1. Theoretical background.

1.1 Pump tests.

1.1.1 Introduction

The term pump test is referred to the body of procedures used to size correctly a water well, based on execution of pumping at constant discharge and on measuring of the drawdown of the water level inside the well.
The main aim is to identify the characteristic curve of the well, that's the curve which correlates the pumping discharge to the water level drawdown inside the well.
Pump tests are usually performed through a set of short-time discharge steps (1-3 hours), to the end of which the final drawdown is measured. When a stable level is reached, it follows a break in the water pumping to allow water to rise up to the initial level. The first discharge step is generally set to the minimum discharge of the pump, next steps have to be twice, three times, etc. the first-step discharge. Execution of the first step has to be preceded by a pumping which allows to empty the well (well capacity effect).
Test steps have to be at least three in case of confined aquifer and four in case of phreatic aquifer. More steps allow to reach a better resolution.
Necessary conditions for the validity of the test are the following:

- it be inside the conditions of validity of the Darcy's law;
- it be a fully penetrating well, that's the draining zone has to reach at least 80% of the aquifer thickness;
- the well has to be correctly executed and equipped;
- the undisturbed piezometric surface has to be horizontal;
- the pumping discharge, during the execution of a single step, has to be effectively constant (maximum allowed error around to 5%);
- the well radius has to be as small as possible

1.1.2 Analysis of the characteristic curve of the well.

Drawing on a linear graphic the values of the discharge steps along the X axis and the final drawdowns along the Y axis, the characteristic curve of the well is generated. This curve can be analytically expressed through the formula:

$$(1)\ s = B \times Q + C \times Q^n;$$

where

s (m) = final drawdown to the end of the discharge step Q;
Q (mc/s) = value of the discharge step;
B = constant value linked to the laminar component of the outflow;
C = constant value linked to the turbulent component of the outflow;
n = exponent usually set equal to 2(Jacob, 1946).

First tract of the curve is generally overlapping to a straight line with equation:

$$(2)\ s = B \times Q.$$

In fact for small pumping discharges the water outflow is of laminar type, whereas the turbulent component is negligible. B value in formula (2) is function of both the hydrogeological parameters of the aquifer (trasmissivity and storage coefficient) and the well characteristics (well screen and sand or gravel filter).
As the discharge increases, the second member of the formula (1) gets dominant. When the outflow velocity gets higher than the critical velocity, that's it passes from a laminar otuflow to a turbulent one, formula (1) gets close to the following form:

$$(3)\ s = C \times Q^n.$$

C value is only function of the characteristics of the well, since independent by the hydrogeological parameters of the aquifer.

The rising of the second member in formula (1) ($C \times Q^n$) causes a loss of the well efficiency, because, when it gets dominant, a small increasing of discharge can result high drawdowns of the water level and a rise of the turbulent outflow, causing dragging of fine soil particles into the well. Pumping discharge for which the second member of the (1) gets dominant is called critical discharge. Then it's possible to define the efficiency of a well, relative to a specific discharge, through the expression:

$$e\% = 100\frac{BQ}{BQ + CQ^n}$$

that's the ratio, express as percentual, between the laminar and total components of the drawdown. Water discharge for which efficiency is exactly 50% represent the critical discharge. Efficiency less than 50 indicates the turbulent outflow is prevalent.
The form of the curve is almost straight where the laminar outflow is dominant and convex where the turbulent outflow is prevalent. In case of concave curve the test is not valid, due to errors in executing measurements or to not applicability of the test conditions seen above.
Characteristic curve tends to modify itself along the well life, displaying generally a rise during the time of the turbulent component , due to, for example, the clogging of the sand or gravel filter caused by finer particles or to the scaling in screen. To estimate the maximum discharge it can be pumping, it has to take in account decreasing of efficiency during the well lifetime.

1.1.3 Estimating B and C parameters by characteristic curve.

Jacob's method (1946)

Assuming that characteristic curve could be expressed in the following form:

$$s = B \times Q + C \times Q^2,$$

where n is set equal to 2, parameters B and C can be computed through the discharge-specific drawdown line, given by:

$$(4)\ s/Q = B + C \times Q;$$

in which the term s/Q is known as specific drawdown (sq). In the discharge-specific drawdown graph, the points relative to the executed measurements tend to dispose themselves along a straight line, having an angular coefficient corresponds to the C value. So C is given taking two points along the straight line and computing the ratio:

$$C = (sq_2 - sq_1) / (Q_2 - Q_1);$$

where:

sq = specific drawdown;
Q = discharge.
B can be computed through the intersection with the Y axis (Q=0).

As an alternative, to take in account the dispersion of the measurements, B and C can be computed by the method of the least squares.

$$C = \sum Q_i \times sq_i / \sum Q_i \times Q_i;$$

$$B = sq_{mean} - C \times Q_{mean}.$$

Analysis of the line expressed by the formula (4) allows to readily identify the outflow characteristics inside the well.

- If the curve passes close to the origin (B=0), the outflow is of turbulent type.
- If the curve is parallel to the Y axis (C x Q =0) the outflow is of laminar type.

Rorabaugh's method (1956)

Rewriting the (1) in the form:

$$\ln\left(\frac{s}{Q}-B\right)=\ln C+(n-1)\ln Q$$

where ln is the natural logarithm, the equation of a straight line having angular coefficient (n-1) is given. Parameter C results from the intersection of the curve with the Y axis. The angular coefficient has to be found instead, varying B to attempts till when the straight line, in logarithmic scale, overlaps the measurement data in a satisfactory manner. The variation interval of B is however small, for it has to satisfy the relation $0 \leq B < s/Q$: negative values of B have not physical meaning, values equal or higher than s/Q are not valid, because they would give negative or null the term (s/Q – B).

Method of the least squares.

Analysis of the curve (1) can be performed, using the method of the least squares. They have to find the B, C and N values which minimise the value of the following expression:

$$\Phi=\sum\left|s_i-s'_i\right|^2$$

where s is the measured drawdown and s' is the theoretical one, computed, with the same discharge value, through the formula (1). By the solution proposed by Dragoni (1990), B and C can be calculated through the following expressions:

$$C=\frac{\left(\sum s_i Q_i^{n'}\right)\sum Q_i^2-\left(\sum s_i Q_i\right)\sum Q_i^{(n'+1)}}{\sum Q_i^{2n'}\sum Q_i^2-\sum Q_i^{(n'+1)}\sum Q_i^{(n'+1)}}$$

$$B=\frac{\left(\sum s_i Q_i-C\sum Q_i^{(n'+1)}\right)}{\sum Q_i^2}$$

using a value of n, here called n', set at attempts, which varies in the interval 0-12, interval inside which generally the n values are included.

1.1.4 Cases with anomalous curve.

Discharge-drawdown curve occasionally may not have the form (1). It may happen that the three proposed methods not be able to interpolate in a satisfactory manner the experimental data. In these cases an estimating of B can be gotten, applying the Rorabaugh or the least squares methods only to the lower discharge steps, whereas C and n can be evaluated, using the same methods to the higher discharge steps.

1.2 Estimating pollutant dispersivity.

1.2.1 Test with two boreholes and continuous injection (Fried, 1975).

It works out through two boreholes, in the first one a constant water discharge, mixed to the chemical tracer, is injected, in the second one, downstream located, the same discharge is pumped to create a steady-state flow condition. In the downstream borehole the tracer concentration is scanned at regular temporal intervals. Calling C_{max} the maximum measured concentration of the tracer, it can be drawn on a graph the trend of the C/C_{max} ratio as a function of time, where C is the measured concentration in a specific instant.
Assuming that the generated curve is gaussian, its standard deviation can be defined through the expression:

$$\sigma_t = \frac{(t_{84} - t_{16})}{2}$$

where t_{84} and t_{16} are, respectively, the instants when concentrations equal to $0.84C_{max}$ and $0.16C_{max}$ are measured. The longitudinal coefficient of dispersion, along the flow direction, is given by:

$$D_L = \frac{v^2 \sigma_t^{\,2}}{2t}$$

where v is the flow velocity, given by the product of the soil permeability by the hydraulic gradient, and t is the instant when a concentration equal to $0{,}5C_{max}$ is measured.
The longitudinal dispersivity is instead calculable through the expression:

$$\alpha_L = \frac{D_L}{v}.$$

1.2.2 Test with two boreholes and intermittent injection (Fried, 1975).

The procedure is the same depicted in the previous case, except that the injection of the chemical tracer is not executed in a continuous way. The expression which gives the longitudinal coefficiente of dispersion if the following:

$$D_L = \frac{\frac{\left[d^2 - v^2 t_{\max}\left(t_{\max} - t_0\right)\right]}{2}}{\left(t_{\max} - t_0\right)}$$

where d is the distance between the two boreholes, t_0 is the duration of the injection and t_{max} is the instant when the maximum concentration of the tracer is measured. The longitudinal dispersivity is given by the formula:

$$\alpha_L = \frac{D_L}{v}.$$

1.3 Estimating the hydrogeological parameters of the aquifer.

1.3.1 Introduction.

A pumping test is a test in which pumping has long duration (42 hours at least), with a one only step of discharge, whose purpose is:
- to estimate the hydrogeological parameters of the aquifer, mainly trasmissivity and storage coefficient;
- to find, if present, possible impervious or inflow limits of the aquifer and its condition of homogeneity.

Interpreting pumping tests usually involves two possible models of development of the depression cone around the well:
- steady-state flow model: it assumes that, after a relatively short period of pumping , the depression cone reaches a practically constant shape and width
- unsteady-state flow model: it assumes that width of the depression cone progressively raises up as a function of the pumping time.

The unsteady-state model fits better the experimental data and consequently is more used. A semi-permanent condition, when increasing of the cone sizes is extremely slow and gradual, might be happen in case of very long pumping time.
As regarding the conditions of execution of the test, they are the same above depicted relatively to the pump test.

1.3.2 Estimating the hydrogeologial parameters..

A) Steady-state flow.

Operating with a steady-state flow model, we can calculate the aquifer transmissivity, but not the storage coefficient.
The test is executed, measuring, at the end of the discharge step, the drawdowns reached inside the piezometers. Having more piezometers along several measurement lines, we can evaluate possible heterogeneity of the aquifer, estimating a transmissivity value for every line.

The test results are drawn on a semi-logarithmic graph, where they stand the piezometer distances along the X axis, in logarithmic scale, and the water drawdown along the Y-axis.
The calculation procedure is different depending on aquifer is phreatic or confined.

I) Confined aquifer (Thiem's formula).

The average transmissivity of the aquifer can be estimated by the expression:

$$T_{av} = 0.366 \times Q / \Delta s;$$

where:

T_{av}(mq/s) = average transmissivity of the aquifer;
Q (mc/s) = test discharge;
Δs(m) = water drawdown relative to a logarithmic cycle (decimal logarithm).

II) Phreatic aquifer (formula di Thiem).

The formula is the same seen in the previous case:

$$T_{av} = 0.366 \times Q / \Delta s;$$

where:

T_{av}(mq/s) = average transmissivity of the aquifer;
Q (mc/s) = test discharge;
Δs(m) = water drawdown relative to a logarithmic cycle (decimal logarithm).

Three cases are possible:

a) $s / H_{fw} \leq 0.05$.

where:

s (m) = measured drawdowns;
H_w (m) = aquifer thickness.

In this case no corrections have to be applied to the formula. The coefficient of permeability can be simply estimated dividing T by H_w :

$$K = T / H_w$$

b) 0.05< s / H_w ≤ 0.15.

In this case the measured drawdowns have to be corrected through the following expression:

$$s = s - s^2 / (2 \times H_w);$$

This correction has to be taken in account, being the discharge equal, because the measured drawdowns in a phreatic aquifer are higher than in a confined aquifer. In this case as well, the coefficient of permeability is given by:

$$K = T / H_w$$

c) s / H_w >0.15.

If this condition occours, we cannot calculate T directly thorugh the Thiem's formula. The permeability coefficient can be estimated by the following formula:

$$k = (1/\pi)Q\ln(r_2/r_1)/\sqrt{[}(H_{falda} - s_1) - sqr(H_{falda} - s_2)];$$

where:
s_1, s_2 = measured drawdowns inside the piezometers 1 and 2;
H_{falda} = H_w;
r_1, r_2 = distances of the piezometers 1 and 2 from the well.

An appromaxitevely value of T might be derived by the following formula:

$$T = K\,[(H_w - s_1) + (H_w - s_2)]/2;$$

B) Unsteady-state flow.

Operating with a unsteady-state flow model, we can calculate both the transmissivity and the storage coefficient.
The pumping test is executed, measuring, with exponential-increasing time steps, the drawdown inside the piezometers and the well. At the end of the test, pumping is stopped and the residual drawdowns (recoveries) are measured, using the same time steps. Having more piezometers, we can evaluate the possible heterogeneity of the aquifer.
The test is depicted on a semilogaritmic graph, where time is along the X axis, in logarithmic scale, and the drawdowns along the Y axis.
The generated curve is generally close to a straight line, at least in case of illimitated aquifer. Only in the initial tract, the curve may be different by a straight line for the capacity effect of the well, which makes an outflow of turbulent kind.
The calculation procedure is different depending on aquifer is phreatic or confined.

I) Confined aquifer.

Theis's formula

The theoretical drawdown is given by (1):

$$s = \frac{Q}{4\pi T} W(u)$$

where W(u) is the well function:

$$W(u) = -0.577216 - Log(u) + u - \frac{u^2}{2 \cdot 2!} + \ldots + \frac{u^n}{n \cdot n!}$$

and (2):

$$u = \frac{r^2 S}{4Tt}$$

r = distance of the piezometer from the well;
t = time since pumping beginning.

The well function W(u) is drawn on a bi-logarithmic graph, overlapping it, by a trial and error process, to the experimental curve (s-t curve). In the overlapping range of the two curves, a common point is chosen. The corresponding values of u and W(u) are read along the axes. Substituting W(u) in (1) and u in (2), the parameters T and S are derived.

Jacob's formula
Taking in account only the first term of the series expansion of W(u), we obtain the simplified expression by Jacob.
This formula allows to evaluate the aquifer average transmissivity through the relation:

$$(1) T_{av} = 0.183 \times Q / \Delta s;$$

where:

T_{av}(mq/s) = average transmissivity;
Q (mc/s) = test discharge;
Δs (m) = drawdown relative to a logarithmic cycle (decimal logarithm).

The storage coefficient is given by:

$$S_{av} = 2.25 \times T_{av} \times t_0 / r^2;$$

where:

S_{av} = average storage coefficient;
t_0 (s) = time given by the intersection of the curve with the time axis;
r (m) = distance of the reference piezometer from the test well.

In case of measurements performed inside the test well, the calculated values of S cannot be considered reliable.
Transmissivity can be also gained through measuring of the recovery after stopping the pumping, using the expression (1). In this case, however, it cannot estimate the storage coefficient S.

II) Phreatic aquifer

Formulas by Theis and Jacob

It works as the confined aquifer, taking in account two different cases.

a) $s / H_w \leq 0.05$.

where:

s (m) = measured drawdowns;
H_w (m) = aquifer thickness.

In this case no corrections have to be applied to the formulas.

b) $0.05 < s / H_w \leq 0.15$.

In this case the measured drawdowns have to correct through the following formula:

$$s = s - s^2 / (2 \times H_{falda});$$

When $s / H_{fw} > 0.15$ T and S cannot be calculated.
Transmissivity can be also gained through measuring of the recovery after stopping the pumping, using the expression (1). In this case, however, it cannot estimate the storage coefficient S.

C) Laterally limited aquifer.

The calculation methods seen above are relative to laterally unlimited aquifer.

In case of laterally limited aquifer, due to either the end of the aquifer layer against a impervious boundary (impervious boundary condition) or the water inflow from a surface source (water channel, lake, etc.) (inflow limit condition), interpretation should be executed only on the straight-line tract of the test curves, drawdown-time curve or drawdown-distance curve.
The existence of a boundary condition displays itself on the curves by the appearance of a second straight-line tract, having a different slope. Inclination of this second interval is higher than the first one when the boundary is impervious and lower in case of water inflow from a surface source
Theoretical distance between the test well and the aquifer boundary can be estimated through the following formulas:

$$d = (x / 2) \sqrt{(t_i / t_0)} \text{ (impervious boundary);}$$

$$d = (x / 2) \sqrt{(t_i / t_0)} + x/2 \text{ (inflow boundary);}$$

where:
t_i = time of intersection of the two straight-line tracts;
t_0 = time of intersection of the first track with the time axis..

1.3.3 Estimating the influence radius of a pumping well.

Steady-state flow

In a steady-state flow condition the radius of influence of a pumping well can be calculated by the following expression:

$$Rf(m) = 3000 s \sqrt{k}$$

where:
s(m) = measured drawdown;
k(m/s) = aquifer coefficient of permeability.

To estimate the shape of the depression cone as a function of the distance from the pumping well, it can use the formula:

$$s_r = \frac{Q}{2\pi T} \ln \frac{Rf}{r}$$

s_r(m) = drawdown at the distance r from the well;
Q(mc/s) = pumping discharge;
T(mq/s) = aquifer transmissivity.
Distance r cannot be set equal to 0. As minimum value, it has to be used a value equal to the well radius. This formula is valid for confined aquifer and for phreatic aquifer where s/H<0.15.

Unsteady-state flow

In an unsteady-state flow condition the radius of influence of a pumping well is a function of time since the beginning of the pumping. In this case Rf can be calculated by the following expression:

$$Rf(m) = 1{,}5\sqrt{\frac{Tt}{S}}$$

where:
T(mq/s) = aquifer transmissivity;
t (s) = time;
S = storage coefficiente.
To estimate the shape of the depression cone as a function of the distance from the pumping well and of time since the beginning of the pumping, it can use the formula:

$$s_r = 0{,}183 \frac{Q}{T} \log_{10} \frac{2{,}25Tt}{Sr^2}$$

s_r(m) = drawdown at the distance r from the well;
Q(mc/s) = pumping discharge;
t(s) = time.

Distance r cannot be set equal to 0. As minimum value, it has to be used a value equal to the well radius. This formula is valid for confined aquifer and for phreatic aquifer where $s/H<0.15$.

1.4 2D groundwater-flow simulation.

1.4.1 Processing the path a water particle inside the aquifer.

In the hypothesis of an homogeneous and unlimited aquifer, we can develop an analytical solution of the differential equations depicting the motion of a fluid across a porous medium. Practically this solution allows to simulate the motion of a single particle, by water or by an other liquid, subject to the influence of pumping wells along a XY surface.

We start out from the assumption that the particle motion is initially undisturbed and that they stream along the X axis with constant velocity. In case of water particles this velocity can be calculated through the product *k x i*, where *k* is the coefficient of permeability of the aquifer and *i* its hydraulic gradient. Average velocity along the Y axis is set equal to zero.

When the water particle reaches the influence radius of pumping wells, the X and Y components of velocity undergo a change as follow (Bear & Verruijt, 1987):

$$v_x = v_{0x} + \sum_{i=1}^{n} \left[\frac{Q_i}{4naH} \left(\frac{N_x}{D_1} + \frac{N_x}{D_2} \right) \right]$$

$$v_y = \sum_{i=1}^{n} \left[\frac{Q_i}{4naH} \left(\frac{N_{y1}}{D_1} + \frac{N_{y2}}{D_2} \right) \right]$$

where: n = number of pumping wells;

Q_i = discharge of the i-th well, setting the sign– if the well is pumping, the sign+ if the well is injecting;

a = width of the area (along Y axis);

H = aquifer thickness;

v_{0x} = initial velocity of the particle along the X axis;

N_x = senh[π(x - x_i) / a];

senh = hyperbolic sine;

x = x of the particle;

x_i = x of the i-th well;

D_1 = cosh[π(x - x_i) / a] - cos[π(y - y_i) / a];

cosh = hyperbolic cosine;

y = y of the particle;

y_i = y of the i-th well;

D_2 = cosh[π(x - x_i) / a] - cos[π(y + y_i) / a];

N_{y1} = sen[π(y - y_i) / a];

N_{y2} = sen[π(y + y_i) / a];

1.4.2 Processing the path a pollutant particle inside the aquifer.

The cumulative effect of the dispersion and adsorption of a pollutant inside the porous medium can be inserted in the calculation through a numerical procedure called random walk (Feller, 1966). Practically, total time of the simulation is divided in a set of same-length steps with the assumption that the particle location inside the XY surface, in a specific instant, involves the sum of two components, one determistic and one probabilistic. The first one is given by:

$$a_x = \frac{v_x \Delta t}{R} \text{ , } a_y = \frac{v_y \Delta t}{R}$$

where v_x and v_y are the fluid velocities estimated through the expressions by Bear e Verruijt (1987) and R is a retardation factor, inserted to take in account the adsorption phenomenon. Parameter R can assume only values higher or equal than 1 and practically involves a slowdown of the particle motion. Setting R equal to 1 means, consequently, to neglite the adsorption effect.

The second component of the motion is considered to take in account the dispersion effect of the pollutant. We assume that this phenomenon involve a random location of the particle, after a probabilistic distribution of gaussian kind, between these two limits:

$$b_x = \pm\sqrt{6 a_l a_x R} \text{ e } b_y = \pm\sqrt{6 a_t a_y R}$$

1.5 Permeability tests.

1.5.1 Introduction

Inside a soil, permeable by porosity, in which the Darcy's law is verified, permeability is defined through a coefficient k which has the dimension of a velocity. Inside a rock mass, permeable due to the rock joints, where the Darcy's law is not verified, permeability is defined thorugh the inflow volumes of water measured in the test borehole, expressed by water amount absorbed per lenght unit, and by the test pressure. Sometime the coefficient k is used to define permeability of rock mass, but in this case it supposes a orientative meaning. The choose of the test method should be made as a function of the soil type and of the desired accuracy.
The test accuracy can be improved through the following measures:

- analysis of the pore pressure distribution before the test execution;
- knowledge of the stratigraphic profile;
- performing test in laminar flow condition;
- using of clear water in all the test involving water injection.

1.5.2 Tests in soil pit.

Permeability tests inside soil pit are suitable essentially in case of granular soils. They allow to estimate the permeability in shallow soils, above the water table and are executed in pits having cylindrical or square plant with vertical or sloped walls.
They are classified as follow:

- constant-head test: they are performed filling with water the soil pit and measuring the discharge necessary to keep constant the water level;
- falling-head test: they are executed measuring the falling of the water level as a function of time after stopping the water injection.

The necessary conditions for the test be reliable are the following:

- soil has to be saturated before executing the test to set a permanent flow condition;
- the depth of the soil pit has to be equal at least to 1/7 of the distance of the bottom to the water table;
- The diameter (or the side width) of the soil pit has to be at least 10-15 times the maximum diameter of the soil grains;
- the soil has to be homogeneous, isotropic and to have a permeability coefficient higher than 10^{-6}m/s.

A) Cylindrical soil pit.

The permeability coefficient K can be calculated through the following expressions:

a) Constant-head test:

$$k = \frac{q}{\pi d h_m}$$

with
q = discharge to keep the constant head;
h_m = head of the water level (h_m > d/4);

d = diameter of the soil pit.

b) Falling-head test:

$$k = \frac{d(h_2 - h_1)}{32(t_2 - t_1)h_m}$$

where
h_m = average water head in the soil pit (h_m > d/4);
d = diameter of the soil pit;
t_2-t_1 = measuring time interval;
h_2-h_1 = head fall during the interval t_2-t_1 .

B) Square soil pit.

The permeability coefficient K can be calculated through the following expressions:

a) Constant-head test:

$$k = \frac{q}{b^2\left(27\frac{h}{b} + 3\right)}$$

with
q = discharge to keep the constant head;
h = head of the water level (h > d/4);
b = width of the side.

b) Falling-head test:

$$k = \frac{h_2 - h_1}{t_2 - t_1} \frac{1 + \left(2\frac{h_m}{b}\right)}{\left(27\frac{h_m}{b} + 3\right)}$$

with
h_m = average water head in the soil pit (h_m > d/4);
b = width of the side.
t_2-t_1 = time interval;
h_2-h_1 = head fall during the interval t_2-t_1 .

1.5.3 Test in borehole

Tests in borehole allow to estimate the permeability coefficient of soil layers located below or above the water table level. They can be executed either at several depth during the drilling or at the end of the drilling in the terminal tract.
To correctly perform the tests is necessary that:

- the drilling walls have to be cover by tubing along the tract not involved by the test;
- In case of unstable layers, the test interval has to be filled with a filtering mass, having an adequate granulometry, and isolated by an impervious tamp.

They are classified as follow:

- constant-head test;
- falling-head test.

A)Constant head test.

A Constant-head test is performed, measuring the discharge necessary to keep constant the water head inside the borehole, in a constant flow condition. They may be executed also in soil layers located above the water table; in this case, however, the soil has to be prevently saturated to reach a permanent flow condition.

1)A.G.I. recommendations(1977)

The permeability coefficient is given by:

$$k = \frac{q}{mh}$$

where

q = injected discharge;
h = water head inside the borehole;

m = shape coefficient = 2,85D
D= borehole diameter

2) Hvorslev (1951) Wilkinson (1968)

As well as in the former case, the permeability coefficient is given by:

$$k = \frac{q}{mh}$$

the parameter m has different values as a function of the inflow conditions, according to the following table:

Conditions	Coefficient m
Spherical filter in uniform soil	$2\pi D$
Semi-spherical soil bottom at impervious boundary	πD
Soil flush with bottom at impervious boundary	$2D$
Soils flush with bottom in uniform soil	$2{,}75D$
Soil in casing with bottom at impervious boundary	$\dfrac{2D}{1+\dfrac{8LK_h}{\pi D K_v}}$
Soil in casing with bottom in uniform soil	$\dfrac{2{,}75D}{1+\dfrac{11LK_h}{\pi D K_v}}$
Cylindrical filter at impervious boundary	$\dfrac{3\pi L}{\ln\left[\dfrac{3L}{D}+\sqrt{1+\left(\dfrac{3L}{D}\right)^2}\right]}$
Cylindrical filter in uniform soil	$\dfrac{3\pi L}{\ln\left[\dfrac{1.5L}{D}+\sqrt{1+\left(\dfrac{1.5L}{D}\right)^2}\right]}$

where:

L= Length of the test tract
K_h= Horizontal coefficiente of permeability;
K_v= Vertical coefficiente of permeability.

In the case the K_h/K_v ratio is unknown, it might be approximately set equal to 10.

3) Zagar (1953)

3a) Saturated soil layers

The permeability coefficient is given by

$$k = \frac{q}{mh}$$

the coefficient m assumes the following values:

$m = 5{,}7r$ If the borehole is open only in the bottom

$$m = \frac{4\pi r\sqrt{\left(\frac{L}{2r}\right)^2 - 1}}{\ln\left[\frac{L}{2r} + \sqrt{\left(\frac{L}{2r}\right)^2 - 1}\right]}$$ If the borehole is open also along the sides

where r=borehole radius and L=length of the test tract.

3b) Unsaturated soil layers

In case where the water head inside the borehole be located shallower than the water table, the former expression cannot be used anymore.
Defining H_u the difference between the water head in the borehole and the depth to the water table and r' the ratio between the borehole radius and the area of the filtering surface, the parameter Y is calculated according to the following expression:

$$Y = -1{,}0556 + 0{,}035\frac{100h}{H_u}$$

where h is the average water head inside the borehole respect to the bottom of the borehole itself. When it results $Log_{10}(H_u/L)>Y$, where L is the length of the filtering tract, K is given through the following formula:

$$k = \frac{q}{Cr'h}$$

where C is calculable by the following expressions:

$$C = C1 + (C2 - C1)Log_{10}\frac{100L}{h}$$

$$C1 = 60{,}96 + 0{,}152\frac{h}{r}$$

$$C2 = 104{,}58 + 0{,}822\frac{h}{r}$$

Coversely, when $Log_{10}(Hu/L) \leq Y$, k is given by:

$$k = \frac{q}{\left(C + 4\frac{r}{r'}\right)r'(H_u + h - L)}$$

where $C = 6{,}247 + 0{,}797\frac{L}{r}$

This procedure is reliable only in cases when h>5L e L>10r'.

B)Falling-head test.

Falling-head tests below the water table are classified in two kinds: recovery test and drawdown test. Recovery tests are performed dropping the water head, whose value is known, and measuring its recovery speed. Conversely, drawdown test are executed pouring water inside the borehole up to a specific head and then measuring the head-fall speed.
Drawdown tests can be performed also in soil layers above the water table; in this case, however, they have to be preventely saturated.

1)A.G.I. recommendations (1977)

In case of falling-head tests, the coefficient of permeability is given by:

$$k = \frac{A}{C_L\left(t_2 - t_1\right)} \ln \frac{h_1}{h_2}$$

with

A = area of the bottom of the borehole;
h_1 and h_2 = depth of the water heads in the borehole respect either to the undisturbed water table or to the borehole bottom during the instants t_1 e t_2;
t_1 and t_2 = instants when h_1 and h_2 are measured;
C_L = shape coefficient depending on the area of the borehole and the length of the uncovered tract.

For the coefficient C_L the following values are suggested:

$L \gg d \quad C_L = L$
$L \leq d \quad C_L = 2\pi d + L$

Where L is the length of the uncovered tract and d is the borehole diameter.

4) Hvorslev (1951) Wilkinson (1968)

The coefficient of permeability is given by::

$$k = \frac{A}{C_L(t_2 - t_1)} \ln \frac{h_1}{h_2}$$

the coefficient C_L can assume different values as a function of the inflow condition, according to the following table:

Condition	Coefficient
Spherical filter in uniform soil	$2\pi D$
Semi-spherical soil bottom at impervious boundary	πD
Soil flush with bottom at impervious boundary	$2D$
Soils flush with bottom in uniform soil	$2{,}75D$
Soil in casing with bottom at impervious boundary	$\dfrac{2D}{1 + \dfrac{8LK_h}{\pi D K_v}}$
Soil in casing with bottom in uniform soil	$\dfrac{2{,}75D}{1 + \dfrac{11LK_h}{\pi D K_v}}$
Cylindrical filter at impervious boundary	$\dfrac{3\pi L}{\ln\left[\dfrac{3L}{D} + \sqrt{1 + \left(\dfrac{3L}{D}\right)^2}\right]}$
Cylindrical filter in uniform soil	$\dfrac{3\pi L}{\ln\left[\dfrac{1.5L}{D} + \sqrt{1 + \left(\dfrac{1.5L}{D}\right)^2}\right]}$

where:

L= Length of the filtering tract;

K_h= Horizontal coefficient of permeability;

K_v= Vertical coefficient of permeability.

In the case the K_h/K_v ratio is unknown, it might be approximately set equal to 10.

5) Zagar (1953)

K is given by:

$$k = \frac{\pi r^2}{m} \frac{\left(\frac{h_2 - h_1}{t_2 - t_1} \right)}{h_m}$$

where r is the borehole radius and h_m is the average water head.
The coefficient m assumes the following values:

$m = 5{,}7r$ If the borehole is open on the bottom;

$$m = \frac{4\pi r \sqrt{\left(\frac{L}{2r} \right)^2 - 1}}{\ln\left[\frac{L}{2r} + \sqrt{\left(\frac{L}{2r} \right)^2 - 1} \right]}$$

If the borehole is open also along the sides.

with r=borehole radius and L=length of the filtering tract.

1.5.4 Lugeon test.

Lugeon tests allows to calculate the permeability and to evaluate the jointing ratio of a rock mass. They are performed injecting water under pressure inside a borehole. Inside the borehole a tubing for the water inlet, having two shutters to isolate the test tract, is inserted. During the test are measured: the injection pressure, the water discharge and the test duration after reaching permanent flow condition. At least 5 different pressure steps are taken in account, everyone kept constant as long as 10-20 minutes.

They can be performed downward during the drilling, stopping the drilling every 2, 5 meters, or upward at the end of the borehole execution.
The pressure in the test tract is given by:

$$P_e = P_m + \gamma_w \left(H - H_p\right)$$

where
P_m = Pressure read on the gauge;
H = head of the water column;
H_p = water-head loss;
γ_w = specific gravity of water.

In a homogeneous and uniform medium, in presence of laminar flow around the borehole, the coefficient of permeability is given by:

$$k = \frac{q\gamma_w}{CP_e}$$

where
q = adsorbed discharge;
P_e = pressure in the test tract;

C = shape coefficient = $2\pi D \frac{\sqrt{\left[\left(\frac{L}{D}\right)^2 - 1\right]}}{\ln\left[\frac{L}{D} + \sqrt{\left(\frac{L}{D}\right)^2 - 1}\right]}$

with :
D = diameter of the borehole;
L = length of the test tract.

Rock mass permeability can be undirectly evaluate through the parameter Lugeon (L.). L represents the water discharge, in liters per minute, adsorbed in a test tract having a length equal to 1 m, at the

pressure of 10 kg/cmq. It correspond to about 10^{-7} m/s. The reference value of L, for the current test, is obtainable by the absorption-pressure graph, which along the X axis has the absorption, in liters per minute and per 1 meter of test tract, and along the Y axis the effective pressure.
Four cases are possible:

a)Laminar flow.

In this case the measured L values result are approximately the same. As reference L value the average of the 5 measured values is taken.

b)Turbulent flow.

The measured L at the maximum pressure has the lowest value of the set and is taken as reference value.

c)Joint dilation.

In this case it is visible a remarkable increasing of L at the maximum pressure step, whereas the values of L measured at the middle pressure steps are approximately the same. The average of the values measured at the middle pressure steps is taken as reference value L.

d)Joint wash-out.

A progressive increasing of L is displayed. As reference value of L, the last measured value, which would be the maximum too, is taken.

e)Void filling.

A progressive decreasing of L is displayed. As reference value of L, the last measured value, which would be the minimum too, is taken.

1.5.5 Estimating permeability by granolumetric curve.

They are numerous empirical correlations which allow to estimate the permeability coefficient of a porous medium, passing through the analysis of granulometric curves. These formulas, though not substituting the on-site tests, can be useful to a first estimation of k in sandy layers. They all are written in the following form:

$$K(m/s) = \frac{g}{v} C\phi(n) d_e^{\,2}$$

where:

g =gravity acceleration=9.81 (m/s^2);

v =water viscosity coefficient, variable as a function of temperature, according to the following table:

T (°C)	0	5	10	15	20	30	50
V (mq/s)	1.78 10^{-6}	1.52 10^{-6}	1.31 10^{-6}	1.14 10^{-6}	1.01 10^{-6}	0.81 10^{-6}	0.55 10^{-6}

C = constant;

$\phi(n)$ = function of soil porosity;

d_e = effective diameter of grains.

The below described formulas differ each other due to the different values of the parameters C, $\phi(n)$ and d_e. Soil porosity, if not directly measured, may be approximately estimated through the following empirical expression:

$$n=0.255(1+0.83^{n})$$

where $\eta = d_{60}/d_{10}$ is the uniform coefficient of the soil layer.

1) Hazen's formula.

In the Hazen's formula C, $\phi(n)$ and d_e have the following expression:

C =$6\ 10^{-4}$
$\phi(n)$ =$\left[1+10(n-0{,}26)\right]$
d_e =d_{10}

This formula is reliable in the following conditions:
0,1 mm < d_e < 3 mm and η<5.

2) Slichter's formula.

In the Slichter's formula C, $\phi(n)$ and d_e have the following expression:

C =$1\ 10^{-2}$
$\phi(n)$ =$n^{3,287}$
d_e =d_{10}

This formula is reliable in case of coarse sand:
0,01 mm < d_e < 5 mm.

3) Terzaghi's formula.

In the Terzaghi's formula C, $\phi(n)$ and d_e have the following expression:

C =$10.7\ 10^{-3}$ for sand having rounded grains and $6.1\ 10^{-3}$ for sand with sharpgrains
$\phi(n)$ =$\left(\frac{n-0{,}13}{\sqrt[3]{1-n}}\right)^2$
d_e =d_{10}

This formula is reliable in case of coarse sand.

4) Beyer's formula.

In the Beyer's formula C, $\phi(n)$ and d_e have the following expression:

C =$6\ 10^{-4}\ Log_{10}(500/\eta)$

$\phi(n)$ =1
d_e $=d_{10}$

This formula is reliable in the following conditions:
$0{,}06\ mm < d_e < 0{,}6\ mm$ and $1<\eta<20$.

5) Sauerbrei's formula.

In the Sauerbrei's formula C, $\phi(n)$ and d_e have the following expression:

C $=3{,}75\ 10^{-3}$
$\phi(n)$ $=\dfrac{n^3}{(1-n)^2}$
d_e $=d_{17}$

This formula is reliable in case of sand and sandy clay with $d_e < 0{,}5$ mm.

6) Krueger's formula.

In the Krueger's formula C, $\phi(n)$ and d_e have the following expression:

C $=4.35\ 10^{-5}$
$\phi(n)$ $=\dfrac{n}{(1-n)^2}$
$1/d_e$ $=\sum \Delta g_i \dfrac{2}{d_i^g + d_i^d}$ where Δg_i is the part of the sample weight encompassed between the maximum and minimum diameters (d_i^g and d_i^d)of i-th pass-through of the granulometric curve

This formula is reliable in case of medium sand with $\eta>5$.

7) Kozeny's formula.

In the Kozeny's formula C, $\phi(n)$ and d_e have the following expression:

C =8,3 10^{-3}

$\phi(n)$ $= \frac{n^3}{(1-n)^2}$

$1/d_e$ $= \frac{3}{2}\frac{\Delta g_i}{d_i} + \sum \Delta g_i \frac{d_i^g + d_i^d}{2 d_i^g d_i^d}$ where Δg_i is the part of the sample weight encompassed between the maximum and minimum diameters (d_i^g and d_i^d)of i-th pass-through of the granulometric curve

This formula is generally valid for coarse sand.

8) Zunker's formula.

In the Zunker's formula C, $\phi(n)$ and d_e have the following expression:

C =2.4 10^{-3} uniform sand with rounded grains
=1.4 10^{-3} coarse sand with rounded grains
=1.2 10^{-3} heterogeneous sand
=0.7 10^{-3} heterogeneous clayey sand with sharped grains
as an alternative, it may be inserted an average value equal to 1.55 10^{-3}

$\phi(n)$ $= \left(\frac{n}{1-n}\right)^2$

$1/d_e$ $= \frac{3}{2}\frac{\Delta g_i}{d_i} + \sum \Delta g_i \frac{d_i^g - d_i^d}{d_i^g d_i^d \left(\ln d_i^g - \ln d_i^d\right)}$ where Δg_i is the part of the sample weight encompassed between the maximum and minimum diameters (d_i^g and d_i^d)of i-th pass-through of the granulometric curve

This formula is generally valid from fine to coarse sand

9) Zamarin's formula.

In the Zamarin's formula C, $\phi(n)$ and d_e have the following expression:

C =8.3 10^{-3}

$\phi(n)$ $= \frac{n^3}{(1-n)^2}(1{,}275 - 1{,}5n)^2$

$1/d_e$ $= \frac{3}{2}\frac{\Delta g_i}{d_i} + \sum \Delta g_i \frac{\ln d_i^g - \ln d_i^d}{d_i^g - d_i^d}$ where Δg_i is the part of the sample weight encompassed between the maximum and minimum diameters (d_i^g and d_i^d)of i-th pass-through of the granulometric curve

This formula is generally valid in case of coarse sand.

10) USBR's formula.

In the USBR's formula C, $\phi(n)$ and d_e have the following expression:

C =4.8 10^{-4} $d_{20}^{0,3}$

$\phi(n)$ =1

d_e $=d_{20}$

This formula is generally valid for medium sand with $\eta < 5$.

1.6 Estimating the potential infiltration ratio.

The potential infiltration ratio (f) is the maximum water volume which can be infiltrated into the ground, if such a volume is available. The actual infiltration water volume may be lower if the surface runoff is not sufficient. Anyway it cannot be higher.
The potential infiltration ratio depends on the ground permeability and on the initial saturation ratio. The higher is the permeability, the higher will be the infiltration. The higher is the saturation ratio, the lower will be the infiltration.
Green & Ampt's method is commonly used to estimate the potential infiltration ratio. This procedure involves that the saturation front moves itself downward as a function of the time, dividing distinctly the saturated ground volume, with a water contents equal to the soil porosity (η), by the deeper one, not yet reached by the saturation front, having a water contents equal to the initial one (θ).
At time t, after beginning of the infiltration process, the cumulative infiltration F, that is the water volume which is infiltrated till that moment, can be express by the following formula:

$$F(t)(mm) = Kt + \Delta\theta(h_0 + \psi)\ln\left(1 + \frac{F(t)}{\Delta\theta(h_0 + \psi)}\right)$$

where:

K(m/h) = vertical permeability of the ground, usually sets equal to the 50% of the horizontal one;
t(h) = calculation time;
ψ(mm) = capillary rise;
h_0(mm) = hydraulic depth, in respect to the bottom of the lowered area.
$\Delta\theta$ = η-θ;

Since the parameter F appears in both the members of the equation, the solution has to be found through an iterative process, imposing a first value inside the second member, solving the equation and then substituting the new calculated value in the second member. Calculation has to be repeated until the difference between two

consecutive values of F will be lower than a prefix limit (for example 0.001).
The value of the capillary rise may be chosen, selecting it by the following table:

Soil type	ψ(m)
Gravel	0.05-0.30
Coarse sand	0.30-0.80
Medium sand	0.12-2.40
Fine sand	0.30-3.50
Silt	1.5-12
Clay	>10

Known the cumulative infiltration, the potential infiltration ratio can be calculated by the following expression:

$$f(t)(mm/h) = K\frac{F(t) + \Delta\theta\left(h_0 + \psi\right)}{F(t)}$$