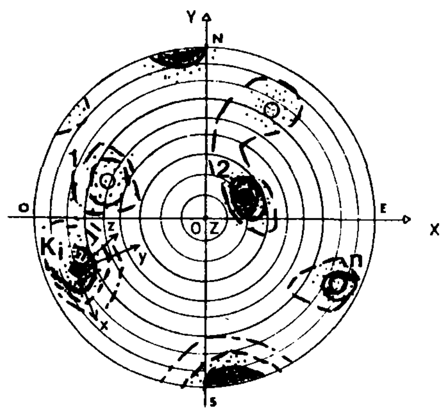


Fig. 32 - Stereographic representation of fracturation in a rock mass with n systems of fractures

6 - DRAINAGE IN FISSURED ROCK

6.1 - Introduction

Development of mining techniques, as well as progress in methods of construction in rocks, have made it imperative for engineers to concentrate on improving rock masses. Apart from rock grouting and the anchoring of rock faces, drainage constitutes one of the most efficient processes used to improve the stability of a mass of rock and to obtain rational and economical operation of mining works or of above ground or underground construction activities.

It is well known that groundwater, flowing or at rest, greatly endangers the stability of rock masses. The basic aim of drainage is to eliminate or reduce the mechanical or physico-chemical action of water flowing within the rock.

We are here touching upon a specific aspect of the problems facing the rock-hydraulics engineer, namely the drawing up of rational projects for drainage in fissured rocks. For their calculations to be correct, they must first of all take into consideration the medium and the laws governing water circulation ; a specific methodology for the solving of drainage problems must also be evolved. The factors guiding the choice of one or the other drainage systems must always take into account the mechanical aspect of the problem ; it must be kept in mind that drainage is to reduce as much as possible, the mechanical effects of water, in order to improve the rock stability. Faced with such needs, the rock-hydraulics engineer must, at the same time, be proficient in mechanics ; all this points to a very specific field of knowledge, which has recently been named "hydrogeotechnics".

Having recalled the hydraulic characteristics of rocks and analysed drainage phenomena in such media, we shall then review some general topics which need to be taken into consideration if a rational drainage network is to be evolved. The methodology suggested may well be illustrated by a concrete practical example : drainage of a slope.

6.2 - Project of a drainage network

6.2.1 - *Preliminary survey*

When planning a drainage system, it is, of course, imperative to make the usual preliminary geological survey with a detailed analysis of the structural aspects involved. A thorough topological survey of all the joints is essential to lay down a rational plan of pumping tests (chapter 3). This will help to map out hydraulic conductivities in the fields studied and thus to define the details of the model that is to represent the medium (chapter 5).

6.2.2 - *Drainage criterion*

Before undertaking drainage calculations, it is necessary to make a proper choice of the drainage criterion which will enable the engineer to determine, from both a qualitative and a quantitative point of view, the relative merits of various solutions. Each given problem has several drainage solutions. Choice of the optimal solution is essential.

A drainage criterion depends on the type of drainage problem faced. It would be different for a slope, a dam or a gallery. In order to define the criterion, one either determines a drainage area at the limit of which the resultant of all mechanical actions of the water are calculated, or one determines a line or a surface along which the pressures will have to be minimal.

In the particular case of a slope, the area to be drained with a minimum number of drains corresponds to the critical sliding area. That area is generally unknown. To simplify the problem, the area to be drained has been bounded, inside the slope, by two segments (fig. 33) so as to make the limit thus defined coincide, as nearly as possible, with the critical sliding curves obtained by the usual classical methods of soil mechanics (fig. 34). Whatever the angle of the slope, the surface of the drainage area thus defined will be constant :

$$A = \frac{3\ H^2}{4} \ .$$

For a given drainage system and given flow conditions, the resultant of all forces due to the water flow along the drainage area limit will thus be calculated as well as the dimensionless coefficient f ($f = \frac{\gamma w A}{F}$), characteristic of the action of ground water, alternatively the efficiency 1/f of the drainage network. It is this coefficient f, or its inverse, which will be taken as a drainage criterion. For perfect drainage, the coefficient of water action will be nil and the drainage efficiency infinite. In fact, f, or its inverse, will be a vector ; it will be related to the direction of the resultant $\vec{F}$. In choosing the best solution, it may be interesting to take into consideration not only the magnitude of this vector, but also its direction ; in problems concerning stability, the direction of the applied force may indeed be of the utmost importance in the stability of the system.

6.2.3 - Theoretical calculations

It will be useful first to determine the flow network without drainage and then to work out the water action coefficient, f_o. Then one can study, for the particular drainage system considered, the different flow networks by determining the potential grid and each coefficient $f_{1,2,\,..n}$ to go with each solution. It will thus be easy, through the f coefficient, to find the optimal solution.

Generally, for a given problem, it is best to begin by determining the optimal direction θ_{opt} of the drainage system for a given length chosen beforehand and then by the same procedure to find the optimal length L_{opt} of the drains that corresponds to the fixed θ_{opt} direction of drains.

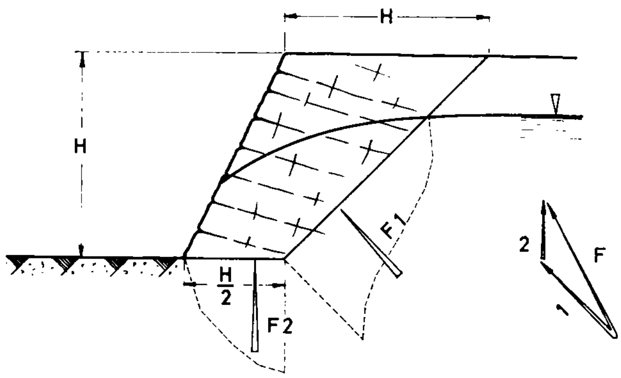


Fig. 33 - Drainage criterion in the case of a slope

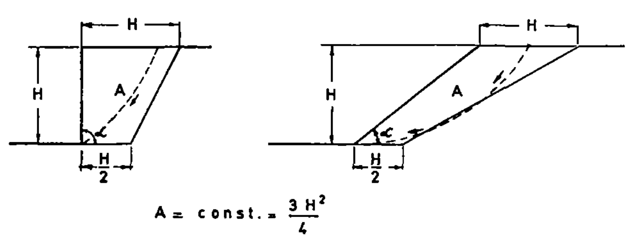


Fig. 34 - Sliding curve and drainage criterion for different slope angles

A study of the curves f (θ, L) first plotted with θ variable and L fixed, then with $\theta = \theta_{opt}$ and L_{opt} as shown in fig. 35 only qualitatively illustrates the case in point. Strictly speaking, a systematic study of the function $f(\theta,L)$ of the two independent variables would be needed in all the two dimensional space θ,L within the fixed limits of the problem at hand.

In most of the actual cases, drainage networks are made with parallel drillings, 5 to 10 cm in diameter. There is no criterion to help determine the distance d between drains ; it is generally set between 2 and 10 metres.

The actual flow in a drained slope is three-dimensional, hence extremely intricate to study. Two-dimensional studies therefore give only approximate solutions. In a cross-section, the drainage is only complete in the planes of the drains. Between drains, the drainage is limited. In consequence, the real depth of drains will have to be set at a value L greater than the theoretical L_{opt} value computed (fig. 36) L_{opt} is the effective length of drains whose real length is L. L_{opt} is also the depth of the continuous drainage trench equivalent to the parallel drain network. One may thus assume that the relationship between the actuel length of the cylindrical drains and the width of the area drained - corresponding to L_{opt} in the theoretical calculations - is of the form :

$$L = \alpha . L_{opt},$$

where the coefficient α is a function of distance d between the drains (for practical purposes, one selects $\alpha = 1.5$).

Strictly speaking, a study of the flow network in the plane of the drains would be necessary if one were to determine the true value of coefficient $\alpha(d)$.

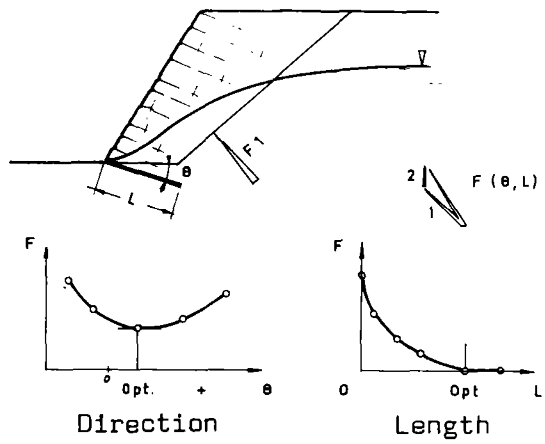


Fig. 35 - Theoretical efficiency of a drainage system

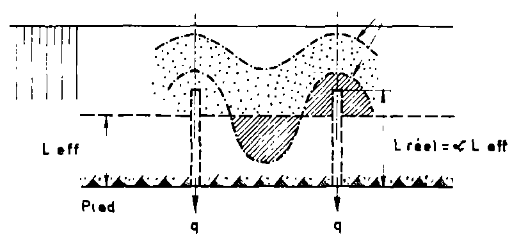


Fig. 36 - Effective length of a parallel drainage network (plan view)

6.3 - Drainage of a slope

6.3.1 - *Data of the problem*

Slope drainage problems are generally solved in practice by one of the three following techniques (fig. 25) :

a) By drainage from the foot of the slope with a network of parallel cylindrical drains.

b) By natural drainage using a drainage gallery parallel to the surface of the slope.

c) By pumping from wells drilled from the top of the slope.

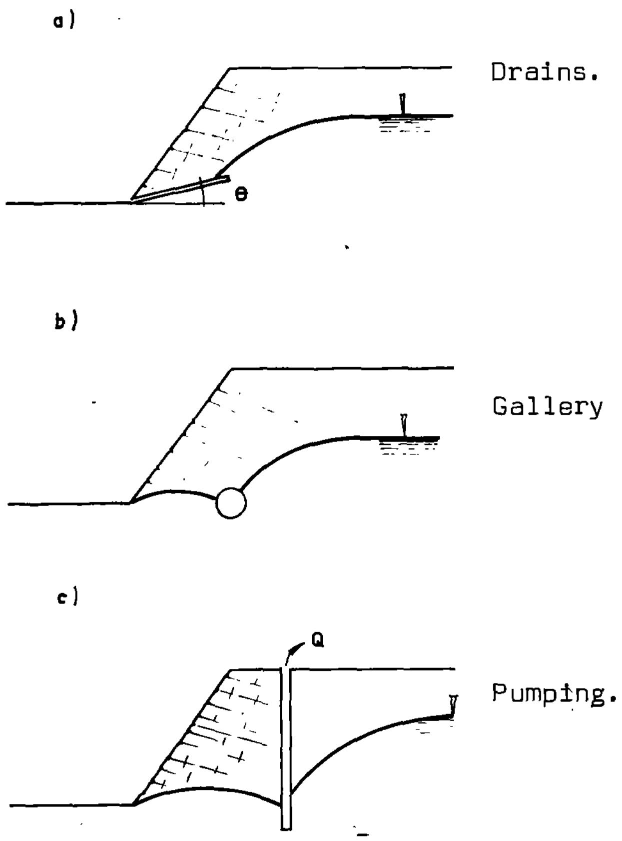


Fig. 37 - Various possible ways for slope drainage

Solution (a) is certainly the most economical. It is thus most frequently adopted. Moreover, it is particularly suitable for a low slope (roughly ten metres high).

Solution (b), which is more costly, is on the other hand more efficient than solution (a) owing to the fact that the drainage gallery is parallel to the surface of the slope. Moreover, the actual width of the area drained is known. This solution can be justified only in the case of a large slope (with a height exceeding 100 m), for instance, in open-cast mines or dam abutments or some slopes. That particular type of drainage has been analysed by SHARP (1970) within the same study program as the present chapter.

Solution (c) must be adopted only where natural drainage (i.e. by gravity) is not possible, that is, when the foot of the slope is on a level lower than that of the natural surrounding ground. These conditions are frequently met in open-cast mining. The techniques used in such cases are very similar to those connected with ground water lowering. This solution is, of course, very costly, first of all because of the length and diameter of the drillings (for a depth exceeding 6 to 7 metres, pumps must be placed in the wells which calls for large diameters), and furthermore because of the energy used in pumping.

Whatever the solution adopted, the basic principle in the study of a drainage system remains the same. In each case, the best direction and area covered by this drainage system must be determined from the different flow nets using the same drainage criterion.

In this report, only solution (a) mentioned above has been taken into account in the case of a slope forming a 75° angle with the horizontal. The geometric and hydraulic data of the examples studied are shown in fig. 38. Boundary conditions are determined by the slope itself, by the surface with a constant potential $H_w = 1.5H$ situated inside the mass, at a distance 3H from the foot of the slope, and finally by two impermeable surfaces or flow-lines for the lowest and highest boundaries in the domain under consideration. Directional hydraulic conductivities vary from case to case as regards both direction and relative value. In practice, the direction of the elementary hydraulic conductivities and the degree of anisotropy - defined as being the ratio of hydraulic conductivities - are to be measured *in situ*. Recent publications give all necessary theoretical and practical details to carry out these measurements (LOUIS and MAINI, 1970 ; LOUIS, 1970).

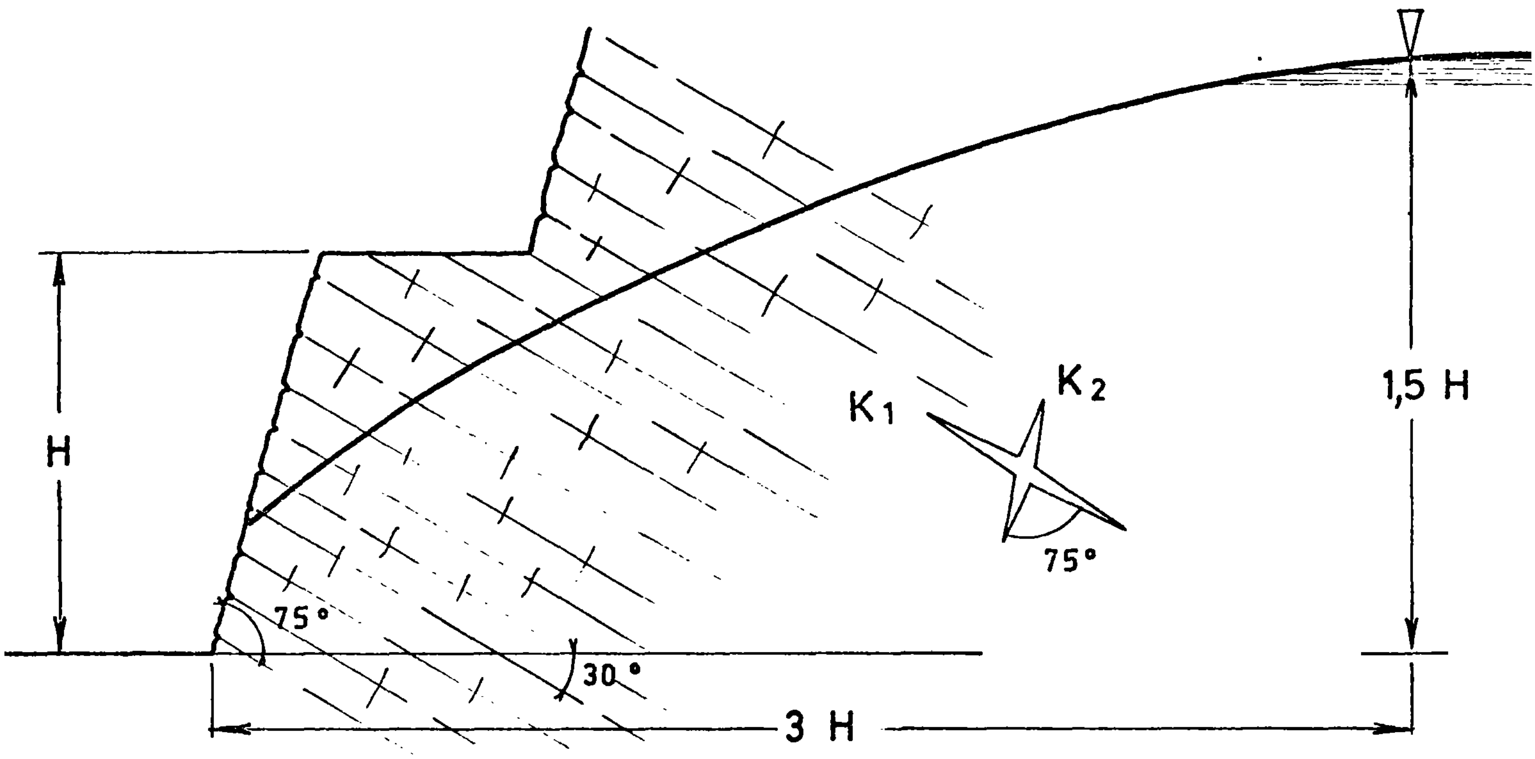
In each case considered, boundary conditions are identical (main hydraulic gradient 0.5) ; only the hydraulic characteristics of the medium vary (such as joint direction and degree of anisotropy). In this study, two main fissure directions K_1 (of maximum hydraulic conductivity), have been taken into account : in the first case, fissures K_1 have a 30° angle of dip upstream ; in the second, the dip of K_1 is 15° downstream. The degrees of anisotropy were taken to be equal to 2 and infinity. So as not to complicate this report, only results concerning one example will be explained in detail below.

6.3.2 - Theoretical results

The example chosen to illustrate the methodology detailed in paragraph 6.2 is outlined in figure 38a. The slope angle is 75°, the system of main fractures has a 30° dip upstream, while the secondary fissures are parallel to the slope surface. Results given in this report concern a degree of hydraulic anisotropy equal to 2. In accordance with the preceding discussion, the ideal direction of the drainage system has been determined first, drain lengths being dealt with subsequently.

a) Optimal direction of drainage network

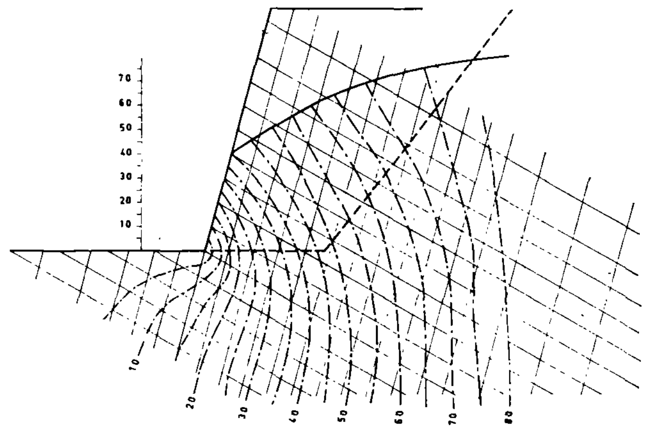
Flow nets (equipotential lines, free surface) have been worked out by the now well-known methods of hydraulics of jointed media (LOUIS, 1970 ; SHARP, 1970, etc.), first in the undrained slope and for draining directions at -23°, +30° and +68°, for a fixed drain-length at 0.45H. With results concerning the drainless slope with -75° and +180° orientations, the first characteristics curve is given by 5 points (fig. 35a). Flow nets for each case considered are shown in figure 39, while figure 40 gives the efficiency curve of the corresponding drainage. From the general behaviour of the curve, it is convenient to determine within a few degrees the value of θ_{opt} at which curve f (θ) is minimum. In the present case, the value θ_{opt} = 15° was found.



$$\frac{K_1}{K_2} = 2$$

Fig. 38 - Geometrical and hydraulic data of the examplesconsidered

WITHOUT DRAINAGE



WITH DRAINAGE

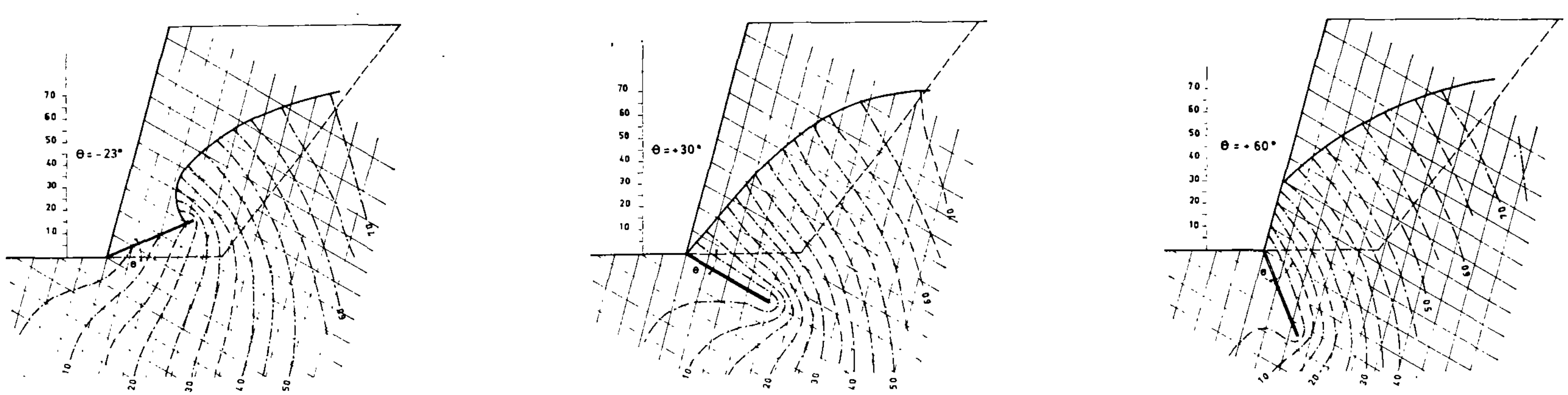


Fig. 39 - Flow nets in a slope with different drain orientations (case of fig. 38a)

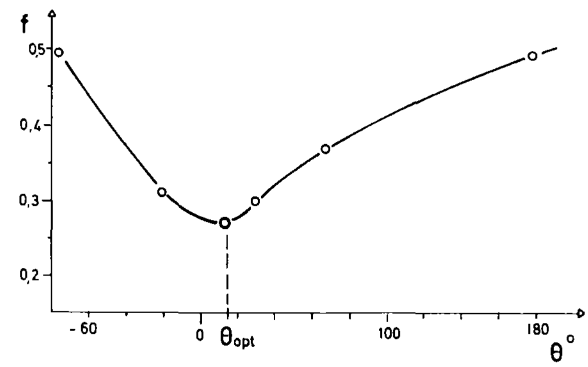


Fig. 40 - Characteristic efficiency curve of drainage (corresponding to fig. 35)

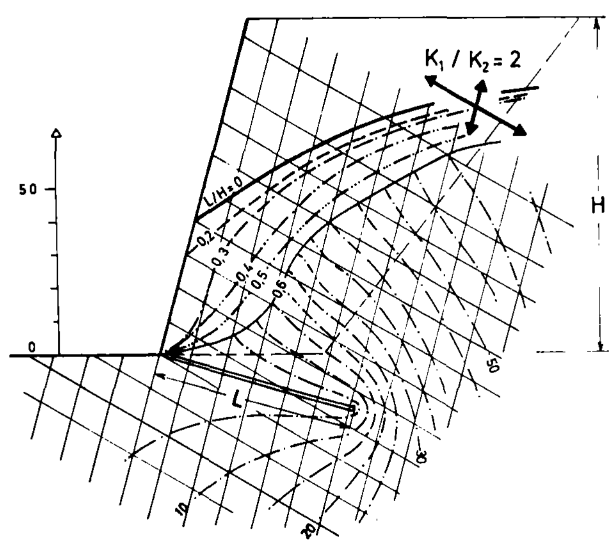


Fig. 41 - Flow networks for various drain lengths

b) Optimal length of drain

Following the same technique, flow nets have been studied for a fixed drain direction ($\theta = \theta_{opt} = 15°$) and various drain lengths, L = 0.20H ; 0.30H ; 0.40H ; 0.50H ; 0.60H. Results have been grouped in figure 41. The draining curve f (L), shown in figure 42, has been worked out from 6 points, knowing that it is tangent to the straight line f = 0. A close examination of this curve makes it possible to conclude that the optimal drain length is situated in this specific example considered, between 0.5 and 0.6H.

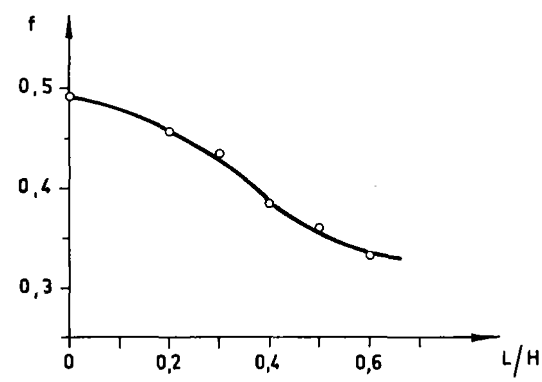


Fig. 42 - A characteristic drainage curve for cases outlined in figure 41

6.4 - Conclusion

Planning a drainage network proves in practice to be a complex problem. It is imperative, in order to solve it - and this essential for safety - to carry out a systematic hydrogeotechnical study of the operations, namely to ensure the stability of a slope by eliminating the disastrous effects of ground water.

Very often, analyses of stability and mechanical problems are carefully dealt with while the working out of network drainage characteristics are not studied in detail. It is advisable to harmonize the relative importance given to each phase of study in the framework of an overall program bringing in geology, mechanics and hydraulics simultaneously.

Practical experience shows that drainage networks are very often ineffectual. Their efficiency could be much increased and their cost reduced simply by studying drainability characteristics and by taking into account the structure of the medium (lithology, fissuring, discontinuity), simple hydraulic conductivities and the geometry of the medium.

Much progress still remains to be made in the fields of drainage. Theoretically, actual flows are often three-dimensional and transient, and their study therefore very intricate. Moreover, the geological aspects of the problem are important : certain masses of rock, where flow proceeds through small channels, for instance, are very difficult to drain by commonly used techniques.

It would appear that the use of explosives in drains could solve certain problems by efficiently increasing the range of action. Many aspects could lead to extremely interesting research projects.

7 - PRACTICAL EXAMPLES

7.1 - Water flow in dam abutments

We shall endeavour to determine the flow net (distribution of the hydraulic potential, free surface and seepage surface) in a fissured rock mass constituting the foundation of a dam abutment (the problem is sketched out in figure 25). The rock mass is taken to be intersected by two sets of fractures K_1 and K_2 (joints) which break up the sedimentary strata. The stratification boundaries K_3 are taken to be of secondary hydraulic importance, since the presence of filling impedes the free circulation of water. The grout curtain is situated in the plane of the upstream facing of the dam (fig. 43).

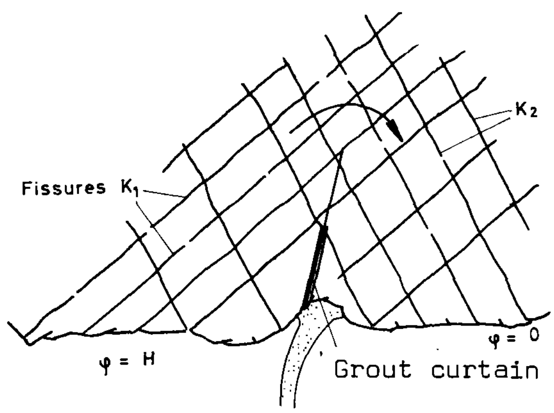


Fig. 43 - Horizontal cross-section of the abutment of the dam shown in fig. 25

The results of the computation are grouped on figure 44 which represents a perspective view of the dam. This representation makes it possible to give an overall view of the free surface of the fractures of the rock mass.

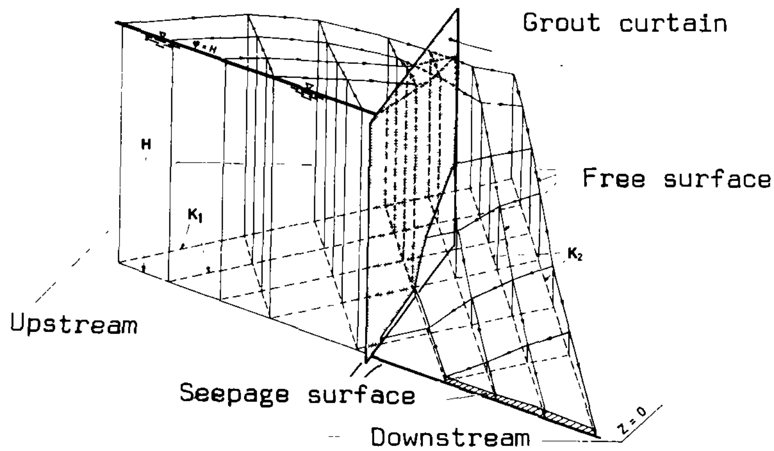


Fig. 44 - Flow network in the dam abutment - case of fig. 43

7.2 - Flow conditions at the Grand Maison damsite

7.2.1 - Introduction

As part of the study plan for the Grand Maison dam in the French Alps, the "Electricité de France" offered the opportunity for undertaking a highly developed analysis of flow phenomena within the foundation mass of the damsite. This analysis, aimed at understanding the hydromechanical behaviour of the dam and its abutments, required both by the foundation force exerted by the arch and the various water actions within the rock mass. A study of the dam's stability formed the last stage of this complex programme which included geological, structural, hydraulic and mechanical studies.

This chapter only deals with the hydraulic treatment which from the start relies greatly on the structural data of the site. The complexity of the water flow phenoma within the fractured rock required the use of a new, and indeed original, methodology, as much for the theoretical approach in the treatment of structural data, as for the testing techniques both in the laboratory and *in situ*.

7.2.2 - *The damsite and the construction*

The damsite is situated in the Eau d'Olle Valley between the crystalline massives of Grandes Rousses and Belledonne, north-east of Grenoble (France). The dam, an arch about 200 m high will rest on a rock mass composed of very compact, fractured amphibolites and gneiss. The transition between the ribbon gneiss and the amphibolites is progressive and does not constitute any hydraulic discontinuity. The schistosity is marked but closed. The possibility of water circulation is determined essentially by the fractures.

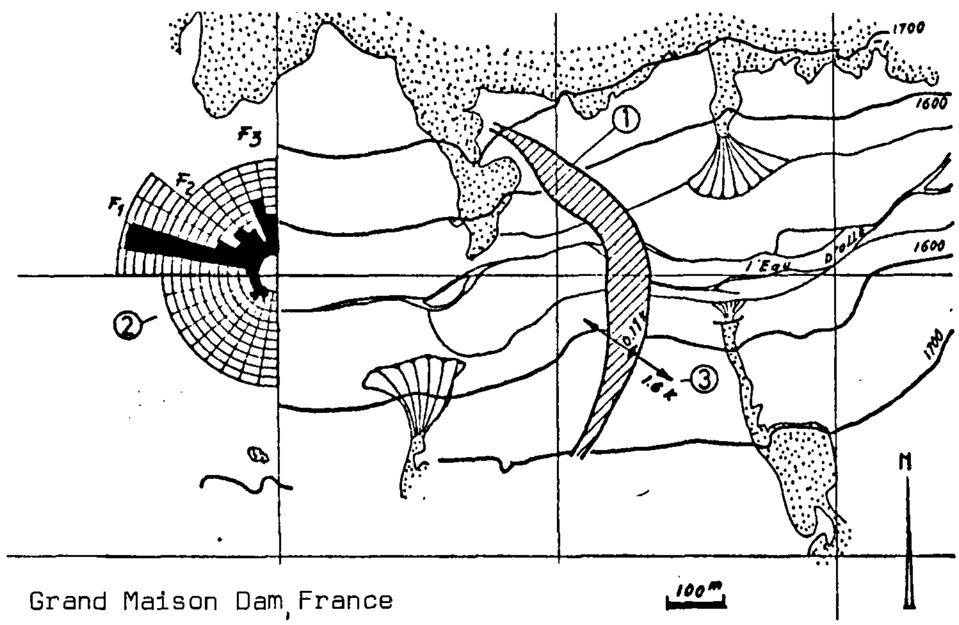
Such a massive, as indeed do most rock masses, constitutes a very complex geometrical medium as a result of its fracturation. The fractures, irrespective of their origin, play a preponderant role in the hydraulic system. The medium, which has a negligible permeability of matrix compared with that of the fractures, is characterised by discontinuities, distributed in an anisotropic and heterogenous manner. It is for this reason that the structural study of the site has a particular importance.

The dam which will rise to the 1700 m contour will have its foundations in unweathered rock, with their base at level 1490. The valley bed is covered by about 50 m of recent alluvia and scree which have a strong permeability compared with that of the bed-rock. Because of this contrast in permeability, the hydraulic study has been carried out without taking into consideration the presence of the alluvia and lateral scree found on the valley slopes.

7.2.3 - *Structural study*

The whole hydrogeotechnical study of the Grand Maison damsite is essentially based on the structural characteristics of the environment. Thus, the hypothesis concerning the hydraulic parameters of the foundation mass of the dam, the nature of the mathematical model used to simulate the water flows and, finally, the kind of hydraulic tests carried out *in situ*, depend very greatly on the interpretation of the surveys of fracturation made on the site.

The preliminary hydraulic study (LOUIS 1970) was carried out from existing structural and geological data. During these reconnaissance compaigns, 7,500 structural elements were noted on the site. From these preliminary studies it was possible to characterize the structure by five families of fractures, two of which may be considered as secondary from the hydraulic point of view. The principal families, F1,2,3 are all subvertical, their direction remaining within an angle of 60°-70° of the bisector plane, approximately parallel to the dam on the right abutment (fig. 45).



Grand Maison Dam, France

outcrops
Gneiss, Amphibolites

scree, alluvia

Fig. 45 - Relative arrangement and principal structural elements
(1) Dam
(2) Fracturation direction (subvertical)
(3) Horizontal anisotropy of the permeability

Certain conclusions may be drawn from this first study :

1) There is a very marked anisotropy of the hydraulic conductivity within the horizontal plane, the strong conductivity being parallel to the right abutment.

2) The vertical hydraulic conductivity, parallel to all the fractures will be the highest.

A very clear dissymetry in the hydraulic behaviour of the two abutments could therefore be foreseen.

Because of the importance of the conclusions arrived at, it seemed necessary to verify the earlier results on the fracturation of the rock mass. This work was carried out by a new survey of the fractures within the zones directly affected by the water flow under the dam. The object of this exercice was very precise - to be able to describe, in the most objective way possible, the potential routes of water flow within the rock mass. It included two distinct phases : (see chapter 3.1).

a) A systematic survey in the left and right abutments and the sub-fluvial reconnaissance galleries and also in the excavations open at that time (fig. 46) on the abutments, using a very detailed check list, which made it possible to describe each structural element observed in the most precise manner possible.

b) A statistical analysis of field data with a possible "hydraulic weighting", designed to reveal the hydraulic characteristics, representative of the site. To facilitate this task and to be able to carry out very varied statistical treatments, the calculations were made by computor.

From this complementary study it was possible to define the homogenity of the structure of the site and also to analyse the validity of the hypothesis arising from the theoretical treatment of the hydraulic problem (hierarchy between the fracture families, characteristics of the anisotropic tensor of the permeability, mathematical model, water tests to be foreseen etc.).

In general, the weighting factors clarify the diagrams by discounting the secondary families which are sometimes numerically rich but not very important from a hydraulic point of view. Five families appear with the traditional treatment, whereas only three principal families (F1,2,3) are seen on the stereonets by using weighting factors.

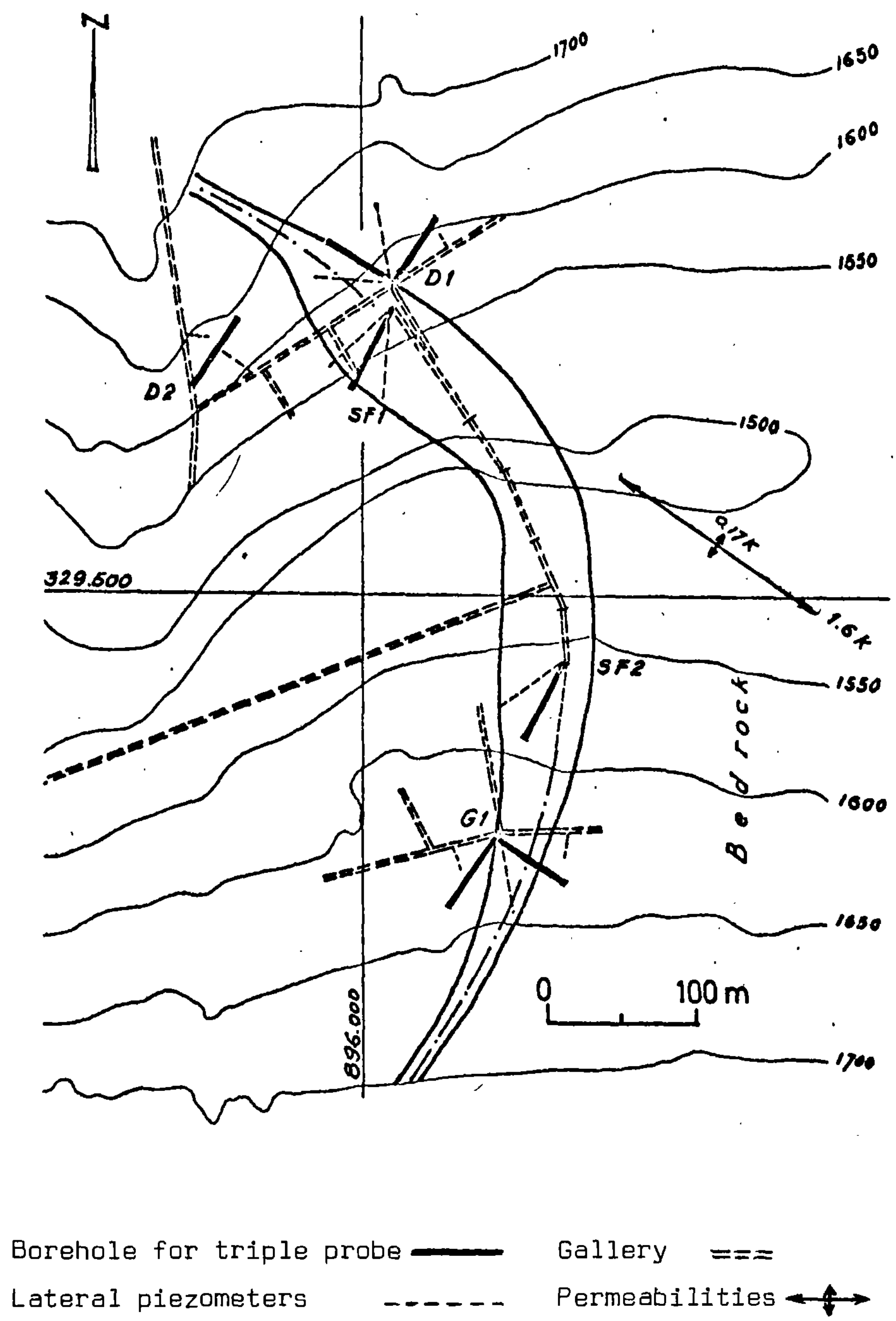


Fig. 46 - Lay-out of reconnaissance means : Galleries and boreholes

Following this structural approach to the problem, a test of the anisotropic permeability tensor was undertaken by introducing various, absolute and relative hydraulic conductivities of the three principal fracture systems. In this way the most favorable orientations for the hydraulic tests to be carried out *in situ* could be chosen in order to determine the quantitative character of the anisotropy of the site.

7.2.4 - *Natural flow conditions*

Before beginning the very elaborate study programme of the damsite, it seemed desirable to first make a piezometric study of the natural flow conditions. The aim of this exercise was on the one hand, to verify whether or not the rock mass had a homogenous distribution of piezometric heads in the sub-fluvial zone beneath the aquifer, and, on the other hand, to obtain all the necessary elements for making a calibration of a mathematical model in a later phase. A simulation of flow conditons around the sub-fluvial gallery can in fact be achieved, given that all the elements, (conditions at the bounderies, permeability and piezometry distribution, flow rates within the galleries) are known. Figure 47 gives a transversal, vertical cross-section showing the situation of the principal elements : the valley bed (1) at level 1550 m, the roof of the bed-rock (2) the sub-fluvial gallery (3) and the boreholes drilled from seven lateral excavations in the gallery.

The flow network between the valley bed with constant piezometric heads 1546 and the gallery at level 1485 m was analysed by the original technique presented in chapter 3.2.5., giving a continuous ponctual piezometry in the boreholes.

The hydraulic examination of the site provided the following information (fig. 47) :

- the fractured rock mass has a good hydraulic homogenity ;
- important fractures may be discovered by hydraulic means ;
- decompression is more marked on the right abutment than on the left ;
- decompression is very little marked in the valley bed. It reaches no further than 5 m beneath the roof of the bed-rock.

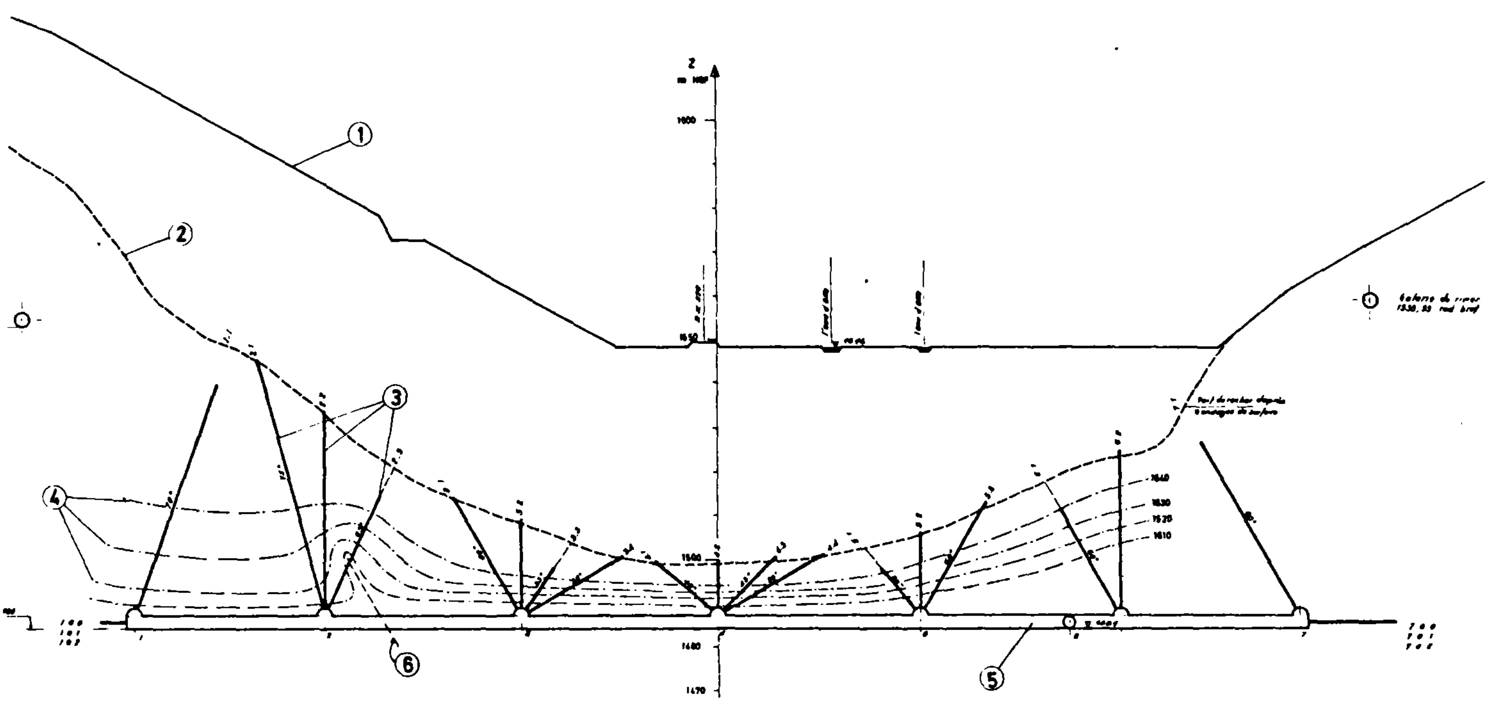


Fig. 47 - Piezometric campaign in the sub-fluvial gallery - Transversal vertical cross-section

(1) Valley section (scree-alluvia)
(2) Roof of the bed-rock
(3) Boreholes
(4) Sub-fluvial gallery
(5) Potential lines
(6) Important fracture

7.2.5 - Hydraulic characteristics of the site

a) Directional hydraulic conductivities

According to the conclusions of the initial geological study, later confirmed by the structural analysis, it seemed that the foundation mass of the Grand Maison dam is cut by three principal families of sub-vertical fractures (F_1 - N110E, F_2-N135E, F_3-N170E) which are not perpendicular (fig. 45). The definition of the hydraulic characteristics of the site and the simulation of the three dimensional flows could therefore only be made by two means (LOUIS 1970) :

a) by a "Surface Elements" model coinciding with the fracturation planes ; simulation being achieved by the finite elements method (with plane elements distributed in space) ;

b) by a "Line Elements" model, using the concept of an equivalent medium to that of the real medium, solving the problem by the finite differences method.

For the present phase of investigation it was the latter technique that was chosen. It implies several hypotheses, in particular :

- to use the finite differences method we must admit that the principal directions of the permeability of the medium are constant throughout the zone under consideration. This supposes that at every point in the medium, the relationships of the directional hydraulic conductivities of each fracture family remain constant.

- the use of tensorial writing must be admitted for the determination of the principal permeability directions which are characteristics of the equivalent medium to the real medium. The "tensor of permeability" concept is only applicable to continuous media. This mathematical tool has only been used in the preliminary phase to define the characteristics of the line elements or the principal permeabilities of the equivalent medium. The simulation was then achieved with an essential discontinuous "Line Elements Model".

The total permeability tensor $\bar{\bar{K}}$ (X,Y,Z), a function of the point M (X,Y,Z) is easily determined from the elementary hydraulic conductivities K, aK, bK, of the fracture families F_1, F_2 and F_3 and from the permeability of the rocky matrix k. In a system of axes x_1, y_1, z, linked to the fracture family F_1, the elementary tensor of permeability is written :

$$\bar{\bar{K}}_1 = \begin{vmatrix} K & 0 & 0 \\ 0 & k & 0 \\ 0 & 0 & K \end{vmatrix} \quad \text{oz vertical.}$$

The same is true for fracture families F_2 and F_3 by replacing K with aK and bK.

By expressing these elements in the same system of axes X,Y,Z (by changing the axes of the matrice P(i)) the total permeability tensor is then :

$$K(X,Y,Z) = \sum_{1=1}^{3} P(i)\ Ki\ P(i)^{-1}$$

It is written in the form :

$$\bar{\bar{K}}\ (X,Y,Z) = K\ (\sigma,t)\bar{\bar{F}}\ (a,b,\theta\ i)$$

The total permeability tensor $\bar{\bar{K}}$ (X,Y,Z) is thus expressed by a tensor F which fixes the principal permeability directions (linked to the relationships of hydraulic conductivities of each fracture family and to the structure of the site). The coefficient K(σ,t) fixes, on the contrary, the absolute rate of permeability, which depends on the position of the point considered (by the intermediary of the depth of the overburdening) and the state of stress σ due to the dam.

In the example, $\bar{\bar{F}}$ (a,b,θi) is constant for the whole model, whilst the absolute rate of permeability was taken in the form k = ko exp (-At), a formula chosen after *in situ* and laboratory tests (see next paragraph). The principal permeabilities were therefore the following (fig. 45 and 46) :

Horizontal directions K_1 = 1.6K N 124° E
K_2 = 0.17K N 34° E
Vertical direction K_2 = 1.75K.

b) Influence of the state of stress

The hydraulic characteristics of the fractured media are very sensitive to the state of stress imposed by the very weight of the rock mass or by exterior forces (tectonic or mechanical). It was an indispensable part of the study of the Grand Maison damsite to evaluate this influence, in order not only to determine the variation of the permeability according to the depth but also to be able to estimate the effects of the pressure exercised by the arch dam at a later date.

The influence of stress was introduced into the formula for the tensor of permeability by the intermediary of the absolute rate of permeability $K(\sigma,t)$ (see preceding paragraph). The object of the exercise was therefore to find out the law governing the variation of K according to a normal stress applied.

In the theoretical expression of the hydraulic conductivity of a family of parallel fractures, three terms vary simultaneously according to the state of stress ; the free opening e, the relative roughness k/Dn and the degree of separation of the fracture K.

For a family of continuous fractures, the hydraulic conductivity is, in fact, expressed by (LOUIS 1970) :

$$K_F = B \frac{Ke^3}{C}$$

with B the independant term of σ

$$C = 1 + 8.8 \; (K/D_n^{\;1.5})$$

When the factures are closed by a stress σ, K decreases from 1 to 0, e tends towards 0 and C increases from 1 to about 2. The laws for the variation of these parameters are unknown, they depend on the mechanical behaviour of each type of fracture. Only an experimental approach seems to be realistic.

For the Grand Mason site the analysis of the phenomenon was carried out *in situ* and in the laboratory on three different scales.

- *In situ* water tests

The influence of stress was analysed after a statistical treatment of the permeability tests made in the boreholes at varying depths in the homogenous fractured formations. The examination of the very numerous results showed that the empirical law that best translated the phenomenon was written :

$$K = K_o \, \text{Exp} \, (-\alpha\sigma)$$

with $\sigma \simeq \gamma \; t$

K_o being the initial permeability (on the surface) γt the weight of the covering (t the depth). Figure 48 shows this law of variation of the permeability according to the depth for a borehole situated on the Grand Maison site. As we shall see, the coefficient $A = \alpha\gamma$ is variable according to the zone (right and left abutments, valley bed) over the whole site.

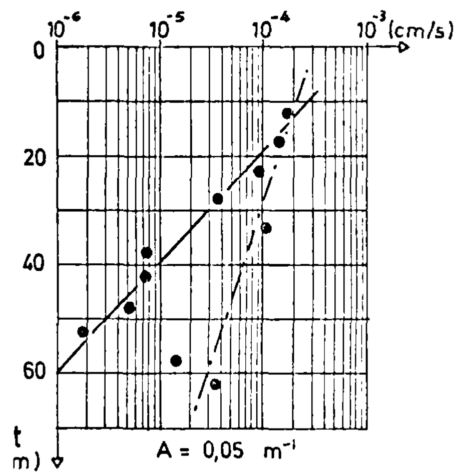


Fig. 48 - The law of variation of the permeability according to the depth (Log Ko - Log K = At)

<u>Laboratory Tests</u>

Permeability tests under stress were carried out in the laboratory on two different scales (see fig. 18) :

- on a 30 x 25 cm fracture,
- on rock cores (ϕ4 cm) parallel to the discontinuities.

Some very varied samples were analysed in this way (LOUIS and RICOME 1974, LOUIS 1974) ranging from those with a very marked fracturation to those which were compact and homogenous.

The laboratory results showed that the exponential type of law obtained from the statistical treatment of the results of the *in situ* permeability tests, translated relatively well the phenomena of permeability variation according to the state of stress, at least within the stress variation zone, having an important role in the vicinity of a construction such as a dam. A detailed analysis of the results is given in the publications mentioned in the bibliography.

Taking into consideration the tests carried out *in situ* in the laboratory, it has been possible to draw up, by cross-sections, a map of the variations of the absolute rate of permeability within the rock mass (fig. 49). Also on this cross-section we see the variations of coefficient A which translates the sensitivity of the medium to a stress variation, (A varies from 7.8 to 3.4 10^{-3} m^{-1}). This distribution is only provisional and was chosen for the preliminary phase of the simulation. It will be refined and improved, taking into consideration the new hydraulic data available (e.g. after the very deep boreholes on the abutments and in the valley bed). Knowledge of the law of variation of the permeability rate will also make it possible to take into consideration the influence of the pressure of the dam on this distribution.

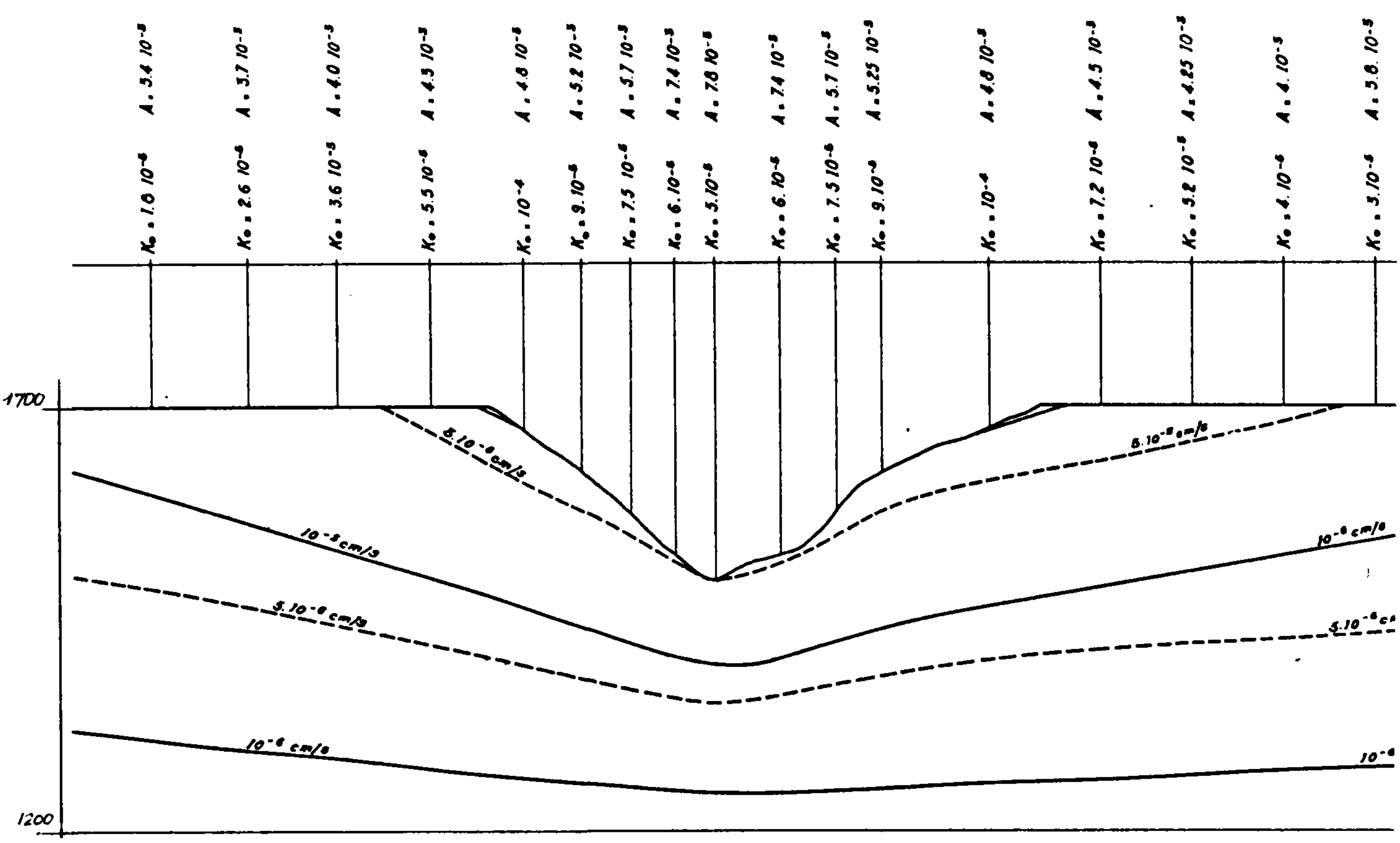


Fig. 49 – Law of the distribution of the absolute rate of permeability in natural conditions. Transversal, vertical cross-section $K = Ko\ Exp\ (-At)$, K in cm/s, A in m^{-1}

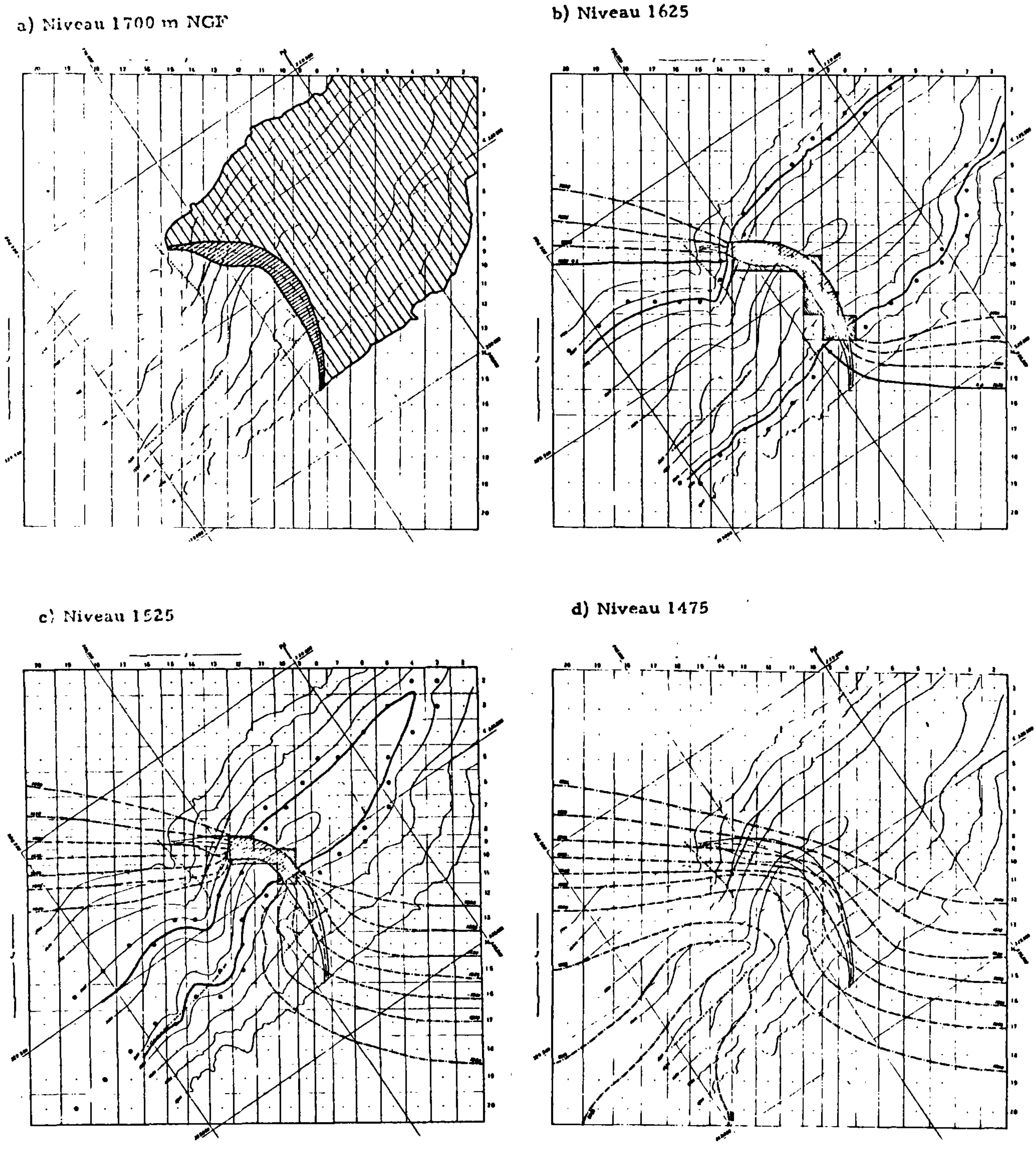


Fig. 50 - Results of simulation by tri-dimensional mathematical model
Horizontal cross-sections

7.2.6 - *Flow simulation*

a) The mathematical model

As stated earlier in paragraph 7.2.5., the simulation was carried out by a tri-dimensional, "Line Elements", mathematical model, using the finite differences method. The flows simulated are of free surface and permanent conditions. The mesh of the model is variable, the permeability is anisotropic, the principal directions of permeability and the degrees of anisotropy are fixed, whilst the rates vary according to the distribution of figure 49.

The mesh chosen for the first phase of calculation is represented by figure 50 which gives four horizontal cross-sections at different levels. The model includes ten horizontal layers, each of them comprising 21 x 21 elements of variable edges. The total number of elements is therefore 4410.

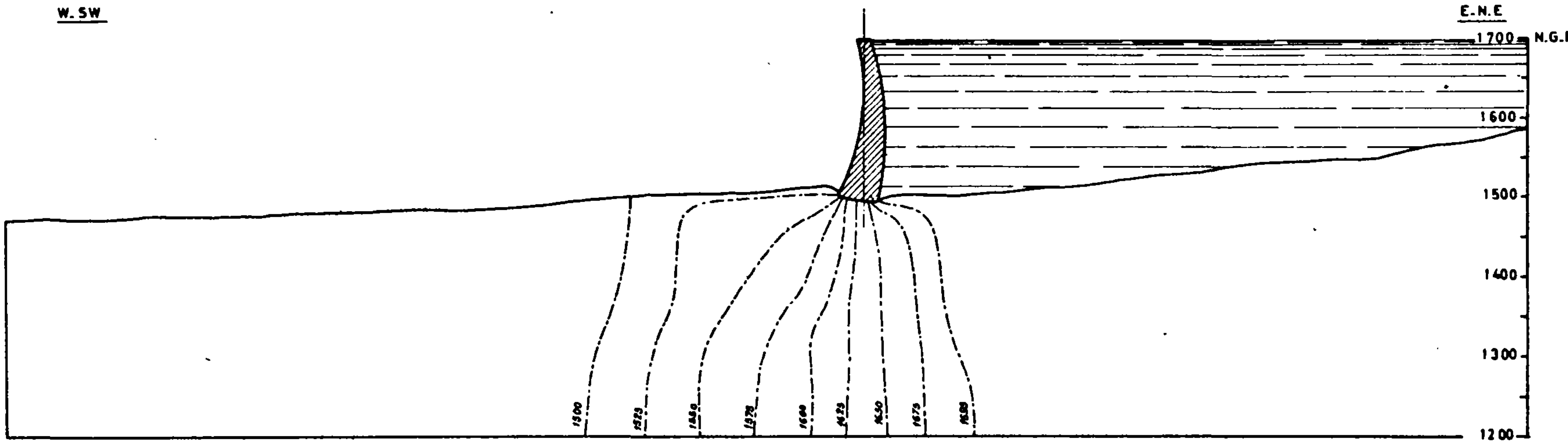
The boundaries of the model are impermeable, the topographical surface is of given potential (ϕ = 1700 m in the reservoir, $\phi = z$ downstream of the dam.

b) Results of the simulation

The aim of the simulation was, on the one hand, to obtain the distribution of the equipotentials, the shape of the free flow surface and the seepage surfaces, downstream of the dam, and, on the other hand, to estimate the leakage flow. Figure 50 gives some horizontal cross-sections through the flow network at the levels 1700 m (maxiumum retained level) 1625 m, 1525 m (in the dam zone developing from 1490 to 1700 m) and 1475 in the valley bed. Ten analagous cross-sections were established.

Anisotropic effects were very marked in the calculation hypotheses. On the right abutment the free surface makes an angle of 10 to 20° with the corresponding level line, whilst on the left abutment, this angle is greater than 130° (cross-section at level 1625 m). This tendancy is confirmed by the other cross-sections. The right abutment is therefore naturally badly drained ; the piezometric head is very high there. The

a) Longitudinal cross-section



b) Transversal cross-section

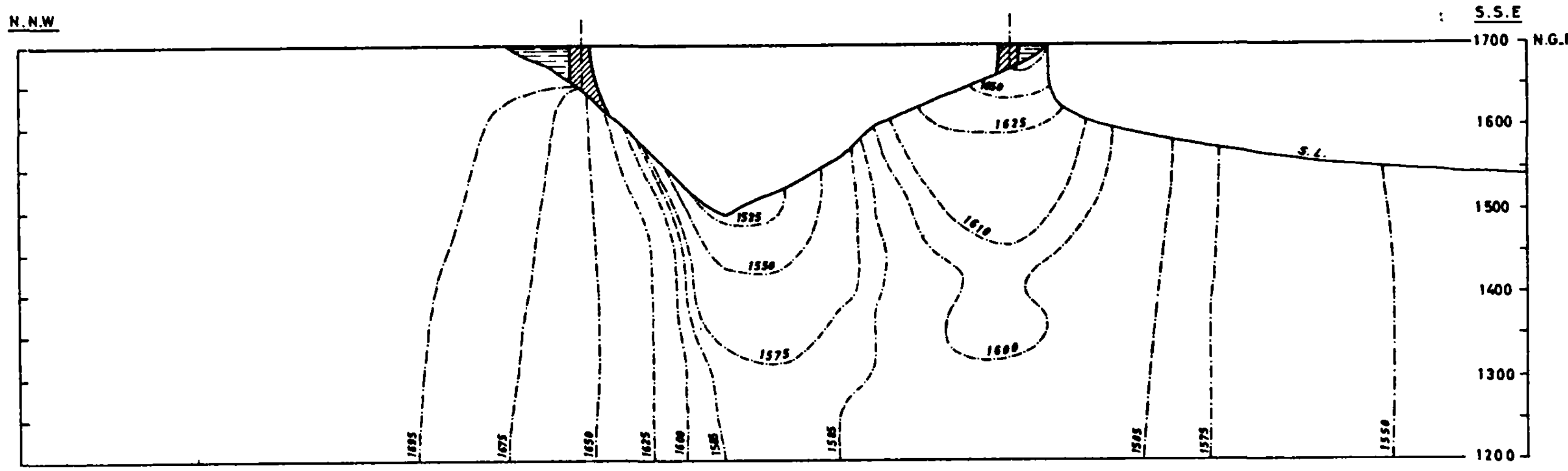


Fig. 51 - Transversal and longitudinal cross-sections of the flow network (see fig. 50)

contrary is true of the left abutment. In the valley bed, the flow network is also very dissymetric. Longitudinal and transversal cross-sections of the flow network are given on figure 51, in the directions defined on figure 50a.

It seems that the gradients below the dam are very steep and also that the flow network remains quite complex (transversal cross-section) because of the particular topography of the site.

c) Consequences of the results

The analysis of the results provided by the model must be made remembering its limitations ; the topography and the geometry of the dam are, in fact, imperfectly reflected, in that the mesh chosen are parallelepipedic and of large dimensions. The results obtained are therefore subject to caution for the direct vicinity of the dam (in a zone having an area of about one mesh). Moreover, possibilities of drainage or injection were not taken into consideration at this stage of the study.

A more detailed simulation could be undertaken by a secondary model, including a number of similar mesh, but applied to a more restricted zone within the vicinity of the dam and taking as boundary conditions for the secondary model, the hydraulic results provided by the first model.

At the present stage of reconnaissance, the examination of the flow network leads to the following qualitative conclusions :

- on the right abutment the flow force will be large ; it should be drained. Leakage flows here are not very great.
- the left abutment on the contrary, will be well drained, the flow pressures will be weak. However the flows below the abutments will be relatively abundant. According to the absolute value of the permeabilities, the left abutment may later have to be treated by grouting.
- in the valley bed, the gradients below the dam seem to be very steep.

The importance of anisotropy is the essential factor in this study, as it conditions the whole of the flow network. This anisotropy was verified by *in situ* measurements.

7.2.7 - *Verification of the hypotheses by in situ tests*

The marked hydraulic anisotropy of the Grand Maison damsite and its particular orientation, causing a total dissymetry in the flow networks have very important consequences in the behaviour of the dam. A serious verification of the hypotheses of calculation, and also of the preliminary results, therefore appears to be very necessary. This verification will naturally concern the evaluation of the anisotropy by *in situ* measurements using a technique particularly adapted for the hydraulic problem of the Grand Maison damsite, (see chapter 3.2.).

The essential aim of the *in situ* hydraulic tests is to verify the characteristics of the tensor of the permeability using the triple hydraulic probe and ponctual piezometers in boreholes (fig. 10b and 12).

The verification of the calculation hypotheses were planned for the two abutments and in the sub-fluvial gallery. Five test lateral excavations were made for this purpose. The emplacement of the boreholes in each excavation is given on figure 46. The injection boreholes, 50 m long, are directed parallel with the presumed orientations of the principal permeabilities. Five boreholes will serve for the estimation of maximal permeability whilst two others (perpendicular to the preceding ones) will provide information concerning the weakest principal permeability. Only the tests on the left abutment have as yet been accomplished.

7.2.8 - *Conclusion*

Some very interesting conclusions concerning the hydromechanical behaviour of the dam abutments have resulted from the hydraulic study forming part of the Grand Maison Dam project. It seems that, at the present stage of the work, the most delicate problem is not the theoretical study of water flows but the determination of the hydraulic parameters. The study cannot, as yet, be considered as concluded. The reconnaissance campaign must be continued with the greatest of care, directing the programmes so as to gain the maximum information concerning the directional hydraulic conductivity values. The structure of the medium seems to be already well defined. Complementary work should be carried out for the study of the influence of stress on the flow networks.

Such a study is enlightening as it makes possible a detailed analysis of the stability of the abutments, the final phase of the geotechnical study for this development. This phase will lead to a precise determination of the drainage network and the grout curtain, which in the case of the Grand Maison site will be dissymetric, with its particular situation. If required, more details on this project are given in the recent papers (LOUIS 1970, 1972, 1974, LOUIS and PERNOT 1972 etc...).

7.3 - Flow in a fissured rock slope

The characteristics of the example now considered are analogous to those of the example in paragraph 7.1. The difference lies in that in a rock slope, the flow is more nearly perpendicular to the surface of the rock mass than in the abutment of a dam. The data of the problem are given in figure 52. The boundary conditions are made up of the equipotential surface $\phi = H$, by the free surface of the rock slope $\phi = Z$, and by the lateral impervious surfaces. In order to emphasize the three-dimensional character of the considered examples the first boundary condition $\phi = H$ has not been chosen parallel to horizontal lines of the slope. As before the fracturation consists of two sets of sub-vertical fractures K_1 and K_2 and of a secondary horizontal fissure.

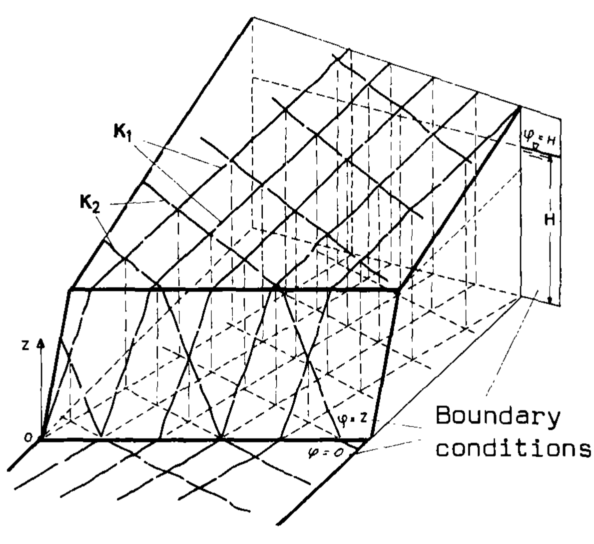


Fig. 52 - Groundwater flow in slope in fractured rock

a)

b)

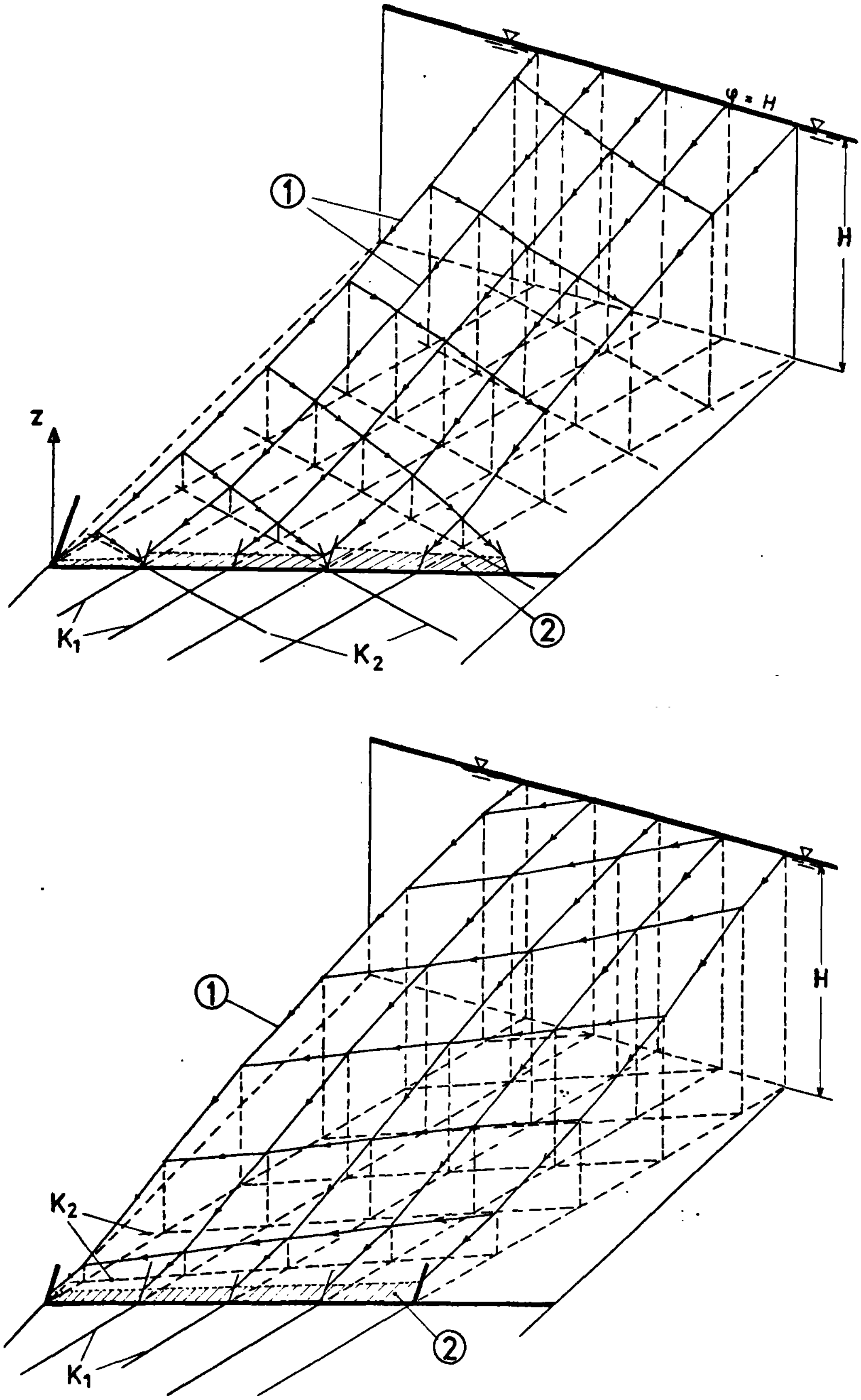


Fig. 53 - Three-dimensional flow nets in the slope in fractured rock
(a) Case of fig. 52
(b) Fracture set K_2 in different direction
(1) Free surface
(2) Seepage surface

The results of the hydraulic simulation are given in figure 53 for two assumed orientations of the fracture set K_2. The distributions of the hydraulic potential are well three-dimensional and the seepage heights are not constant along the bottom of the slope. These simple examples prove again that, for given boundary conditions, the flow net depends strongly on the structure of the medium.

7.4 - Hydraulic problems connected with dam projects

The two considered practical examples, which are studied in detail in other papers (LOUIS and WITTKE 1971 and WITTKE and LOUIS 1969), are very similar. Both are connected with two large hydroelectrical projects : one in Formosa (Tachien Project) and the second in West Germany (Bigge Dam Project). In each case the topographical situation is identical : upstream, the reservoir is connected through a very narrow rock ridge, with a valley which is parallel to the axis of the river. On the one hand, the leakage flow arising from this situation could have been very high, and on the other hand, because of the possibility of considerable hydraulic gradients, the stability of the rock mass was in danger.

7.4.1 - Pitan Ridge (Tachien project)

The dam on the Tachia river is designed to retain water to a height of 1400 to 1420 metres. Immediately below the dam the river swings around and the valley extends in a direction parallel to the dam axis. The Pitan ridge thus constitutes a narrow separation between the reservoir (level 1420) and the valley below, at level 1230 (figure 54).

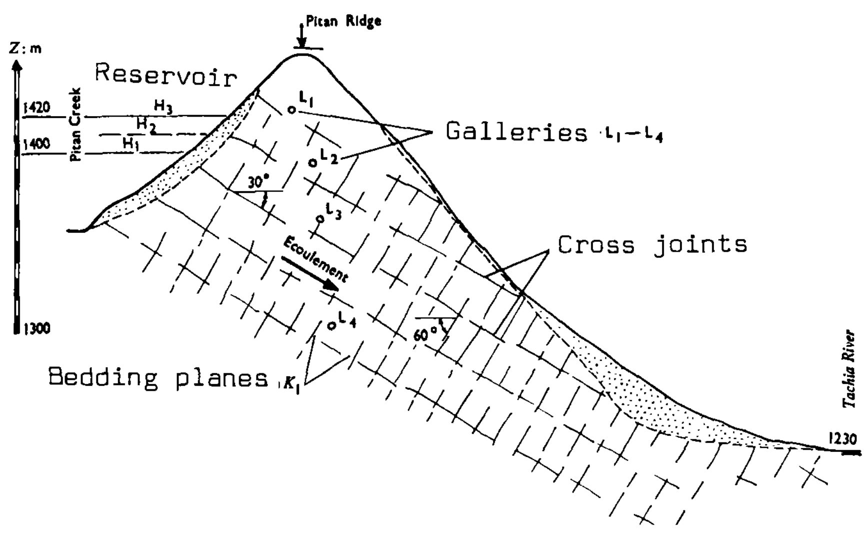
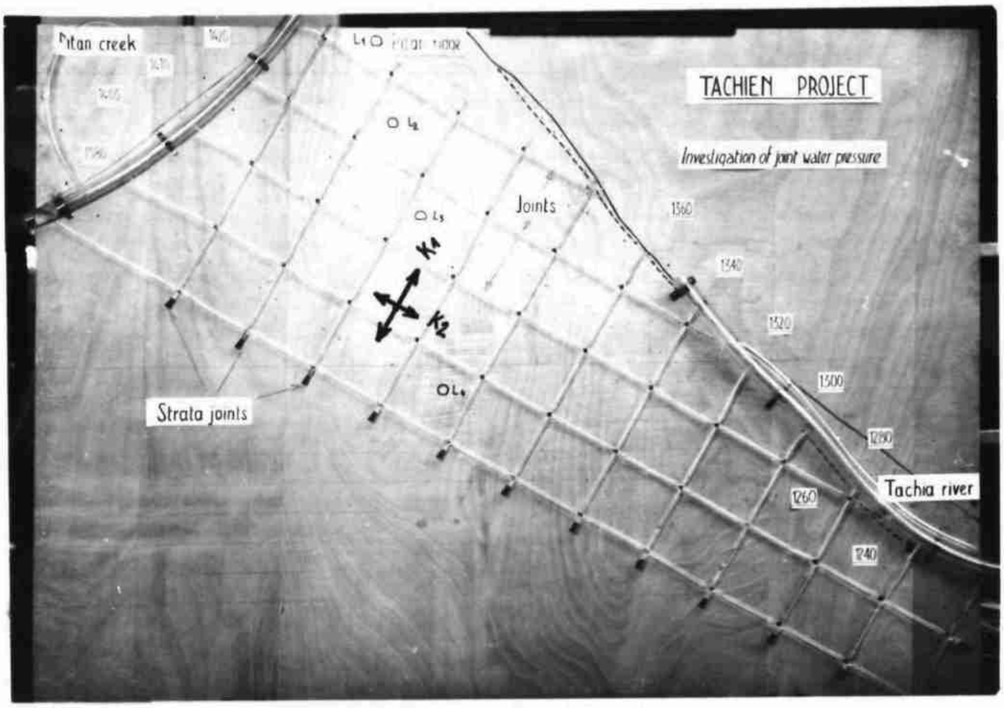


Fig. 54 - Schematic cross-section of Pitan Ridge (Tachien project, Formosa) Hydraulic and geometric data

Beneath the superficial soils there is a jointed rock mass (made up or alternating quartzite and schistose strata). The fracturation is characterized by two perpendicular systems of parallel fractures.

The study of the flow within the rock mass has been performed with hydraulic models made up of PVC pipes (figure 55). In order to be sure that the study included the actual case, two different hypotheses of relative permeabilities were considered (cases a and b, figure 55). The advantage of the technique used was primarily to allow a low cost study of a large number of cases, as shown in figure 56 (different heights of the reservoir, different relative permeabilities, different drainage systems). An example of the results obtained, showing the flow in the rock mass without drainage, is given figure 57. The study showed that the flow forces within the undrained rock mass are very large, and therefore that a network of drains was necessary to ensure stability.

a



b

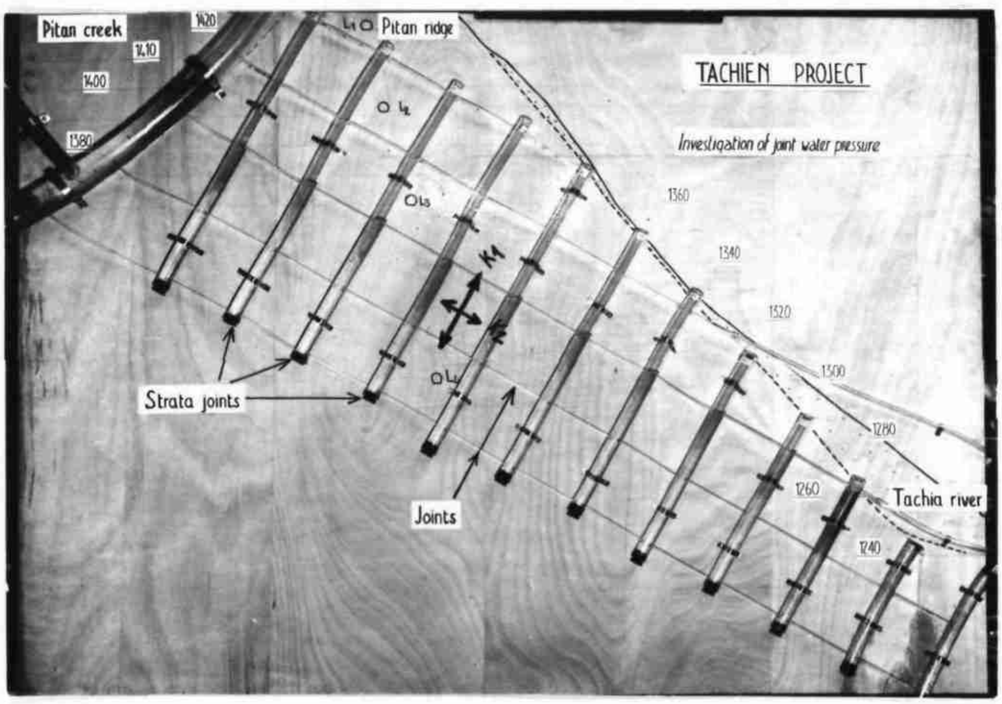


Fig. 55 - Hydraulic models for the hydrogeotechnical study of the Pitan Ridge
(a) Ratio of hydraulic conductivities : $K_1/K_2 = 2$

(b) Ratio of hydraulic conductivities : $K_1 \gg K_2$

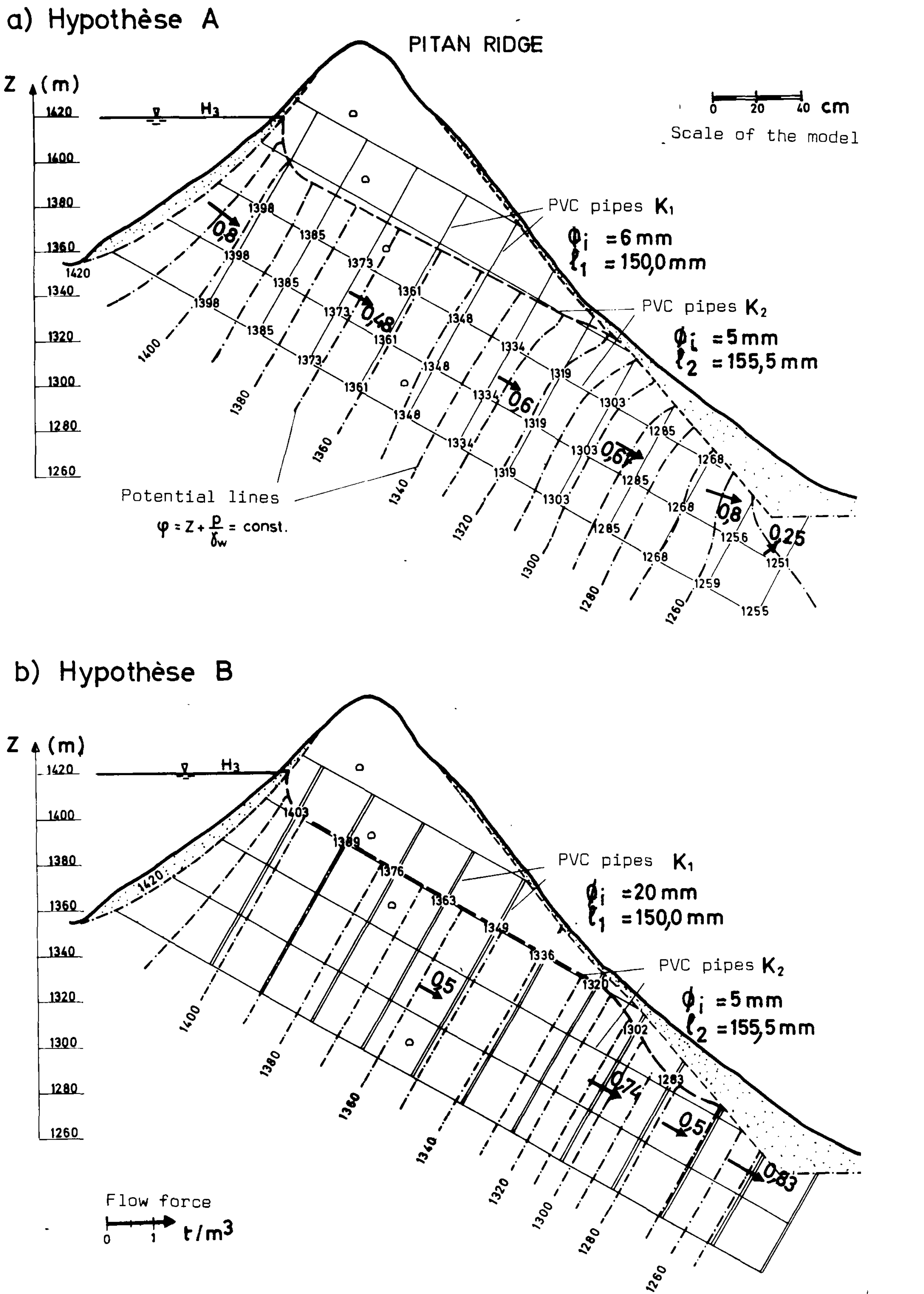


Fig. 55 – Hydraulic models for the hydrogeotechnical study of the Pitan ridge
(a) Ratio of hydraulic conductivities : $K_1/K_2 = 2$
(b) Ratio of hydraulic conductivities : $K_1 >> K_2$

7.4.2 - *Kraghammer Sattel (Bigge dam)*

The Biggetalsperre hydro-electric complex is in the Sauerland, which is part of the schist zone of the West German Rhineland. The rock pass known as the Kraghammer Sattel separates, over an area of 300 m, the Bigge Valley reservoir from the Ihnetal, a parallel valley upstream of their confluence (figures 57 and 58). Contrary to the preceding case, the level of the Kraghammer Sattel was below that planned for the reservoir. The rock mass, weathered at the surface and therefore quite permeable, was first to be given an increased height, than waterproofed to decrease losses. The stability of the downstream spillway was equally a problem because of water seepage.

A geological hydro geotechnical survey of the rock pass was made with the help of drillings, with water-pumping tests, of deep drill holes at the apex of the rock mass and of a longitudinal gallery. On the basis of the survey conclusions it was decided to install concrete piling in the decompressed zone (which has a well-defined lower boundary, as seen during the water-pumping tests, figure 59), and also a grout curtain within the heart of the rock mass to a depth corresponding to the zones with very low permeability (as revealed by the water-pumping tests).

From the hydraulic point of view it was possible to distinguish, within the rock mass, two zones with clearly different characteristics (figure 59). The water-pumping tests and the statistical survey of fractures showed that flows within zone 1 occurred essentially through the bedding planes, which are quite open, also through sub-vertical transverse joints. As in the case of the Pitan Ridge, continuous bedding planes that are oriented parallel to the longitudinal axis of the rock ridge are prone to open up cavities or voids and thus to have a hydraulic conductivity markedly greater than that of the cross joints which are discontinuous ; this fact was verified *in situ*. Because of this, a special hydraulic model was used to study flow phenomena in this domain (figure 60) ; the bedding planes themselves form equipotential surfaces. Zone II, a narrow faulted domain, is perpendicular to the longitudinal axis of the rock ridge and constitutes a large joint (with isotropic permeability) between the reservoir and the Ihne Valley. Flow phenomena were studied in zones I and II and various

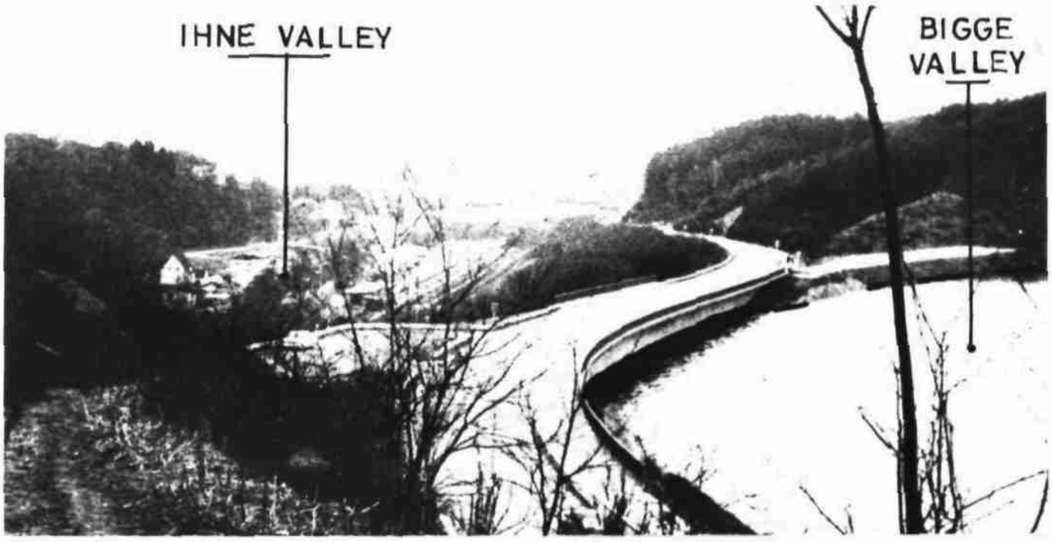


Fig. 57 - View of the ridge "Kraghammer Sattel" at the Bigge damsite, West Germany

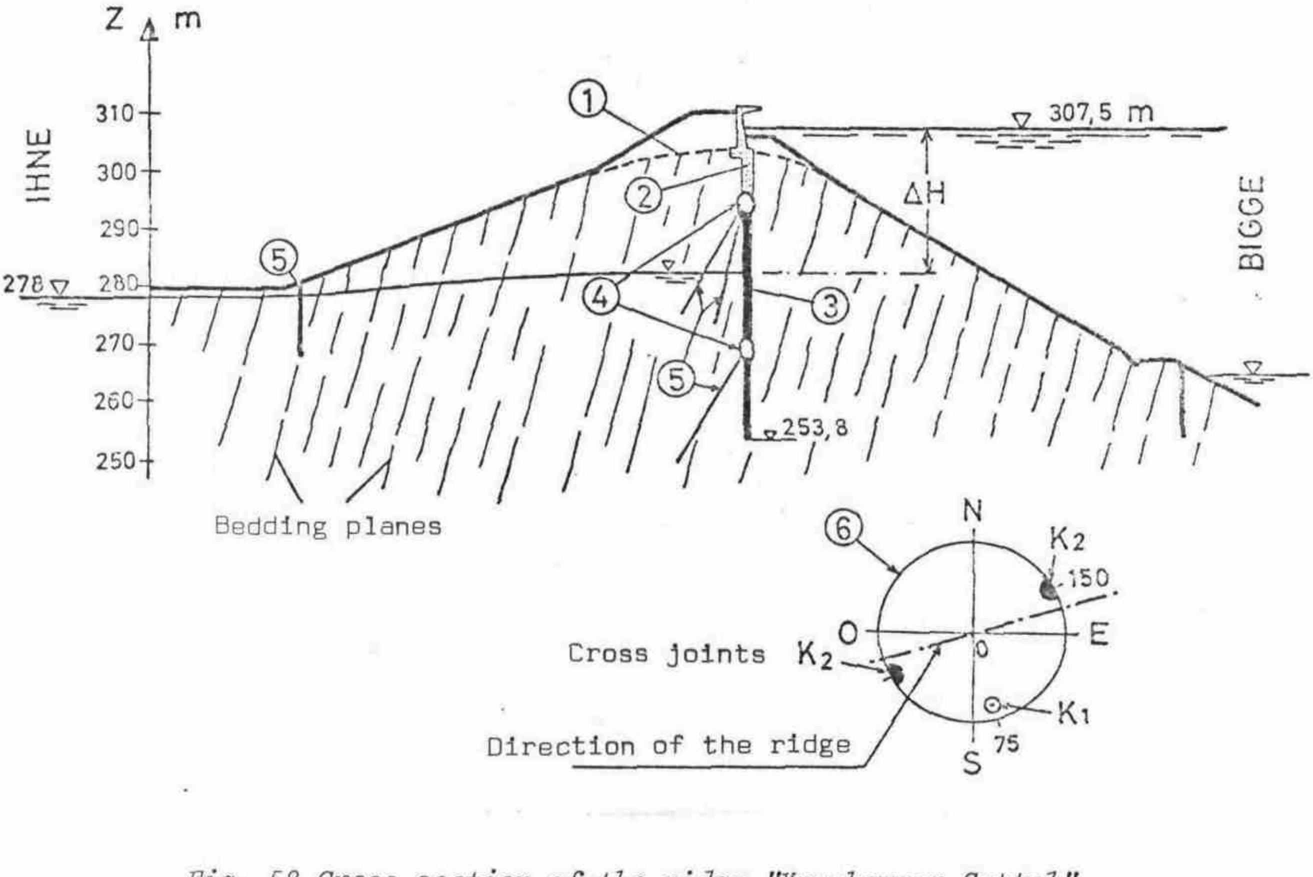


Fig. 58 Cross section of the ridge "Kraghammer Sattel"
(1) Natural profile *(4) Controlling galleries*
(2) Concrete piling *(5) Piezometers and drainage*
(3) Grout curtain *(6) Main fractures in zone I.*

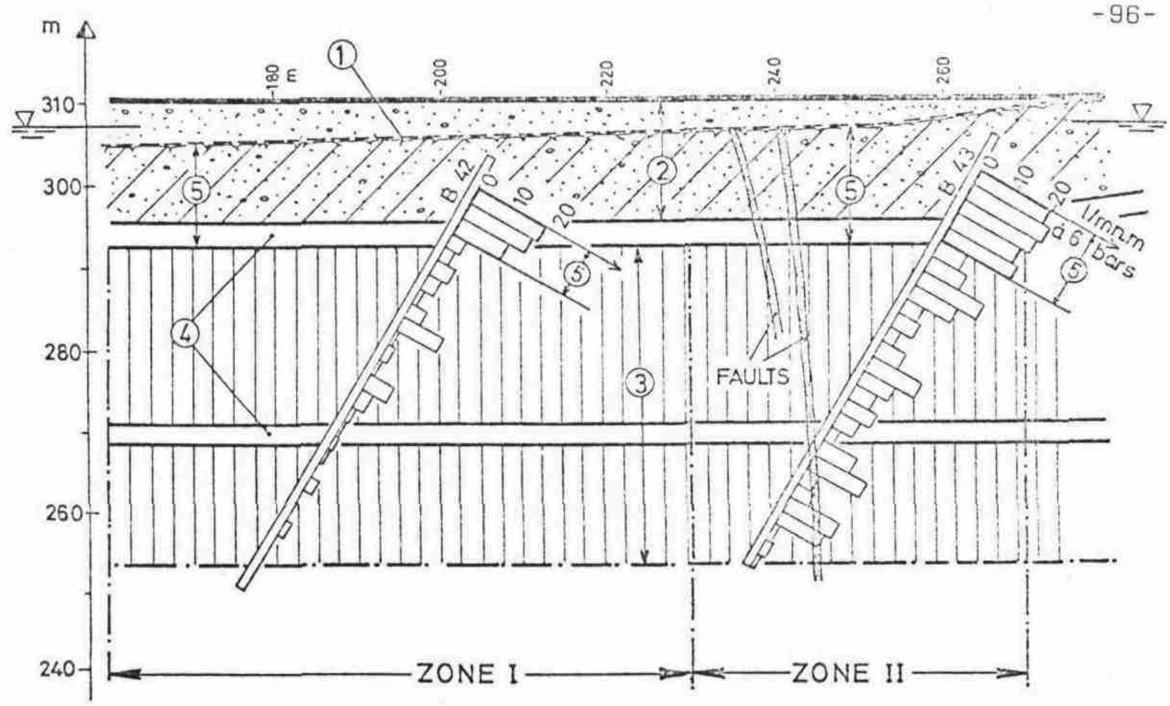
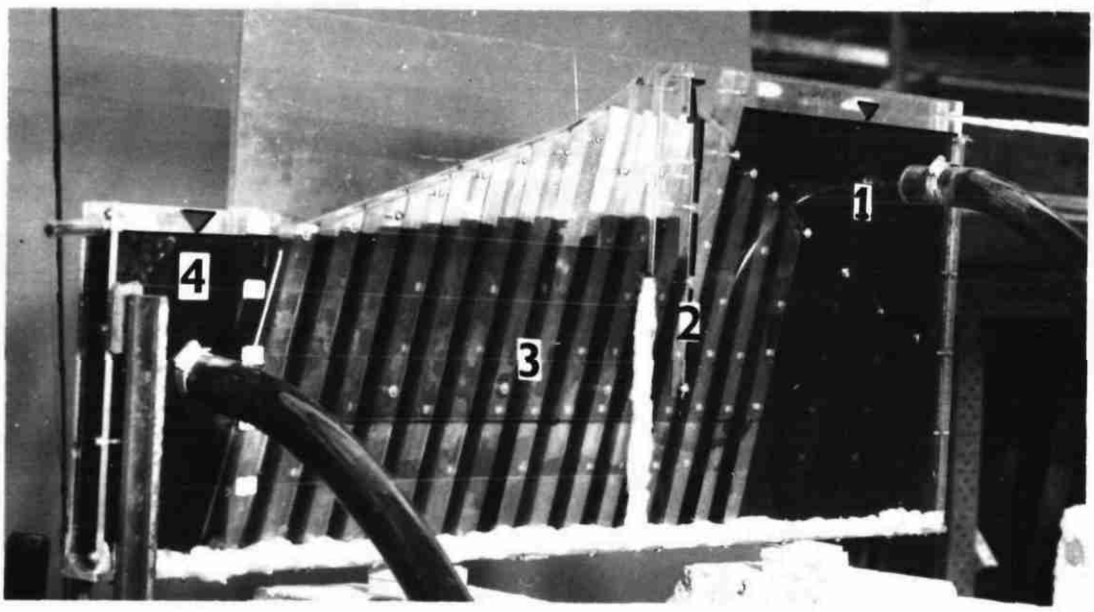


Fig. 59 - Longitudinal cross-section of Kraghammer Sattel
(1) Natural surface *(4) Control galleries*
(2) Concrete wall *(5) Decompressed zone*
(3) Grout curtain *(6) Water tests*



Cross section

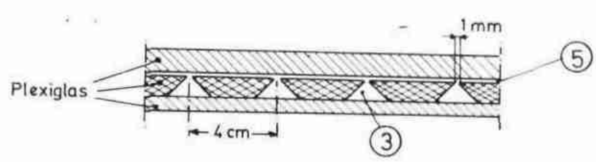


Fig. 60 - Hydraulic model for study of flows in zone I of Kraghammer Sattel
(1) Reservoir at 307.5 m level *(4) Downstream level 278 m*
(2) Grout curtain *(5) Simulation of cross joints K_2*
(3) Simulation of the bedding

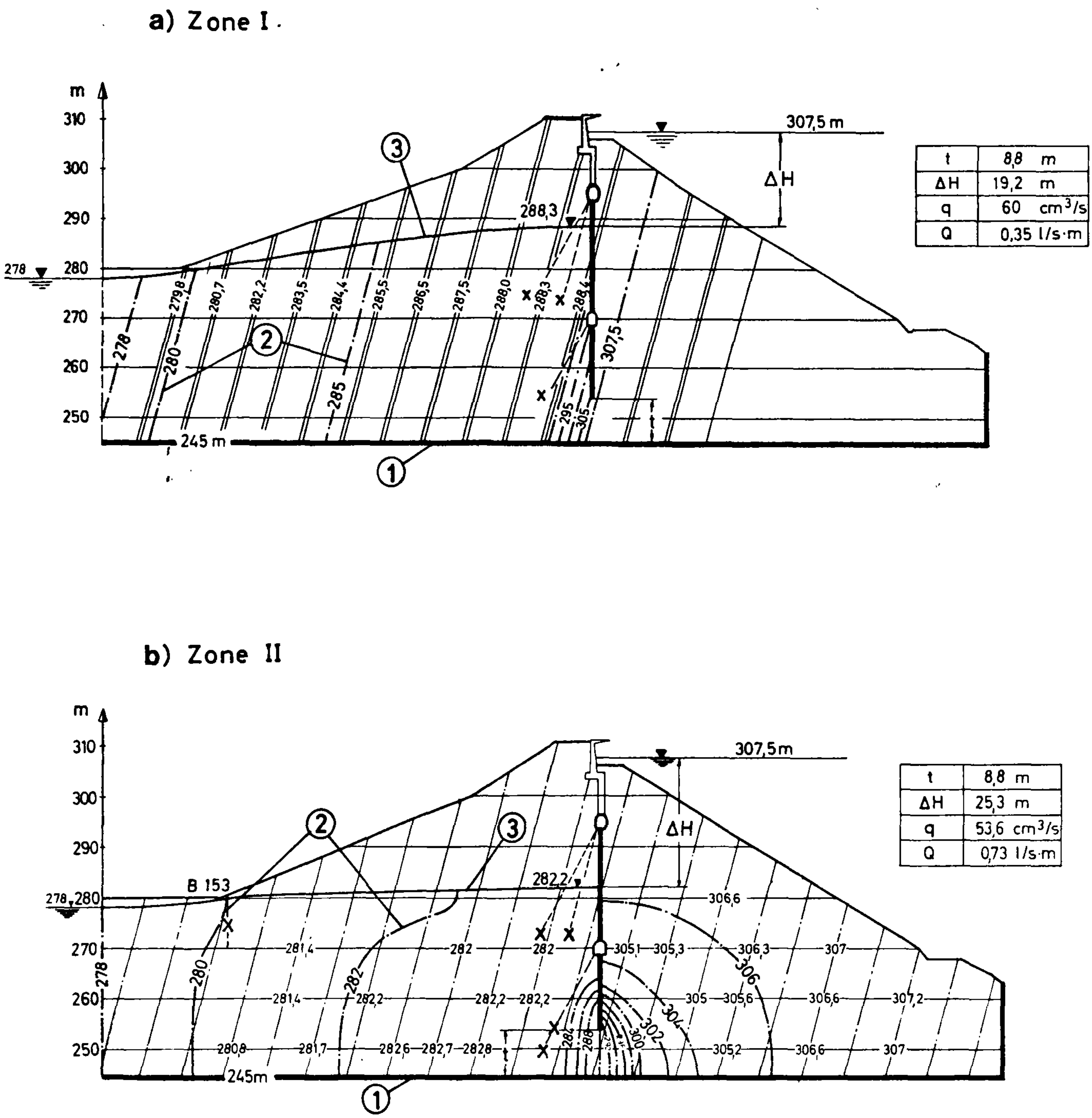


Fig. 61 - Flow nets in the rock ridge "Kraghammer Sattel"
(Two of 13 cases considered)

(1) Impervious boundary — *x = Piezometers*
(2) Equipotential lines — *q = Flow rate measured on model*
(3) Free surface — *Q = Flow rate measured in situ*

different boundary conditions were considered. Figure 61 shows, as an example, the flow networks in zones I and II for one of the cases considered.

Along with the theoretical studies, a number of piezometric measurements were performed *in situ* in boreholes at the foot and in the heart of the rock mass. These measurements taken *in situ* proved to be in complete agreement with the hypotheses concerning zones I and II, and showed a gratifying concordance with the results of laboratory studies.

These two hydrogeotechnical studies were performed at the "Theodor Rehblock Flussbau laboratorium", of the University of Karlsruhe, in close collaboration with Professor L. MULLER for the Pitan Ridge, and with the "Ruhrtalsperrenverein" for the Kraghammer Sattel.

8 - CONCLUSION

The recent developement of computer techniques, particularly in the field of scientific calculation, has greatly contributed to the progress achieved these last few years in the hydraulics of fractured media and in hydrogeotechnical studies. The power of computers has considerably increased, and it is now possible to obtain representative results through the use of mathematical models with a sufficient number of elements.

Physical models, (electric or hydraulic), can also give quite serviceable and practical solutions to certain problems which had so far seemed insoluble.

The validity of results thus obtained depends, whatever the method used, on care in applying the techniques and on careful *in situ* determination of the basic parameters, namely the geometric data on fracturation and the hydraulic conductivities on which the model will be based. It is here necessary to adopt a realistic view : in rock hydraulics, it is inconceivable that one could describe in detail the geometry of the fractures in a rock mass because of their random and therefore unpredictable characteristics.

The methods suggested are nevertheless applicable, for the essential requirement of mathematical or physical models is a knowledge of the overall hydraulic effect scaled down to the mesh of the model (as measured *in situ* with the help of appropriate testing techniques) and not a detailed knowledge of the fracture network structure.

On the basis of information obtained in the field the model can of course, be subdivided into elements as small as necessary. Hence, critical zones with a high gradient and also heterogenous zones may be subdivided into a large number of elements, while other zones, less important hydro-dynamically speaking and more homogenous, may be represented by an expanded mesh. The choice also depends on the precision required in final results. With this methodology, anisotropy, discontinuity and heterogeneity in naturally fractured media may be taken into account.

Before concluding, it is necessary to emphasize the need to carry out piezometric control measurements *in situ*, along with all theoretical or experimental studies. It is only through this method that one can test a mathematical or physical model and draw conclusions on the applicability of theoretical results thus obtained.

ACKNOWLEDGEMENTS

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May I also express my gratitude to the "Electricité de France" for their constant support, which made it possible for me to carry out *in situ* hydrogeotechnical studies at some major dam sites.

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