The water-pressure test for determining hydraulic conductivity

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In geological and geophysical site explorations in connection with the foundations of large dams, underground chambers, and other constructions in or on the rock, water-pressure test are used to determine the relative tightness of the bedrock. Here, the water-pressure test fulfils an important function. However, this method may also be used to increase knowledge of the general hydrogeological properties of the bedrock.

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Introduction

The measurement of water loss to determine the relative tightness of the bedrock is a test method which has long been used in geological and geophysical site explorations for constructional work. In these measurements (Lugeon-tests), attention has been concentrated on obtaining a comparative value of the tightness, and the absolute hydraulic conductivity in standarized dimensions has not been determined.

In the course of many years of constructional work, the Swedish State Power Board has worked out routines of testing and judgement which are directly adapted to specific problems. Some of these routines can be revised for use in other hydrogeological investigations, such as for determinations of the hydraulic conductivity.

In some earlier publications (Carlsson & Olsson 1976, 1977a, 1977b, and 1977c), we have tried to illustrate the variations of and the reasons for the hydraulic conductivity on the basis of the water-pressure test which have been carried out in some Swedish crystalline-rock types.

Generally, the crystalline basement is of little importance for water supply in Sweden. In all, only 3 per cent of the water consumed in Sweden is supplied from wells drilled in rock (Knutsson & Fagerlind 1977). Regular pumping tests in connection with ground-water prospecting in rock are seldom carried out as the financial investment would be too large, compared with the quantity of water obtained. In these circumstances, it is easy to realize the difficulties in determining the hydrogeological properties of the bedrock by the usual methods of hydrogeological exploration.

The water-pressure test

The water-pressure test can be described as a reversed pumping test. In the pumping test (pumping out), the level of the ground water is lowered, which results in a potential gradient. In the water-pressure test (pumping in), however, water under a certain pressure is forced into the rock mass, which gives rise to a potential gradient. Both methods cause a controlled disturbance of the normal state of ground-water flow. In measuring the effect of a given disturbance, the hydraulic properties of the water-bearing medium can be determined.

Field performance

The method used at the Power Board is as follows. In a drill-hole, water is forced into the rock mass under a certain pressure. The pressure is increased stepwise up to a maximum of about 800 kPa, after which the pressure is gradually relieved. In each load increment, the pressure is constant for a certain period of time (normally 1—3 minutes, but some tests have been carried out with a duration of 10 minutes and the time can naturally be prolonged for special purposes), and the quantity of water consumed is measured. Packers are normally used to demarcate the drill-hole and to locate large water-bearing zones. In this connection, double packers at a distance of 3 m are used. Single packers are used for tests in drill-holes which are partly blocked or in tests at greater depths. Single-packer measurements are also used as a check on tests with double-packers. In double-packer tests, the water is forced into a section of the rock mass

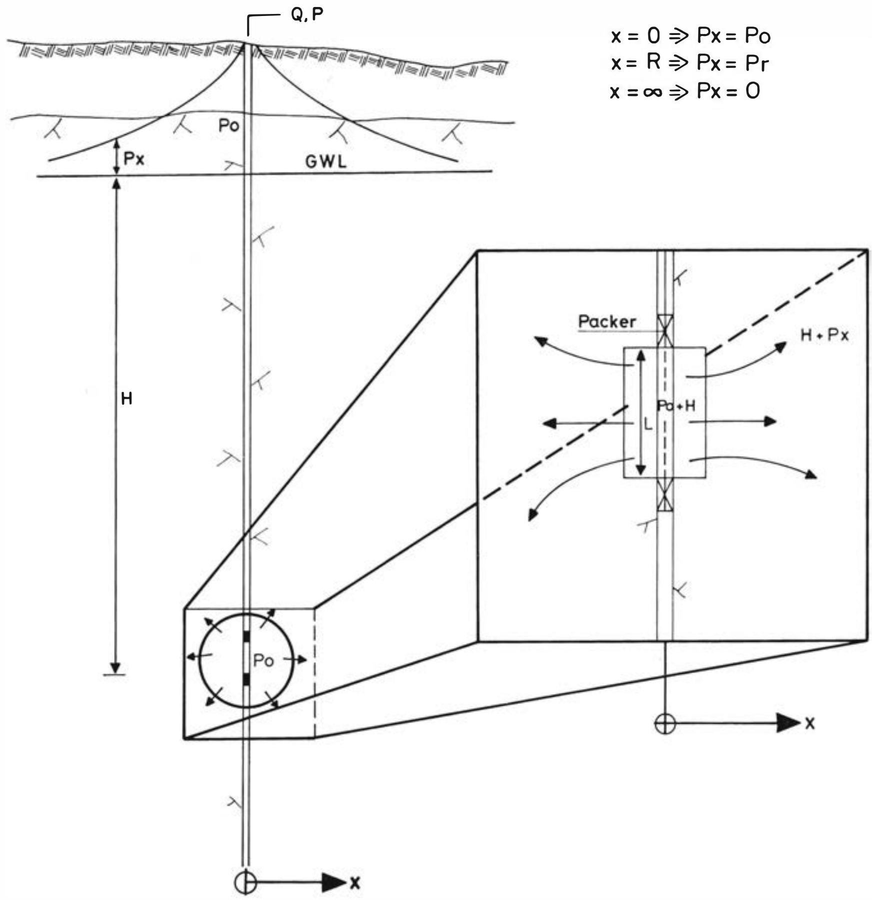


Fig. 1. The principle of the flow conditions in a rock mass during a water-pressure test. In the vicinity of the test section, the flow is radial from the bore-hole outwards (two-dimensional flow, visualized by the enlargement of the test section). At some distance from the test section, the flow has turned out to the three-dimensional and the effects of the directed hydraulic pressure have vanished.

limited by the two packers, while the corresponding limitations for single-packer measurements will be the packer and the bottom of the drill-hole.

As a result of the tests, the following parameters are obtained: numerical values of the quantity of water, measuring time and hydraulic pressure. These parameters and the geometrical data of the test section are the basis of an analytical calculation of the hydraulic conductivity of the rock mass.

Theory of the water-pressure test

The use of the water-pressure test for determining the rock tightness was first described by Lugeon (1933) and has since been used worldwide for engineering purposes.

The water flow around a test section during a water-pressure test is shown in Fig. 1. The symbols used are explained in Table 1. The flow pattern in Fig. 1 is valid, provided that the water-bearing medium is homogeneous and isotropic. In a fractured medium, such as a crystalline rock, the water flow will be governed by the intersecting joints.

This condition will generally create hydraulic conductivity of the rock mass without any allowance being made for the anisotropy. Hence, the flow conditions illustrated in Fig. 1 are in many cases false but can be used as a fairly good approximation in hydraulic-conductivity determinations. If

Table 1. Explanations of the symbols used in the equations and their dimensions.

Symbol	Explanation	Dimension
A	Area	L^2
C	Constant	1
d	Diameter of drill-hole	L
	Diameter of pipe	L
H	Static ground-water level	L
h	Head loss	L
I	Hydraulic gradient	1
k	Coefficient of hydraulic conductivity	LT^{-1}
L	Length of test section	L
l	Length of pipe	L
M	Manning's coefficient	1
P	Initial pumping pressure	L
P_0	True pressure in the test section	L
P_R	Pressure in the rock at distance R	L
Q	Flow rate	L^3T^{-1}
R	Radius of the influence area	L
	Hydraulic radius	L
r	Radius of the drill-hole	L
t	Duration of water-pressure test	T
V	Volume of water	L^3
v	Velocity of the water flow	LT^{-1}

the anisotropy is of vital importance, the method described by Louis (1967, 1974) may be used.

Under the assumption that Darcy's Law is valid, the water flow in the water-pressure test can be represented by the equation

$$Q = \frac{V}{t} = vA = kIA. \tag{1}$$

At the test hole, the potential gradient of the ground water in the rock mass (I) can be denoted by

$$I = -\frac{\partial P}{\partial x}. \tag{2}$$

The percolation area near the bore-hole can be assumed to be the same as the test-section concentric cylinder, according to the equation:

$$A = 2\pi xL. \tag{3}$$

If these definitions of I and A are inserted in equation (1), we get the following result:

$$Q = -k\frac{\partial P}{\partial x}2\pi xL. \tag{4}$$

After integration,

$$P_R - P_0 = -\frac{Q}{2\pi Lk}\ln\frac{R}{r}. \tag{5}$$

If $P_R - P_0 \approx -P_0$

$$k = \frac{Q}{2\pi LP_0}\ln\frac{R}{r}. \tag{6}$$

This equation applies in conditions of stationary, two-dimensional, ground-water flow. However, there are difficulties in determining the distance (R), where the result of the increase of the pressure in the drill-hole is hardly noticeable. For ordinary exploration purposes, the following approximation is used:

$$\frac{\ln R/r}{2\pi} = 1 \tag{7}$$

which gives

$$k = \frac{Q}{P_0L} = \frac{V}{P_0Lt} \tag{8}$$

This approximation is valid if the water-pressure test influences an area with a diameter of about 535 times the diameter of the drill-hole; the diameter of the area influenced is about 30 m for a drill-hole with a diameter of 57 mm.

In order to develop equation (6) further, Moye (1967) has assumed that the water flow is spherical at a great distance from the test section. In this case, the percolation area will be

$$A = 4\pi x^2. \tag{9}$$

After integration, this expression, in combination with equations (1) and (2), gives

$$P_R = \frac{Q}{4\pi kR}. \tag{10}$$

Now, equations (5) and (10) can be combined in the form

$$P_0 = \frac{Q}{2\pi k}\left(\frac{1}{2R} + \frac{1}{L}\ln\frac{R}{r}\right). \tag{11}$$

An empirical relation, according to Moye, is that $2R \approx L$. This will give

$$P_0 = \frac{Q}{2\pi kL}\left(1 + \ln\frac{L}{d}\right). \tag{12}$$

After solution with regard to the value of the hydraulic conductivity, we have:

$$k = \frac{Q}{2\pi LP_0}\left(1 + \ln\frac{L}{d}\right). \tag{13}$$

This equation is the standard equation normally used for determining the hydraulic conductivity of the basement on the basis of water-pressure tests (Carlsson & Olsson 1976, 1977a, 1977b, and 1977c).

The conditions of the test method, source of errors, and correction possibilities

For the validity of this method, stationary conditions have to prevail. Usually, due to the very short measuring times, transient conditions prevail and this is the basic disadvantage of the water pressure test. With a more exact instrumentation it is possible to analyse the transient ground water flow, which will increase the application possibilities of the method.

In equation (13), P_0 is the most difficult factor to determine exactly. In measuring the pressure at the ground level, the head losses in the pipe system of the test equipment may be of such a magnitude that the true pressure in the test section is considerably lower. Furthermore, if turbulent flow appears in the investigated rock mass, this will also influence the value of the potential gradient.

In the following pages, we shall try to describe the reasons for disturbances and inaccuracies and also the available possibilities of correcting these types of errors in the measurement.

Head loss

The following parameters are part of the empirically determined equation (13):

1. Measuring time,
2. Length of the test section,
3. Diameter of the test section,
4. Quantity of water forced into the rock mass (the water take of the rock mass), and
5. Hydraulic pressure.

Parameters (1), (2) and (3) have a constant value in each single test and only a direct measuring error can make the result incorrect. If there is a fault in the flow meter or if there are leakages, either in the pipe system or at the packers, this will lead to an incorrect value of the quantity of water. It is possible to diminish the risk of leakage at the packers by using four packers instead of two, which will give three separate test sections. Experimentally, Maini (1971) shows that, by using four packers, the leakage is reduced to a minimum. However, the technical work is more complicated.

The head loss within the testing equipment causes a decrease of the hydraulic pressure. At low flow rates, the head loss is comparatively small, but it increases with increasing water flow. The difference between the initiated hydraulic pressure at the ground level and the true pressure down in the test section is caused by the head loss. The relation can be described as:

$$P_0 = P - h. \tag{14}$$

To make an accurate determination of the hydraulic conductivity of the water-bearing medium, it is necessary to know the magnitude of the real hydraulic pressure acting in the test section. The true acting pressure can be determined in two different ways, either by a direct measurement of the pressure down in the hole or by calculating the head loss of the measuring equipment. The first method gives the most correct result, but on the contrary, the technical work is much more complicated than in the second method. Which of the methods is to be chosen is merely a question of accuracy, i.e. the purpose for which the water-pressure test is being made.

By using a manometer in the test section, the true pressure can be given directly. In this case, the effects of the head loss are of no importance, since the pressure is measured after the reduction caused by the frictional forces within the pipe system.

Normally, for the usual site-exploration purposes, the pressure measured is the pressure produced by the pumping equipment at the ground level. In this case, the frictional forces within the pipe system cause a decrease of the pressure head, so that the acting pressure down in the drill-hole is lower than the initial pressure. To get an accurate value of the true pressure down in the hole, a reduction according to equation (14) has to be made. According to Manning's Formula, the head loss in a hydraulic system is:

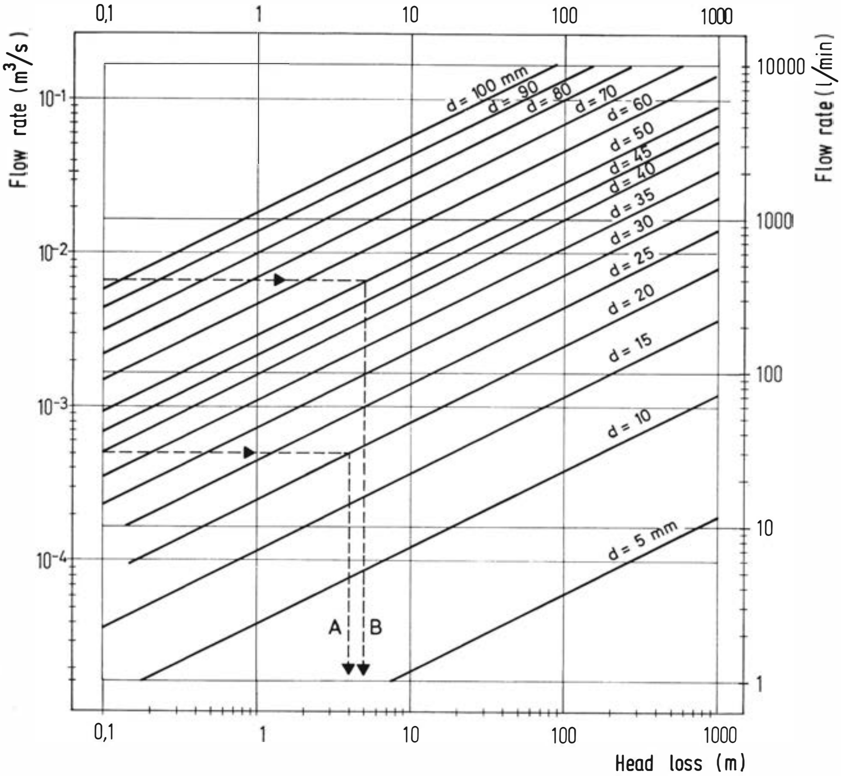
$$h = \frac{Q^2 l}{M^2 A^2 R^{4/3}}. \tag{15}$$

For a pipe with a circular cross-section, $A = \pi d^2/4$ and $R = d/4$. This gives:

$$h = \frac{Q^2 l\, 16}{M^2 \pi^2 d^4 (d/4)^{4/3}}. \tag{16}$$

Manning's coefficient for normal pipe material is in the range of 80—90.

Besides the plain head loss according to equation (16), losses will arise from changes of the cross-section (splices and changes of the pipe diameter, among other things) and also from bends in the pipe system. However, these types of additional losses are difficult to estimate. If the splices are made in such a way that the loss is minimal,



Examples:
l = 100 m ø = 20 mm
p= 8 kp/cm q = 30 l/min
give a head loss of
4 metres. This results
in
Po = 80 - (10·4) =40 m

l = 50 m ø = 50 mm
p= 6 kp/cm² q = 400 l/min
give a head loss of
5 metres. This results
in
Po = 60 - (5·5) = 35 m

Fig. 2. Manning's formula diagrammatically shown. The formula may be used to determine the head loss within the measuring equipment. The different lines in the diagram give the head loss as a function of the flow rate in a 10-metre-long, straight pipe with a certain diameter. In the examples, the use of the diagram is shown.

this additional loss may be neglected. Generally, the additional loss from pipe bends is less than 10 per cent of the head loss of a straight pipe 10 m long. These additional losses are calculated for each bend in the system.

In Fig. 2, the head losses for straight pipes are shown. The diagrams are calculated for 10-metre-long pipes of varying diameters and the head loss is shown for varying flow rates. If we know the diameter of the pipe system and its length, the diagram may be used to determine the head loss for a certain testing equipment. The true pressure in the test section can be determined with a fairly good accuracy by using equation (14).

Skin effect

The skin effect is well known in pumping tests, both for petroleum production and for water supply. This effect causes a marked drop in the draw-down curves in the vicinity of the well. Within the skin, the hydraulic conductivity is significantly lower than in the rest of the formation. This skin effect is generally thought to be a result of the drilling operation, the completion technique and the pumping practices used, which damage the aquifer (cf. van Everdinger 1953, Hurst 1953, and others).

It is probable that this effect also influences the results obtained in water-pressure tests in boreholes. The hydraulic-conductivity value calculated from the test results may be the conductivity of the skin and not the conductivity of the natural rock mass. According to Carlsson & Carlstedt (1977), the additional draw-down due to the skin effect is usually less than 25 per cent for the normal types of Swedish rock. It seems reasonable to assume that the effect is about the same magnitude in the water-pressure tests. If this assumption holds, the hydraulic conductivity of the natural rock material will generally be less than 30 per cent higher than the value obtained from the test results.

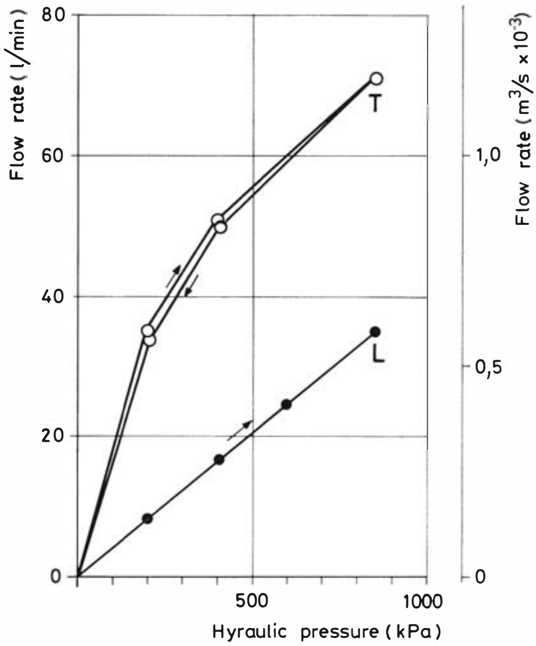


Fig. 3. The relation between the hydraulic pressure and the flow rate under laminar (L) and turbulent (T) flow conditions. In this linear relation between the two variables, the laminar-flow condition has a linear correspondence, while in the case of turbulent-flow conditions, the correspondense is parabolic.

Turbulent state of flow in the rock mass

The calculation method is based on Darcy's Law and has therefore the same assumptions as regards validity. One of the most important of these assumptions is the demand that a laminar state of flow shall exist.

The essential difference between laminar and turbulent flow is that the velocity of the laminar state is in proportion to the potential gradient, while the velocity of the turbulent state is in proportion to the square root of the gradient. Furthermore, the velocity is influenced by the hydraulic radius (R). According to Chezy, the relation for a turbulent state of flow can be given by the following expression:

$$v = C\sqrt{RI}. \qquad (17)$$

If the water flow can be assumed to take place in a number of capillaries with the diameter d, Chezy's formula will be:

$$v = \tfrac{1}{2}C\sqrt{dI}. \qquad (18)$$

For the water-pressure test, this means that the pressure P_0 in equation (13) will be exchanged for $\sqrt{P_0}$ if there is a turbulent state of flow. However, the difficulty is to decide when the flow conditions change from the laminar to the turbulent state. Here, the gradual increase of the pressure is of great help. In Fig. 3, two P-Q curves are shown, representing turbulent and laminar flow. The same curves are inserted in Fig. 4, but in this diagram proportionately to $\sqrt{P_0}$. If the curves in Fig. 4 are compared with those of Fig. 3, it will be seen that the shape of the curves has changed. The straight curve in Fig. 3 is curved in Fig. 4 and vice versa.

Accordingly, on the basis of the gradual increase of the pressure, it is possible to decide if there is a turbulent state of flow and thus to compensate for this state. However, it is also possible that the normal linear course of the P-Q curves is influenced by other factors. Some of these factors are discussed in the next section.

Test influence

In connection with the water-pressure test, several effects are produced by the increase of the hydrau-

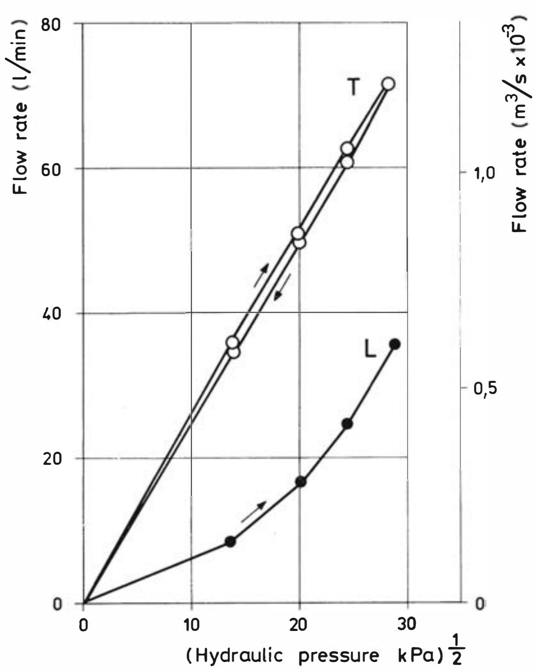


Fig. 4. The same measurements as in Fig. 3, with laminar (L) and turbulent (T) flow conditions. In this case, the flow rate is related to the square root of the hydraulic pressure. The turbulent condition shows a linear correspondence, while the laminar flow gives a parabolic relationship.

lic pressure. These effects cannot be attributed to losses or turbulence but possibly to changes in the water-bearing medium. Figs. 5—7 show some of the relationships between the hydraulic pressure and the quantity of water which may orginate from such changes.

An outline will now be given of the effects which arise during the course of the test and of the effects which disturb the normal linear relation between the flow rate and the hydraulic pressure.

Rock-heaving

From the petroleum industry, it is known that the hydraulic conductivity of the fractures in a fractured aquifer is stress-dependent. As Cooke (1973) and Holditch & Morse (1976) have shown, the hydraulic conductivity in a fracture can be reduced by a factor of 10 at high closure pressures. This reduction is caused by the significant reduction of the hydraulic pressure within the fracture due to pumping in the well. Analogously, the hydraulic conductivity may be increased during a water-pressure test, as the pressure in the fracture increases. The possibilities of establishing this effect are naturally especially favourable at very high testing pressures.

In Fig. 5, the curves describe a drastic increase of the flow rate at a fairly moderate increase of the pressure. The measurements were made at depths of 10 m and 20 m respectively in a granitic type of rock, which has a well-developed, horizontal-joint set. The effect is probably due to a widening of the fissures and consequently an increase

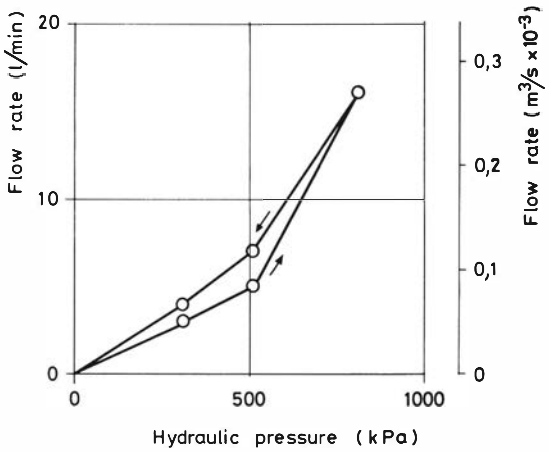


Fig. 5. An example of a measurement in which the hydraulic pressure has caused a movement within the rock mass (rock-heaving).

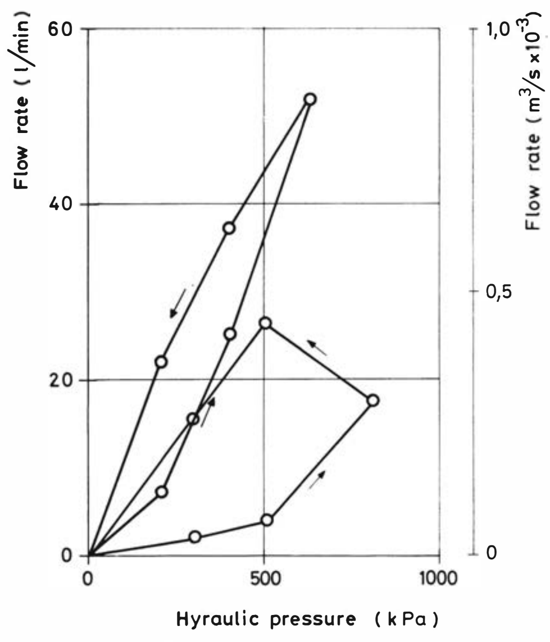


Fig. 6. An example of a measurement in a rock mass containing easily eroded joint fillings. The flow rate increases very rapidly as the hydraulic pressure increases. In the stepwise unloading, the flow rate is still very high.

of the hydraulic conductivity. Depending on the hydraulic pressure, a movement of the rock appears. From a theoretical point of view, if no attention is paid to the development of shear stresses, a hydraulic pressure of 800 kPa may heave a rock mass about 30 m. When the rock mass is subjected to this pressure, besides the vertical load, shear stresses will be produced, and the magnitude of these stresses will correspond to the effect of the hydraulic pressure. Thus, on the assumption that the applied pressure overcomes the weight of the rock mass and the shear stresses produced, a movement which relieves the water pressure will take place in the rock mass. These movements may be great if the rock mass has a well-developed, horizontal-joint system which admits of certain limited movements even within the rock mass without heaving the whole of the overburden. In this case, the whole applied rock load is not contributed and the resistant factor is lower than the true weight of the rock mass. Under these conditions, rock movements may appear at depths exceeding the theoretical maximum.

During the release of the hydraulic pressure, the result of the rock-heaving may either remain or result in a re-tightening of the fissures. In the first case, the flow rate will have a higher value

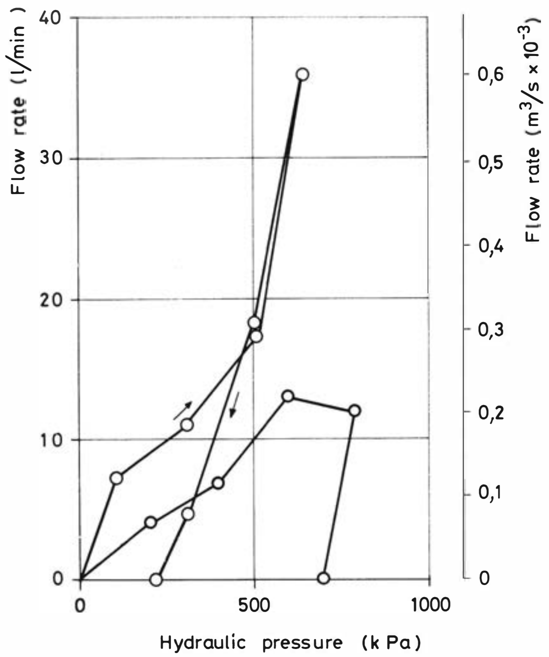


Fig. 7. An example of a measurement in a rock mass containing easily eroded filling materials. By increasing the hydraulic pressure, the material is compressed, which causes a decreased flow rate. The effect persists during the release of the hydraulic pressure.

during the release than during the increase of the hydraulic pressure. If there is a re-tightening of the fissures, the flow rate at the release will, on the whole, follow the loading (either as a elastic deformation of the rock mass or as a sudden tightening which is the result of a breaking deformation).

Wash-out

Fig. 6 describes a relation between the hydraulic pressure and the flow rate, where each load increment produces an increasing water flow which is not in proportion to the increase of the pressure. In contrast to the rock-heaving, where the increase of water flow was momentary, the increase here is continuous. This effect is obtained when the joint system in the rock mass includes easily eroded, infilling materials. To obtain this effect, besides the occurrence of the infilling material, there must be possibilities of redepositing the eroded material. If there are no redepositing possibilities, the result is somewhat different. An example of this is shown in Fig. 7, where the curve at the beginning shows a wash-out and an increasing flow rate and later shows a stop-up and compaction of the material and thereby a decrease of the flow rate. This combined effect is obtained during the increase of the pressure, which first of all leads to an increase of the hydraulic conductivity. A further increase of the pressure, however, leads to a decrease of the hydraulic conductivity and the reduction will also remain during the release.

Other effects

Except rock-heaving and wash-out, other effects of the water-pressure test may arise, although these types of effects are less usual. Moreover, the connction between the cause and the effect is not clearly indicated. Generally, the effects tend to increase with increasing hydraulic pressure in the rock mass.

Conclusions

The effects earlier mentioned show that the possibilities of disturbances increase with increased measuring pressure, i.e.

1. The condition for turbulence increases,
2. The friction loss increases,
3. The conditions for rock-heaving increase, and
4. The condition for wash-out increases.

These conditions indicate that the hydraulic conductivity determined from the results of water-pressure tests tends to be larger than the true hydraulic conductivity of the rock. The difference depends on the amount of the hydraulic pressure used. Thus, the value of the hydraulic conductivity should be based on the results of the lowest pressure increment at the loading, in order to be as representative as possible of the true hydraulic conductivity of the bedrock. The other increments of the pressure are measures more of the deformation and of the fissure characteristics than of the water-bearing properties. However, that is not to say that it is sufficient to use only the lowest increment at the loading; on the contrary, the results obtained during the gradual increase of the pressure are essential, among other things, for determining the state of flow.

In the introduction, we discussed the difficulty of determining the hydrogeological properties of the bedrock by accurate methods. The reason for this is, as we mentioned, that the crystalline basement is hardly of any importance for the water supply in Sweden, since the financial investment would be to large, compared with the quantity of water obtained. Therefore, from this point of view, the water-pressure test is not only a method of importance in site exploration for engineering and geological purposes but also an important method of increasing our knowledge of the hydrogeological properties of the bedrock.

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