



VODOPROPUSNOST STIJENSKE MASE

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Sažetak:

Prisustvo vode u stijenskoj masi odosno procjeđivanje ima važnu ulogu pri izgradnji inženjerskih objekata (brane, tuneli, ...), kao i oblikovanju stijenskih kosina, uslovljenih realizacijom različitih projekata. Stijenska masa kakvu nalazimo u prirodi (in-situ) je heterogena, anizotropna, raspucala i nalazi se u prirodnom stanju naprezanja. U numeričkom modeliranju za potrebe planiranog geotehničkog projekta, pored ostalih ulaznih parametara, neophodno je, što tačnije, definisati i vodopropusnost stijenske mase. U radu se opisuju mehanizmi tečenja vode kroz stijenski masiv i metodologija definisanja parametra vodopropusnosti za takvu geološku sredinu.

Ključne riječi: Stijenska masa, vodopropusnost.

WATER PERMEABILITY OF ROCK MASS

Abstract:

The presence of water in the rock masses, or water filtering, has an important role in the construction of engineering facilities (dams, tunnels, ...), as well as in formation of rock slopes, caused by the implementation of various projects. The rock mass, that we find in nature (in-situ), is heterogeneous, anisotropic, jointed and is in the natural state of stress. On numerical modeling of planned geotechnical project, in addition to other input parameters, it is necessary, as more precisely, to define the water permeability of the rock mass. The paper describes the mechanisms of water flow through the rock mass and the methodology for defining the waterpermeability parameter of such a geological environment.

Keywords: rock mass, water permeability.

1. INTRODUCTION

Water in the rock mass appears in the voids (pores) of intact rock and in the discontinuities. Such as porosity of the intact of the rock is small, we can say that water flow in the rock mass to a large extent unfolds through discontinuities. Because of that we can say that characteristic for the rock mass is crack porosity. Presence of water in the rock mass has important role in the construction of the engineering facilities as well as design the rock slopes, conditioned by realization different projects. Big challenge for the engineers (hydraulics and hydrogeologists) is to describe flow in the rock mass jointed porosity in solving engineering tasks.

An important factor in the construction of the tunnel is seepage flow which has important role on the stability of the temporary face of the excavation. Hydrostatic and hydrodynamic pressure of the water in the rock mass results in difficulty such as sliding and dropping of rock wedges. At dam construction leakage around the sides of the dam and below the foundations has important role on the stability of the dam, and for this reason it is necessary to know very well the rock mass (permeability and capacity) in order to carry out appropriate analyzes and brought a quality solution of problems of leakage.

On the slopes in the rock mass the size of the seepage forces has significant impact on stability. The assumption is that through the jointed rock mass leaking takes place only through discontinuities. Water flow through the rock mass is therefore in the function of discontinued, their connection and the size of the aperture of discontinuity. Classical hypothesis of groundwater flow in the analyzes rock mass includes two assumptions:

- The flow takes place only through discontinuities (the permeability of the intact rock is negligible)
- The flow is laminar.

2. THE IMPACT OF WATER ON THE PROPERTIES OF THE ROCK MASS

The presence of water in the rock mass it aggravates the conditions of stability, just by creating hydrostatic pressure and affects mechanical behavior rock mass, it reduces its strength. The most important water effects are the following:

- Pore pressure decreases the strength of the intact rock
- Filling discontinuities reduces the solidity of the total rock mass
- Causes physical and chemical changes and as a result of the weakening of the intact rock and the rock mass (process of weathering)
- Acts as an erosive medium (in clay and soft materials) by washing out fine particles
- Causes chemical reactions that change the composition of the rock
- In soft (clayey and clayey-sandy) rocks the pores of the pores of water work on reducing the strength parameters (cohesion and friction angle).

2.1. POROSITY OF INTACT ROCK

As we know each intact rock contains a part of the voids, in order to allow for a seepage flow, at least part of these voids must be interconnected to form passages through which water can permeate. Total porosity is defined as:

$$n = \frac{V_P}{V_P + V_S} = \frac{V_P}{V} \quad (1)$$

where:

V_P is pore volume,

V_S is volume of solids,

V is reference volume.

In the description of flow through the porous intact rock, the seepage velocity of v_s defined as a quotient water quantities in the unit of time Q , and is referred to is "discharge" or "flow-rate" through the total cross-sectional area A , normal in relation to the flow direction (Fig.1).

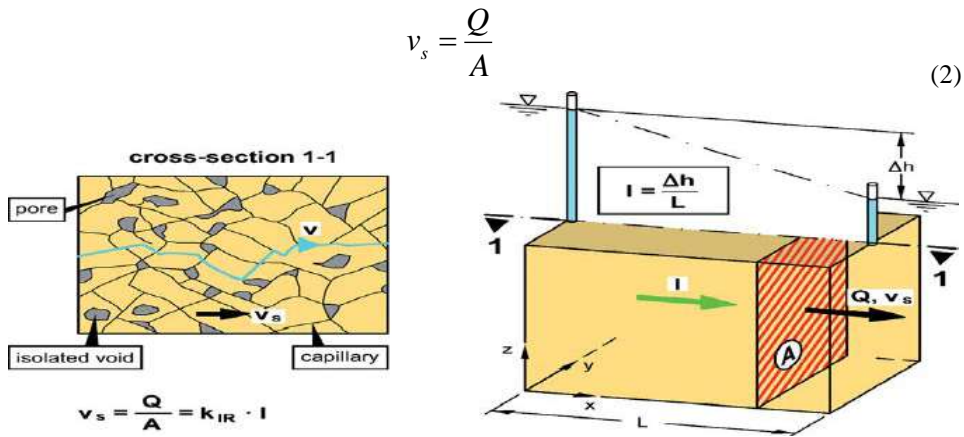


Figure 1. Seepage flow of intact rock- Darcy's Law [15]

It was found that the seepage velocity is proportional to the hydraulic gradient I (Darcy 1856). In the example of one-dimensional flow represented in Fig.1 this relationship, known as "Darcy's law", takes the following form:

$$v_s = k_{IR} \cdot I = -k_{IR} \cdot \frac{\partial h}{\partial x} = k_{IR} \cdot \frac{\Delta h}{L} \quad (3)$$

where:

k_{IR} is coefficient hydraulic permeability (intact rock),

I is hydraulic gradient.

Equation can be generalized to the three-dimensional case at:

$$\{v_s\} = k_{IR} \cdot \{I\} \quad (4)$$

where:

$\{v_s\}$ and $\{I\}$ are vectors of seepage velocity and hydraulic gradient.

$$\{v_s\} = (v_{sx}, v_{sy}, v_{sz})^T \quad (5)$$

$$\{I\} = (I_x, I_y, I_z)^T = - \left(\frac{\partial h}{\partial x}, \frac{\partial h}{\partial y}, \frac{\partial h}{\partial z} \right)^T = - \{\nabla\} h \quad (6)$$

Equation is valid if the intact rock permeability is isotropic.

2.2. FLOW TROUGH DISCONTINUITIES

Due to the low water particle velocity, in most cases the flow of water in discontinuities can be considered laminar. As is already known from Darcy's law that the velocity of the proportional hydraulic gradient, the velocity of flow in discontinuity is counted as:

$$v_s = k_d \cdot I \quad (7)$$

$$k_d = \frac{g \cdot (2a_i)^2}{12\nu} \quad (8)$$

and is called "coefficient permeability" of the discontinuity,

$(2a_i)$ is aperture discontinuity,

ν is kinematic viscosity of water and it is usually $1.3 \cdot 10^{-6} \text{ m}^2/\text{s}$ at water temperature of 10°C .

Discontinuities encountered in nature usually are not smooth as it has been assumed so far. According to Lomize (1951) [5], the streamlines in laminar flow of water between two

$$\frac{k}{D_h} \leq 0.032$$

parallel glass plates remain parallel if $\frac{k}{D_h} \leq 0.032$. Such flow is referred to as "irrotational". This implies that flow remains practically uninfluenced by the discontinuity

$$\frac{k}{D_h} \leq 0.032.$$

walls if $\frac{k}{D_h} \leq 0.032$. Such a discontinuity wall is denoted as "hydraulically smooth".

$$\frac{k}{D_h}$$

If $\frac{k}{D_h} > 0,032$ the streamlines diverge and sometimes whirl due to local rotation of water particles. Lomize (1951) [5] found the following relationship between flow rate per unit

width (specific flow) Q_s and mean aperture $2\bar{a}_i$ of rough fissures:

$$Q_s = \bar{v} \cdot 2\bar{a}_i = k_d \cdot I \cdot 2\bar{a}_i = \frac{g \cdot (2\bar{a}_i)^3}{12\nu \cdot f} \cdot I \quad (9)$$

where:

k is maximum amplitude of the roughness wall discontinuity

D_h is hydraulic diameter

$2\bar{a}_i$ is averaged aperture of the discontinuity over the entire surface

f is friction factor.

$$\frac{k}{D_h} = \frac{k}{2 \cdot 2\bar{a}_i} \quad (10)$$

On the basis of flow experiments conducted by Louis (1967) [6], for laminar flow the following relationship has been found for k_d as a function of the relative roughness of the discontinuities' walls:

$$k_d = \frac{g \cdot (2\bar{a}_i)^2}{12\nu} \quad \text{if it's } \frac{k}{D_h} \leq 0.032 \quad (11)$$

$$k_d = \frac{g \cdot (2\bar{a}_i)^2}{12\nu \cdot \left[1 + 8.8 \cdot \left(\frac{k}{D_h} \right)^{1.5} \right]} \quad \text{if it's } \frac{k}{D_h} > 0.32 \quad (12)$$

$$f = 1 \quad \text{if it's } \frac{k}{D_h} \leq 0.032 \quad (13)$$

$$f = 1 + 8.8 \cdot \left(\frac{k}{D_h} \right)^{1.5} \quad \text{if it's } \frac{k}{D_h} > 0.32 \quad (14)$$

Thus, k_d can be expressed as:

$$k_d = \frac{g \cdot (2\bar{a}_i)^2}{12\nu \cdot f^3} = \frac{g \cdot (2\bar{a}_i)_h^2}{12\nu} \quad (2\bar{a}_i)_h = \frac{2\bar{a}_i}{f^{\frac{1}{3}}} \quad (15)$$

2.3. HOMOGENEOUS MODEL

The permeability of jointed rock can be described by means of a homogeneous model. For this purpose, a representative elementary volume (REV) must be defined in which the rock mass can be considered as statistically homogeneous in the sense that an increase of this volume does not change the mean permeability of the rock mass which is referred to as "equivalent permeability" (Baghbanan & Jing 2007) [1].

To evaluate the dimension of the REV discrete models can be helpful. For this purpose, seepage flow through networks of discontinuities so-called "discrete fracture networks" (DFN) has been simulated (Baghbanan & Jing 2007) [1].

In Baghbanan & Jing (2007) a crystalline rock in the Sellafield area (England) with four discontinuity sets was examined by simulating flow through a large number of two-dimensional DFNs using the distinct element model (DEM). In this study DFNs with constant and statistically distributed apertures and trace lengths of discontinuities were investigated. Accordingly, the size of the REV was found to be dependent not only on the mean values of apertures and trace lengths but also on their statistical distribution and degree of correlation. It was shown that homogeneous models are applicable if the statistical distributions of the discontinuity properties exhibit sufficiently low standard deviations. This requirement is normally fulfilled in regularly jointed rock.

3. PERMEABILITY

To describe the permeability of jointed rock with the aid of a homogeneous model we assume impermeable intact rock and laminar flow in the discontinuities. In the same way

as for porous rocks, is introduced seepage flow v_s defined as the quotient of the discharge Q and the cross-sectional area A perpendicular to the direction of flow (Fig.2). The cross-sectional area A incorporates the cross-section of both the discontinuities and the intact rock. Fig.2 illustrates Darcy's law for a rock mass containing one discontinuity set D with persistent, open joints, which are assumed to be oriented parallel to the hydraulic gradient I . The aperture $2a_i$ and the spacing s of the discontinuities are assumed to be constant. According to Wittke (1990) [13], in this case seepage flow is:

$$v_s = \frac{Q}{A} = k_D \cdot I, \quad k_D = k_d \cdot \frac{2a_i}{s} = \frac{g \cdot (2a_i)^3}{12\nu \cdot s} \quad (16)$$

k_D is the coefficient permeability of the discontinuities, which can also be described by means of a cubic law with regard to the aperture of the discontinuities.

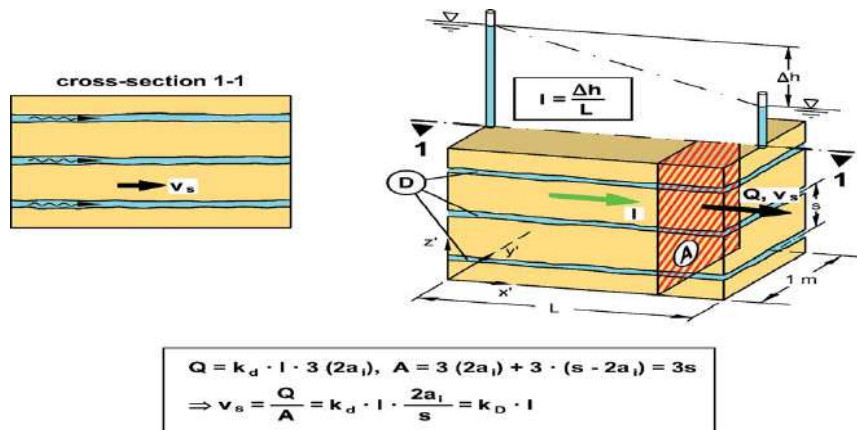


Figure 2. Darcy's law for a rock mass with one set of persistent, open discontinuities

If the aperture and spacing of the discontinuities are not constant, the corresponding values

in the previous equation must be replaced by the hydraulic aperture $(2a_i)_h$ and the mean spacing \bar{s} .

$$k_D = k_d \cdot \frac{(2a_i)_h}{\bar{s}} = \frac{g \cdot (2a_i)_h^2}{12\nu} \cdot \frac{(2a_i)_h}{\bar{s}} = \frac{g \cdot (2a_i)_h^3}{12\nu \cdot \bar{s}} \quad (17)$$

To obtain the hydraulic aperture, the mean apertures and friction factors of all discontinuities must be subjected to an averaging procedure. To illustrate the huge influence of aperture on the magnitude of the permeability of jointed rock, in Fig. 3 the

permeability coefficients k_D parallel to a discontinuity set with constant spacing of $s = 1m$ and with differing mean apertures $2a_i$ and relative roughnesses k/D_h are compared with the permeability coefficients of soil (Witke 1990) [13].

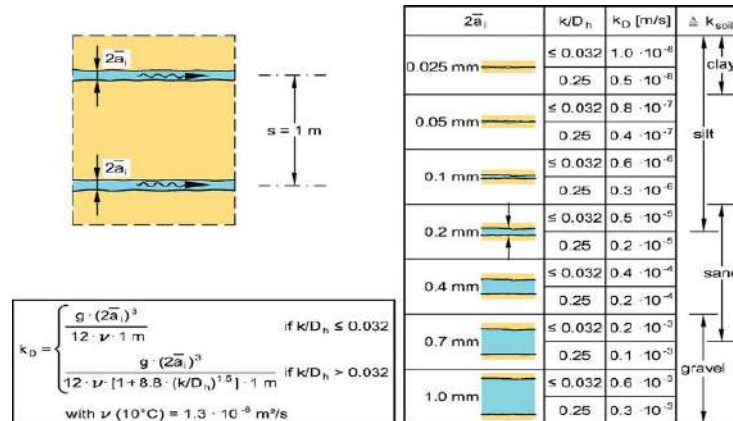


Figure 3. Comparison of coefficients of permeability of a rock mass containing one set of persistent, open discontinuities with those of a soil [15]

A typical example is a rock mass which we encounter in nature is that we have a set of bedding-parallel discontinuities related with a jointed set oriented perpendicular on them. If the traces of the bedding-parallel discontinuities and the joints are oriented parallel to the x axis and y axis, respectively, as illustrated in Fig. 4, the permeability of such a rock mass can be expressed by the following equivalent permeability tensor:

$$[K] = [K_B] + [K_j] = \begin{bmatrix} k_{Bxx} & 0 \\ 0 & 0 \end{bmatrix} + \begin{bmatrix} 0 & 0 \\ 0 & k_{Jyy} \end{bmatrix} = \begin{bmatrix} k_{Bxx} & 0 \\ 0 & k_{Jyy} \end{bmatrix} = \begin{bmatrix} k_{xx} & 0 \\ 0 & k_{yy} \end{bmatrix} \quad (18)$$

where:

k_{xx} and k_{yy} are the permeability of the rock mass parallel and normal to the bedding.

Assuming steady-state, laminar flow through the discontinuities, Doolin & Mauldon (2001) [2] derived an approximation for the permeability coefficient k_{yy} normal to the bedding.

Accordingly, this is dependent on both aperture $(2a_i)_{hBj}$ and spacing s_{Bj} , of the bedding-parallel discontinuities as well as aperture and spacing of the joints $(2a_i)_{hJij}$ and s_{Jij} (Fig. 4).

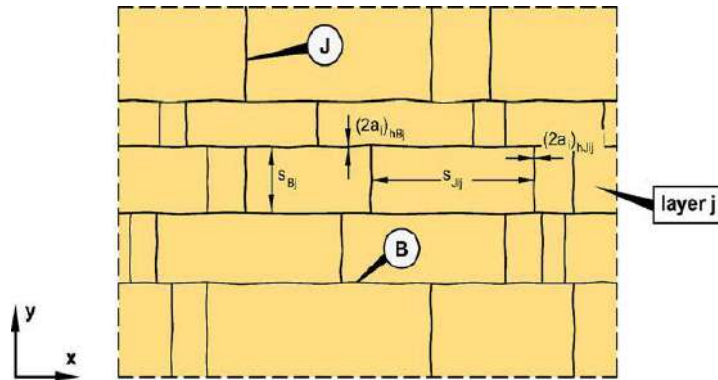


Figure 4. Rock mass with persistent horizontal bedding-parallel discontinuities and vertical joints of negative exponential spacing ending at the bedding [2]

When the individual values of these quantities are replaced by the mean values, $(\bar{2a}_i)_{hj}$, $(\bar{2a}_i)_{hB}$, \bar{s}_B , \bar{s}_J , k_{yy} can be calculated as:

$$k_{yy} = \frac{g \cdot c \cdot \bar{s}_B \cdot (\bar{2a}_i)_{hB} \cdot (\bar{2a}_i)_{hj}^3}{12\nu \cdot [c \cdot \bar{s}_B \cdot \bar{s}_J \cdot (\bar{2a}_i)_{hB} + \bar{s}_J^2 \cdot (\bar{2a}_i)_{hj}]} \quad (19)$$

The permeability parallel to the bedding can be expressed by

$$k_{xx} = k_B = \frac{g \cdot (\bar{2a}_i)_{hB}^3}{12\nu \cdot \bar{s}_B} \quad (20)$$

Equations reflect connectivity of joints and bedding-parallel discontinuities. c - is a factor depending on the distribution of the joints' spacing (Doolin & Mauldon 2001) [2]. If the spacing constant then $c = 2$ is valid. If the spacing of the joints follows a negative exponential distribution which has been validated in a large number of discontinuity measurements in the field then $c = 4$ (Priest & Hudson 1976, Beacher et al., 1977, Hudson & Priest 1979, Priest & Hudson 1981, Wittke 1990) [12] [13].

If $(\bar{2a}_i)_{hB} \gg (\bar{2a}_i)_{hj}$ the permeability k_{yy} normal to the bedding is dominated by the permeability k_j of the joints and can be approximated by (Doolin & Mauldon 2001) [2].

$$k_{yy} = k_j = \frac{g \cdot (\bar{2a}_i)_{hj}^3}{12\nu \cdot \bar{s}_J} \quad (21)$$

In this case the connectivity of joints and bedding-parallel discontinuities can be neglected.

If $(2\bar{a}_i)_{hB} \ll (2\bar{a}_i)_{hJ}$, k_{yy} is controlled by the permeability k_B of the bedding-parallel discontinuities. k_{yy} can be approximated (Doolin & Mauldon 2001) [2].

$$k_{yy} = c \cdot \left(\frac{\bar{s}_B}{\bar{s}_J} \right)^2 \cdot k_B = c \cdot \left(\frac{\bar{s}_B}{\bar{s}_J} \right)^2 \cdot \frac{g}{12\nu} \cdot \frac{(2\bar{a}_i)_{hB}^3}{\bar{s}_B} \quad (22)$$

4. DETERMINATION OF WATER - PERMEABILITY IN ROCK MASS

Two methods for determining the water-permeability of the rock mass are described below:

- Lugeon test
- Tracer test.

4.1. LUGEON TEST

Lugeon test, sometimes referred to as the Packer test, is a on-site test method for estimating the average hydraulic conductivity of the rock mass. The test is named after Maurice Lugeon (1933), a Swiss geologist who first formulated the test. Lugeon test is performed in an isolated borehole. The results provide information on the hydraulic conductivity of rock mass including intact rock and discontinuity.

4.1.1. Description and procedure

The Lugeon test can be carried out in boreholes with arbitrary orientation. The borehole diameter should be at least 46 mm. The test section may be located below or above the groundwater table. The test is carried out in the borehole section which is isolated by pneumatic packer. The water is injected in isolated part of borehole. The packers can be inflated using a gas compressor on the surface, and so they can isolate and seal that portion of the borehole. A pressure transducer is also located in that portion to measure the pressure with a help of reading station on the surface. The sealing length should not fall below 0.5 m, usually of length from 1 m to 5 m. The testing device consists of a water supply system comprising a water reservoir, a pump, a flow meter, a regulating valve and a delivery line to carry water from the pump down to the borehole and into the test section (Fig.5). Prior to the beginning of the test a maximum test pressure (Pmax). Pmax is chosen such that it does not exceed the confinement stress (σ_3) expected at the depth (D) where the test is being conducted, thus avoiding the development of hydraulic fracturing or hydraulic jacking:

$$P_{\max} = D \cdot 0.226 \text{ bar/m} \quad (23)$$

The test is conducted in five stages, with a particular water pressure magnitude associated with each stage. A single stage consists of keeping a constant water pressure at the test interval for 10 minutes by pumping as much water as required. The first stage is held at a low water pressure, increasing the pressure in each subsequent stage until reaching Pmax. Once Pmax is reached, pressures are decreased following the same pressure stages used on the way up, thus describing a “pressure loop”. Table 1 shows the pressure magnitudes customarily used during the five test stages.

Table 1. The magnitude of pressure typically used for Lugeon test

Test stage	Pressure
1 st	0.5 Pmax
2 nd	0.75 Pmax
3 rd	Pmax
4 th	0.75 Pmax
5 th	Pmax

During the execution of each stage, both water pressure (P) and flow rate (Q) values are recorded every minute. Subsequently, average values for P and Q are then used to compute the hydraulic conductivity for each stage. The hydraulic conductivity is expressed in terms of the Lugeon value, which is empirically defined as the hydraulic conductivity required to achieve a flow rate of 1 liter/minute per meter of test interval under a reference water pressure equal to 1 MPa (10 bar). Lugeon value is defined by next equation [6]:

$$Lugeon\ value = \left(\frac{Q}{L} \right) \times \left(\frac{P_0}{P} \right) \quad (24)$$

where:

Q is flow rate (lit/min),

L = Length of the borehole test interval,

P₀ is reference pressure of 1 MPa,

P is test pressure (MPa).

Lugeon's value is not only representing the hydraulic conductivity of the rock mass but can also be a condition indicator on the joints of the rock, as can be seen in Table 2.

Table 2. Condition of rock mass discontinuities associated with different Lugeon values

Lugeon range	Classification permeability	Hydraulic conductivity range (cm/sec)	Condition of rock mass discontinuities
< 1	Very low	$< 1 \times 10^{-5}$	Very tight
1-5	Low	$1 \times 10^{-5} - 6 \times 10^{-5}$	Tight
5-15	Moderate	$6 \times 10^{-5} - 2 \times 10^{-4}$	Few partly open
15-50	Medium	$2 \times 10^{-4} - 6 \times 10^{-4}$	Some open
50-100	High	$6 \times 10^{-4} - 1 \times 10^{-3}$	Many open
>100	Very high	$> 1 \times 10^{-3}$	Open closely spaced or voids

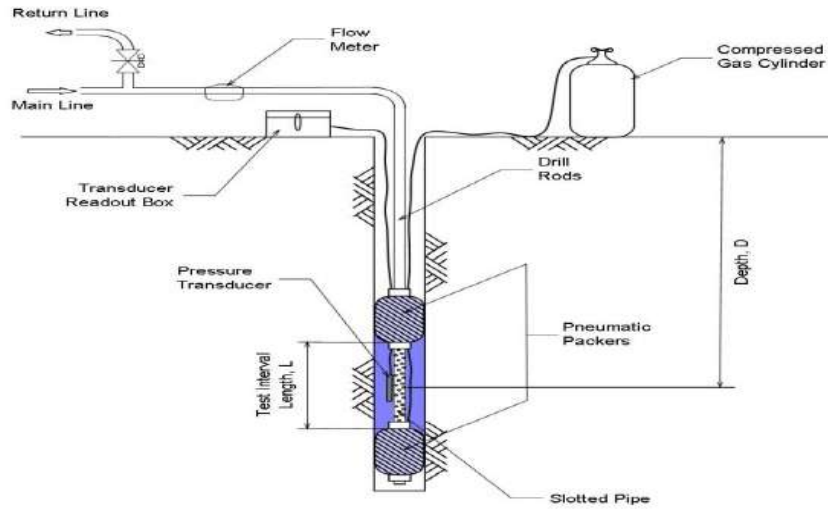


Figure 5. Lugeon test configuration

With Lugeon tests, as mentioned above, only the equivalent rock mass permeability in the direction normal to the borehole can be measured, so tests in boreholes of different orientation may provide an indication of rock mass anisotropy. In many cases and with the aid of additional information obtained from the evaluation of core drillings and the observation of borehole walls with respect to discontinuity data, it may be possible to measure the permeability of a discontinuity set. For this purpose, Lugeon tests are carried out in a borehole oriented perpendicular to the set. Figure 6 shows the determination of the local permeability of a horizontal discontinuity set D by means of a Lugeon test carried out in a vertical borehole assuming laminar and radial flow (Wittke 1990)[13]. The permeability coefficient k_D can be calculated according to replacing k_{RM} by k_D (Fig. 6, DGEG 1984c, Wittke 1990)[13].

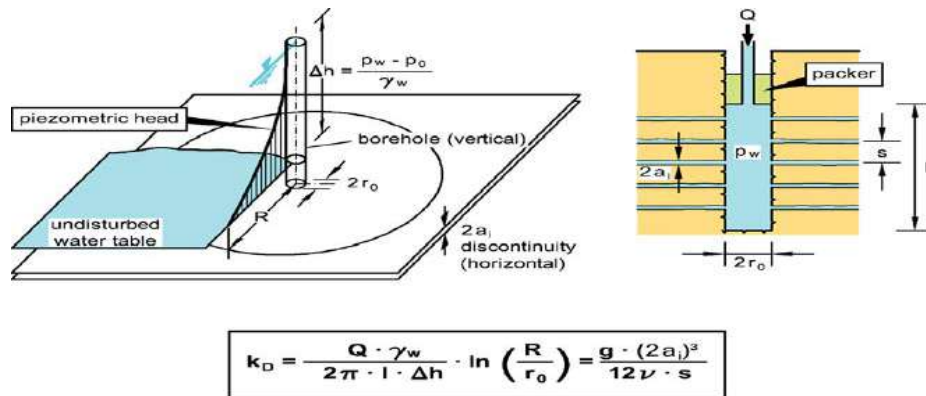


Figure 6. Lugeon test, determination of the permeability of a horizontal discontinuity set in a vertical test hole [13]

4.2. TRACER TEST

Testing with tracer consists from that you insert tracer into borehole or well. Like tracer usually is using uses fluorescent dyes. Dilution rate of the tracer like the time appearing of the tracer in control well, can be used to calculate the permeability as it has been described (Lewis, 1966) [4] and (Maini, 1971) [8]. Monitoring or precision measurement of the concentration can be performed manually (by probing sampling) or by probes.

The test traser is based on the assumption that we have a steady-state seepage flow of ground water intersection through the vertical well, the tracer moves under the influence of the hydraulic gradient and the flow is conducted according to Darcy's law:

$$v = k \cdot I \quad (25)$$

where:

I is hydraulic gradient which can be determined by observing the water level in wells (injected and controlled) and the distance between the wells (borehole).

The flow velocity is obtained from the equation:

$$v = \frac{l}{t} \quad (26)$$

where:

l is distance between boreholes,

t is tracer travel between boreholes.

In analyzing a tracer dilution test, the flow velocity is related to the rate at which the tracer concentration diminishes within the test section of the injection well. For an assumed homogeneous isotropic porous medium (Lewis, 1966) [4] gives the following equation of velocity:

$$v = \frac{\pi \cdot w_d}{8 \cdot t_d} \cdot \ln(C_r) \quad (27)$$

where:

w_d is well diameter,

C_r is ratio of the final to the initial tracer concentration,

t_d is dilution time period.

(Maini, 1971) [8] was in analysis of tracer dilution test in a fissure system which he applied same criterion like analysis of flow between parallel plate. It is assumed that a set of equally spaced horizontal fissures, each with the same equivalent parallel plate aperture e , intersect the test section of the injection well. The flow velocity through the fissures is given by

$$v = \frac{\pi \cdot w_d \cdot \ell}{8 \cdot n \cdot e \cdot t_d} \cdot \ln(C_r) \quad (28)$$

where:

n is number of fissures intersecting the test section,

ℓ is test section length.

Coefficient jointed permeability is defined equation:

$$k_j = \frac{\gamma_w \cdot e^2}{12\mu_w} \quad (29)$$

where:

γ_w is unit weight of water,

μ_w is dynamic viscosity of water.

Test of tracking the travel time from tracer includes large portion of the rock mass, reflecting high or low permeability zones in order to determine the average coefficient of permeability. Test of dilution is convenient as such tests are rapid, conducted in one hole. These methods avoid unnatural conditions which can occur at testing under the pressure of water. The main disadvantage of tracer tests is their limitation to use below the groundwater table.

5. TRANSIENT SEEPAGE FLOW

The equation describing transient seepage flow is based on the application of the equation continuity condition and Darcy's law and has a shape (Wittke, 2000b) [14]:

$$\frac{\partial}{\partial x} \left(k_{xx} \cdot \frac{\partial h}{\partial x} + k_{yy} \cdot \frac{\partial h}{\partial y} + k_{zz} \cdot \frac{\partial h}{\partial z} \right) + \frac{\partial}{\partial y} \left(k_{yx} \cdot \frac{\partial h}{\partial x} + k_{yy} \cdot \frac{\partial h}{\partial y} + k_{yz} \cdot \frac{\partial h}{\partial z} \right) + \frac{\partial}{\partial z} \left(k_{zx} \cdot \frac{\partial h}{\partial x} + k_{zy} \cdot \frac{\partial h}{\partial y} + k_{zz} \cdot \frac{\partial h}{\partial z} \right) = S_0 \cdot \frac{\partial h}{\partial t} \quad (30)$$

where:

S_0 is specific coefficient storage defined as:

$$S_0 = \frac{\partial n_{eff}}{\partial h}. \quad (31)$$

6. STEADY - STATE SEEPAGE FLOW

The equation of steady-state seepage flow in a rock mass is obtained by setting $\frac{\partial h}{\partial t} = 0$:

$$\{\nabla\}^T ([K] \cdot \{\nabla\} h) = 0 \quad (32)$$

The following applies if the permeability of the rock mass is homogeneous:

$$[K] \cdot \{\nabla\}^T \cdot \{\nabla\} h = 0 \quad (33)$$

If the permeability of the rock mass is not only homogeneous but also isotropic, the distribution of piezometric heads is independent of permeability

$$\{\nabla\}^T \{\nabla\} h = 0 \quad (34)$$

Therefore, the specific storage coefficient of homogeneous unconfined aquifers can be expressed as:

$$S_0 = \frac{n_{eff}}{H} \quad (35)$$

where:

H is thickness aquifer.

If H is the thickness of the water-bearing rock formation that is affected by seepage flow, for example, the thickness of an aquifer penetrated by a well, the effective porosity n_{eff} can be expressed as

$$n_{eff} = \int_{h=0}^{h=H} S_0 dh \quad (36)$$

The effective porosity of jointed rock corresponds to the volume of open interconnected discontinuities V_D related to the reference volume V referred to as the relative joint volume:

$$n_{eff} = n_D = \frac{V_D}{V} \quad (37)$$

For a rock mass with m - orthogonal discontinuity sets ($n = 1, 2, 3$) with mean aperture $2\bar{a}_i$ and mean spacing \bar{s} , n_D may be calculated by way of approximation as:

$$n_D \approx \frac{m \cdot (2\bar{a}_i)}{\bar{s}} \quad (38)$$

Thus, for this particular case the following relationship is obtained:

$$S_0 = \frac{\partial n_D}{\partial h} = \frac{n_D}{H} \approx \frac{m \cdot (2\bar{a}_i)}{\bar{s} \cdot H} \quad (39)$$

The specific storage coefficient of a joint aquifer is normally considerably smaller than that of a pore aquifer. As a consequence, the time required for a lowering of the water table in a joint aquifer is much smaller than that in a pore aquifer of the same permeability.

The drawdown of the water table $\Delta h(r, t)$, due to a pumping rate Q_p around a well as a function of distance r from the wells axis and time t in a homogeneous aquifer with isotropic permeability k can be calculated as follows (Theis 1935) [9]:

$$\Delta h(r, t) = \frac{Q_p}{4\pi \cdot k \cdot H} \cdot W(u) \quad (40)$$

$$W(u) = \int_u^{\infty} \frac{e^{-v}}{v} dv \quad (41)$$

$$u = \frac{S_0 \cdot r^2}{k \cdot 4t} \text{ is the so-called well function.} \quad (42)$$

where:

S_0 - is the specific storage coefficient of the aquifer.

The upper equation is valid if the level of groundwater table is smaller compared to with the thickness H of the aquifer penetrated by the well. In this case, the vertical velocity component of the seepage velocity can be neglected. If $u < 0.01$, it can be approximated as

$$\Delta h(r, t) \approx \frac{Q_p}{4\pi \cdot k \cdot H} \cdot \ln \left(\frac{2,25 \cdot k \cdot t}{r^2 \cdot S_0} \right). \quad (43)$$

If we take it $\Delta h(r, t) = 0$, will get $(r) = R$ radius of influence on well (borehole), in time t :

$$r(t) \approx \sqrt{\frac{2,25 \cdot k \cdot t}{S_0}} \Rightarrow t = t_0 = \frac{R^2 \cdot S_0}{2,25 \cdot k}. \quad (44)$$

To estimate the radius of influence $(r) = R$ on borehole (well) when requires drawdown Δh , using a formula for well gotten from (Dupuit, 1863) [3], (Thieme, 1870.) [10], which is valid at steady state of seepage flow

$$Q_p = \frac{\pi \cdot (H^2 - h_0^2) \cdot k}{\ln R - \ln r_0} \quad (45)$$

where is:

$$h_0 = H - \Delta h_0. \quad (46)$$

7. CONCLUSION

The presence of water in rock masses respectively permeability is a great influence on the realization of many engineering projects. In order to define the water permeability of the rock mass, it is necessary to have data on characteristics of the same, among which are the most important characteristics of discontinuity. The assumption is that through the jointed rock permeability the estimation takes place only through discontinuities. The flow of water through the rock mass is therefore a function of discontinuity, their connection and the size of discontinuity aperture. Due to the low water particle velocity, in most cases the flow of water in discontinuities can be considered laminar. What needs to be emphasized is that the discontinuities we encounter in nature are usually not as smooth as we suppose. Therefore, the conditions of turbulent flow occur only at relatively large discontinuity apertures or at high hydraulic gradients, and the assumption of laminar flow which is

reasonable for most practical problems in rock engineering. One way to establish a model of permeability and seepage flow currents in a jointed rock is to simulate any discontinuity in rock mass with aperture, roughness, filling. Permeability is then treated as a flow through a network of discontinuities. The description of the water permeability jointed rock by using a homogeneous model can be assumed to be the impermeability of the intact rock and the laminar flow through the discontinuities, respectively it was shown that homogeneous models are applicable if the statistical distributions of the discontinuity properties exhibit sufficiently low standard deviations. This requirement is normally fulfilled in regularly jointed rock.

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