Arba Minch Water Technology Institute

Faculty of Water supply and Environmental Engineering

Course title; Ground Water Hydrology

Course code; WSEE3132

Target groups; 3rd Year Water Supply and Environmental Engineering Students

Academic year; 2019/20

Semester; II

Instructor; Melsew E.

Course syllabus

Objective:

- ✓ This course is designed to aware students about;
- ✓ The scope and occupancy of groundwater, hydrologic cycle, different types of aquifers and their characteristics.
- ✓ The Study of groundwater movement, Darcy's law, laboratory and field determination of hydraulic conductivity, hydraulics of wells, steady and unsteady states of flow in confined and unconfined aquifers are thecore of this course

Course syllabus

Outcomes:

- ✓ After completion of this course students will be able to
- ✓ Know the basic concepts of occurrence, laws of movement and distribution of groundwater in relation to natural and artificial influences.
- ✓ Understand the directional flow of groundwater, well hydraulics, how to explore groundwater, pumping tests of wells, tube wells and tube well development
- ✓ Analyze pumping test data and interpretation of the result.
- ✓ Have a clear idea of groundwater resource development for various purposes.
- ✓ Aware of the complex nature of groundwater balance and its management.
- ✓ Understand artificial recharge, need of artificial recharge and methods employed to apply artificial recharge

Course Contents

- Ground water resources: Scope and occurrence; ground water in hydrologic cycle; different types of aquifers and their characteristics. Ground water movement:
- Darcy's law, mathematical treatment of frequently occurring flow problems, one-, two- and three-dimensional flow in phreatic, confined and semi-confined aquifers.
- Laboratory and field determination of hydraulic conductivity, determination of ground water flow parameters.
- Hydraulics of wells: steady and unsteady states of flow in, phreatic, confined and unconfined aquifers. Solution methods; graphical methods, use of images; numerical analysis, application of mathematical models to the study of ground water flow problems; unsteady flow in leaky aquifers; partially penetrating wells; multiple well system.
- Pumping test, design of piezometer, analysis and interpretation of data. Ground water exploration.
- Design of tube wells (water wells): screened and gravel wells; methods of construction based on drilling equipment's; well development and maintenance; well failures and rehabilitation. Ground Water balance and ground water management. Artificial recharge of ground water.

Chapter 1
Groundwater resources

- Groundwater is commonly understood to mean water that occupies all the void spaces of a geologic formation below the surface of the ground.
- The study of groundwater flow is equally important as studying the surface water resources since about 22% of the world's fresh water resources exist in the form of groundwater.

Con,t…

- Because of its importance as significant source of water supply for various aspects, the exploration, development and utilization of groundwater have been extensively studied by workers from different disciplines, such as geology, geophysics, geochemistry, irrigation engineering, hydraulic engineering and civil engineering etc.

Groundwater in Hydrological cycle

- **Hydrology:-** is the science that deals with the water of the earth: the **occurrence**, circulation, distribution, chemical and physical properties, and water reaction with the environment, including relation to living things.
- That is, it deals with the full history of water on earth.
- **Groundwater Hydrology**:- is a subdivision of the science of hydrology that deals with the occurrence, movement and quality of water beneath the earth’s surface.

Hydrologic cycle

- Water on earth circulates in a space called the hydrosphere which extends about 15km up in to the atmosphere and 1km down into the lithosphere and this constant circulation and exchange of water is known as hydrologic cycle.
- **The hydrologic cycle** represents the sequence of events when water drops from the atmosphere to the earth and hydrosphere (water bodies such as rivers, lakes, seas and oceans covering the earth's surface) and then goes back to the atmosphere.

Con,t…

- In other words hydrologic cycle can be defined as the circulation of water evaporated from the sea through the atmosphere to the land and then by surface and subsurface routes back to the sea.
- River flow to the sea shows the existence of this cycle. without rising of water level,seas receive water again and again.
- The sun generates the energy to sustain the cycle.

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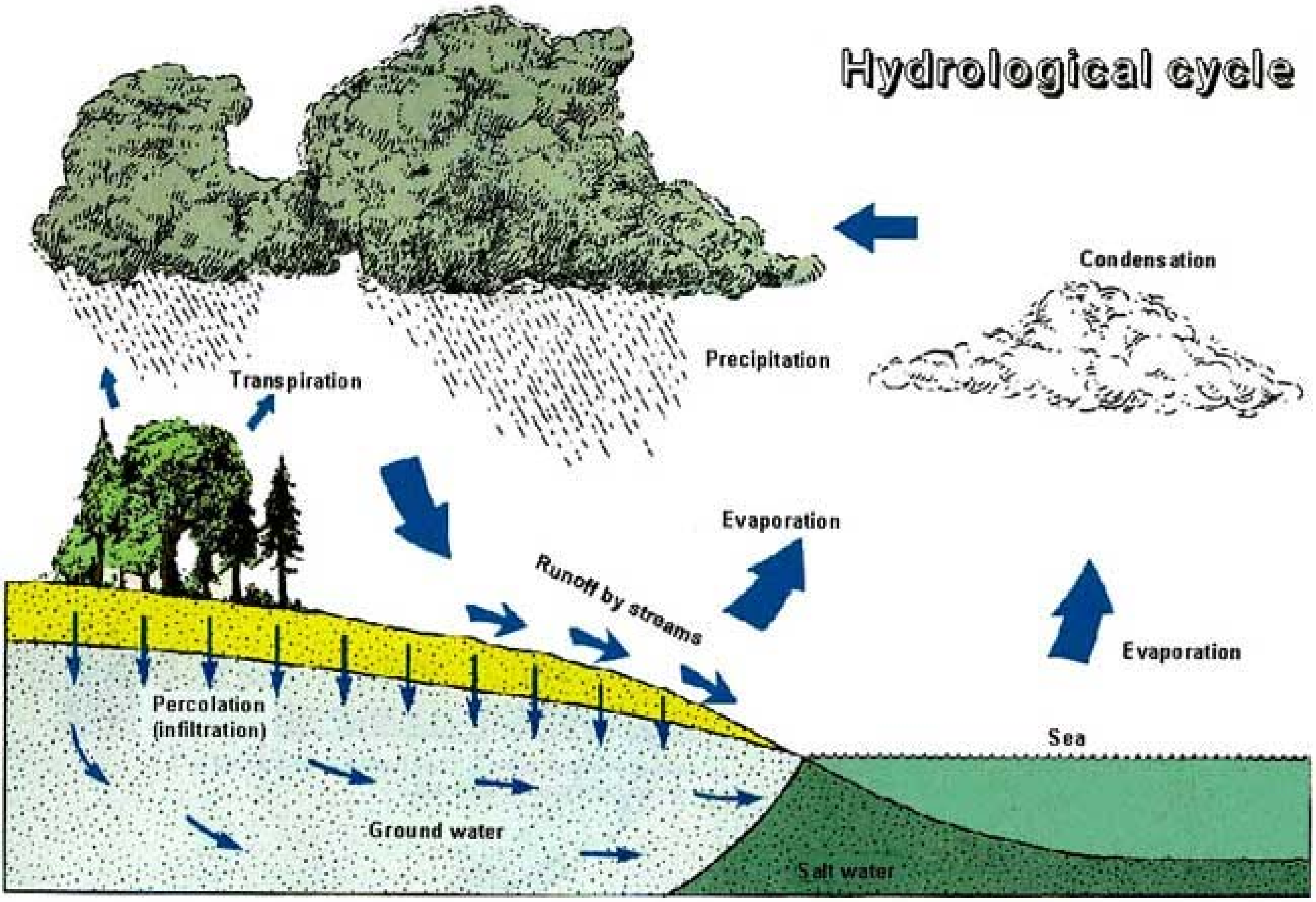
Hydrological cycle
Condensation
Precipitation
Transpiration
Evaporation
Runoff by streams
Evaporation
Percolation
(infiltration)
Sea
Ground water
Salt water

Major processes involved in the hydrologic cycle

- **Precipitation:-** Any form of water which falls on the earth.
- **Evaporation :-** The transfer of water into the atmosphere from a free water surface, a bare soil or interception on a vegetal cover
- **Transpiration: -** The process by which water in plants is transferred to the atmosphere as water vapor.
- **Infiltration:-** The process of water entry into a soil from rainfall, snow melt, or irrigation
- **Percolation: -** The process of water entry into the saturated zone or the groundwater table.
- **Surface runoff:-** The flow of water over the land surface
- **Groundwater flow:-** The movement of water in the subsurface

Groundwater resource

Of all water on earth, 97% is contained in the oceans and seas as saline or salt water. The total volume of water on earth is distributed as Ocean(97.3%) and Fresh water (2.7%).

The distribution of fresh water on earth

Ice caps and glaciers	**77.2%**
Groundwater and soil moisture	22.4%
Lakes and swamps	0.35%
Atmospheric water	0.04%
Stream channel	0.01%

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- The above figure shows that the water available for domestic, industrial and agricultural purposes is very limited as compared to the total volume of water on the planet.
- Still the existing usable fresh water is by far enough to satisfy the current demand. However, this resource is unevenly distributed both in space and time.

History of Groundwater use

- Since early days of humanity people have used groundwater for domestic water supply and irrigation. Both springs and water wells were utilized for this purpose.
- Well construction was carried out in old civilization in china, Middle East and Egypt.
- 2500 years ago, Khanates were installed in Persia (Iran) and latter practiced in Afghanistan and Egypt.
- Since 12^{th} century, due to modern technology many African countries including Ethiopia were utilizing groundwater from wells.

The end

Chapter 2
Groundwater occurrences

- **Groundwater system:-** is the zone in the earth's crust where the open space in the rock is completely filled with groundwater at a pressure greater than atmospheric pressure.
- Groundwater is found below the groundwater table.
- **Groundwater table:-** is the top most part of groundwater. may be located near or even at land surface . not fixed meaning it fluctuate seasonally.

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Two zones in which Groundwater occurs in the ground,

A. **The unsaturated zone/ Zone of aeration/**

B. **The saturated zone**

- The process of water entering into the ground is called infiltration.
- Downward transport of water in the unsaturated zone is called percolation,
- The upward transport in the unsaturated zone is called capillary rise.
- The flow of water through saturated porous media is called groundwater flow.
- The out flow from groundwater to surface water is called **seepage**.

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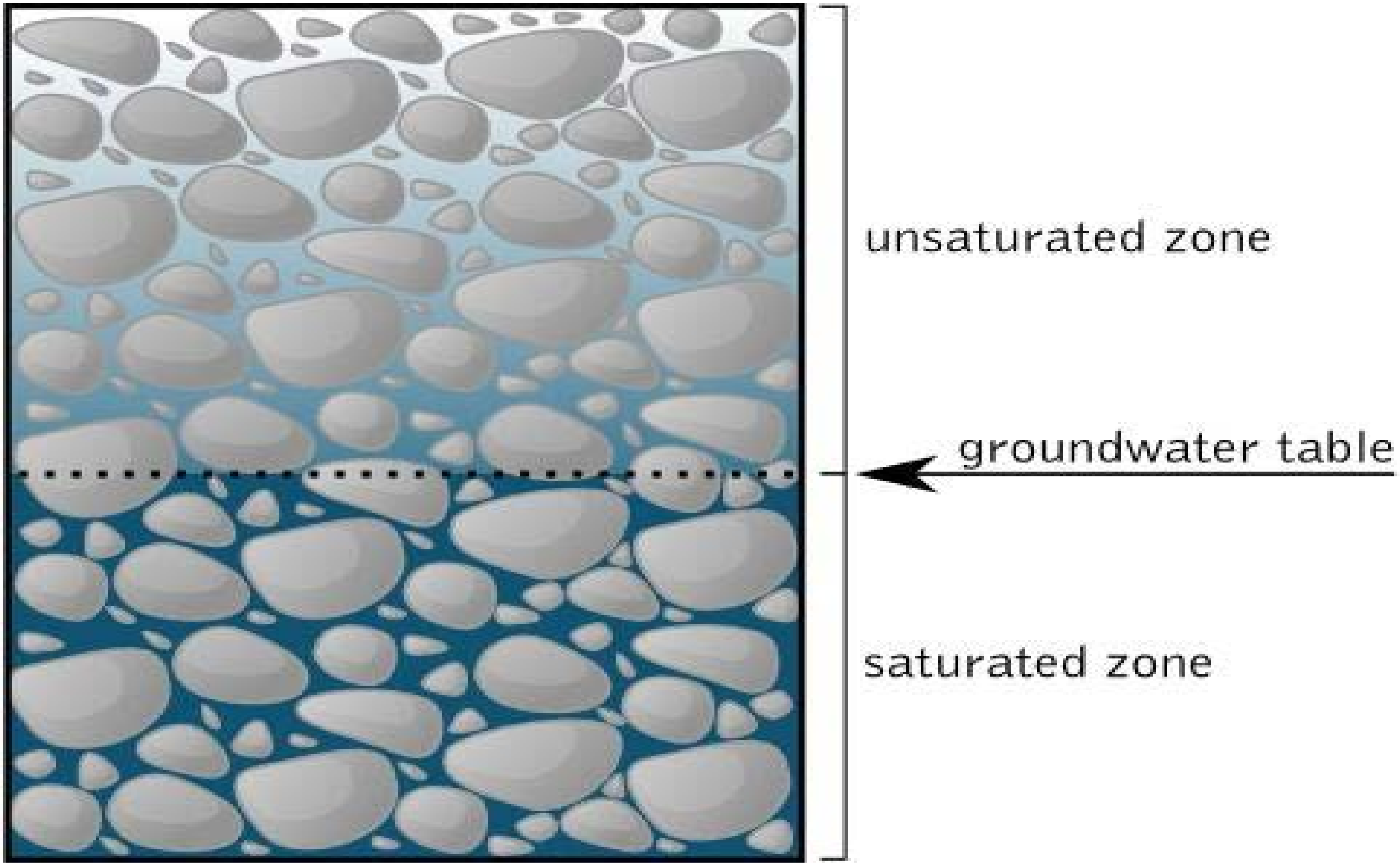

Con,t…

The type of openings (voids or pores) in which groundwater occurs.

1.Pores:-

- Openings between individual particles as in sand and gravel.
- Pores are generally interconnected and allow capillary flow for which Darcy's law can be applied.

2.Fractures, Crevices or joints:

- This means fractures and crevices in hard rock which have developed from breaking of the rock.
- The pores may vary from super capillary size to capillary size.

Con,t…

- Only for the rare situation application of Darcy’s law is possible.
- Water in these fractures is known as fissure or fault water

3. Solution channels and caverns in limestone (karst water):

- are resulting from gas bubbles in lava.
- These large openings result in a turbulent flow of groundwater which cannot be described with Darcy’s law.

Unsaturated Zone/ Zone of aeration

- In this zone the soil pores are only partially saturated with water.
- The space between the land surface and the water table marks the extent of this zone.
- Further, the zone of aeration has three sub zones:

A. soil water zone,
B. capillary fringe and
C. intermediate zone

Con,t…

- The soil water zone lies close to the ground surface in the major root band of the vegetation from which the water is lost to the atmosphere by evapotranspiration.
- The intermediate zone lies between the soil water zone and the capillary fringe.

Con,t…

- Capillary fringe on the other hand hold water by capillary action. This zone extends from the water table upwards to the limit of the capillary rise.
- The thickness of the zone of aeration and its constituent sub-zones depend upon the soil texture and moisture content and vary from region to region.
- The soil moisture in the zone of aeration is important in agricultural practice and irrigation engineering.

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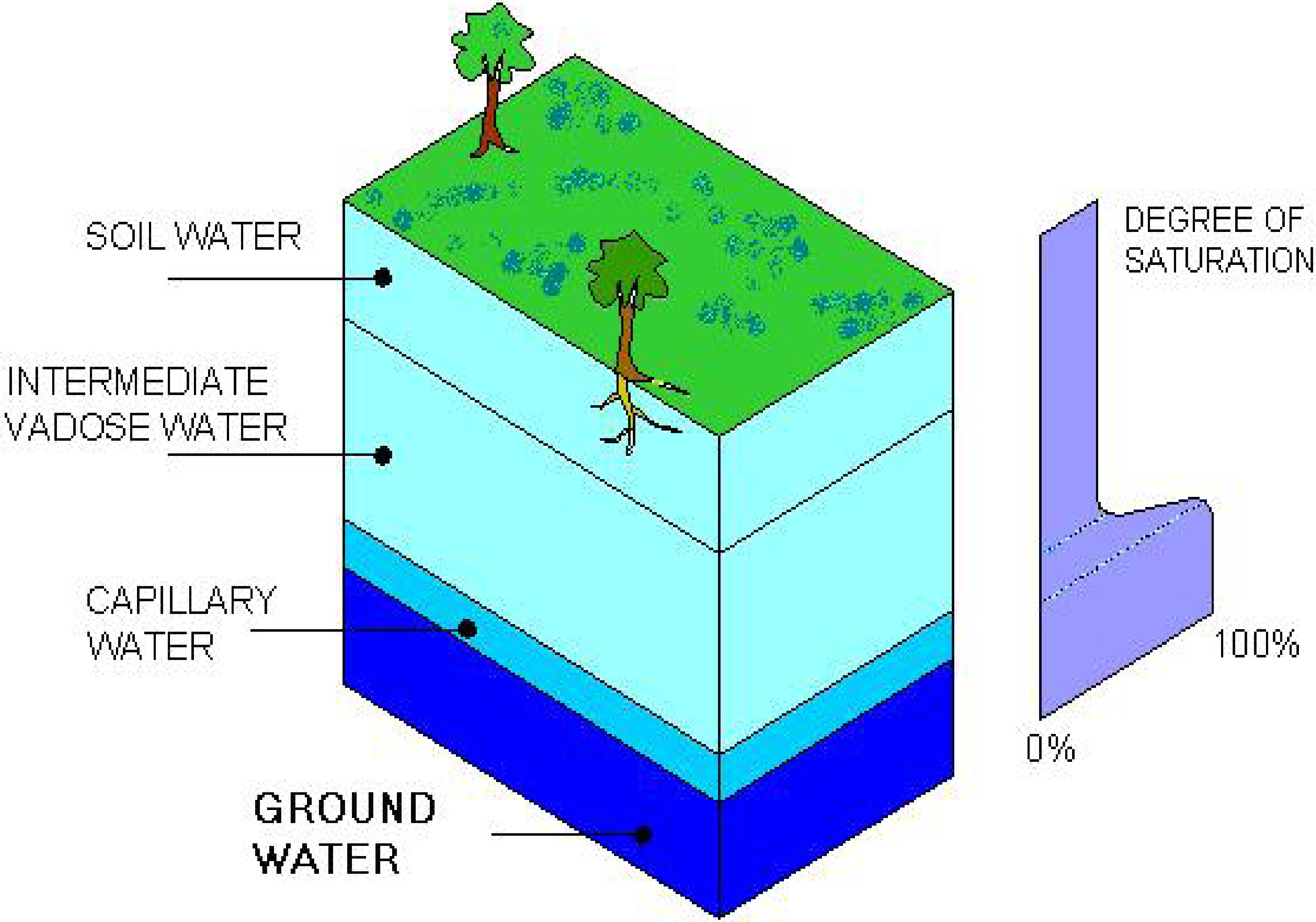

SOIL WATER
INTERMEDIATE
VADOSE WATER
CAPILLARY
WATER
GROUND
WATER
DEGREE OF
SATURATION
100%
0%

Con,t…

Important conditions in the unsaturated zone are the wilting point and the field capacity.

- **Field capacity:-** is the moisture content in the soil a few days after irrigation or heavy rainfall, when excess water in the unsaturated zone has percolated.
- **Wilting point:-** is a minimum soil moisture content for which the plant is no longer capable of taking up the soil moisture and dies.

Saturated Zone

- In this zone all earth materials, from soils to rocks pores are completely saturated with water below the groundwater table or phreatic surface (GWT).
- Natural variations in permeability and ease of transmission of groundwater in different saturated geological formations lead to the recognition of Aquifer, Aquitard, Aquiclude and Aquifuge.

Con,t…

1. Aquifer: -

- This is a water-bearing layer for which the porosity and pore size are sufficiently large.
- not only stores but yields sufficient quantity of water due to its high permeability.
- Unconsolidated deposits of sand and gravel form good aquifers (e.g. sand, gravel layers).

2. Aquitard: -

- It is less permeable geological formation which may be capable of transmitting water (e.g. sandy clay layer).
- It may transmit quantities of water that are significant in terms of regional groundwater flow.

Con,t…

3. Aquiclude:

- is a geological formation which is essentially impermeable to the flow of water.
- It may be considered as closed to water movement even though it may contain large amount of groundwater due to its high porosity (e.g. clay).

4. Aquifuge:

- is a geological formation, which is neither porous nor permeable.
- There are no interconnected openings and hence it neither stores nor transmit water.
- Massive compact rock without any fractures is an aquifuge.

Aquifers and their characteristics

The aquifers can be categorized into one of the following types.

1. Unconfined aquifer (also called phreatic or water table aquifer

- Unconfined aquifer consists of a pervious layer underlain by a (semi-) impervious layer.
- This type of aquifer is not completely saturated with water. The upper boundary is formed by a free water-table that is indirect contact with the atmosphere.
- In most places it is the uppermost aquifer.

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2. Confined aquifer

- Such an aquifer consists of a completely saturated pervious layer bounded by impervious layers.
- There is no direct contact with the atmosphere.
- The water level in wells tapping these aquifers rises above the top of the pervious layer and sometimes even above soil surface (artesian wells).

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3. Semi-confined or Leaky aquifers

- Consists of a completely saturated pervious layer, but the upper and/or lower boundaries are semi-pervious.
- They are overlain by aquitard that may have inflow and outflow through them.

4. Perched aquifer

- These are unconfined aquifers of isolated in nature.
- They are not connected with other aquifers.

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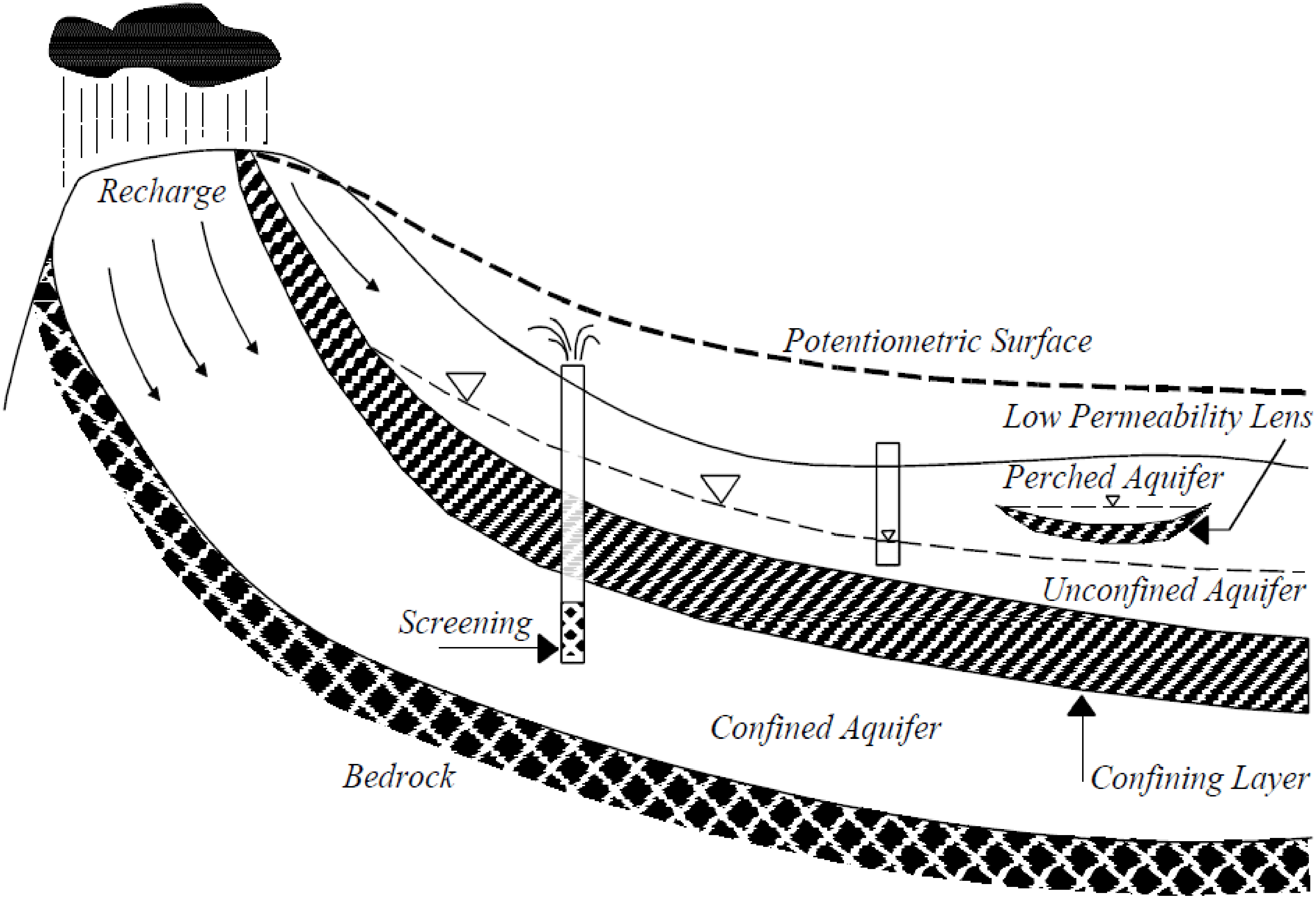

Recharge
Potentiometric Surface
Low Permeability Lens
Perched Aquifer
Unconfined Aquifer
Screening
Confined Aquifer
Confining Layer
Bedrock

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- The height to which the water rises with respect to a certain reference level (e.g. the impervious base, mean sea level, etc.) is called the **hydraulic head**.
- For unconfined aquifers, the hydraulic head may be taken equal to the height of the water table.
- Water moves from locations where the hydraulic head is high to places where the hydraulic head is low.
- The hydraulic head will be split into its gravitational and pressure components.

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Generally the head can be written as $h = z + \frac{p}{\gamma_w}$

Where:-

- z is the gravitational elevation head and the
- p/γ_w the pressure head

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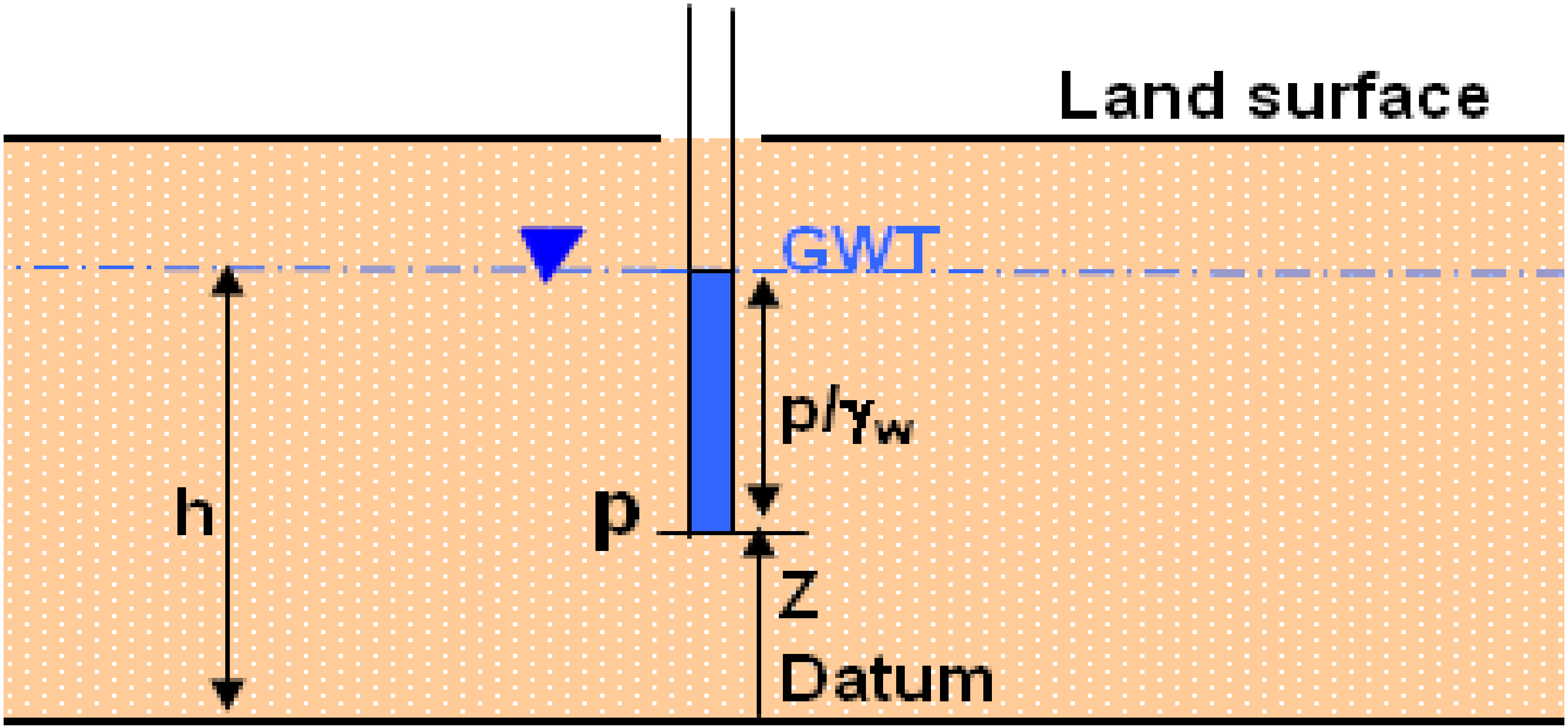

Determination of groundwater flow parameters

1.Porosity (n)

The porosity, n is the ratio of volume of the open space in the rock or soil to the total volume of soil or rock.

$$n = \frac{V_V}{V_T} * 100,$$

Where:-

V_v = the pore volume or volume of voids

V_T = the total volume of the soil

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- Porosity is also the measure of water holding capacity of the geological formation.
- The greater the porosity means the larger is the water holding capacity.
- Porosity depends up on the shape, size, and packing of soil particles.
- While porosity gives a measure of the water storage

Con,t…

2. Specific yield (S_y)

- When water is drained by gravity from saturated material, only a part of the total volumes is released.
- The ratio of volume of water in the aquifer which can be extracted by the force of gravity or by pumping wells to the total volume of saturated aquifer is called Specific yield (S_y).

$$s_y = \frac{V_W}{V_T} * 100$$

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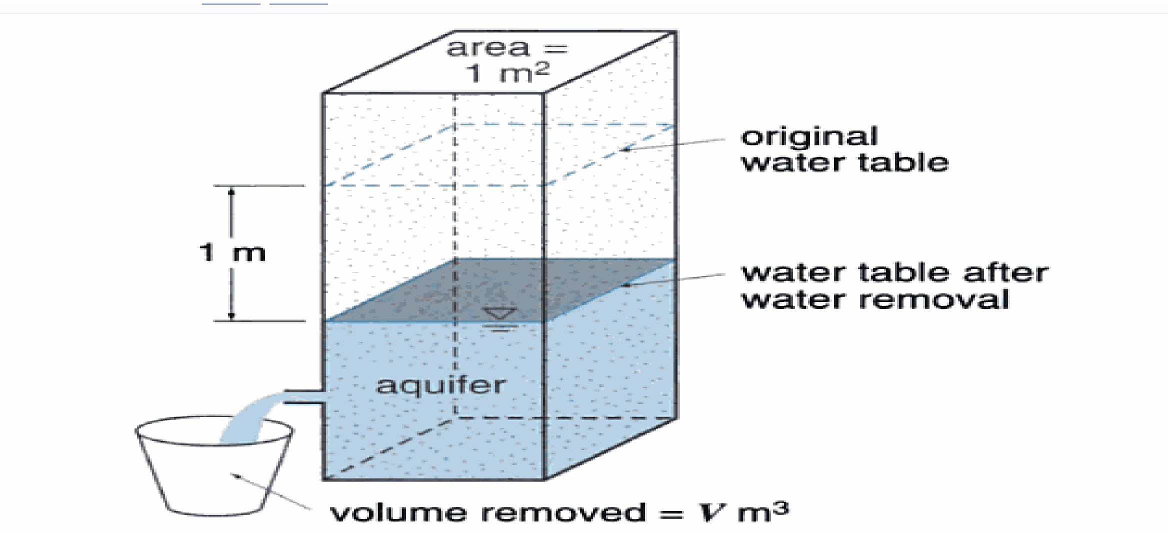
- For unconfined aquifers the specific yield (S_y) is defined as the amount of water stored or released in an aquifer column with a cross-sectional area of $1m^2$ as a result of a 1m increase or decrease in hydraulic head.



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- All the water stored in the water bearing formations can't be extracted by gravity drainage or pumping; a portion of water remains held in the voids of the aquifer by molecular and surface tension forces.

3. Specific retention (S_r)

- The water which is not drained or the ratio of volume of water that cannot be drained (Vr) to the total volume (V_T) of a saturated aquifer is called specific retention (S_r).

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$$S_r = \frac{V_r}{V_T} * 100$$

- In fine-grained material the forces that retain water against the force of gravity are high due to the small pore size.
- Hence, the specific retention of fine-grained material (silt or clay) is larger than that of coarse material (sand or gravel).

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- The total volume of voids in saturated (Vv) equals to the sum of volume of water drained out (Vw) and volume of water retained (Vr); that is Vv=Vw+Vr.

From the above expression we can get:

$\leftrightarrow n = S_y + S_r$

- Meaning sum of S_y and S_r is equal to the porosity.
- It should be noted that; it is not necessarily the soil with a high porosity will have a high specific yield because of its permeability.

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4. Coefficient of permeability (k)

- Coefficient of permeability is also called hydraulic conductivity reflects the combined effects of the porous medium and fluid properties.
- It is the capacity of geological formation to transmit water. Coefficient of permeability is primarily dependent on the soil property and water contained in it.
- Unconsolidated rocks are permeable when the pore spaces between grains are sufficiently large.

Con,t…

$K = k_i . kw$

Where:-

- K = Coefficient of permeability,
- k_i = Intrinsic permeability; depending on rock properties (such as grain size & packing)
- k_W = Permeability depending on fluid properties (such as density and viscosity of water)

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5. Transmissivity (T)

- Transmissivity is the product of horizontal coefficient of permeability and saturated thickness of the aquifer. For an isotropic aquifer ($K_x = K_y = K$):

$T = KB$

Where:- T = aquifer Transmissivity (m^2 / day),

B = aquifer thickness (m).

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6. Vertical Resistance (C)

- The **vertical resistance** of an aquitard is defined as the ratio of the thickness of the aquitard and its permeability in the vertical direction (k_z):

$$C = \frac{D}{kz}$$

Where: - C = vertical resistance (days),

D = thickness of the aquitard (m)

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7. Storage Coefficient (Sc)

- The amount of water stored or released in an aquifer column with a cross sectional area of $1m^2$ for a 1m increase or drop in head is known as storage coefficient.
- Storage coefficient of unconfined aquifer is equal to the specific yield.
- In confined or semi-confined aquifers water is stored or released from the whole aquifer column is mainly as a result of elastic changes in porosity and groundwater density.

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- Common values for the storage coefficients for confined and semi-confined aquifers range form 10^{-7} to 10^{-3}.
- The volume of water drained from an aquifer, Vw may be found from the following equation.
- $V_w = S_x A_x \Delta h$

Where :-

- A is horizontal area and
- Δh is fall in head

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8. Specific Storage (S_s)

- In a saturated porous medium that is confined between two transmissive layers of rocks, water will be stored by a combination of two phenomena; water compression and aquifer expansion.
- As water is forced in to the system at a rate greater than it is being extracted, the water will compress and the matrix will expand to accommodate the excess.

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- In a unit of saturated porous matrix, the volume of water that will be taken in to storage under a unit increase in head, or the volume that will be released under a unit decrease in head is called specific storage.
- It is also the storage coefficient per unit saturated thickness of an aquifer.
- For confined aquifer, the relation between the specific storage and the storage coefficient is as follows:
- $Sc = S_s * b$

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Where: -

S = Storage coefficient (dimensionless),

b = aquifer thickness (m)

Specific Storage is also called elastic storage coefficient and is given by the following expression.

$$Ss=\rho g\ (\alpha+n\beta)$$

Where:-

Con,t…

- ρ =fluid (water) density,
- g=gravitational acceleration,
- α=aquifer compressibility,
- n= porosity,
- β=water compressibility.

Elastic storage is the only storage occurring in semi-confined and confined aquifers.

The end

Chapter three

1 Groundwater movement

1.1 Darcy's Law

Groundwater in its natural state is invariably moving. This movement is governed by established hydraulic principles. The rate of groundwater movement depends upon the slope of the hydraulic head or hydraulic gradient, and intrinsic aquifer and fluid properties. The flow through aquifers can be expressed by Darcy's law, which is one of the established hydraulic principles.

Henry Darcy, a French hydraulic engineer, observed that the rate of laminar flow of a fluid (of constant density and temperature) between two points in a porous medium is proportional to the hydraulic gradient (dh/dl) between the two points. The equation describing the rate of flow through a porous medium is known as Darcy's Law.

$Q = -KA\frac{dh}{dl}$ Where: - Q = volumetric flow rate (m^3/s)

K = hydraulic conductivity (m/s)

A = cross-sectional area of flow (m^2)

1.1.1 Formulation of Darcy's Law

The experimental verification of Darcy's law can be performed with water flowing at a rate Q through a cylinder of cross-sectional area A packed with sand and having a piezometric distance L apart. Total Energy head, or fluid potentials, above the datum plane may be expressed by Bernoulli equation as:

$$\frac{P_1}{\gamma_w}+\frac{{V_1}^2}{2g}+Z_1 = \frac{P_2}{\gamma_w}+\frac{{V_2}^2}{2g}+Z_2 + hL$$

Where: -

p = the pressure,

v= the velocity of flow,

g = the acceleration of gravity,

z = the elevation,

hL = the head loss and

γ_w = the specific weight of water.

Subscripts refer to the points of measurement. Since the velocity of flow in porous media is very small, the velocity head can be neglected ($\frac{{V_1}^2}{2g}$ ≈0) and thus the head loss can be obtained as:

$$hL = \left(\frac{P_1}{\gamma_w} + Z1\right) - \left(\frac{P_2}{\gamma_w} + Z2\right)$$

Therefore, the resulting head loss is defined as the potential loss with in the sand cylinder. This head loss is due to the energy loss by frictional resistance dissipated as heat energy. It follows that the head loss is independent of the inclination of the cylinder

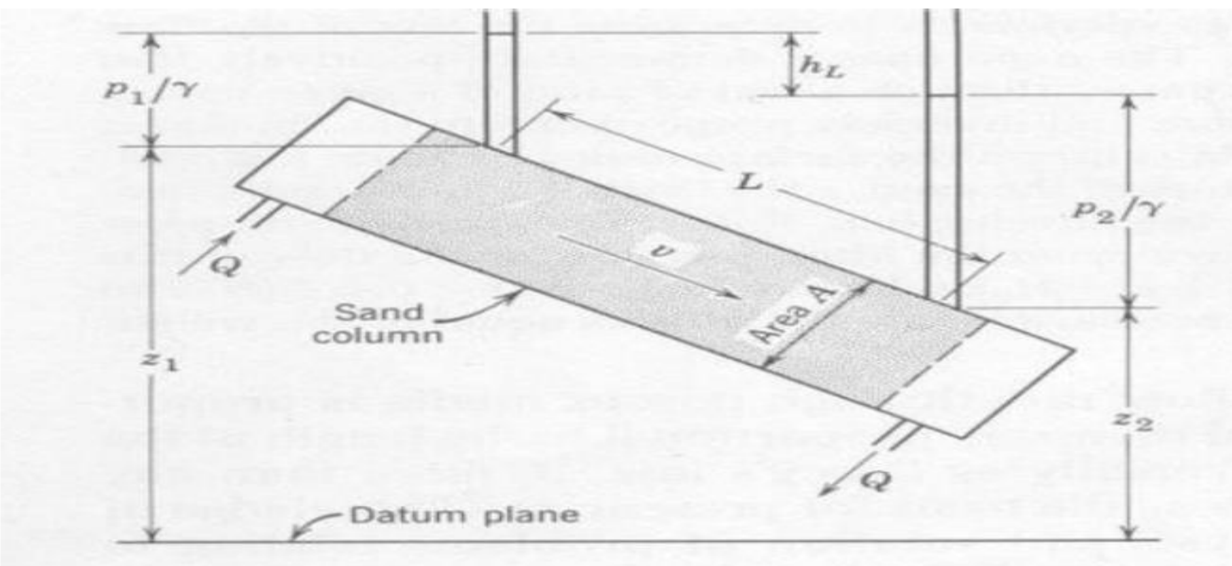


Fig 3.1 Pressure distribution and head loss in flow through a sand column

1.1.2 Specific Discharge

Specific discharge is also called as the Darcy Velocity. It is the discharge Q per cross-section area, A. The specific discharge is designated by q.

Form Darcy's equation, q = $Q/A = -k\dfrac{\Delta h}{\Delta \ell}$

Taking the limit as $\Delta\ell \rightarrow$ 0 i.e. $\lim_{\Delta\ell\rightarrow 0} it \; -K\dfrac{\Delta h}{\Delta \ell} = -k\dfrac{dh}{dl}$

$$- q = -k\frac{dh}{dl}$$

The Darcy velocity (v) or the specific discharge (q) assumes that flow occurs through the entire x-section of the material without regard to solids & pores. Actually, the flow is limited to the pore space only so that is the average interstitial velocity.

$V_a = \dfrac{Q}{nA}$ where n = porosity

$$V_a = \frac{Q_1 + Q_2 + Q_3 + ... + Q_n}{A_{actual}}$$

$$V_a = \frac{Q}{A * n}$$

$V_a > v$

To define the actual flow velocity (V_a), one must consider the microstructure of the rock material. The actual velocity is non-uniform, involving endless accelerations, decelerations, and changes in direction. Thus the actual velocity depends on specifying a precise point location within the medium.

1.1.3 Validity of Darcy's law

In general the Darcy's law holds well for

a. Saturated & unsaturated flow.
b. Steady & unsteady flow condition
c. Flow in aquifers and aquitards.
d. Flow in homogenous & heterogeneous media

e. Flow in isotropic & an isotropic media.

f. Flow in rocks and granular media.

Darcy's law is valid for laminar flow condition as it is governed by the linter law.

$$v = -k\left(\frac{dh}{d\ell}\right)^m, m = 1.0$$

In flow through pipes, it is the Reynolds number(R) to distinguish b/n laminar flow & turbulent flow.

$$N_R = \frac{Inertial\,force}{visouse\,force} = \frac{\rho v D}{\mu}$$

For the flow in porous media, v is the Darcy velocity and D is the effective grain size (d_{10}) of a formation/media. D_{10} for D is merely an approximation since measuring pore size distribution a complex research task.

Experiments show that Darcy's law is valid for $N_R < 1$ and does not go beyond seriously up to N_R =10. This is the upper limit to the validity of Darcy's laws.

Fortunately, natural underground flow occurs with $N_R < 1$. So Darcy's law is applicable. Deviations from Darcy's law can occur where steep hydraulic gradients exist; such as near pumped wells. Turbulent flow can contain large underground openings.

1.2 HYDRALIC CONDUCTIVITY

The famous Darcy's law (1856), which describes the flow of fluids through (inter-granular) porous media, was derived experimentally after hundreds of laboratory tests with the apparatus. In its basic form, this linear law states that the rate of fluid flow (Q) through a sand sample is directly proportional to the x-sectional area of the flow (A) and the loss of hydraulic head b/n two points of measurements (Δh), and it is inversely proportional to the length of the sample L.

$$Q = \frac{KA\Delta h}{L}\left[m^3/s\right]$$

Therefore, K is the proportionality constant of the law & called hydraulic conductivity and has the unit of velocity. It can also be called as the coefficient of permeability.

A medium has a unit hydraulic conductivity if it will transmit in unit time a unit volume of GW at prevailing kinematic viscosity through a cross section of unit area, measured at right angles to the direction of flow, under unit hydraulic gradient.

Generally, hydraulic conductivity is a coefficient of proportionality describing the rate at which water can move through a permeable medium. The density and kinematic viscosity of water must be considered in determining the hydraulic conductivity.

The general hydraulic equation of continuity of flow, which results from the principle of conservation of mass, is (for incompressible fluids).

$$Q = V_1 A_1 = V_2 A_2 = constant$$

From which $V = Q/A$ and relating it with Darcy's equation,

$$V = K \frac{\Delta h}{L}$$

Where

$\Delta h / L$ Is the hydraulic gradient

Δh Is the head loss along the distance L.

The hydraulic gradient is given by i.

$$i = \Delta h / L \quad \text{(dimensionless)}$$

v = Ki (another form of Darcy's equation)

Finally from above equation; hydraulic conductivity K can be determined as.

$$K = v / i$$

1.3 Intrinsic Permeability

The permeability of a porous media is the ease with which a fluid can flow through that medium. In other words, permeability characterizes the ability of a porous medium to transmit a fluid. In hydrogeology, the permeability is referred to as the intrinsic permeability. It is dependent only on the physical properties of the porous medium: grain size, grain shape and arrangement, pore interconnections etc… On the other hand hydraulic conductivity is dependent on the properties of both porous media and the fluid. The relationship between intrinsic permeability (K_i) and hydraulic conductivity (K) is expressed through the following formula.

$$K_i = K\mu/ \rho g$$

Where:-

μ: absolute viscosity (dynamic viscosity)

ρ: density of fluid

ki: intrinsic permeability (L^2)

Viscosity of a fluid is the property which describes its resistance to flow.

The dynamic viscosity (μ) and the density of fluid (ρ) are related through the kinematic viscosity (ν): $\nu = \mu/\rho g$

Therefore, knowing the kinematic viscosity, which is a function of temperature, the intrinsic permeability can be determined from field experiments having the value of hydraulic conductivity.

$$K_i = K\ \nu/g$$

Although it is much better to express the permeability in units of area (m^2 or cm^2), for reasons of consistency and easier use in other formulas, in practice it is more commonly given in Darcy's (which a tribute to oil industry).

1 Darcy = $9.87x10^{-9}cm^2$ (or approximately $1x10^{-8}cm^2$)

1.4 Determinations of Hydraulic Conductivity

Hydraulic conductivity in saturated zones can be determined by variety of techniques. These include, analytical or empirical methods, laboratory methods, tracer tests, augur hole tests and pumping tests of wells.

1.4.1 Empirical formulas

Numerous investigators have studied the relationship of hydraulic conductivity or permeability to the properties of porous media.

Most commonly used relationship of such a formula has the following general formula.

$K = Cd^2$ Where:- C is the dimensionless constant

And in some specific terms the formula is expressed as

$K = f_s f_n d^2$ Where

f_s = the grain shape factor

f_n = the porosity factor

d= the characteristic grain diameter.

Few formulas give reliable estimates of results because of the difficulty of including all possible variables in porous media.

It should be clearly understood that these empirical formulas have various limits of application and give just approximate values of hydraulic conductivity for small point samples. Since they are derived for different experimental materials and conditions, it is very common that several formulas applied to the sample will yield several very different values of hydraulic conductivity (K). For preliminary works the value of C is often taken as 100 and d is the effective grain size (d_{10})

1.4.2 Laboratory methods

In the laboratory, hydraulic conductivity is determined by permeameters in which flow is maintained through a small sample of material while measurements of flow rate and head loss are made. A permeameter is a laboratory device used to measure the intrinsic permeability and hydraulic conductivity of a soil or rock sample.

There are two types of permeameters:

a) Constant head permeameter
b) Variable head permeameter

a) Constant head permeameter

The constant head permeameter is the one which can measure the hydraulic conductivities of consolidated and unconsolidated formations under low heads. Water enters the medium cylinder from the bottom and is collected as overflow after passing upward through the material/sample (See figure 3.3). The hydraulic conductivity is determined from the equation of Darcy as

$Q = \frac{V}{t} = \frac{KhA}{L}$ from this

$K = \frac{V*L}{A*h*t}$ Where:-

L = length of sample;

t = time of measurement;

A = Area of sample;

h = head loss for the flow through the sample for a given particular test and

V = Volume of water collected through time t after passing through sample.

b) Variable(falling) head permeameter

Here water is added to the fall tube; it flows upward through the cylindrical sample and collected as an overflow. The test in falling head permeameter consists of measuring the rate of fall of the water level in the tube and collecting volume of water overflow through time.

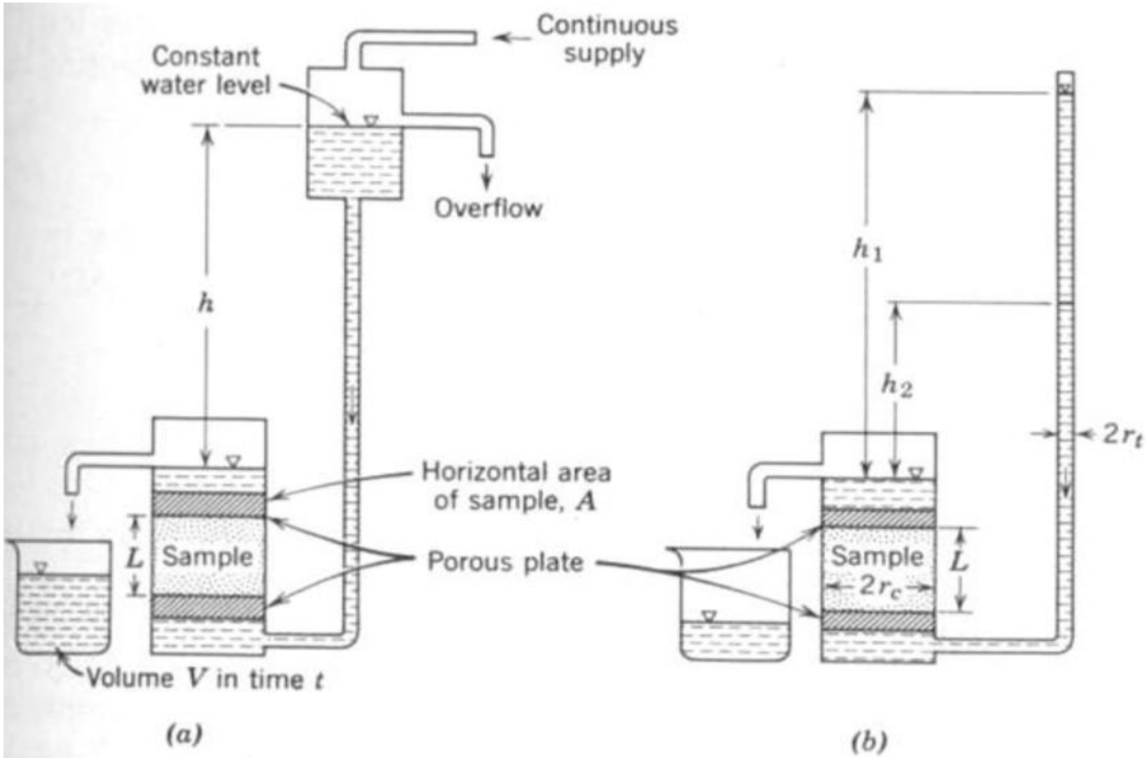


Fig 3.2 Arrangement of constant (a) and Variable (b) head permeameters

The flow rate in the tube is

$Q_{tube} = a_{tube}$ x dh/dt

Where: - a is the area of the tube and

dh/dt is the rate of fall of head in the tube

And the rate of flow in the sample is governed by Darcy's law.

Thus the flow rate through the sample is

$Q_{sample} = -KiA$

Where A is the area of the sample

$i = h/L$

From continuity equation, $Q_{tube} = Q_{sample}$

Therefore, adh/dt = -KAh/L

aLdh/h = -KAdt

$$-aL\int_{h1}^{h2} dh/h = KA\int_{0}^{t} dt$$

Therefore, from integration,

$$K = \frac{a\,L\ln(h1/h2)}{At}$$

1. **Tracer tests**

Field determination of hydraulic conductivity can be made by measuring the time interval for a water tracer to travel b/n two observation wells or test holes. The tracer can be a die such as sodium fluorescein or salt.

Consider the unconfined aquifer case below where the GW flow is from point A to point B.

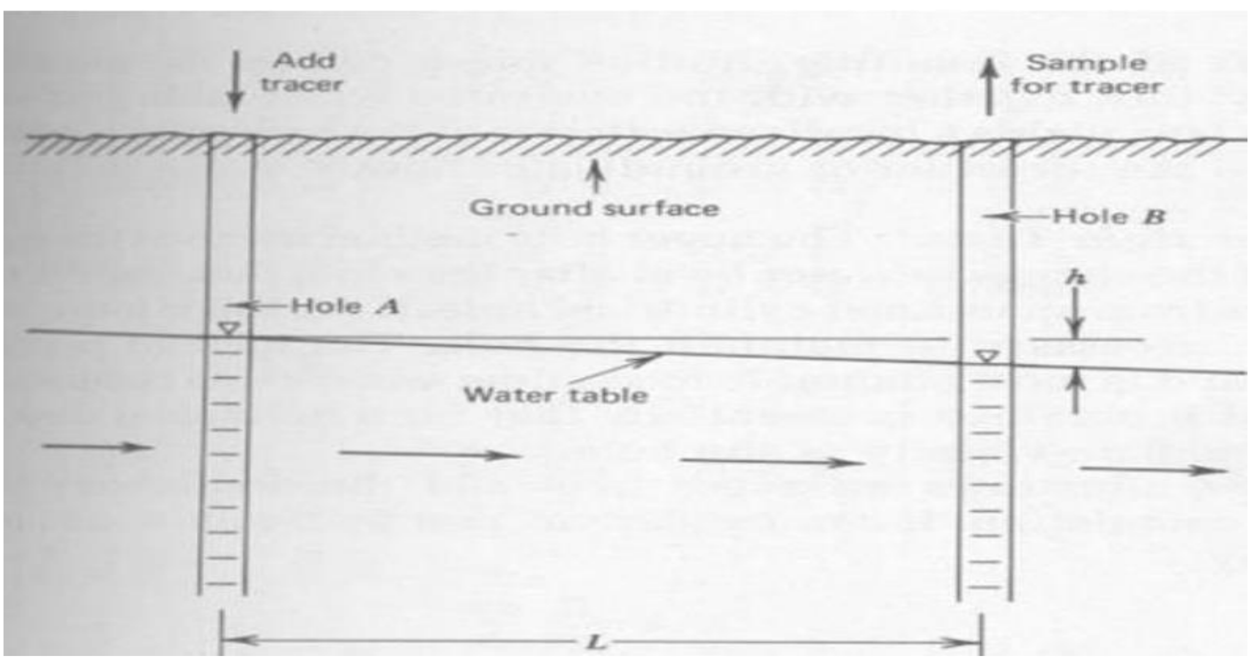


Fig 3.4 Determination of hydraulic conductivity using tracer tests

The tracer is injected in hole A as a slug after which samples of water are taken from hole B to determine the time passage of the tracer. B/c the tracer flows through the aquifer with the average interstitial velocity, va, then;

va = Kh/(nL)

Where K is the hydraulic conductivity, n is the porosity, L is the distance b/n two points and h is the difference in head causing flow b/n the points.

But va = L/t

Where

t: is the travel time interval of tracer b/n two holes

Equating the two equations above;

K= nL^2/ht

Though the method is simple, the results of this method may face serious limitations in the field. Such as

1. The holes need to be close together; otherwise, the travel time interval can excessively be long. For this requirement, the value of K is highly localized.
2. Unless the flow direction is accurately known, the tracer may miss the d/s hole entirely. Multiple sampling holes may help, but costly.
3. If the aquifer stratified with layers having different hydraulic conductivities, the first arrival of the tracer will result in conductivity considerably larger than the average for the aquifer.
4. **Auger hole method**

This method is relatively simple method and most adaptable to shallow water table conditions. The value of K obtained is essentially that for a horizontal direction in the immediate vicinity of the hole.

The value of K is given by

K = C/864 (dy/dt)

Where

dy/dt is measured rate of rise (cm/sec)

C = Constant (dimensionless)

K = hydraulic conductivity (m/day)

The factor 864 yields k values in m/day. And the value of C which is defined based on $\frac{Lw}{rw}$ and $\frac{y}{Lw}$ can be obtained from tables.

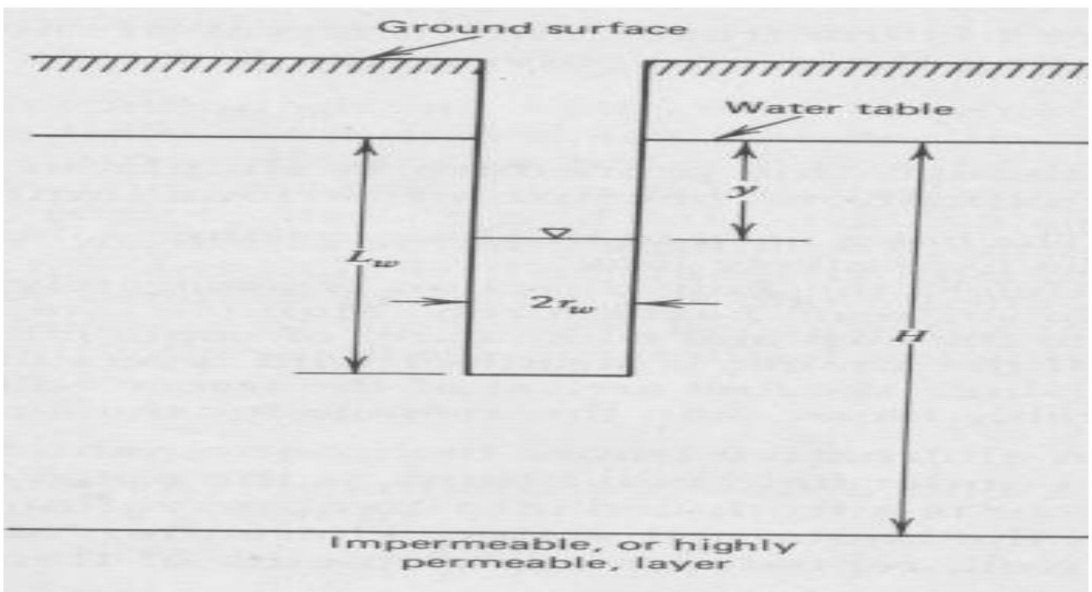


Fig 3.5 Diagram an Auger hole for determining the hydraulic conductivity

5. **Pumping tests of wells**

The most reliable method of estimating aquifer hydraulic conductivity is the pumping test of wells. Based on observations of water levels near pumping wells an integrated K value over sizable aquifer section can be obtained. It is the superior method where the sample is not disturbed.

1.5 Aquifer flow and transmissivity

1.5.1 Aquifer Flow

Aquifer flow can be one dimensional, two dimensional or more. Darcy's equation can be used to calculate one dimensional flow in aquifers. To obtain the volume rate of flow in aquifer, Darcy's velocity is multiplied by cross sectional area of an aquifer normal to the flow.

$Q = Av = -AKdh/dl = Aki$ i is the hydraulic gradient

$Q = -WbKi$

1.5.2 Transmissivity(T)

Transmissivity T is a measure of the amount of water that can be transmitted horizontally through a unit width by the fully saturated thickness of an aquifer under a hydraulic gradient equal to 1. Transmissivity is equal to the hydraulic conductivity multiplied by the saturated thickness of the aquifer and is given by:

$T = Kb$

Where:-

K=hydraulic conductivity [LT-1]

b = is the saturated thickness of an aquifer [L].

The saturated thickness for confined aquifer is fairly constant and hence the value of T is constant; however, the saturated thickness for unconfined aquifers is variable as the water table varies. Hence the transmissivity for unconfined aquifers vary as a function of the water table variation.

1.5.3 Homogeneity and Isotropy

If hydraulic conductivity is consistent throughout a formation, regardless of position, the formation is homogeneous. If hydraulic conductivity within a formation is dependent on location, the formation is heterogeneous. When hydraulic conductivity is independent of the direction of measurement at a point within a formation, the formation is isotropic at that point. If the hydraulic conductivity varies with the direction of measurement at a point within a formation, the formation is anisotropic at that point. Figure 2-4 is a graphical representation of homogeneity and isotropy.

Geologic material is very rarely homogeneous in all directions. A more probable condition is that the properties, such as hydraulic conductivity, are approximately constant in one direction. This condition results because: a) of effects of the shape of soil particles and b) different materials incorporate the alluvium at different locations. As geologic strata are formed, individual particles usually rest with their flat sides down in a process called imbrication. Consequently, flow is generally less restricted in the horizontal direction than the vertical and Kx is greater than Kz for most situations. Layered heterogeneity occurs when stratum of homogeneous, isotropic materials are overlain upon each other

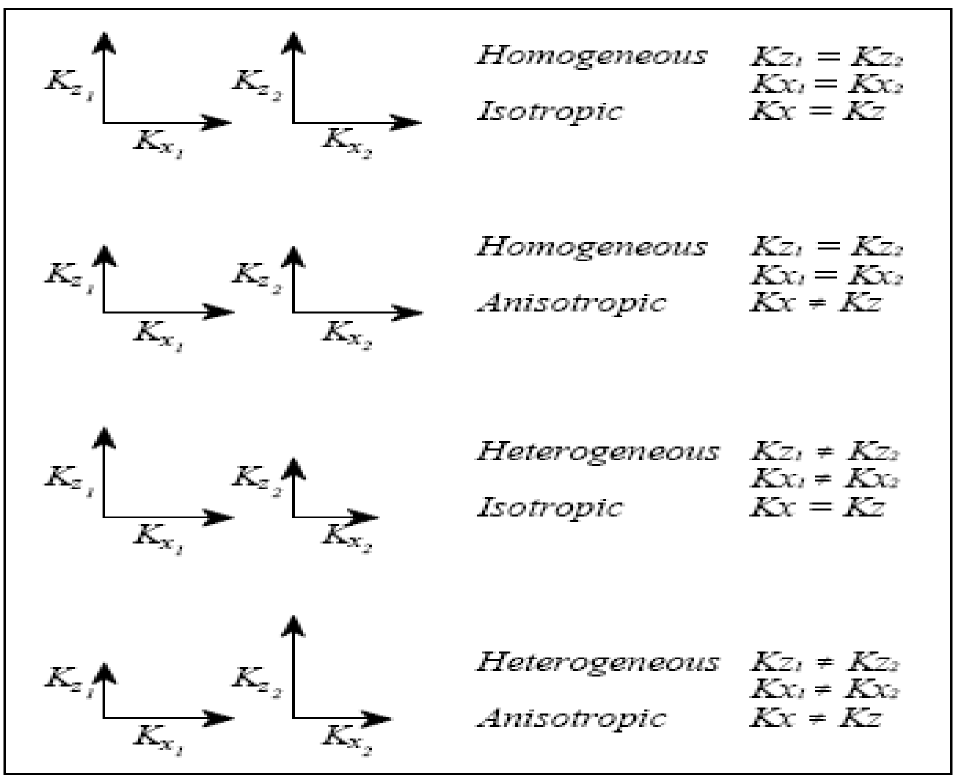


Figure 3.6 Homogeneity and isotropy

1.6 Flow in Stratified Media

Flow through stratified media can be described through the definition of a hydraulically equivalent conductivity (or effective hydraulic conductivity). Expressions for horizontal and vertical equivalent conductivities can be generalized from expressions developed for flow through porous media comprised of three parallel homogeneous, isotropic strata.

1.6.1 Horizontal flow

Consider an aquifer of n horizontal layers each individually isotropic, with different thickness a hydraulic conductivity (refer figure 2.6).

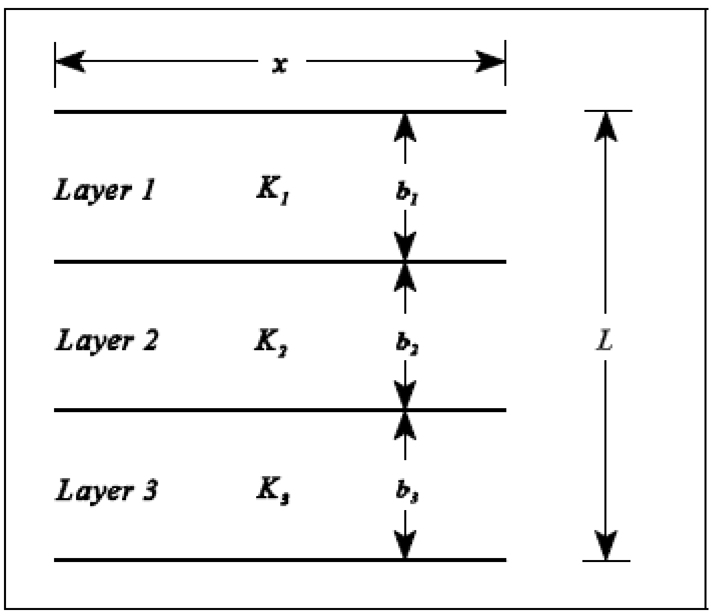


Figure 3.7 Stratified media

For horizontal flow parallel to the layers, the flow per unit width in the upper layer, q1 is given by $q_1 = K_1 i b_1$ Where i is the hydraulic gradient

K_1 and b_1 are indicated in the figure.

Similarly, $q_2 = K_2 i z_2$ and the total flow q_x in the horizontal direction is given by:

$$q_x = \sum_{i=1}^{n} q_i = i(K_1b_1 + K_2b_2 + \ldots\ldots\ldots + K_nb_n) \qquad (1)$$

i for the horizontal flow is the same in each layer.

If the whole aquifer system is taken as homogeneous; then the total flow is:

$$q_x = K_xi(b_1 + b_2 + \ldots\ldots\ldots + b_n) \qquad (2)$$

Where K_x is the horizontal hydraulic conductivity for the entire system

Equating the two equations and solving for the K_x yields:

$$K_x = \frac{(K1b1 + K2b2 + \cdots\ldots\ldots + Knbn)}{(b1 + b2 + \ldots\ldots + bn)}$$

If the thicknesses are equal, then

$K_x = (K_1 + K_2 + \ldots\ldots\ldots. + Kn)/n$ (Arithmetic mean) Where n is the number of thicknesses.

1.6.2 Vertical flow

Consider an aquifer system consisting of n horizontal layers each individually isotropic, with different thickness values. If there is a vertical flow through the system, the flow q per unit horizontal area for the top layer can be expressed as: $q_z = K_1\Delta h_1/b_1$

Where Δh_1 is the total head loss across the first layer

Solving for Δh_1, $\Delta h1 = q_zb_1/K_1$

Similarly for the second layer and n layer:

$\Delta h_2 = q_zb_2/K_2 \qquad \Delta h_n = q_zb_n/K_n$

The total head loss ($\Delta h_t = \Delta H$) for vertical flow through all the layers of the system can be calculated as the sum of the head losses in each layer,

$$\Delta h_t = \Delta H = q_z(b_1/K_1 + b_2/K_2 + \ldots\ldots\ldots + b_n/K_n) \qquad (3)$$

In homogenous system, the vertical flow can be expressed as

$q_z = K_z \, \Delta H / (b_1 + b_2 + \ldots . b_n)$ and $\Delta H = q_z \underline{(b_1 + b_2 + \ldots . b_n)}$ (4) K_z

Equating the two equations (3 and 4) and solving for K_z,

$K_z = (b_1 + b_2 + \ldots . b_n) / ((b_1/K_1 + b_2/K_2 + \ldots \ldots + b_n/K_n)$

And for equal thickness; $K_z = n / ((1/K_1 + 1K_2 + \ldots \ldots + 1/K_n)$ (Harmonic mean)

The statement, horizontal hydraulic conductivity (K_x) is greater than the vertical hydraulic conductivity (K_z) can also show with the help of the above derivations.

Mathematically, harmonic mean is less than arithmetic mean; thus, $K_x > K_z$.

For two dimensional flows in anisotropic media, the approximate value of K must be selected for the direction of flow. For directions other than horizontal (K_x) and vertical (K_z) the K value K_β can be obtained from:

$1/K_\beta = \cos^2\beta / K_x + \sin^2 \beta / K_z$ Where β is the angle b/n K_β and the horizontal

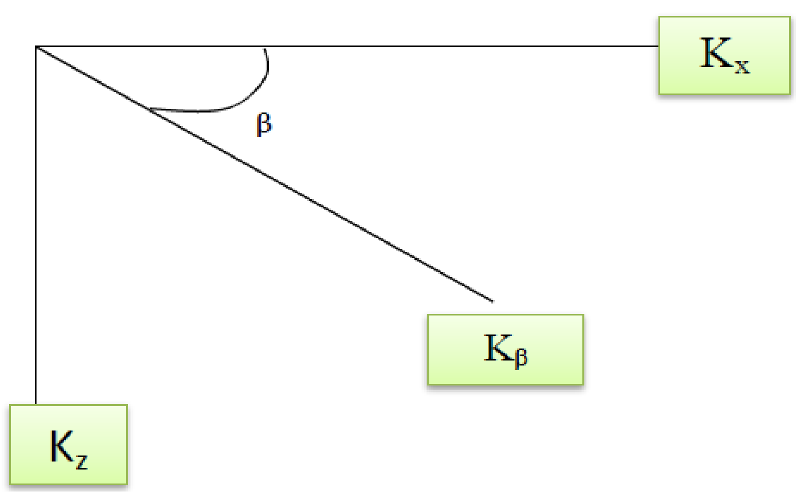


Fig 3.8 Hydraulic conductivity in other directions

1.6.3 Average Hydraulic Conductivity

The hydraulic conductivity in horizontal direction (Kx) and in the vertical direction (Kz) defined previously were the average hydraulic conductivities in their respective directions. However, it is not customary to determine the hydraulic conductivities of each layer and determine the average hydraulic conductivities, unless in rare circumstances as limited in research and academic purposes. Field methods such as pumping test, auger hole methods, tracer tests etc… allow the computation of average hydraulic conductivity of a formation.

The overall average hydraulic conductivity is computed from the geometric mean or the arithmetic mean of the logarithm of the average horizontal and vertical hydraulic conductivities.

$K_{av} = \sqrt{K_x . K_z}$ or $\log K_{av} = (\log K_x + \log K_z)/2$

1.7 Flow nets and Groundwater flow directions

1.7.1 Flow nets

Flow net is a network of flow lines and equipotential lines intersecting at right angles to each other. The imaginary path which a particle of water follows in its course of seepage through a saturated soil mass is called flow line. An equipotential line is the line which joins points with equal potential head. Equipotential lines are lines that intersect the flow lines at right angles. At all points along the equipotential line, the water would rise in a piezometric tube to a certain elevation known as piezometric level.

For specified boundary conditions, flow lines and equipotential lines can be mapped in to two dimensions to form a flow net. The two sets of lines form an orthogonal pattern of small squares. See fig. below.

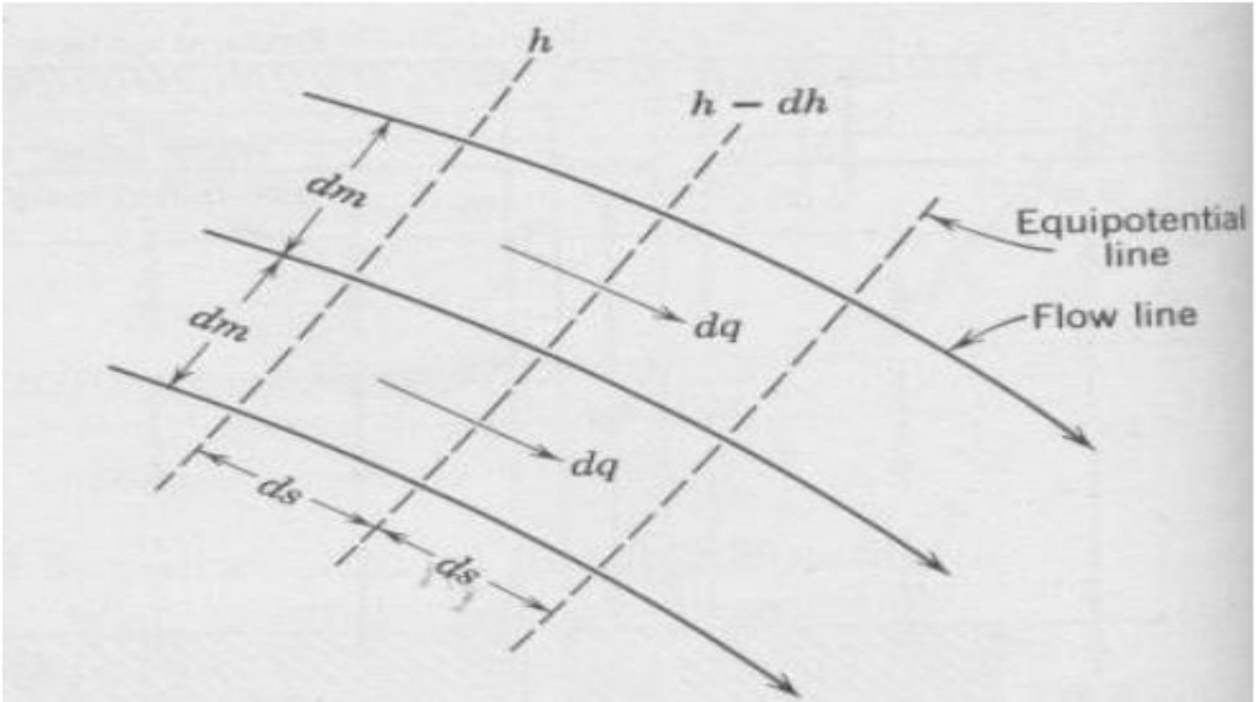


Fig 3.9 Portion of an orthogonal flow net formed by flow and equipotential lines

Basically flow net is constructed to quantify the flow rate through a medium. Consider the portion of a flow net shown in figure above. The hydraulic gradient is given by:

i = -dh/ds and the constant flow rate, between two adjacent lines is given by

q = -K.dm.dh/ds for unit thickness. But for the squares of the flow net, the approximation ds ~ dm can be made. Therefore, the above equation reduces to

q = Kdh . Applying this to an entire flow net, where the total head loss h is divided in to n squares b/n two adjacent flow lines, then

dh = h/n

If the flow field is divided in to m channels by flow lines, then the total flow rate is:

Q = mq = Kmh/n

Thus the geometry of the flow net, together with the hydraulic conductivity and head loss, enables the total flow to be computed directly.

Properties of a flow net

- Flow lines and equipotential lines are smooth curves.
- Flow lines and equipotential lines meet at right angles to each other.
- No two flow lines or equipotential lines cross each other
- No two flow lines or equipotential lines start at the same point.

1.7.2 Boundary Conditions

Flow of water in earth mass is in general three dimensional. Since the analysis of three dimensional flows is too complicated, the flow problems are usually solved in the assumption that the flow is two dimensional.

The three common types of boundaries of GW flow are:

- Impermeable (No flow boundary)
- Constant head boundary(head not varies)
- Water table (Variable head boundary)

i) Impermeable(No flow boundary)

There is no flow through such a boundary. Flow lines run parallel to the boundary and GW head contour lines (equipotential lines) are perpendicular to this boundary.

ii) Constant head boundary

This could be the boundary with open water bodies such as perennial rivers, lakes or seas. The flow lines are perpendicular to this open water bodies.

iii) Water table (variable head) boundary

This boundary may be influenced by recharge or discharge from an aquifer. Water table may be served as constant head boundary if there is no recharge/discharge and not influenced by other phenomena in which water table is fairly constant.

Flow net construction

There are many methods in use for the construction of flow nets. Some of the important methods are:

a. Analytical methods
b. Electrical analog methods
c. Scaled model method
d. Graphical method

The usual method of obtaining flow nets is a graphical trial and error sketching method, sometimes called a Forchiemer solution. This is the quickest method and the most practical of all the available methods.

1.7.3 Ground water Flow Direction

The direction of GW flow in a localized area of an aquifer can be determined if at least three recordings of water table (piezoelectric surface) elevations are available. Figure below illustrates the principle of finding the position of water table in three dimensions using data from three monitoring wells. In a map or two dimensional views, the water table is represented by the contour lines which connect points with the same hydraulic head. The fastest way to construct contours is by linear triangulation as shown in the figure (three well methods). The direction of GW flow is indicated by the arrow drawn perpendicular to and 'down' the contour lines. This is also the direction of the dip of the water table approximated by the plate. It is very important to understand that the flow direction determined this way is representative only of the local area covered by the three monitoring wells. Depending on the hydrologic condition this direction may change in nearby aquifer portion. For that reason more observation points are usually established during a hydro geologic investigation to construct a reliable contour maps.

Contouring Methods

- Manual contouring
- Contouring with computer programes

Manual contouring

Manual contouring is practically always used in GW studies, either as the only method or in conjunction with computer based methods. A complete reliance on software contouring could lead to erroneous conclusions since computer programes are unable to recognize interpretations apparent to a GW professional such as presence of geologic boundaries, varying porous media, influence of surface water bodies or principles of GW flow. Thus manual contouring and/or manual reinterpretation of a computer generated maps are essential and integral parts of hydro geologic studies.

Manual contouring is essentially based on triangular linear interpolation combined with the hydro geologic experience of the interpreter. The first draft map is not necessarily an exact linear interpolation b/n data points. Rather it is an interpretation of the hydro geologic and hydrologic conditions with contours that roughly follow numeric data on water table (or piezometric surface) elevations.

One of the most important aspects of constructing contour maps in alluvial aquifers is to determine the relationship between ground and surface waters. Sometimes water flows from river to an aquifer (which in hydraulic contact with a river) and vice versa. If the flow is from aquifer to river/stream, then it is called gaining (effluent) stream. However, if the flow of water is from river to an aquifer, then the stream is said to be losing (influent) stream. In some complicated situations, the two basic cases can co-exist.

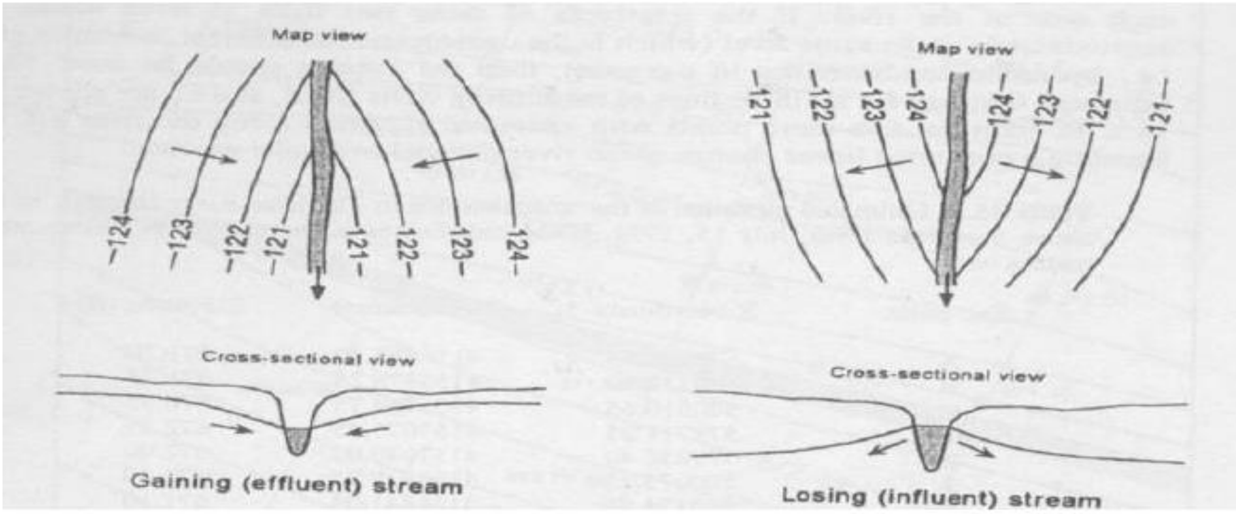


Fig 3.10 Explanation of losing and gaining streams

In hydraulic terms, the contact between the aquifer and the surface stream is equipotential boundary. This does not mean that everywhere along the contact the hydraulic head is the same. Since both the river water and the groundwater are flowing, there is a hydraulic gradient along the contact. If enough measurements of a river stage are available, it is relatively easy to draw the water table contours in the vicinity of the river and to finish them along the river aquifer contact.

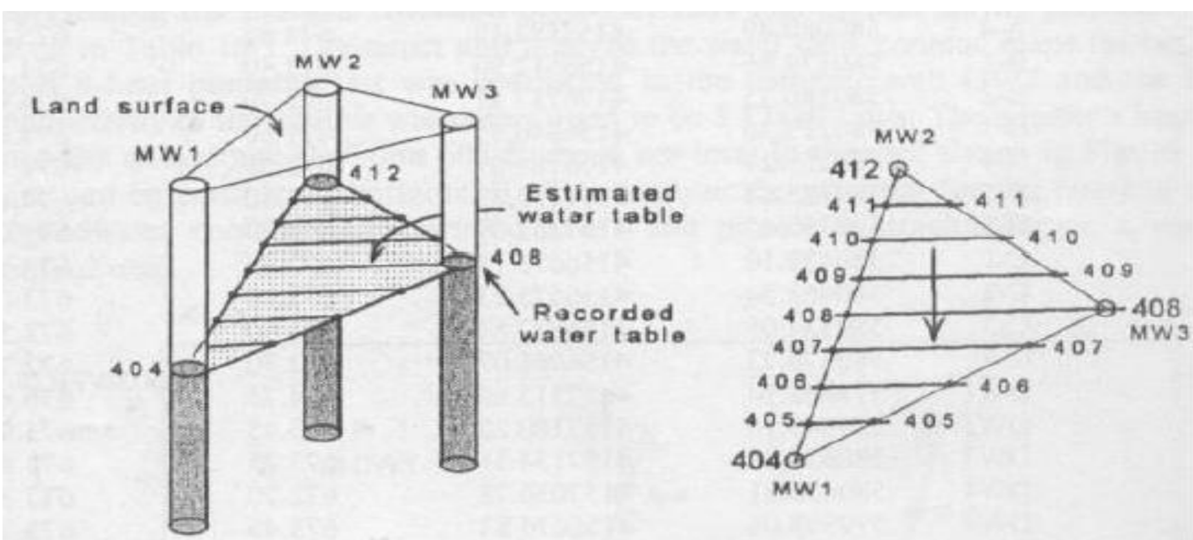


Fig 3.11 Contouring using the three well methods

Contouring with computer programs

In this computer era, we can have to different programs which undertake the contouring of groundwater levels. Some of them may be Arc View GIS, Surfer Golden software, AutoCAD and so on. Most of them need elevation of GW levels as an input. But the drawn contours should be checked against the true conditions in some situations since software usually depend on the given data and not able to catch up the natural condition in the field

1.8 General Flow Equations

1.8.1 Confined aquifer

The governing flow equation for confined aquifers is developed from application of the law of mass conservation (continuity principle) to the elemental volume shown in Figure 3.12.

Continuity is given by:

Rate of mass accumulation = Rate of mass inflow - Rate of mass outflow

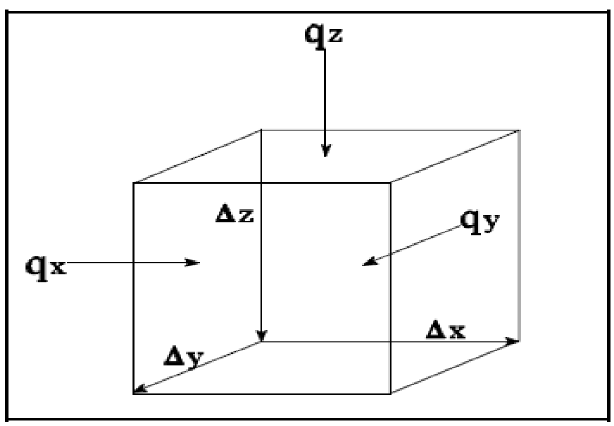


Fig 3.12: Elemental control volume

Integrating the conservation of mass (under constant density) with Darcy's Law, the general flow equation in three dimensions for a heterogeneous anisotropic material is derived:

$$S_s \frac{\partial h}{\partial t} = \frac{\partial}{\partial x}\left(kx\frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(ky\frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(kz\frac{\partial h}{\partial z}\right) \text{-----------------} (1)$$

Equation above is the general flow equation in three dimensions for a heterogeneous anisotropic material. Discharge (from a pumping well, etc.) or recharge to or from the control volume is represented as volumetric flux per unit volume (L3/T/L3 = 1/T):

$$S_s \frac{\partial h}{\partial t} + W = \frac{\partial}{\partial x}\left(kx \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(ky \frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(kz \frac{\partial h}{\partial z}\right)$$

Where: - W = volumetric flux per unit volume

Assuming that the material is homogenous, i.e K does not vary with position, equation [1] can be written as: - $S_s \frac{\partial h}{\partial t} = Kx \frac{\partial}{\partial x}\left(\frac{\partial h}{\partial x}\right) + Ky \frac{\partial}{\partial y}\left(\frac{\partial h}{\partial y}\right) + Kz \frac{\partial}{\partial z}\left(\frac{\partial h}{\partial z}\right)$--------------- (2)

If the material is both homogenous and isotropic, i.e Kx=Ky=Kz, then equation 2 becomes :

$$S_s \frac{\partial h}{\partial t} = K\left[\frac{\partial}{\partial x}\left(\frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(\frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(\frac{\partial h}{\partial z}\right)\right]$$

Or, combining partial derivatives

$$S_s \frac{\partial h}{\partial t} = K\left[\left(\frac{\partial h^2}{\partial x}\right) + \left(\frac{\partial h^2}{\partial y}\right) + \left(\frac{\partial h^2}{\partial z}\right)\right] \text{-------------------------------- (3)}$$

Using the definitions for storage coefficient,($S=bS_s$), and transmissivity ,(T=Kb), where b is the aquifer thickness and equation 3 becomes

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \frac{S}{T}\frac{\partial h}{\partial t} \text{ -------------------------------------- (4)}$$

If the flow is steady-state, the hydraulic head does not vary with time and equation 4 becomes

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0 \qquad \text{----------------------------------- (5)}$$

Equation 5 is known as the Laplace equation

1.8.2 Unconfined aquifer

In an unconfined aquifer, the saturated thickness of the aquifer changes with time as the hydraulic head changes. Therefore, the ability of the aquifer to transmit water (the transmissivity) is not constant:

$$\frac{\partial}{\partial x}\left(kxh\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(kyh\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(kzh\frac{\partial h}{\partial z}\right)=S_y\frac{\partial h}{\partial t} \text{ --------------------- (6)}$$

Where:- $S_{Y=}$ specific yield [dimensionless]

For a homogeneous, isotropic aquifer, the general equation governing unconfined flow is known as the Boussinesq equation and is given by:

$$\frac{\partial}{\partial x}\left(h\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(h\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(h\frac{\partial h}{\partial z}\right)=\frac{S_y}{K}\frac{\partial h}{\partial t} \text{ -------------------------- (7)}$$

If the change in the elevation of the water table is small in comparison to the saturated thickness of the aquifer, the variable thickness h can be replaced with an average thickness b that is assumed to be constant over the aquifer. Equation -7 can then be linearized to the form:

$$\left(\frac{\partial h^2}{\partial x}\right)+\left(\frac{\partial h^2}{\partial y}\right)+\left(\frac{\partial h^2}{\partial z}\right)=\frac{S_y}{Kb}\frac{\partial h}{\partial t} \text{ ----------------------------------- (8)}$$

Regional ground water flow occurs in ground water basins which usually occupy large areas. Regional flow may influenced by local groundwater flow phenomena. For example, the construction of canals may influence the regional natural flow. Another example concerns wells. Pumping from well May also affects the regional flow and this can often be observed on groundwater head contour lines parallel to the canals on the maps. Estimates on local flow of groundwater can be obtained using groundwater head contour maps, flownets or numerical groundwater models. Traditionally, however, these local flow problems were also solved by analytical methods. In these methods the differential Darcy and continuity equations are solved in a direct way; either separately or combined. We will briefly discuss the analytical methods by presenting cases of the flow between canals or two water bodies and the flow to a well.

1 Chapter four

1.1 Groundwater flow in confined aquifer between two water bodies

Fig. 4.1 shows a very wide confined aquifer of depth H connecting two water bodies. A section of the aquifer of unit width is considered. The piezometric head at the upstream end is H_0 and at a distance X from the upstream end is h. The relevant Darcy equation is: $q_x = -K_x (\partial h / \partial X)$.

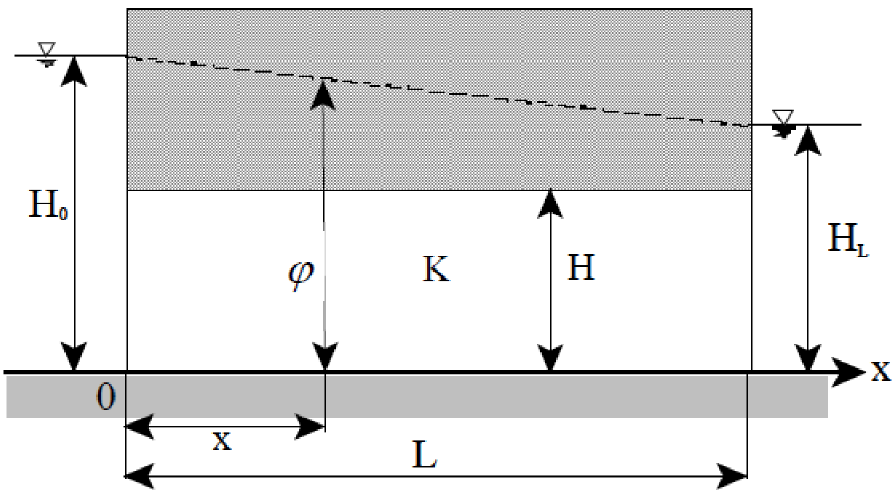


Figure 1.1 Confined groundwater flow between two water bodies

For one-dimensional flow in the X-direction only the continuity equation for steady flow simplifies to:

$\partial^2 h / \partial X^2 = 0$

Integrating twice $\Rightarrow$ $h = C_1 X + C_2$

The boundary conditions are:

(i) At X= 0, $H = H_0$ hence, $C_2 = H_0$

(ii) At X = L, $H = H_L$ hence, $C_1 = -\left(\dfrac{H_0 - H_L}{L}\right)$

Up on substitution of the boundary conditions C_1 and C_2

$$H = H_0 - \left(\frac{H_0 - H_L}{L} \right) X$$

This is the equation of the hydraulic grade line, which is shown to vary linearly from H_0 to H_L.

By Darcy Law, the discharge per unit width of the aquifer is:

$$q = -K\left(\frac{\partial h}{\partial X} \right) = -K * -\left(\frac{H_0 - H_L}{L} \right)$$

$\Rightarrow q = K\left(\frac{H_0 - H_L}{L} \right)$, And the total discharge with a thickness, H is $Q = KH\left(\frac{H_0 - H_L}{L} \right)$

Where KH= Transmissivity (m^2/s)

Average groundwater velocity, $V = \frac{q}{n_e} = \frac{K}{n_e}\left(\frac{H_0 - H_L}{L} \right)$

To compute travel time, $V = \frac{dS}{dt} \Rightarrow t = \int \frac{dS}{V} = \int_0^L \frac{dX}{V} \Rightarrow t = \frac{n_e L}{q} = \frac{n_e L^2}{K(H_0 - H_L)}$

Example 4.1: In order to determine the groundwater discharge, velocity and travel time, hypothetical aquifer parametric values are given below. H_0=20 m, H_L=19 m, B=10 m, K=10 m/d, L= 1000 m, n_e=0.2. Find Q, q, V, and t.

Solution: $q = K\left(\frac{H_0 - H_L}{L} \right)$=10*1/1000=0.01 m/day

Q=q*H=0.01*10=0.1 m^3/day

$$V = \frac{q}{n_e} = \frac{0.01}{0.2} = 0.05 m/day$$

$$t = \frac{n_e L}{q} = \frac{0.2*1000}{0.01} = 20{,}000 days$$

1.2 Groundwater flow in an unconfined aquifer

In unconfined aquifers the free surface of the water table, known as phreatic surface, has the boundary condition of constant pressure equal to atmospheric pressure. These boundary conditions cause considerable difficulties in analytical solutions of steady unconfined flow problems by using the Laplace equation.

Consider an unconfined aquifer is above a horizontal impermeable base;

- The porous medium is homogeneous (K = constant);
- The aquifer receives uniform recharge (w = constant) on the top; w is defined as amount of water entering to aquifer per unit length and width per unit time.
- The aquifer is bounded by two rivers of constant stages h0 and hL.
- Although flow is two-dimensional in the cross-section, vertical flow velocity is much smaller than the horizontal flow so that the flow is assumed to be one-dimensional horizontal flow

A simplified approach based on the assumptions suggested by Dupuit (1863) which gives reasonably good results in relatively easier manner is described below.

1. The curvature of the free surface is very small so that the streamlines can be assumed to be horizontal at all sections.
2. The hydraulic grade line is equal to the free surface slope and does not vary with depth.
3. The flow in aquifers is horizontal and that in aquitard is vertical.

As shown in Fig. 4.2 Further, there is a recharge at a constant rate of W (m^3/s) per unit horizontal area due to infiltration from the top of the aquifer. The aquifer is of infinite length and hence one dimensional method of analysis is adopted. For a unit width of aquifer:

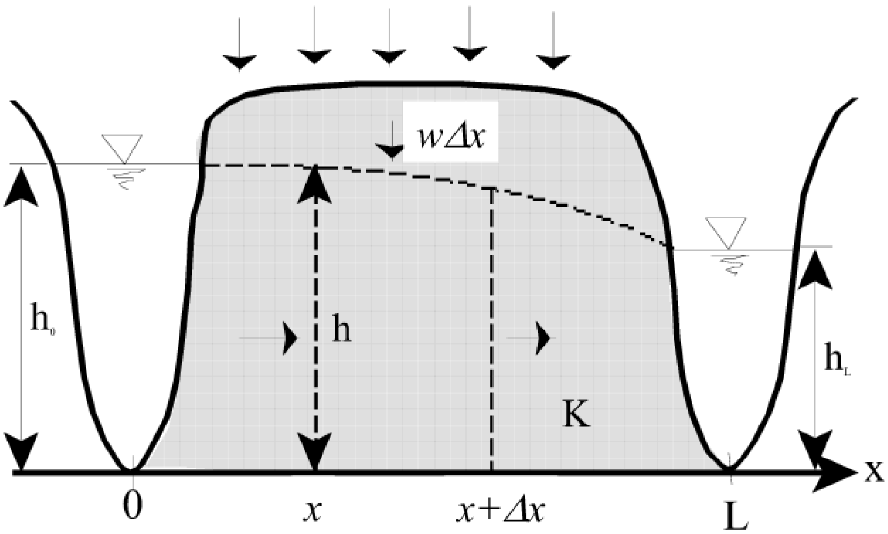


Figure 1.2: Unconfined groundwater flow between two water bodies

The water balance of the control volume taking the reference level of an aquifer at the bottom of the aquifer and the height equals to saturated thickness of the aquifer is:

$$hq_x + w\Delta x = hq_x + \frac{\partial (hq_x)}{\partial x}\Delta x$$

Substituting Darcy's equation and rearranging terms we get:

$$\frac{\partial}{\partial x}\left(h\frac{\partial h}{\partial x}\right) + \frac{w}{K} = 0$$

The equation is the nonlinear governing groundwater flow equation in unconfined aquifer.

Rearranging we get, $\left(\frac{\partial^2 h^2}{\partial x^2}\right) + \frac{2w}{K} = 0$

Integrating this equation twice w.r.t x gives,

$$h^2 + \left(\frac{w}{K}\right)x^2 = C_1 x + C_2 \qquad (2.23)$$

Where C_1 and C_2 are constants of integration and must be determined from the boundary conditions.

The boundary conditions are:

(iii) At X= 0, h = h_0 hence, $C_2 = h_0^2$

(iv) At X = L, h = h_L hence, $C_1 = -\left(\frac{h_0^{\ 2} - h_L^{\ 2}}{L}\right) + \frac{w}{K}L$

Thus substituting C_1 and C_2 in Eq. (2.23) gives:

$$h^2 = h_0^{\ 2} - \left(\frac{h_0^{\ 2} - h_L^{\ 2}}{L} - \frac{wL}{K}\right)x - \frac{w}{K}x^2$$

This is not equation of a straight line.

The unit width discharge in the aquifer will be:

$q_x = -Kh\frac{\partial h}{\partial x} = \frac{K}{2}\left(\frac{h_0^{\ 2} - h_L^{\ 2}}{L} - \frac{wL}{K}\right) + wx$, Which is not a constant through the aquifer. It is obvious that the discharge q_x varies with X. At the upstream water body, X = 0 and discharge to this left river will be:

$$q_L = -\left(\frac{wL}{2} - \frac{K}{2}\frac{h_0^{\ 2} - h_L^{\ 2}}{L}\right) = \frac{K}{2}\left(\frac{h_0^{\ 2} - h_L^{\ 2}}{L}\right) - \frac{wL}{2}$$

The (-) sign do mean flow is in opposite x-direction.

Discharge to this right river when x-L will be:

$$q_R = \left(\frac{wL}{2} + \frac{K}{2}\frac{h_0^{\ 2} - h_L^{\ 2}}{L}\right) = \frac{K}{2}\left(\frac{h_0^{\ 2} - h_L^{\ 2}}{L}\right) + \frac{wL}{2}$$

When *h0= hL*, discharge to two rivers will be the same.

$q_L = q_R = \left(\frac{wL}{2}\right)$ And the total discharge to the two rivers, $q_L + q_R = wL$ which is equals to the total aquifer recharge.

The water table is thus not a straight line. The value of h will in general rise above h_0, reaches a maximum at X = d and falls back to h_L at X = L as shown in Fig 4.2. The value of d is obtained by equating dh/dX = 0 and q_x=0, and is given by:

$$q_x = \frac{K}{2}\left(\frac{h_0^{\,2} - h_L^{\,2}}{L} - \frac{wL}{K}\right) + wx = 0$$

$$d = \frac{L}{2} - \frac{K}{2}\left(\frac{h_0^{\,2} - h_L^{\,2}}{wL}\right)$$

The location X = d is called the *water divide*. In Fig 4.2, the flow to the left of the divide will be to the upstream water body and the flow to the right of the divide will be to the downstream water body.

In the case where there is no recharge to the aquifer, *w = 0*, water table reduces to:

$$h^2 = h_0^{\,2} - \left(\frac{h_0^{\,2} - h_L^{\,2}}{L}\right)x$$

This has a parabolic shape. In this case, the flow occurs only from the left river to the right river with unit width discharge as:

$q_x = \frac{K}{2}\left(\frac{h_0^{\,2} - h_L^{\,2}}{L}\right) = K\frac{h_0 + h_L}{2}\frac{h_0 - h_L}{L}$, This is the well-known Dupuit formula derived in 1863. It indicates that the unit width discharge is a constant and can be obtained using Darcy's law with average aquifer thickness, *(h0 + hL)/2*, and average hydraulic gradient, *(h0 - hL)/L*.

Example 2.4: Two rivers located 1000 m apart fully penetrate a phreatic aquifer. The parameters of the aquifer are: K= 0.5 m/d, w=1.4*10^{-4} m/day, h_0=20 m, h_L=18 m.

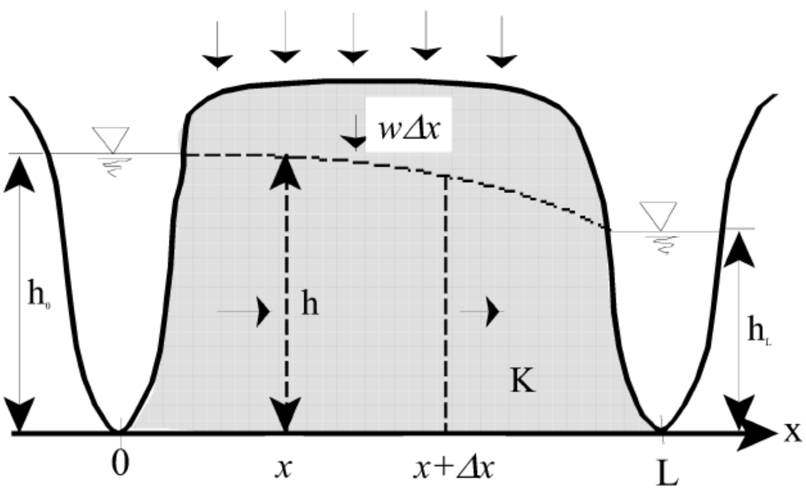
A. Derive a formula for calculating a unit width discharge.
B. Determine the location of d and the maximum height ($h_{max.}$) of the water divide.
C. What is the unit width discharge of the aquifer to the right river?

Solution:

a. $q_x = -Kh\frac{\partial h}{\partial x} = \frac{K}{2}\left(\frac{h_0^{\,2}-h_L^{\,2}}{L} - \frac{wL}{K}\right) + wx = \frac{0.5}{2}\left(\frac{20^2-18^2}{1000} - \frac{1.4*10^{-4}*1000}{0.5}\right) + 1.4*10^{-4}x$ q= -0.051+1.4*10^{-4}x

At q_x=0, x=0.051/1.4*10^{-4}=364.3 m, which is the location of water divide.

b. $d = \frac{L}{2} - \frac{K}{2}\left(\frac{h_0^{\,2}-h_L^{\,2}}{wL}\right) = \frac{1000}{2} - \frac{0.5}{2}\left(\frac{20^2-18^2}{1.4*10^{-4}*1000}\right) = 364.3m$



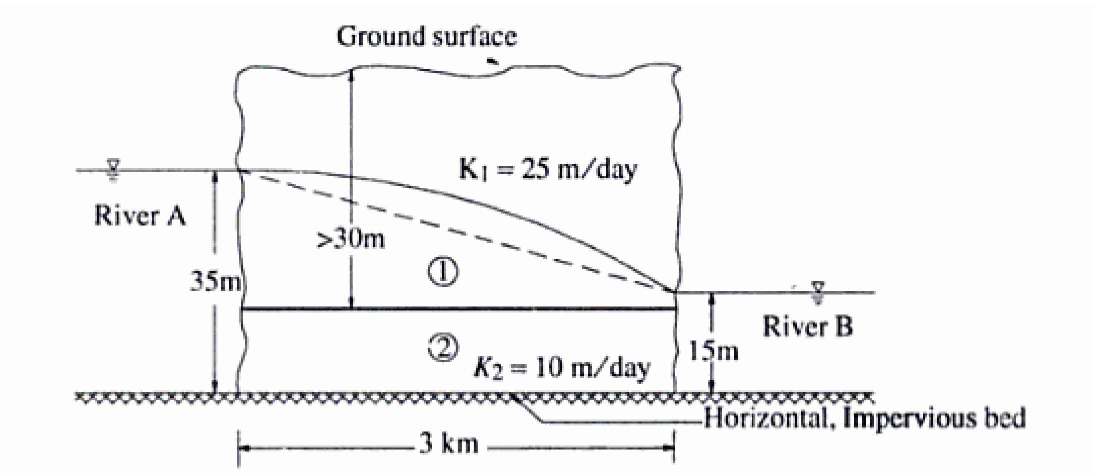
$$h_{max}^{\;2} = h_0^{\;2} - \left(\frac{h_0^{\,2}-h_L^{\,2}}{L} - \frac{wL}{K}\right)d - \frac{w}{K}d^2$$

$$h_{max}^{\;2} = 20^2\left(\frac{20^2-18^2}{1.4*10^{-4}*1000} - \frac{1.4*10^{-4}*1000}{0.5}\right)*364.3 - \frac{1.4*10^{-4}*364.3^2}{0.5} = 437.14$$

h_{max}=20.9 m

c. $q_R = \left(\frac{wL}{2} + \frac{K}{2} \frac{h_0^{\ 2} - h_L^{\ 2}}{L} \right)$=-0.051+1.4*10^{-4}x= 0.089 m/d

Example 2.5: Two parallel rivers A and B are separated by a landmass as shown in the figure below. Estimate the seepage discharge from river A to River B per unit length of the rivers.



Solution:

The aquifer system is considered as a composite of aquifers 1 and 2 with a horizontal impervious boundary at the interface. This leads to the assumptions:

a) Aquifer 2 is a confined aquifer with K_2 = 10 m/day,

b) Aquifer 1 is an unconfined aquifer with K_1 = 25m/day. Consider a unit width of the aquifers.

For the confined aquifer 2:

Here, h_1 = 35.0 m, h_2 = 15m, L = 3000 m, K_2 = 10 m/day and B = 10 m.

From Eq. (2.20), $q = K\left(\frac{H_0 - H_L}{L}\right)$

$\Rightarrow q_2$ = 0.667 m^3/day/meter width

For the unconfined aquifer 1:

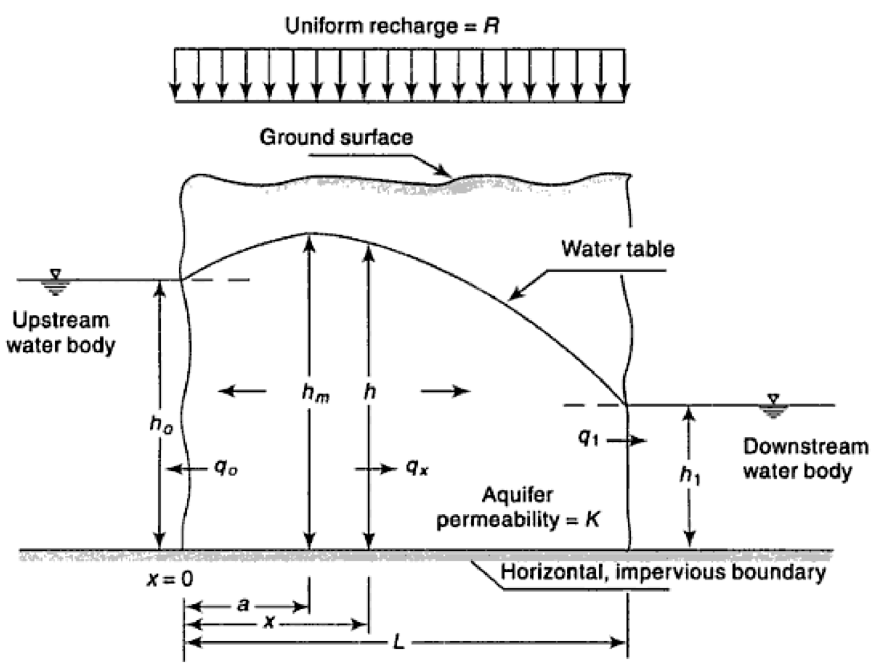
Here, $h_1 = (35 - 10) = 25$ m, $h_2 = (15 - 10) = 5$ m, L = 3000 m, K_1= 25 m/day

From Eq. (2.25), $q_x = \frac{K}{2}\left(\frac{h_0^{\,2} - h_L^{\,2}}{L}\right) = K\frac{h_0 + h_L}{2}\frac{h_0 - h_L}{L}$ $\Rightarrow q_1$ = <u>2.5 m^3/day/meter width</u>

Total discharge from river A to river B = $q_1 + q_2 \Leftrightarrow q = 0.667 + 2.5 =$ <u>3.167m^3/day/ unit length of the rivers</u>

Example 2.6: An unconfined aquifer (K = 5m/day) situated on the top of a horizontal impervious layer connects two parallel water bodies M and N which are 1200 m apart. The water surface elevations of M and N measured above the horizontal impervious bed are 10.00 m and 8.00 m. If a uniform recharge at the rate of 0.002m^3/day/ m^2 of horizontal area occurs on the ground surface, estimate:

a) The water table profile
b) The location and elevation of the water table divide
c) The seepage discharges into the lakes and
d) The recharge rate at which the water table divides coincides with the upstream edge of the aquifer and the total seepage flow per unit width of the aquifer at this recharge rate.



Solution:

Consider unit width of the aquifer referring to the figure below: h_0 = 10.00 m, R=w = 0.002 $m^3/day/m^2$, h_1= 8.00 m, L = 1200 m, K = 5 m/day.

a) The water table profile: By Eq. (2.24), $h^2 = h_1^{\,2} - \left(\frac{h_1^{\,2} - h_2^{\,2}}{L} - \frac{wL}{K}\right)x - \frac{w}{K}x^2$

$\Rightarrow$h = <u>-0.0004X^2 + 0.45X +100</u>

b) Location of water table divide: From Eq. (2.27), $d = \frac{L}{2} - \frac{K}{2}\left(\frac{h_0^{\,2} - h_L^{\,2}}{wL}\right)$ $\Rightarrow$d = <u>562.5 m</u>

At x = a=d = 562.5m, h = h_m = height of water table divide

$\Rightarrow h_m^2$ = - 0.0004 (562.5)^2 + 0.45 (562.5) +100 = 226.56

$\Rightarrow h_m$ = <u>15.05m</u>

c) Discharge per unit width of the aquifer: Form Eq. (2.25) $q_x = \frac{K}{2}\left(\frac{h_0^{\,2} - h_L^{\,2}}{L} - \frac{wL}{K}\right) + wx$

At x =0, $q_1 = \frac{K}{2}\left(\frac{h_0^{\,2} - h_L^{\,2}}{L} - \frac{wL}{K}\right) = -1.125 m^3/d/m$

The negative sign indicates that the discharge is in (-X) direction i.e, into the water body M.

At X = L, $q_2 = q_L$ and from Eq (2.25) $q_2 = wL + q_1$

Hence, q_2 = discharge into water body N = 0.002*1200 + (-1.125) = 1.275 m^3/day/meter width

d) When the distance of the water table divide d= 0

$$d = \frac{L}{2} - \frac{K}{2}\left(\frac{h_0^{\,2} - h_L^{\,2}}{wL}\right) \Rightarrow w = \frac{K}{L^2}\left(h_0^{\,2} - h_L^{\,2}\right)$$

$\Rightarrow w = 5/1200^2(10^2 - 8^2) = 1.25*10^{-4} m^3/day/m^2$

Since d = 0, q_1 = 0 and by Eq. (2.24) $q_2 = q_L = w*L = 1.25*10^{-4}*1200 = 0.15m^3$/day/meter width

1.3 Well Hydraulics

Since ancient times, wells have been dug or drilled into the subsurface to access groundwater. Prior to the development of drilling technologies, buckets were used to collect water from shallow hand-dug wells. Modern groundwater wells can be thousands of meters deep and allow extraction of large quantities of water with electric pumps.

While wells are used in a number of different applications, they find extensive use in water supply and irrigation engineering practice. Drinking water for example is obtained in many communities from groundwater wells. As water is extracted from a well, the water level within the well drops and the water in the surrounding aquifer flows towards the well causing a lowering of the water level extending outward from the well. The drop in water level is greatest immediately adjacent to the well and decreases radially outward creating a feature called the cone of depression. As pumping continues, the cone of depression extends out farther gathering water from a larger cylindrical volume surrounding the well. The expansion of the cone of depression will continue until the volume of water intercepted or drawn by the well equals the pumping rate. Besides aquifer water, the water drawn by a well can also be recharge from the ground surface, adjacent aquifers, streams, lakes or oceans.

Impermeable boundaries formed by low hydraulic conductivity materials (bedrock, faults, etc.) will halt the progression of the cone of depression at their location.

Knowledge of the drop in water level and pattern of groundwater flow resulting from well pumping is necessary for assessing environmental impacts in many situations.

Excessive drops in groundwater levels over regional scales can result in adverse impacts to stream flows, vegetation and the use of shallow wells. At sites of groundwater contamination, the cone of depression can expand outward from the pumping well and “capture” the contaminated water.

1.4 Steady Radial Flow to a well

When a well is pumped, water is removed from the aquifer surrounding the well, and the water table or piezometric surface, depending on the type of aquifer, is lowered. The drawdown at a given point is the distance the water level is lowered. A drawdown curve shows the variation of drawdown with distance from the well. In three dimensions the drawdown curve describes a conic shape known as the cone of depression. Also the outer limit of the cone of depression defines the area of influence of the well.

1.4.1 Steady Flow to a well in Confined Aquifer

The radial flow equation which relates the well discharge to drawdown for a well completely penetrating a confined aquifer can be derived by referring fig 3.1. In this case the flow is assumed to be two-dimensional to a well centered on a circular island and penetrating a homogeneous and isotropic aquifer.

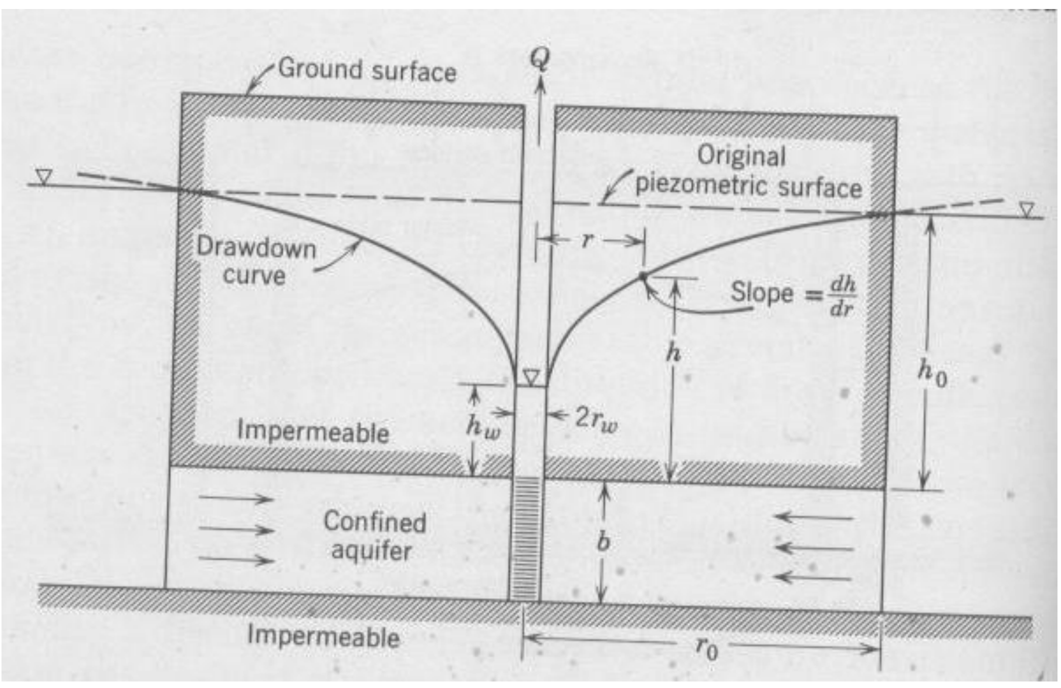


Figure 1.3: Steady radial flow to well penetrating confined aquifer

Since the flow is horizontal everywhere in confined aquifer case, the Dupuit's assumption applies without error. Using the plane polar coordinates for the well and its surrounding; the well discharge at any distance r from the well equals

$$Q = AV = -2\pi rbK\frac{dh}{dr}$$

for steady radial flow to a well. Rearranging and integrating for boundary conditions at the well,h=hw and r=rw, and at the edge of the islands, h=ho and r=ro yields

$$Qdr = -2\pi rbKdh$$

$$Q\int_{r_w}^{ro}\frac{dr}{r} = -2\pi bK\int_{h_w}^{ho}dh$$

$$Q = \frac{2\pi Kb(ho - h_w)}{\ln(ro/r_w)}$$

With the negative sign neglected.

For a well penetrating an extensive confined aquifer, this equation shows that h increases as r increases. Yet, the maximum h is the initial uniform h_O. Thus from theoretical point of view, steady radial flow in an extensive aquifer does not exist b/c the cone of depression must expand indefinitely with time. However, from practical stand point, h approaches ho with distance from the well, and the drawdown vary with the logarithm of the distance from the well

- If the values of head(h) are known (h_1 and h_2) at the respective positions of distance r_1 and r_2 respectively from the well, then the flow equation can be written as :-

$$Q = \frac{2\pi bK(h_2 - h_1)}{\ln(r_2/r_1)}$$

Where $r_2 > r_1$ and $h_2 > h_1$ (Refer Todd, fig 4.5)

The above equation is known as an equilibrium equation or Theim Equation enables one to determine the values of hydraulic conductivity (K) and Transmissivity (T) of a confined aquifer from pumping test data. Because any two points define the logarithmic drawdown curve, the method consists of measuring drawdowns in two observation wells at different distances from a well pumped at constant rate.

The term ‘steady flow’ in well hydraulics, hence, refers to the state of flow in which the change b/n two consecutive drawdowns/water levels become negligibly small. That is the time after long period of pumping, after which aquifer flow and rate of pumping become almost equal.

Exercise: - Prove that equation for flow of water in confined aquifer towards a well is given by:

$$Q = \frac{2\pi T(s_1 - s_2)}{\ln(r_2/r_1)}$$

1.4.2 Steady Flow to a well in Unconfined Aquifer

An equation for steady radial flow to a well in an unconfined aquifer also can be derived with the help of the Dupuit assumptions. As shown in figure 3.2 the well completely penetrates the aquifer to the horizontal base and concentric boundary of constant head surrounds the well.

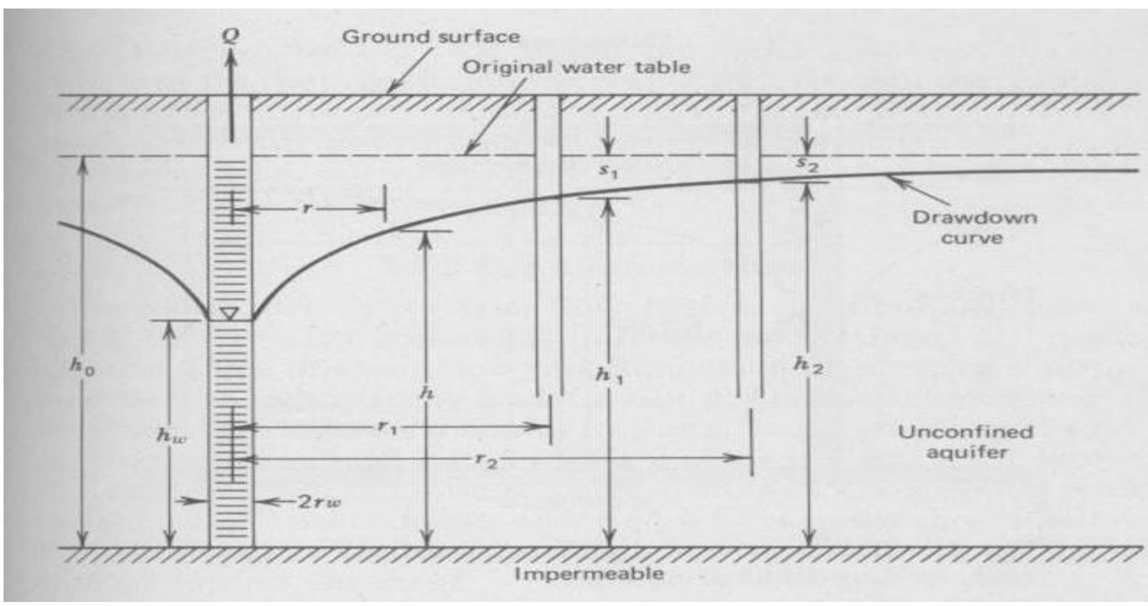


Figure 1.4: Steady radial flow to well penetrating unconfined aquifer

The well discharge Q is given by:-

$$Q = AV = 2\pi rKh\frac{dh}{dr}$$

$$\frac{Qdr}{r} = 2\pi Kdh$$

$$\int_{r_w}^{ro}\frac{Qdr}{r} = 2\pi K\int_{h_w}^{ho} hdh$$

$$Q\ln(ro - rw) = 2\pi K\frac{(ho^2 - h_w{}^2)}{2}$$

$$\text{Therefore, } Q = \pi K\frac{(ho^2 - h_w{}^2)}{\ln\left(\frac{ro}{rw}\right)}$$

Converting heads and radii at two observation wells (as shown in figure)

$$\text{Therefore, } Q = \pi K\frac{(h_2{}^2 - h_1{}^2)}{\ln\left(r2/r1\right)}$$

This equation fails to accurately estimate the drawdown curve near the well because large vertical flow components contradict with the Dupuit's assumption; however, estimates of hydraulic conductivity (k) values for a given heads are good. In practice drawdowns should be small in relation to the saturated thickness of the confined aquifer. Then the average Transmissivity can be estimated from the equation:-

$$T = K\frac{(h1 + h2)}{2}$$

Where drawdowns are appreciable, the heads h_1 and h_2 in the above equation can be replaced by $(ho-s_1)$ and $(ho-s_2)$, Then the Transmissivity for the full thickness expressed as:

$$T = Kh_0 = \frac{Q}{2\pi\left[\left(s_1 - \frac{s_I^{\ 2}}{2h_0}\right) - \left(s_2 - \frac{s_2^{\ 2}}{2h_0}\right)\right]} \ln\left(r_2 / r_1\right)$$

1.4.3 Unsteady Flow to a well in a confined aquifer

When a well penetrating an extensive confined aquifer is pumped at a constant rate, the influence of the discharge extends outward with time. The rate of decline of head times the storage coefficient summed over the area of influence equals the discharge. Since the water must come from a reduction of storage within the aquifer, the head will continue to decline as long as the aquifer is effectively infinite; therefore, unsteady flow exists. The rate of decline, however, decreases continuously as the area of influence expands.

The non-steady GW flow equation in two dimensions is given by:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = \frac{S}{T}\frac{\partial h}{\partial t}$$

or in polar coordinates the above equation ,to represent the radial flow in to a well ,takes the form

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} = \frac{S}{T}\frac{\partial h}{\partial t}$$

Where h is head, r is radial distance from the pumped well, S is storage coefficient, T is transmissivity and t is the time since beginning of pumping.

Theis obtained a solution for the above equation based on the analogy between groundwater flow and heat induction. By assuming that the well is replaced by a mathematical sink of constant strength and imposing the boundary conditions h=ho for t=0 and h → ho as r →∞ for t>=0, the following solution is derived.

$$s = \frac{Q}{4\pi T}\int_u^\infty e^{-u}\frac{du}{u}$$

Where s = drawdown, Q = well discharge and u = dummy variable (dimensionless).

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

Where S = storage coefficient, T = Transmissivity, t = time of pumping and r = radius of observation well from test well where drawdown is observed.

Equation (3.2) is called the Theis equation and it is non-linear equation. The integral term, $\int_u^\infty e^{-u} \frac{du}{u}$ is a function of the lower limit u and is known as an exponential integral and can be expressed by a convergent series as:

$$s = \frac{Q}{4\pi T}[-0.5772 - \ln u + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - ....]$$

Equation (3.3) is widely used in practice and preferred over equilibrium equation because

1. a value of S can be determined
2. only one observation well can suffice
3. Shorter period of pumping generally is required
4. No assumption of steady state flow condition is required.

The assumption made to steady state case holds good except the flow is taken as unsteady state (Refer Todd P- 124).

Because of the mathematical difficulties encountered in applying equation (3.3) several investigators developed simpler approximate solutions that can be readily applied for field purposes. Three of the methods namely Theis, Cooper and Jacob and Chow are discussed in the subsequent sections.

Theis method of solution

Equation 3.2 may be simplified to

$s = \frac{Q}{4\pi T} W(u)$ Where W (u) is termed as a well function and $u = \frac{r^2 S}{4Tt} \Rightarrow \frac{4T}{S} u = \frac{r^2}{t}$

It can be seen that the relation between W (u) and u must be similar to b/n s and (r^2/t) because the terms in parentheses in the two equations are constants. Having these trends of similarities in mind, Theis suggested an approximate solution for S and T based on a graphic method of superposition. The procedure for finding parameters by Theis Method

1. Prepare the logarithmic plot of W (u) vs u or W (u) vs 1/u, known as type curve
2. Plot values of drawdown(s) against values (r^2/t) on logarithmic paper of the same size as for the type curve.
3. Superimpose the observed time-drawdown data on the type curve, keeping the coordinate axes of the two curves parallel and adjusting the graphs till most of the plotted points of the observed data fall on the segment of the type curve.
4. Select any convenient match point and record the coordinates i.e. obtain the values of W (u), u, r^2/t and s on the match point.
5. Determine the values of T and S by inserting the values (step 4) in the above equations.

N.B The match point doesn't have to be on the type curve. In fact calculations are greatly simplified if the point is chosen where W (u) = 1 and 1/u=10.

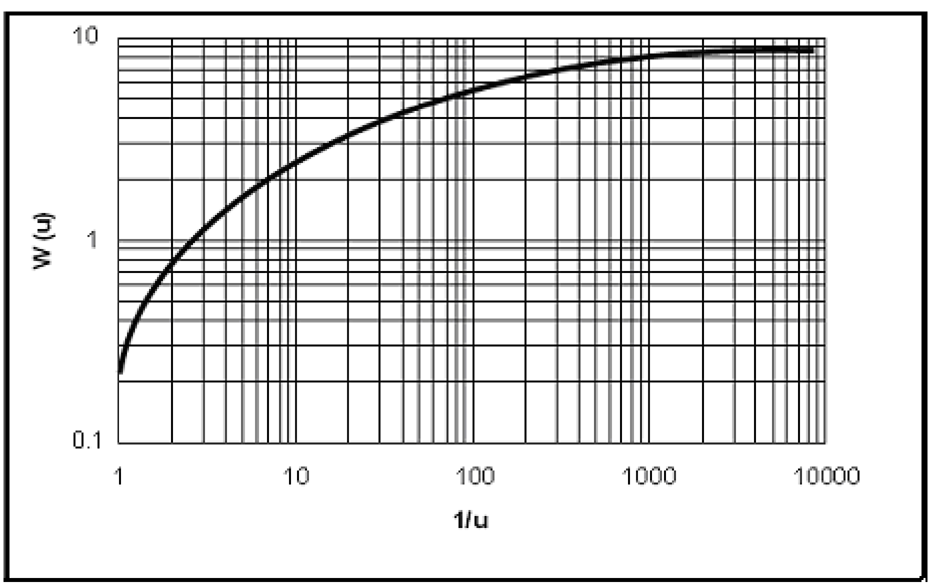


Figure 1.5: The non-equilibrium reverse type curve (Theis curve) for a fully confined aquifer

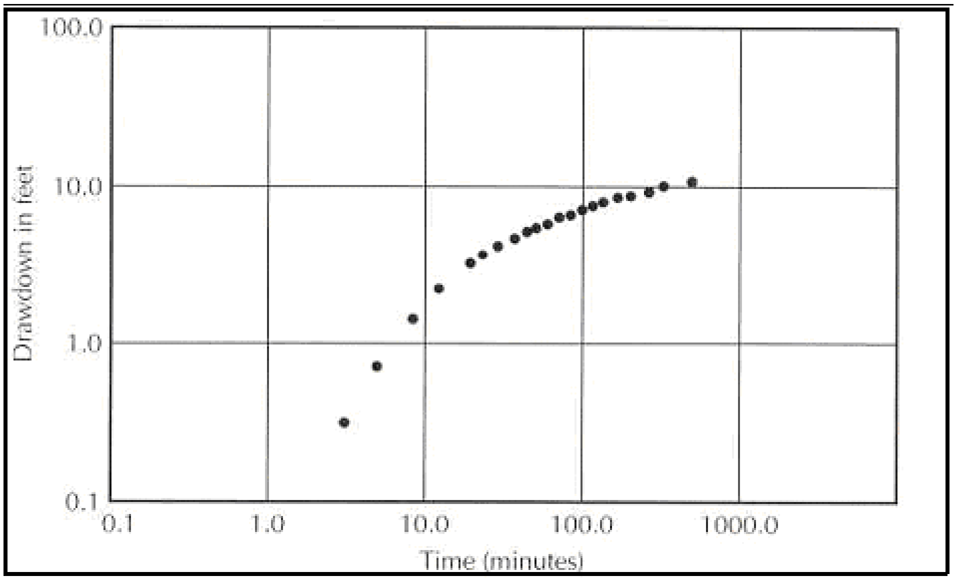


Figure 3.4 Field data plot on logarithmic paper for Theis curve-marching technique

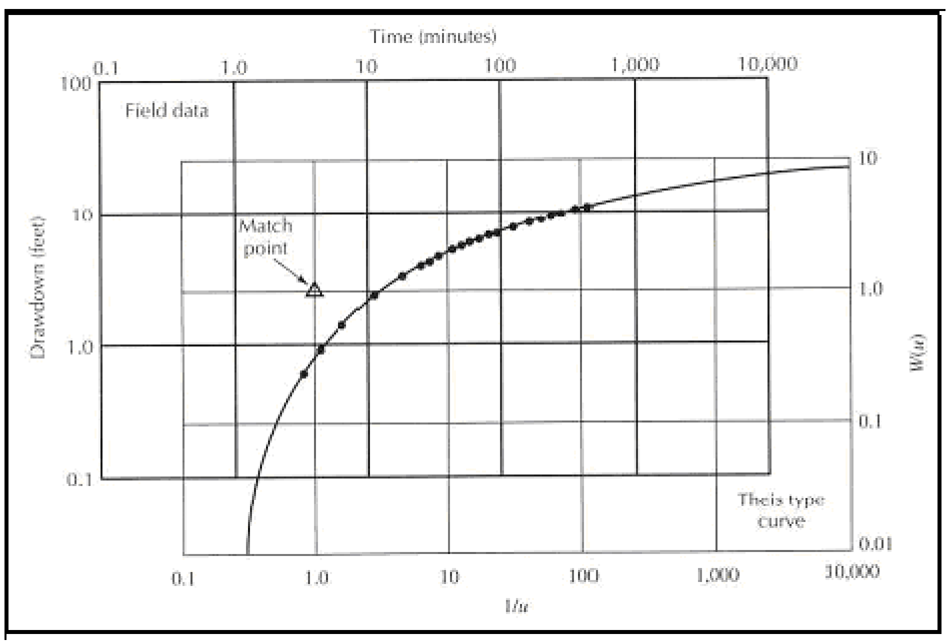


Figure 1.6: Match of field data plot to Theis Type curve

Exercise Do the example given in Todd book (page 125 to 127) and show the procedures.

Cooper - Jacob method of solution

The analysis presented here is of a pumping test in which drawdown at a piezometer distance, r from the abstraction well is monitored over time. This is also based upon the Theis analysis

$$s = \frac{Q}{4\pi T}[-0.5772 - \ln u + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - \text{................}]$$

From the definition of u it can be seen that u decreases as the time of pumping increases and as the distance of the piezometer from the well decreases. So, for piezometers close to the pumping well after sufficiently long pumping times, the terms beyond lnu become negligible. Hence for small values of u, the drawdown can be approximated by:

$$s = \frac{Q}{4\pi T}\left[-0.577216 - \ln\frac{r^2 S}{4Tt}\right]$$

Changing to logarithms base 10 and rearranging produces

$$s = \frac{2.303Q}{4\pi T}\log_{10}\left(\frac{2.25T}{r^2 s}t\right)$$

And this is a straight line equation

$$s = (\frac{2.303Q}{4\pi T}\log_{10}\left(\frac{2.25T}{r^2 s}\right)) + (\frac{2.303Q}{4\pi T})\log t$$

Y=c+ax

It follows that a plot of s against log t should be a straight line. Extending this line to where it crosses that t axis (i.e. where s is zero and t=to) gives

$$\frac{2.25Tto}{r^2 S} = 1$$

The value of T can be obtained by nothing that if t/to=10, then log t/to=1 ;therefore replacing s by Δs, where Δs is the drawdown differences per log cycle of t and the value of T can be computed from : $T = \frac{2.30Q}{4\pi\Delta s}$

The gradient of the straight line (i.e. the increase per log cycle, Δs) is equal to

$$\Delta s = \frac{2.30Q}{4\pi T}$$

Note: - The procedure in this method is first to solve for T and then solving for S. To avoid large errors the straight-line approximation is also restricted for small values of u ($u<0.01$)

Chow Method of Solution

In these method measurements of drawdown in an observational well near a pumped well are made and the observational data are plotted on semi logarithmic paper in the same manner as for the Cooper-Jacob method. On the plotted curve arbitrary point is chosen and the coordinates, t and s are noted. Then tangent line to the curve at the chosen point is drawn and the drawdown difference per log cycle time is computed. The value of F (u) is computed using the following formula.

$$F(u) = \frac{s}{\Delta s}$$

Having the value of F(u) the corresponding value of W(u) and u can be obtained from graph of F(u),W(u) and u (after Chow). (Refer Figure 4.11Todd book page 131)

1.4.4 Unsteady Flow to a well in unconfined aquifer

The first and by far the simplest approach is to use the same flow situation as for the case of confined aquifer provided the basic assumptions are satisfied. In general, if the drawdown is small in relation to the saturated thickness (unconfined aquifer) good approximations are possible with the methods developed for the confined aquifer.

If the drawdown in the monitoring well does not exceed 25% of the saturated thickness, the Theis equation can be applied to unconfined aquifers with certain adjustments. For the drawdown that is less than 10% of the aquifer’s pre-pumping thickness, it is not necessary to adjust the recorded data since the error introduced by using the Theis equation is small. When the drawdown is kept between 10% and 25%, it is recommended to correct the measured values using the following equation derived by Jacob:-

$s' = s - s^2/2h$

Where

s’ = is the corrected drawdown

s = measured drawdown in monitoring well

H = the saturated thickness before pumping started

This correction is needed since the Transmissivity of aquifer changes during the test as the saturated thickness decreases (remember that for unconfined aquifers, T = Kh where h is the saturated thickness liable for variation)

If the drawdown in the monitoring well is more than 25%, the equation (Theis and Theis based) should not be used in the unconfined aquifer analysis.

There are different methods of analysis for unconfined aquifer, when the drawdown due to pumping is remarkably large. Neuman, Boulton, Hantush etc.,methods which are but beyond the scope of the class.

3.3 Unsteady Radial Flow in a Leaky Aquifer- Hantush-Jacob Method and Walton Graphical Method

Leaky aquifer bounded to and bottom by less transmissive horizons, at least one of which allows some significant vertical water “leakage” into the aquifer.

Unsteady radial flow for leaky aquifer can be represented in the following equation:

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} + \frac{e}{T} = \frac{S}{T}\frac{\partial h}{\partial t}$$

Where

r is the radial distance from a pumping well (m)

e is the rate of vertical leakage (m/day)

When a leaky aquifer, as shown in figure (3.6), is pumped, water is withdrawn both from the aquifer and from the saturated portion of the overlying semipervious layer. Lowering the piezometric head in the aquifer by pumping creates the hydraulic gradient within the semi pervious layer; consequently, groundwater migrates vertically downward in to the aquifer. The quantity of water moving downward is proportional to the difference between the water table and piezometric head.

Steady-state flow is possible to a well in leaky aquifer because of the recharge through the semipervious layer. The equilibrium will be established when the discharge rate of the pump equals the recharge rate of the vertical flow in to the aquifer, assuming the water table remains constant. Solution for this special steady-state situation area available, but a more general analysis for unsteady flow follows.

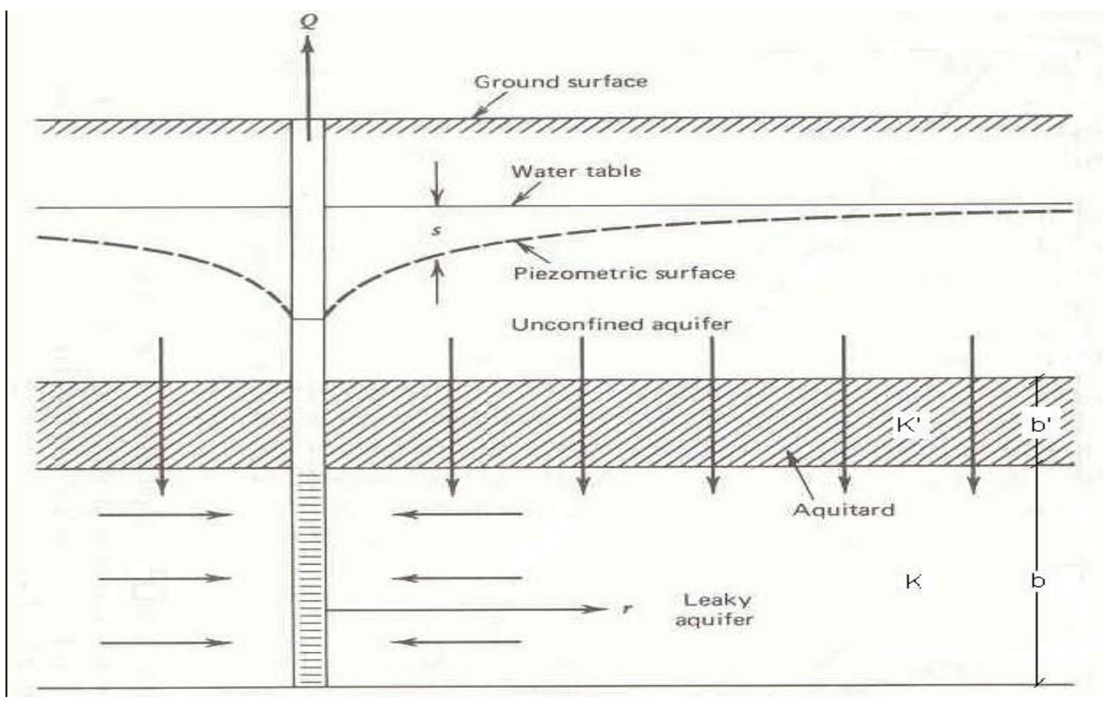


Figure 1.7: well pumping from a leaky aquifer

Normal assumption leakage rate into aquifer = $k^{'}\dfrac{\Delta H}{b^{'}}$ and ΔH = sw, where b' is the thickness of the saturated semipervious layer.

The Hantush and Jacob solution has the following assumptions:

1. The aquifer is leaky and has an "apparent" infinite extent,
2. The aquifer and the confining layer are homogeneous, isotropic, and of uniform thickness, over the area influenced by pumping,
3. The potentiometric surface was horizontal prior to pumping,
4. The well is pumped at a constant rate,
5. The well is fully penetrating,
6. Water removed from storage is discharged instantaneously with decline in head,
7. The well diameter is small so that well storage is negligible,
8. Leakage through the aquitard layer is vertical.

The Hantush and Jacob (1955) solution for leaky aquifer presents the following equations (seeFigure 3-6):

$$s = \frac{Q}{4\pi T} W\left(u, {}^{r}\!/\!_{B}\right) \rightarrow \quad T = \frac{Q}{4\pi s} W\left(u, {}^{r}\!/\!_{B}\right)$$

Where s,Q and r are as shown in the figure and $u = \frac{r^2 S}{4Tt} \rightarrow S = \frac{4Ttu}{r^2}$

Where

$W(u, r/B)$ is the well function for leaky confined aquifer

B : is the leakage factor given as $B = \sqrt{T' b' / K'}$

Where

b' is thickness of the aquitard (m)

K' is hydraulic conductivity of the aquitard (m/day)

Walton Graphical Solution

1. Type curves $W(u, r/B)$ vs 1/u for various values of 1/u and r/B , see Figure 3.7.
2. Field data are plotted on drawdown (s) vs. time on full logarithmic scale.

3. Field data should match one of the type curves for r/B (interpolation if between two lines)

4. From a match point, the following are known values $W(u, r/B)$, 1/u, t, sw, and r/B
5. Substitute in Hantush-Jacob equation:

$$T = \frac{Q}{4\pi s} W\left(u, r/B\right)$$

$$S = \frac{4Ttu}{r^2}$$

$$\frac{r}{B}(\,frommatch) = \frac{r}{\sqrt{\dfrac{T}{\left(K'/b'\right)}}} \quad \text{then}$$

$$K' = \frac{Tb'\left(\dfrac{r}{B}\right)^2}{r^2} \quad \text{where}$$

Q is the pumping rate (m3/day)

t is the time since pumping began (day)

r is the distance from pumping well to observation well (m)

b' is the thickness of aquitard (m)

K' is the vertical hydraulic conductivity of confining bed (aquitard) (m/day)

B is the leakage factor (m)

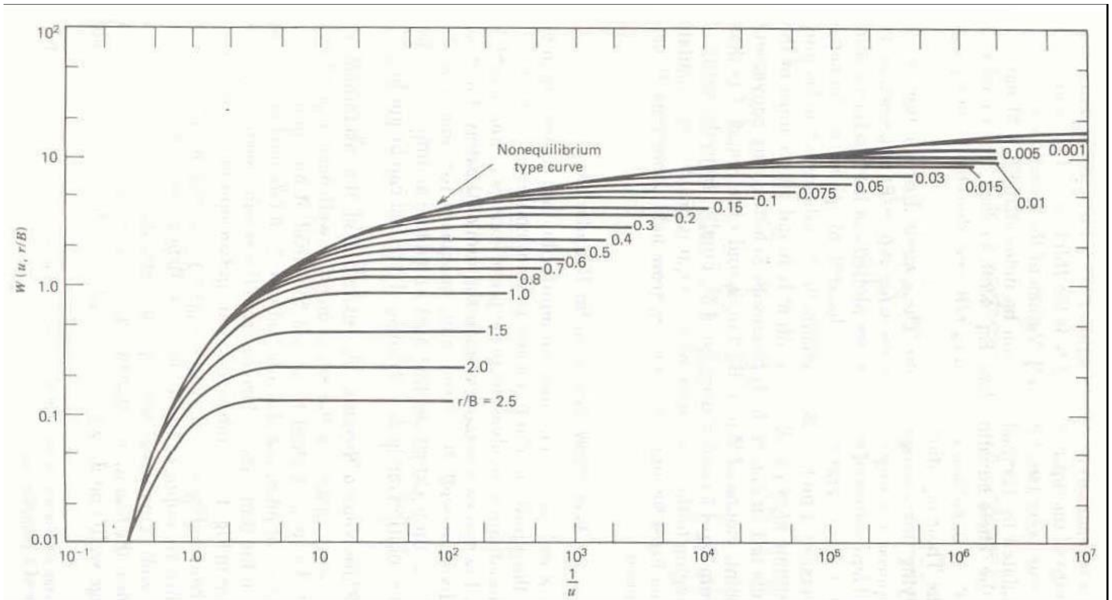


Figure 1.8: Log-log plot for Hantush method

1.5 Multiple Well Systems

Where the cones of depression of two nearby pumping well over lap, one well is said to interfere with another because of the increased drawdown and pumping lift created. For a group of wells forming a well field, the drawdown can be determined at any point if the well discharges are known, or vice versa. From the principle of superposition, the drawdown at any point in the area of influence caused by the discharge of several wells is equal to the sum of the drawdowns caused by each well individually. Thus,

$$S_T = s_a + s_b + s_c + s_d + ... + s_n$$

Where

S_T is the total drawdown at a given point,

$s_a + s_b + s_c + s_d + ... + s_n$: are the drawdowns at the point caused by the discharge of wells a, b, c …n respectively.

The summation of drawdowns may be illustrated in a sample way by the well line of Figure 3.8; the individual and composite drawdown curves are given for $Q_1 = Q_2 = Q_3$ clearly, the number of wells and the geometry of the well field are important in determining drawdowns.

Solutions of well discharge for equilibrium or non-equilibrium equation. Equations of well discharge for particular well patterns have been developed.

In general, wells in a well field designed for water supply should be spaced as far apart as possible so their areas of influence will produce a minimum of interference with each other. On the other hand economic factors such as cost of land or pipelines may lead to a least-cost well layout that includes some interference. For drainage

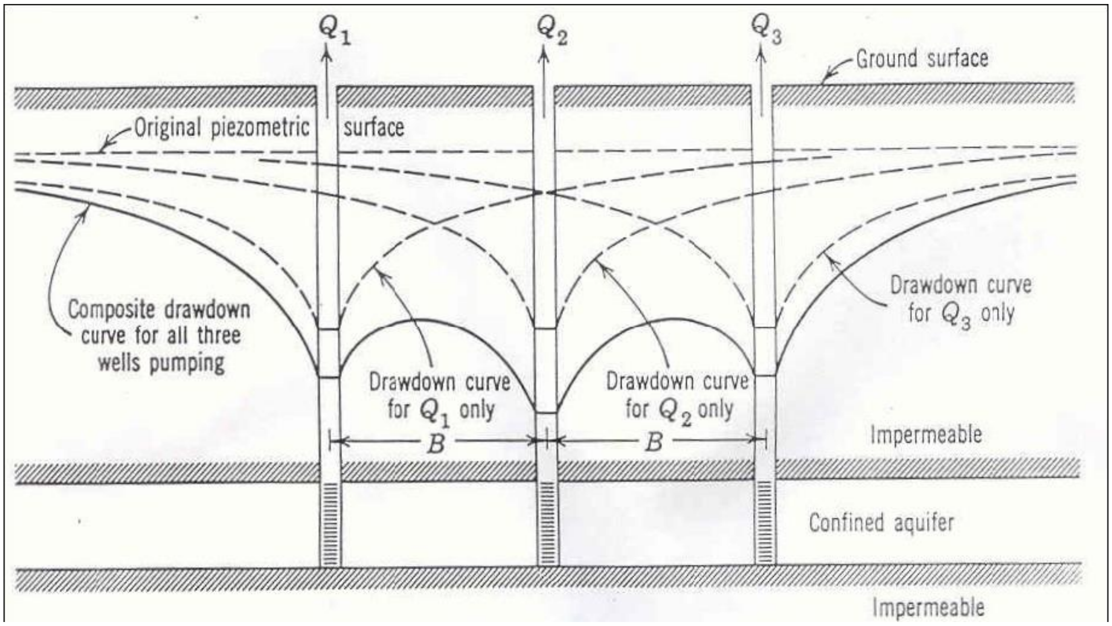


Figure 1.9: Individual and composite drawdown curves for three wells in a line

1.6 Recovery of a well/aquifer

At the end of a pumping test, when the pump is stopped, the water levels in the pumping and observation wells will begin to rise. This is referred to as groundwater levels, while measurements of draw down below the original static water level (prior to pumping) during the recovery period are known as residual draw downs. A schematic diagram of change in water level with time during after the pumping is shown the figure.

It is a good practice to measure residual drawdowns b/c analysis of the data enable transmissivity to be calculated, thereby providing an independent check on pumping test results. Also, costs are nominal in relation to the conduct of pumping test. Furthermore, the rate of recharge to the well during recovery is assumed constant and equal to the mean pumping rate, whereas pumping rates often vary and are difficult to control accurately in the field. During recovery, one can measure the water level variation in the pumped well itself.

If the well is pumped for a known period of time and then shut down, the draw down there after will be identically the same as if the discharge had been continued and a hypothetical recharge well with the flow were superposed on the discharging well at the instant the discharge is shut down. From this principle, Theis showed that, the residual draw down s’ can be given as:

$s' = \frac{Q}{4\pi T}\left[W(u) - W(u')\right]$ Where $u = \frac{r^2 S}{4Tt}$ $and\ u' = \frac{r^2 S}{4Tt'}$

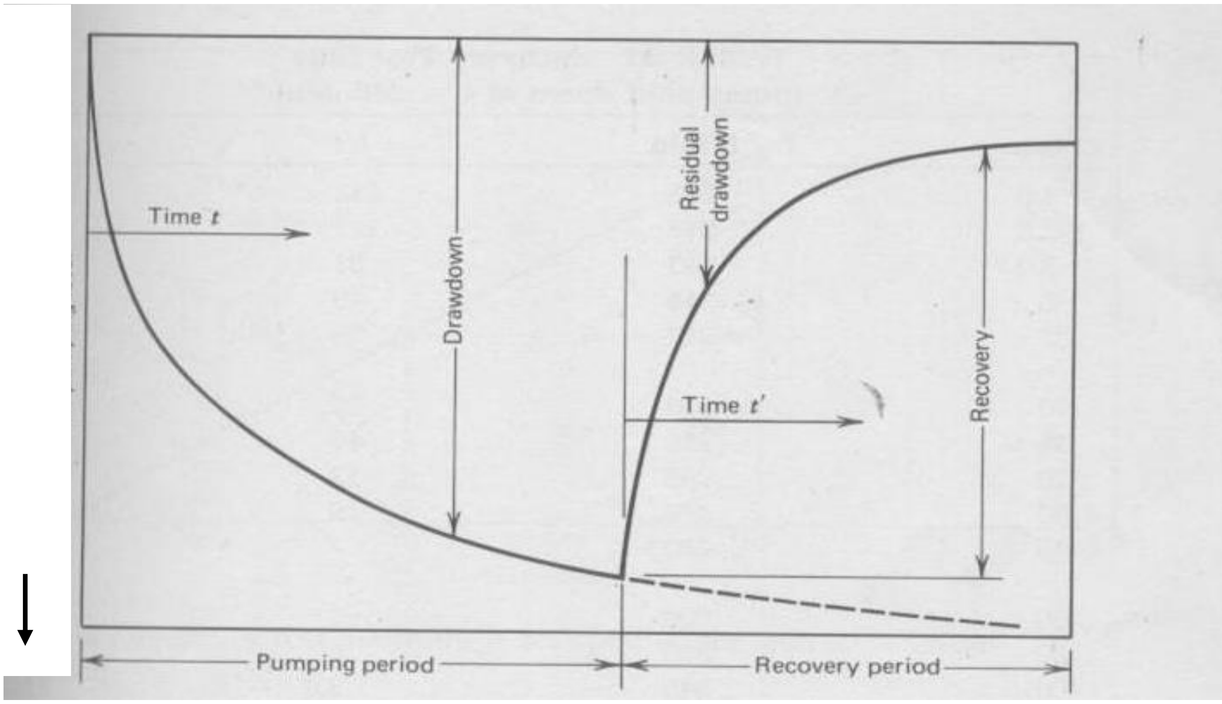
and t and t’ are defined in figure. For r small and t’ large, the well functions can be approximated by the first two terms of the Thies equyation and can be written as

$$s' = \frac{2.30Q}{4\pi T}\log_{10}\frac{t}{t'}$$

Thus, a plot of residual draw down s’ versus the Logarithm of t/t’ forms a straight line. The slope of the line equals 2.30Q/4πT so that for Δs’, the residual draw down per log cycle of t/t’, the transmissivity becomes

$$T = \frac{2.30Q}{4\pi\Delta s'}$$

Note: No comparable value of S can be determined by this recovery test method



1.7 Well Losses and Specific Capacity

1.7.1 Well Loss

The total DD (s_w) at the well face is made up of:

a. Head loss resulting from laminar flow in the formation,
b. Head loss resulting from turbulent flow in the zone close to the well face where Re > 1.
c. Head loss through the well casing and screen

The components under (ii) and (iii) are contributing to the so called well loss.

Therefore, well loss can be expressed as the difference between the actual measured DD in the pumping well and the theoretical DD which is expressed by the Theis equation and as the result of GW flow through the aquifer in the undisturbed zone only.

The additional DD, or well loss, which is always present in pumping wells, is created by a combination of various factors such as: improper well development (drilling fluid left in the formation, mud cake along the bore hole is not removed, fines from formation are not removed, poorly designed gravel pack and well screen), turbulent flow near the well and others.

To summarize, the drawdown at a well includes not only that of the logarithmic drawdown curve at the well face, but also a well loss caused by flow through the well screen and flow inside of the well to the pump intake.

Because the well loss is associated with turbulent flow, it may be indicated as being proportional to an nth power of the discharge, as Qn , where n is a constant greater than one. Jacob suggest that a value n=2 might be reasonably assumed.

Taking account of the well loss, the total draw down sw at the well may be written for the steady state confined case

$$s_w = \frac{Q}{2\pi T} \ln \frac{r_2}{r_1} + CQ^n \qquad (1)$$

Where C is a constant governed by the radius, construction and condition of the well. For simply

let $$B = \frac{\ln(r_2/r_1)}{2\pi T} \qquad (2)$$

So that the total drawdown in a well can be represented by:

$$sw = s_{aquifer} + s_{wellloss} = BQ + CQ^n \qquad (3)$$

Therefore, as shown in figure 3-10, the total drawdown s_w consists of the formation loss BQ and the well loss CQ^n

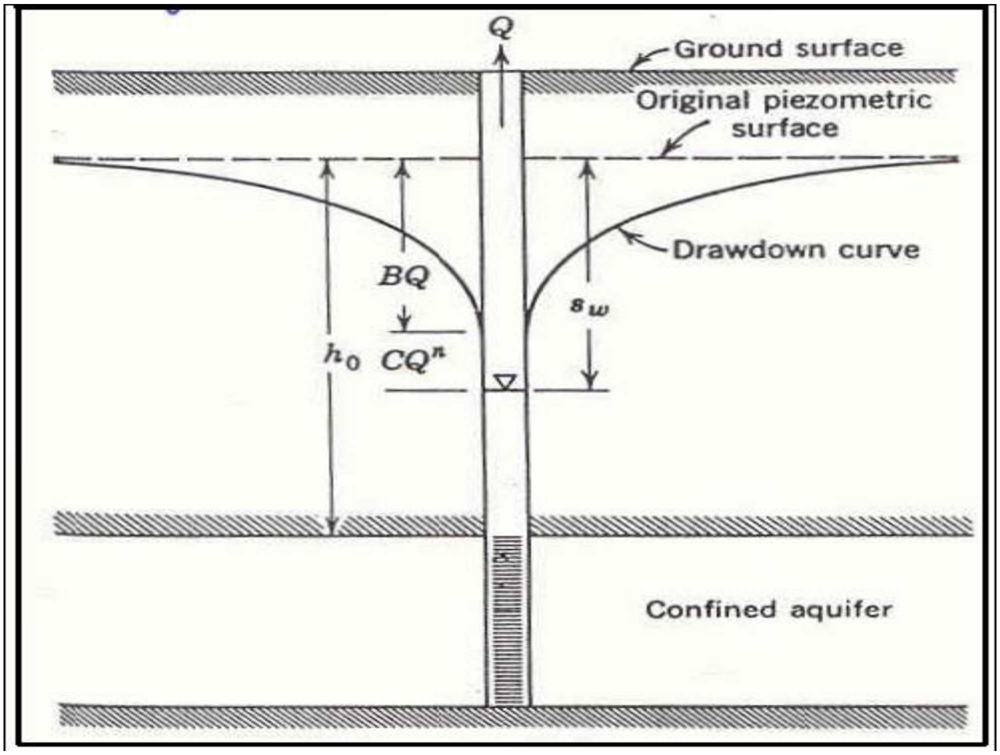


Figure 1.10: Relation of well loss CQn to draw-down for a well penetrating a confined aquifer

Consideration of Equation (2) provides a useful insight to the relation between well discharge and well radius. From equations confined aquifer it can be seen that Q varies inversely with $\ln r_2/r_1$ if all other variables are held constant. This shows that the discharge varies only a small amount with well radius. For example, doubling a well radius increases the discharge only 10 percent. When the comparison is extended to include well loss, however, the effect is significant. Doubling the well radius doubles the intake area, reduces entrance velocities at almost half, and (if n=2) cuts the frictional loss to less than a third. For axial flow within the well, the area increases four times, reducing this loss an even greater extent.

It is apparent that the well loss can be a substantial fraction of total drawdown when pumping rates are large, as illustrated in Figure 3.11. With proper design and development of new wells, well losses can be minimized. Clogging or deterioration of well screens can increase well losses in old wells. Based on field experience

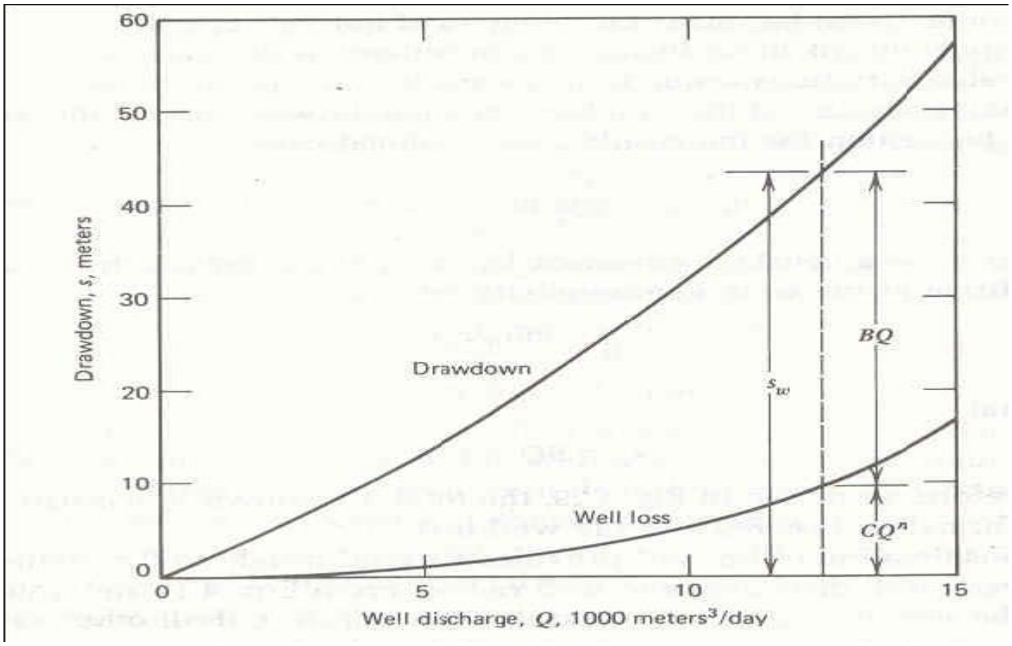


Figure 1.11:Variation of total drawdown w s , aquifer loss BQ , and well loss CQn with well discharge

1.8 Evaluation of Well Loss

To evaluate well loss a step-drawdown pumping test is required. This consists of pumping a well initially at a low rate until the drawdown within the well essentially stabilize. The discharge is then increased through a successive series of steps as shown by the time-drawdown data in Figure 3.12. Incremental drawdowns Δs for each step are determined from approximately equal time intervals. The individual drawdown curves should be extrapolated with a slope proportional to the discharge in order to measure the incremental drawdowns.

For n=2 in equation (3) and by dividing the equation by Q yields:

$$\frac{s_w}{Q} = B + CQ$$

Therefore, by plotting s_w/Q versus CQ (see Figure 3.13) and fitting a straight line through the points, the well loss coefficient C is given by the slope of the line and the formation loss coefficient B by the intercept Q=0

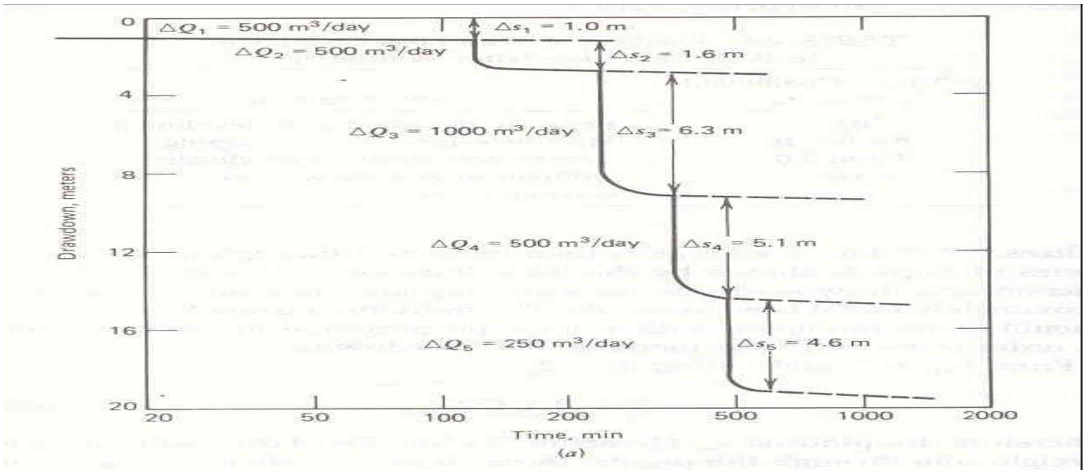


Figure 1.12:Step-drawdown pumping test analyses to evaluate well loss. (a) Time-drawdown data from step-drawdown pumping test

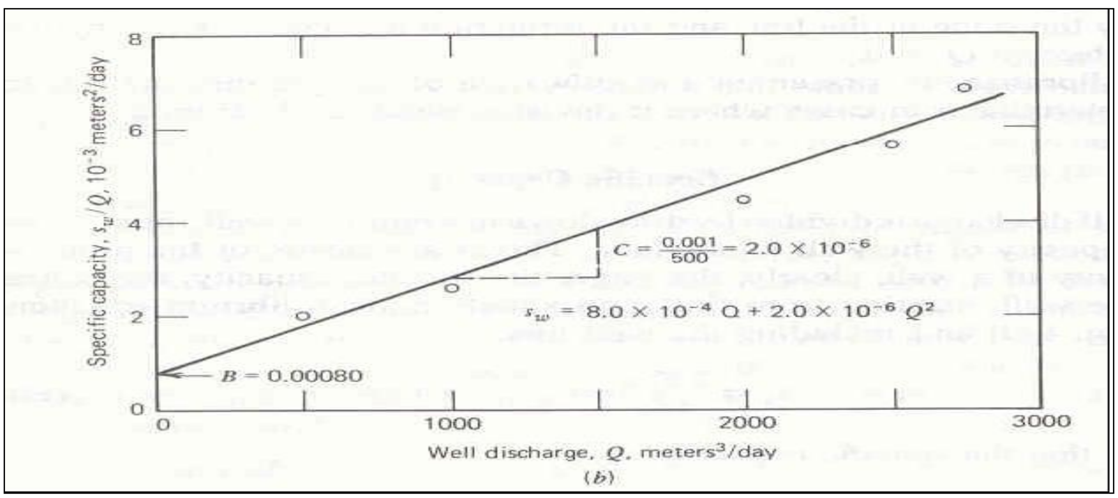


Figure 1.13:Step-drawdown pumping test analyses to evaluate well loss. (b) Determination of B and C from graph s Q w versus Q

1.9 Specific Capacity

It is the ratio of discharge to drawdown in a pumping well. It is the measure of the productivity of a well. The larger the specific capacity, the better the well is.

Using the non-equilibrium equation:

$$s_w = \frac{2.30Q}{4\pi T}\log\frac{2.25Tt}{r_w{}^2 S} + CQ^n$$

So, the specific capacity:

$$\frac{Q}{s_w} = \frac{1}{\left(\frac{2.30}{4\pi T}\right)\log\left(\frac{2.25Tt}{r_w{}^2 S}\right) + CQ^{n-1}} \text{---------}(*)$$

This indicates that the specific capacity decreases with Q and t; the well data plotted in Figure 3.14 demonstrate this effect. For a given discharge a well is often assumed to have a constant specific capacity. Although this is not strictly correct, it can be seen that the change with time is minor.

Any significant decline in the specific capacity of a well can be attributed either to a reduction in transmissivity due to a lowering of the groundwater level in an unconfined aquifer or to an increase in well loss associated with clogging or deterioration of the well screen.

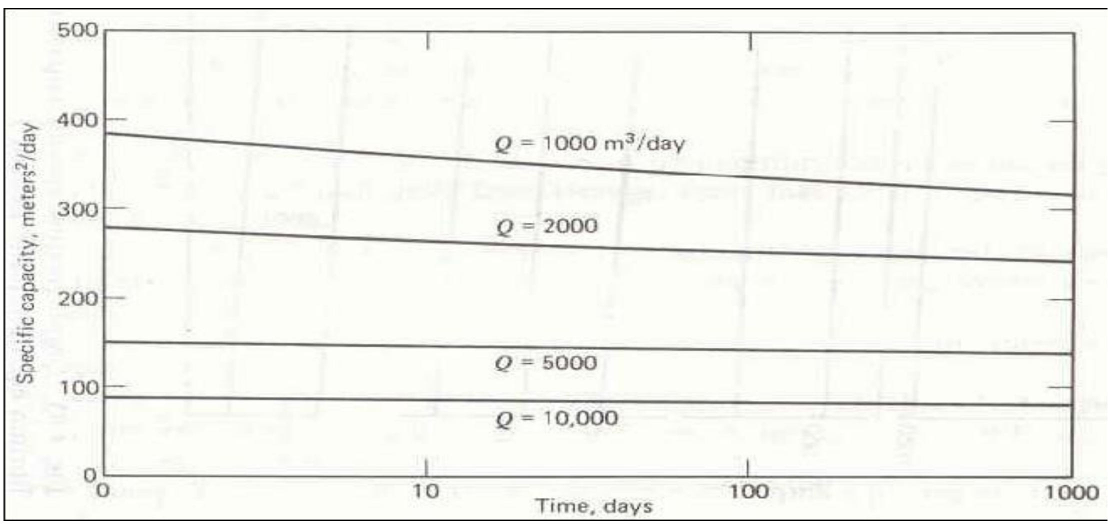


Figure 1.14: Variation in specific capacity of a pumping well with discharge and time

If a pumping well is assumed to be 100 percent efficient ($CQ^n = 0$), then the specific capacity from equation (*) can be presented in the graphic form of Figure 3.15. Here specific capacity at the end of one day of pumping is plotted as a function of S, T, and a well diameter of 30 cm. This graph provides a convenient means for estimating T from existing pumping wells; any error in S has a small effect on T.

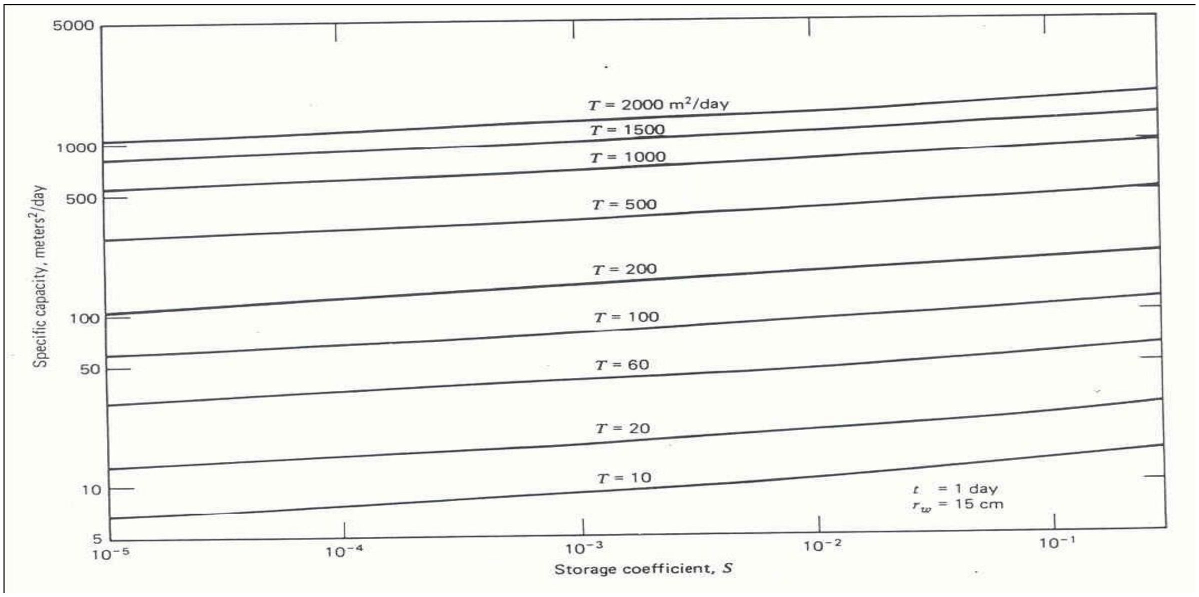


Figure 1.15: Graph relating specific capacity to transmissivity and storage coefficient from the non-equilibrium equation.

1.10 Well Efficiency

Figure 3.14 yields a theoretical specific capacity (Q/ BQ) for known values of S and T in an aquifer. This computed specific capacity, when compared with one measured in the field (Q/sw), defines the approximate efficiency of a well. Thus, for a specific duration of pumping, the well efficiency w E is given as a percentage by:

$$Ew = 100\frac{Q/Sw}{Q/BQ} = 100\frac{BQ}{Sw}$$

Another method for recognizing an inefficient well is to note its initial recovery rate when pumping is stopped. Where the well loss is large, this drawdown component recovers rapidly by drainage in to the well from the surrounding aquifer. A rough rule of thumb for this purpose is: if a pump is shut off after 1 hour of pumping and 90 percent or more of the drawdown is recovered after 5 minutes, it can be concluded that the well is unacceptably inefficient.

Chapter 5
Pump test of wells

Pumping test:-

Pumping tests (or aquifer tests) are in situ methods that can be used to determine hydraulic parameters such as

- Hydraulic conductivity,
- Transmissivity
- Storage coefficient,
- Specific capacity and
- Well efficiency.

The objectives of the pumping test are:

- Determination of well yield,
- Determination of well efficiency,
- Determination of aquifer parameters
- Examination of water chemistry

Con,t…

General notes about pumping test:

- Pump testing is major investigative tool-but expensive.
- Proper planning, observations, interpretation is essential
- It is cheaper (much) if existing wells can be used.
- Pump testing is also carried out in newly constructed wells, as a well test.

Test well & Observation Well

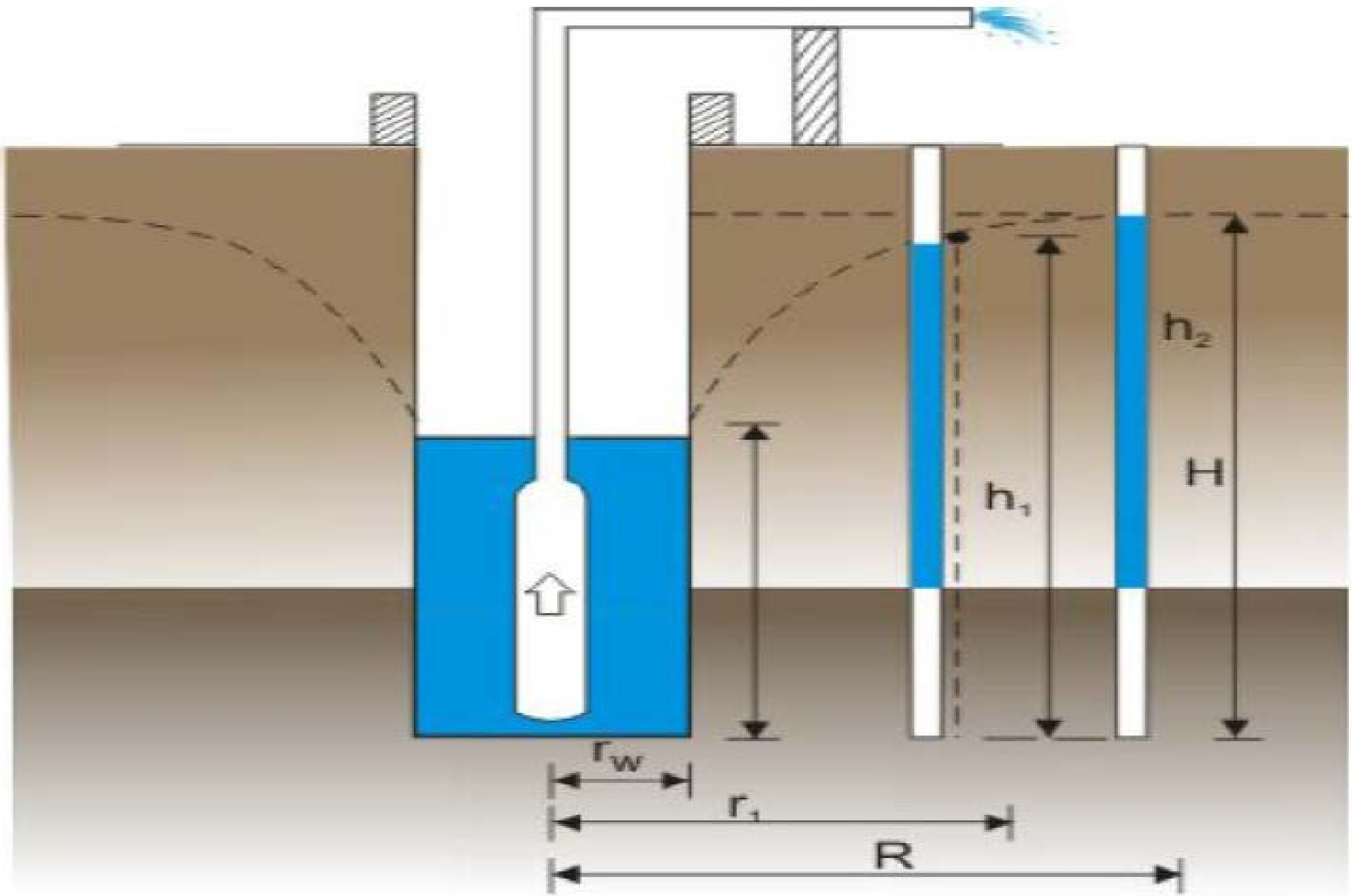
- Test Well – Pumping Well where one measures flow rate/discharge rate (Q [L/T^3])
- Observation well/s are well/s where observation of variation of water level/head being undertaken (also called monitoring wells)
- pumping well itself can be an Observation well (most practically).
- Observation well/wells are located at some distance/s (r) from pumping well.

Piezometers (observatories)

- A piezometer is an open-ended pipe, placed in a borehole that has been drilled to the desired depth in the ground. The bottom tip of the piezometer is fitted with a perforated or slotted screen, 0.5 to 1 m long, to allow the inflow of water.
- The water levels measured in piezometers represent the average head at the screen of piezometers.
- The question of how many piezometers to place for the pumping test depends on the amount of information needed and the funds available for the test.

Con,t…

- Drawdown data from the well itself or from one single piezometer often permit the calculation of aquifer hydraulic characteristics; it is nevertheless always better to have as many piezometers as condition permit.
- The advantage of having more than one piezometer is that draw downs measured in them can be analyzed in two ways: by the time-drawdown relationship and by the distance-drawdown relationship.
- For financial reasons often a single well test is made (no piezometer)


h2
h1
H
rw
r1
R

Conceptual model

Before conducting pumping test one has to carry out the following preliminary investigations.

- The geophysical characteristics of the subsurface
- The type of aquifer and confining beds
- The thickness and lateral extent of aquifers and confining beds
- Boundary conditions
- Data on the groundwater flow system (horizontal or vertical), flow of groundwater, water table gradients, trends in water level, etc.
- Data on any of existing wells in the area
- Good idea on the well set up!

Before conducting pumping test

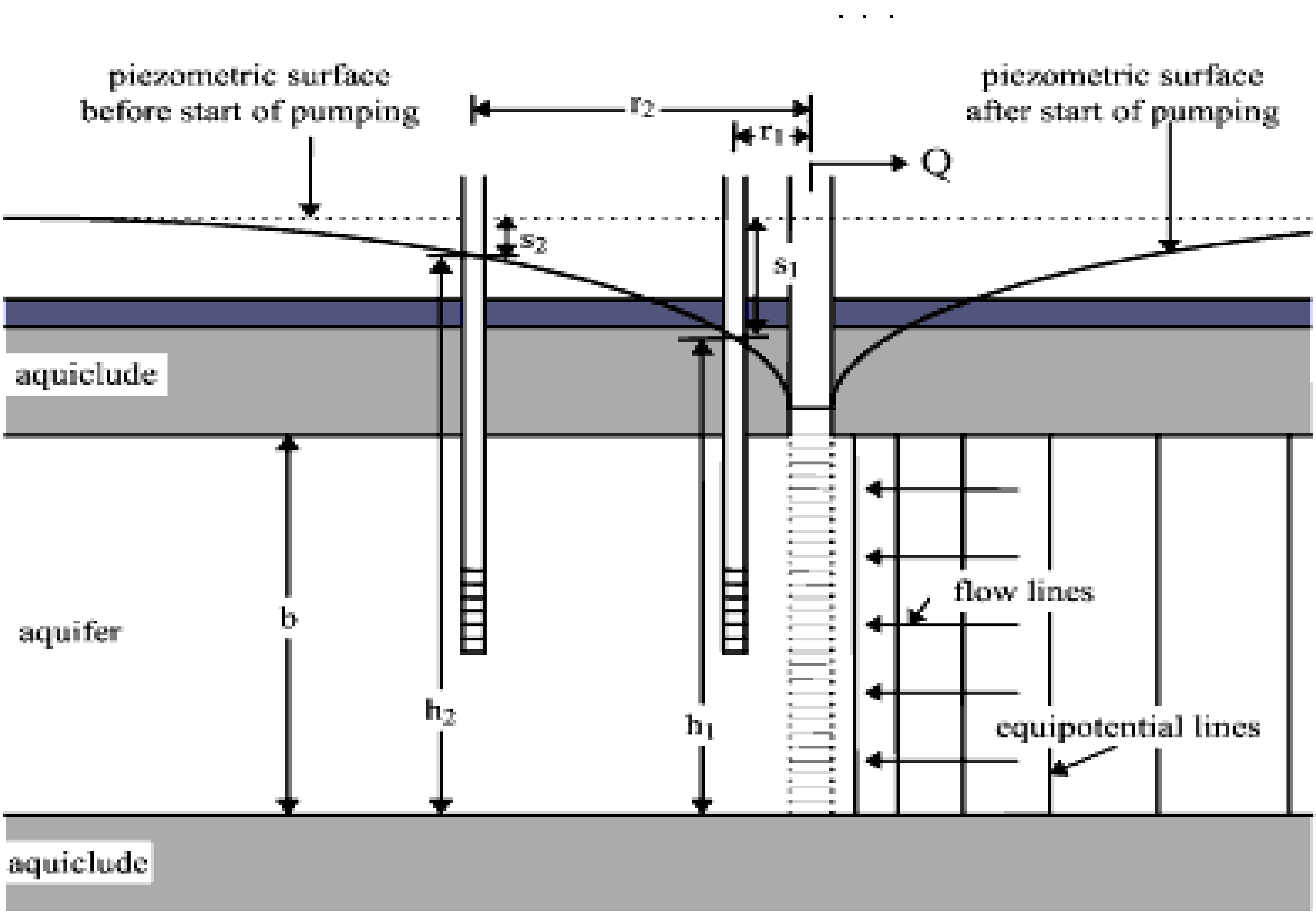
Assess:

- The geophysical characteristics of the subsurface
- The type of aquifer and confining beds
- The thickness and lateral extent of aquifers and confining beds
- Boundary conditions
- Data on the groundwater flow system (horizontal or vertical), flow of groundwater, water table gradients, regional trends in water level, etc.
- Data on any of existing wells in the area

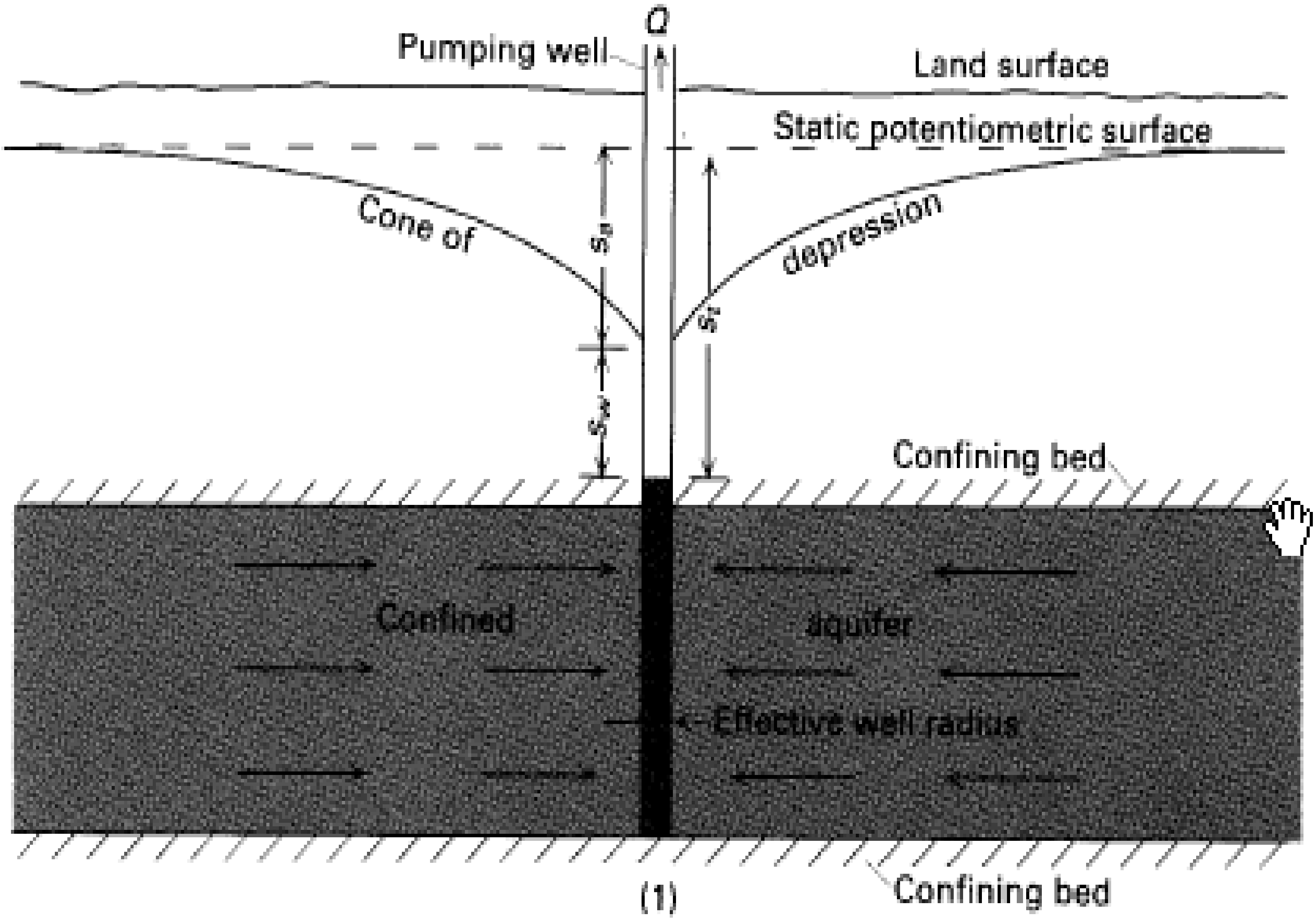
Measurements

- Water level (dynamic and static)
- Discharge rate, water quality samples
- Duration and steps of pumping
- Distance between the well and piezometers
- Pump position
- Aquifer thickness
- Lithologic logs
- Set up of blind and screen casings
- Etc.

illustration Of Measurement



Cone of depression



Selection of Test well site

While locating a test well, the following points should be considered.

- The hydrogeological conditions of test well should not change over short distances and should be representative of the area.
- The gradient of water level should be low.
- The well should be located away from any other recharging or discharging well if such are not used for the testing purpose

Con,t…

- The well should be located way from high and heavy traffic sites, where variation in head may occur.
- The pumped water should be discharged sufficiently away from the well site.
- The well should be far from any recreational areas, buildings and areas of cultural value.
- The site should be easily accessible for machinery, labor and construction material transport.

Observation Wells

The following points should be considered for installation and design of the observation well

- Number of Observation wells
- Spacing of the observation wells
- *Length of Well Screen (aquifer thickness)*
- *Depth of Observation wells*

Number of Observation wells

The number of observation wells depends on the ***amount of information desired*** and ***degree of precision expected***. If it is required to obtain more precise value of hydraulic conductivity (K), and if it is needed to get such information on a large area of extent, a large number of observation wells are needed. However, most of the time only one observation well is used due to economy.

Spacing of the observation wells

The spacing of observation wells depends on:-

- Type of aquifer
- Hydraulic Conductivity
- Length of well screen

Type of aquifer

- In confined and semi confined aquifers the head drop (draw down) is higher than that of unconfined aquifer.
- Therefore, the nearest observation wells should be placed a little further in confined and semi-confined aquifers than in unconfined aquifer for the same discharge rate of a test well.

Con,t…

Hydraulic Conductivity

Observation wells should be placed further in aquifer with higher values of hydraulic conductivity (K) as compared to an aquifer of lower conductivity. The cone depression will be higher when the value of K is high.

Length of Well Screen (aquifer thickness)

The minimum distance of the observation well in partially penetrating confined and semi-confined aquifers should be greater than 0.5 to 2m times the thickness of the aquifer (length of the well screen). In the case of unconfined aquifer, a lesser distance can be used.

Con,t…

Depth of Observation wells

If a test well is fully or partially penetrating, then an observation well should be partially penetrating. This is because in an observation well, only draw down is measured and such work could be done in both cases (whether the well is fully or partially penetrates) and to economize it is recommended to have partially penetrating wells for observation purposes.

Con,t…

Duration of Pumping Test

- The Duration of a pumping test depends on:
- Type of aquifer
- Hydraulic properties of Aquifer
- The method to be used fro analyzing the pump test data

The usual times for pumping in until steady condition are maintained in:

- Semi-confined aquifer is 15 to 20 hours
- Confined aquifers is 24 to 40 hours
- Unconfined aquifers is usually >= 72 hours

Types of pumping tests

Constant-rate test.

- A constant-rate pumping test consists of pumping a well at a constant rate for a set period of time (usually 24 or 72 hr), and monitoring the response in at least one observation well.
- The number and location of observation wells is dependent upon the type of aquifer and the objectives of the study.
- Values of storage coefficient, transmissivity, hydraulic conductivity, and specific capacity can be obtained.

Con,t…

Step-drawdown test.

- During a step- drawdown test, the pumping rate is increased at regular intervals for short time periods.
- The typical step- drawdown test lasts between 6 and 12 hr, and consists of three or four pumping rates.
- Because step -drawdown pumping tests are typically much shorter than constant-rate pumping tests, transmissivity and storativity values are not as accurate for these tests. T
- he primary value of the step-drawdown test is in determining the reduction of specific capacity of the well with increasing yields.

Con,t…

Recovery test.

- A recovery test consists of measuring the rebound of water levels towards preexisting conditions immediately following pumping.
- The rate of recovery is a valuable source of data which can be used for comparison and verification of initial pumping test results.
- The results from properly conducted tests are the most important tool in groundwater investigations.

Con,t…

The following measurements for both well tests and aquifer tests are important:

- The static water levels just before the test is started
- Time since the pump started
- Pumping rate
- Pumping levels or dynamic water levels at various intervals during the pumping period
- Time of any change in discharge rate
- Time the pump stopped

pumping test data sheet

PUMPING TEST DATA SHEET

Project ____

Name of abstraction well ____________________________________

Distance from observation well (m) ____________________________

Well depth ______________________ Well diameter ______________________

Date of test: Start ___________________ Finished __________________________

Depth of pump _____________________ SWL ____________________________

Remarks ____

Date	Actual time	Elapsed time "t" (min)	Depth to water table (m)	Drawdown (m)	Discharge Q (m^3/hr)	Remark

Recovery test data sheet

RECOVERY DATA SHEET

Project ____________________________ Date _________________ Sheet ____________________

Name of abstraction well ____

Distance from pumped well (m) ____

Discharge rate during pumping (m^3/hr) _________________ SWL ____________________________

Remarks ____

____

Actual time	Time (t) since pumping began (min)	Depth to water level (m)	Discharge rate (m^3/hr)	Drawdown (m)	t/r^2

The End

Thank you

Groundwater exploration

6. Introduction

- Groundwater is a hidden resource.
- One has to locate a proper site for getting good amount of Water for the purpose.
- To this end Groundwater exploration (investigation = search for GW) is a first task before developing a water well.
- To Explore = To search through extensive and technical/scientific ways for a material, a place, etc…

- A program of groundwater investigations is to obtain information on the resource through systematic collection, synthesis, interpretation and compilation of data.
- It seeks information on its occurrence, movement, storage, recharge, discharge, quality & quantity etc…

A comprehensive program for hydrological investigations may comprise the following activities:

- Surface Investigations
- Subsurface Investigation of Groundwater
- Hydrological Investigations

Surface Investigations

- Geological field reconnaissance, including observations and collection of data from excavations, bore holes and wells. The appraisal includes information on geological factors, particularly tectonics, lithology, permeability, fissuring and outcrop area.
- Geophysical surveys

  a) Electrical resistivity method

  b) Seismic refraction method

Subsurface Investigation of Groundwater

- Test drilling and preparation of lithological logs
- Sub-surface/bore hole geophysical logging
  a) Electric logging
  b) Radial logging
- Collection of lithological & other logs of existing bore hole & correlation of lithological logs.

Hydrological Investigations

- Preparing inventory of existing wells, giving their location, depth, depth of water, construction features, type of pumping equipment used, pumping records and water analysis.
- Study of groundwater levels - preparation of water table contour maps, water level profiles, hydrographs and setting up of observation grids.
- Collection and analysis of water samples
- Aquifer tests - to appraise transmissibility and storage property of aquifers.
- Hydrologic appraisal of the geological framework: Geometry of aquifers & boundaries affecting recharge & discharge of groundwater.

- Correlation of stream flow factors with groundwater recharge and discharge.
- Estimation of seepage & recharge contribution from canals, lakes and ponds.
- Study and analysis of meteorological factors; precipitation and evapotranspiration
- Rainfall and infiltration studies to estimate contribution of rainfall to groundwater recharged.
- Hydrologic analysis of groundwater systems through analytical & other techniques.

6.1 Surface investigations of Ground water

- Variety of techniques can provide information concerning Groundwater occurrence and under certain conditions even its quality from surface or above-surface locations.
- Surface investigations of groundwater are seldom more than partially successful in that results usually leave the hydrogeologic picture incomplete.
- However, such methods are normally less costly than subsurface investigations.

6.1.1. Geologic Methods

- Geologic studies enable large areas to be rapidly and economically appraised on a preliminary basis as to their potential for groundwater development.
- A geologic investigation begins with the collection, analysis, and hydrogeologic interpretation of existing topographic map, aerial photographs, geologic maps and logs, and other pertinent records.

- Geologic field reconnaissance and evaluation of available hydrologic data on: stream flow and springs; well yields; groundwater recharge & levels; and water quality.
- Such an approach should be regarded as a first step in any investigation of subsurface water because no expensive equipment is required; furthermore, information on geologic composition and structure defines the need for field exploration by other methods.
- Knowledge of the depositional and erosional events in an area may indicate the extent and regularity of water-bearing formations.

6.1.2 Geophysical Exploration

- Geophysical exploration is the scientific measurement of physical properties of the earth’s crust for investigation of mineral deposits or geologic structure.
- Geophysical methods are used to obtain more accurate information about subsurface conditions, such as type and depth of materials (consolidated or unconsolidated), depth of weathered or fractured zone, depth to groundwater, depth to bed rock, and salt content of groundwater.
- The method is mimic to laboratory testing for identifying a patient’s disease/ bacteria, virus or other foreign substance… as such are hidden from physical sight.

- These methods detect differences (or anomalies), of physical properties within the earth’s crust. Density, magnetism, elasticity, and electrical resistivity are properties most commonly measured.
- Experience and research have enabled pronounced differences in these properties to be interpreted in terms of geologic structure, rock type and porosity; and water content & water quality.

The most common techniques for groundwater investigation are:

- Electric resistivity and
- Seismic Refraction methods,

will be discussed in the following sections

1. Electric resistivity Method

- Resistivity survey is a method where by electric current is sent through the ground and the potential difference is measured between two points. It is the most commonly adopted method for the determination of saline and fresh water zone.
- It is cheapest method and relatively easy to interpret. It is easy to employ and the equipment is easy to transport from place to place.

- The concept of this method is based on the reality that different earth materials display a characteristic resistance to flow of a set of electrical current.
- As the geologic layers are functions of (Porosity, Fluid content, Nature of fluid etc…), which have different values of electric resistivity, one can determine the location, the thickness and the state of such layers making use of Electric resistivity method.

Material	**Resistivity Ωm (Ranges)**
Clay and marl	1-140
Loam	6 -130
Top soil	40-100
Clayey soils	100-200
Sandy soils	400-1950
Typical mine water	1-10
Typical surface water	6 -130
Shale	10-40
Limestone	40-1000
Sandstones	20 - 3500
Coal	180 - 1680

The principle of measuring electric resistivity is based on the electric fundamentals.

Having the assumptions:

- Isotropic and homogeneous medium
- Point electrodes
- Earth is semi-infinite (Z=0)

- $V = I.R$ or $R = V/I$ (Ohm's Law)
- And Resistivity (ρ) , characteristic property of a material, is give by:

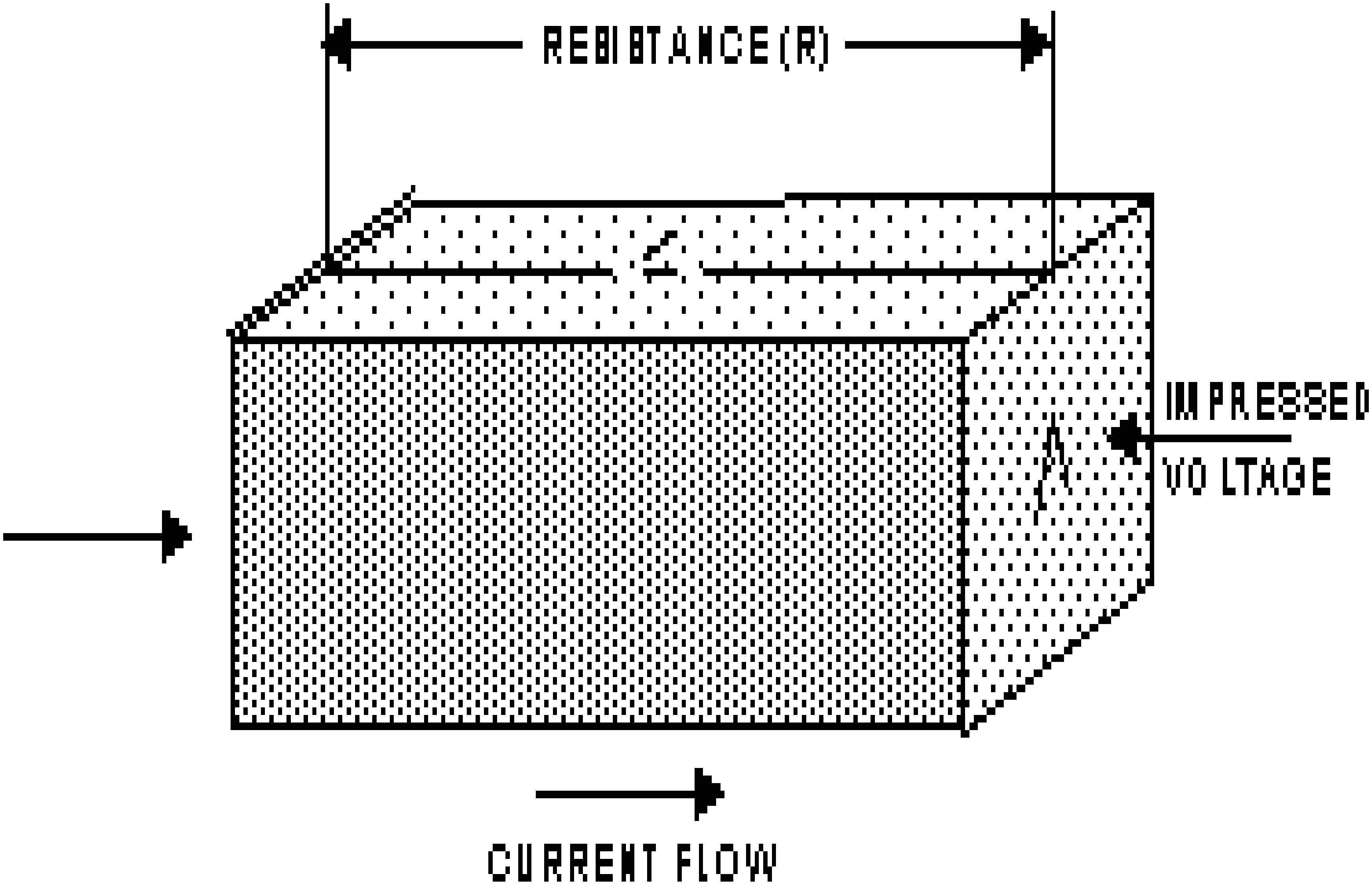
$$\rho = RA/L$$

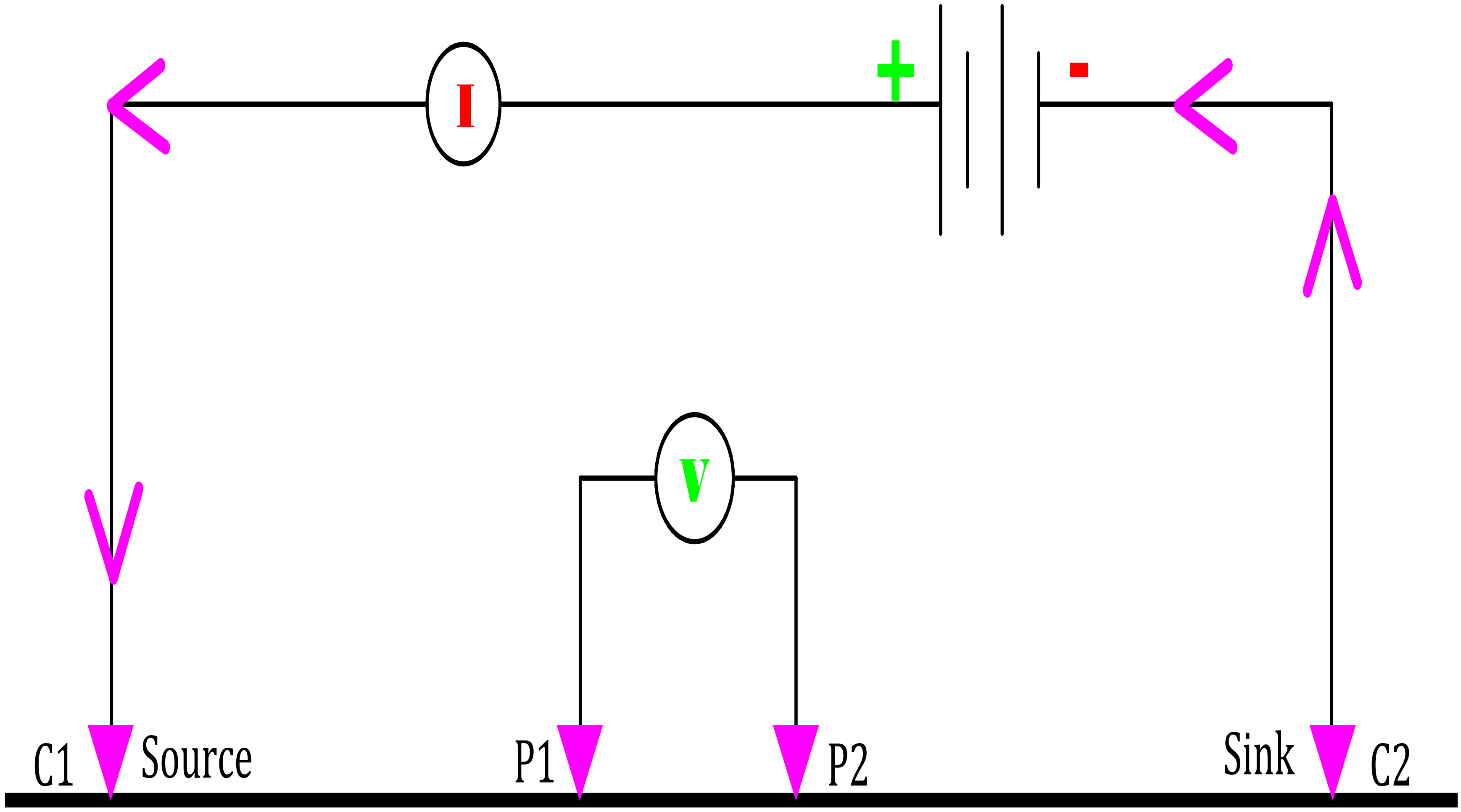
Where: ρ = Resistivity of a material

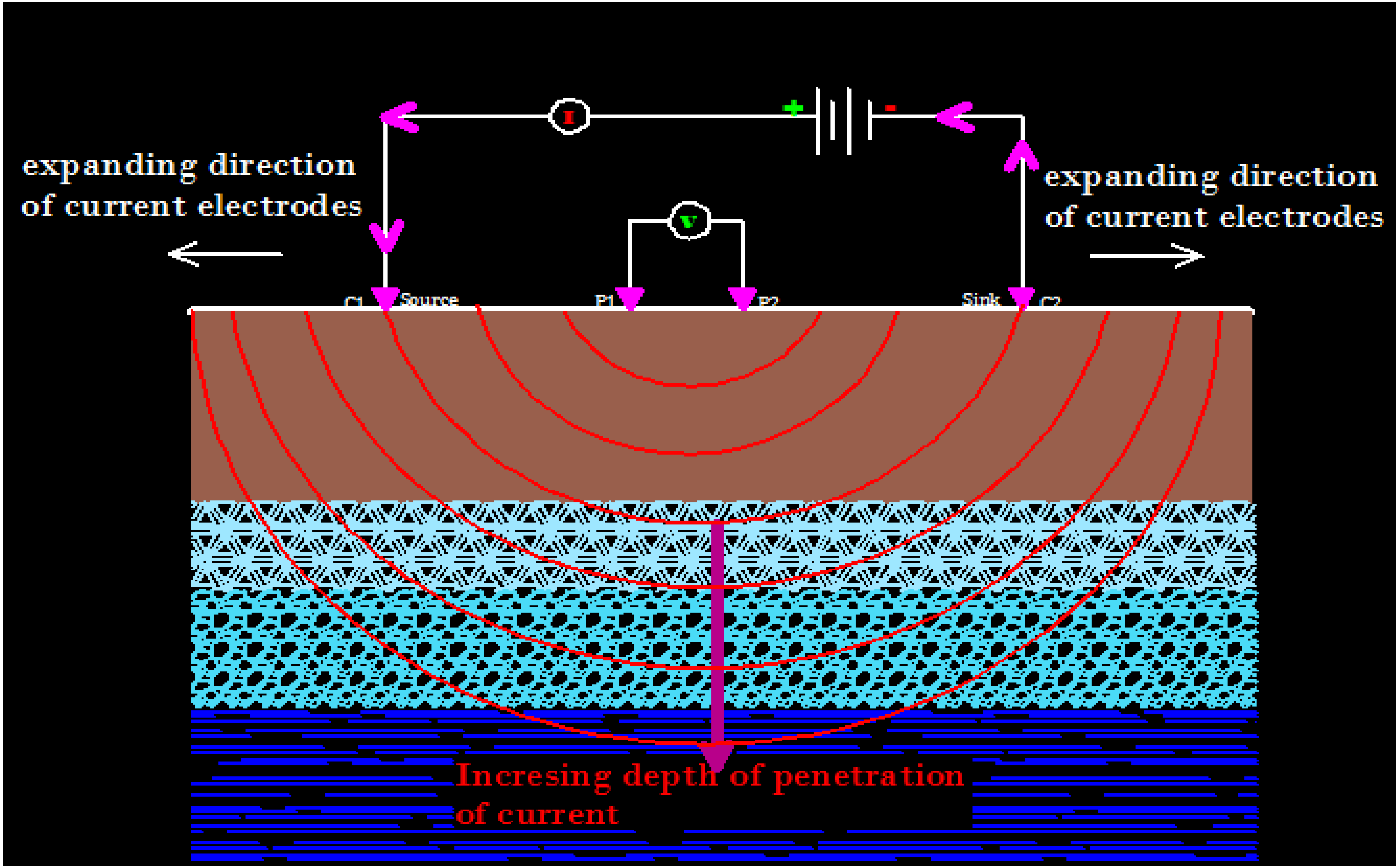
R= Resistance to Electric Current

A = Area of a material

L = Length of a material


RESISTANCE (R)
IMPRESSED VOLTAGE
CURRENT FLOW

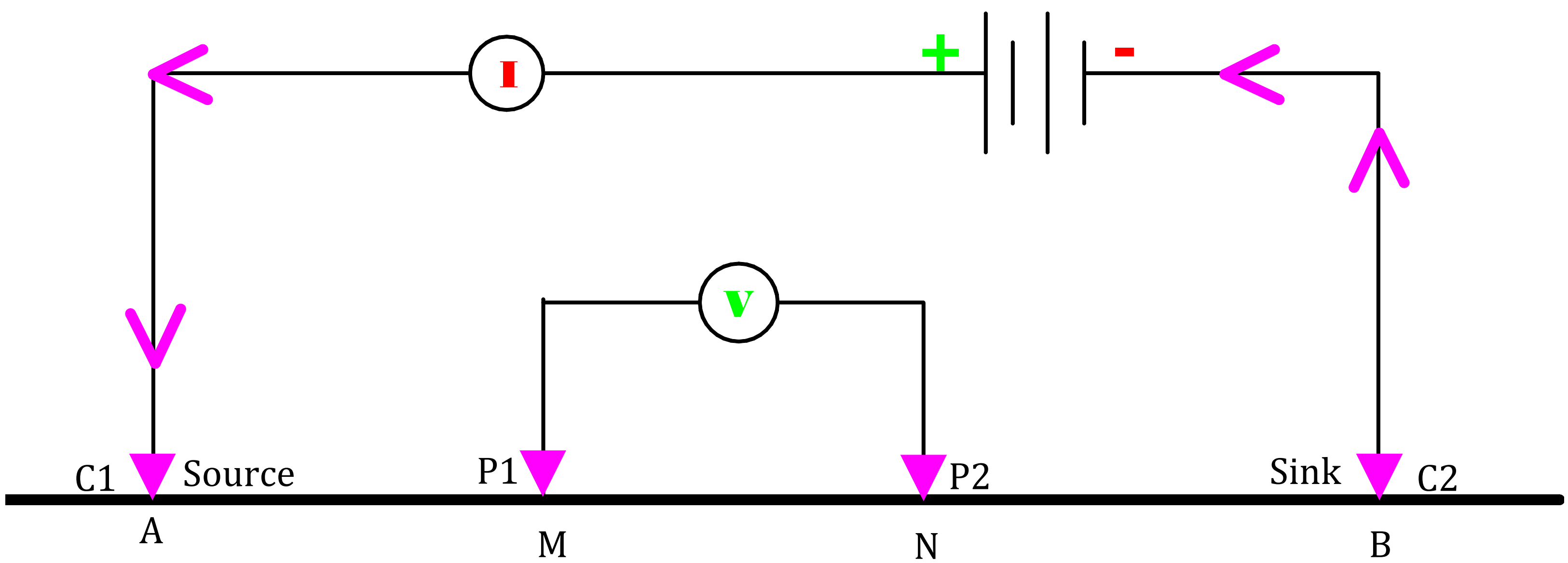

I
+
-
V
C1
Source
P1
P2
Sink
C2


I
V
+
-
expanding direction
of current electrodes
expanding direction
of current electrodes
C1
Source
P1
P2
Sink
C2
Incresing depth of penetration
of current

- Referring Back to the fundamental equations: and considering the depth of of penetration of electric current as r, the radius of semi-sphere:
- $V = \rho I / 2\pi r$; considering the area (A) of semi-sphere (along the depth of penetration of current flow in the subsurface) = $2\pi r^2$ and the length also as r.
- When the spacing of current electrodes widens, then the depth of penetration of current flow increases and pass through different formations.

Computation of Potential Difference

Consider the resistivity set as follows:



Typical Resistivity Set-Up

- C1 and C2 – Current electrodes (metal stakes)
- P1 and P2 – potential electrodes
- AB is the distance between current electrodes
- MN is the distance between potential electrodes
- The potential of electrode P1 at M due to current electrode C1 at A is:

$$V_M^A = \frac{\rho I}{2\pi AM}$$

The potential of electrode P2 at N due to current electrode C1 at A is:

$$V_N^A = \frac{\rho I}{2\pi AN}$$

- And similarly,

$$V_M^B = \frac{\rho I}{2\pi BM} \, and \, V_N^B = \frac{\rho I}{2\pi BN}$$

- The potential difference, ΔV can be calculated as follows:

$$V_M^{AB} = V_M^A - V_M^B = \frac{\rho I}{2\pi}\left[\frac{1}{AM} - \frac{1}{BM}\right]$$

$$V_N^{AB} - V_N^A - V_N^B = \frac{\rho I}{2\pi}\left[\frac{1}{AN} - \frac{1}{BN}\right]$$

$$\therefore \Delta V = V_M^{AB} - V_N^{AB} = \frac{\rho I}{2\pi}\left[\frac{1}{AM} - \frac{1}{BM} - \frac{1}{AN} + \frac{1}{BN}\right]$$

$$\therefore \rho = \frac{\Delta V}{I}\left[\frac{2\pi}{\frac{1}{AM} - \frac{1}{BM} - \frac{1}{AN} + \frac{1}{BN}}\right]$$

Where, ΔV= measured potential difference
I = Measured electric current

And the term in bracket is referred as shape/geometric factor = K

$$\therefore \quad \rho = \frac{\Delta V}{I}.K(ohm - m)$$

As the earth material are almost never homogeneous and electrically isotropic, the resistivity found by the above equation is an apparent resistivity, ρa.

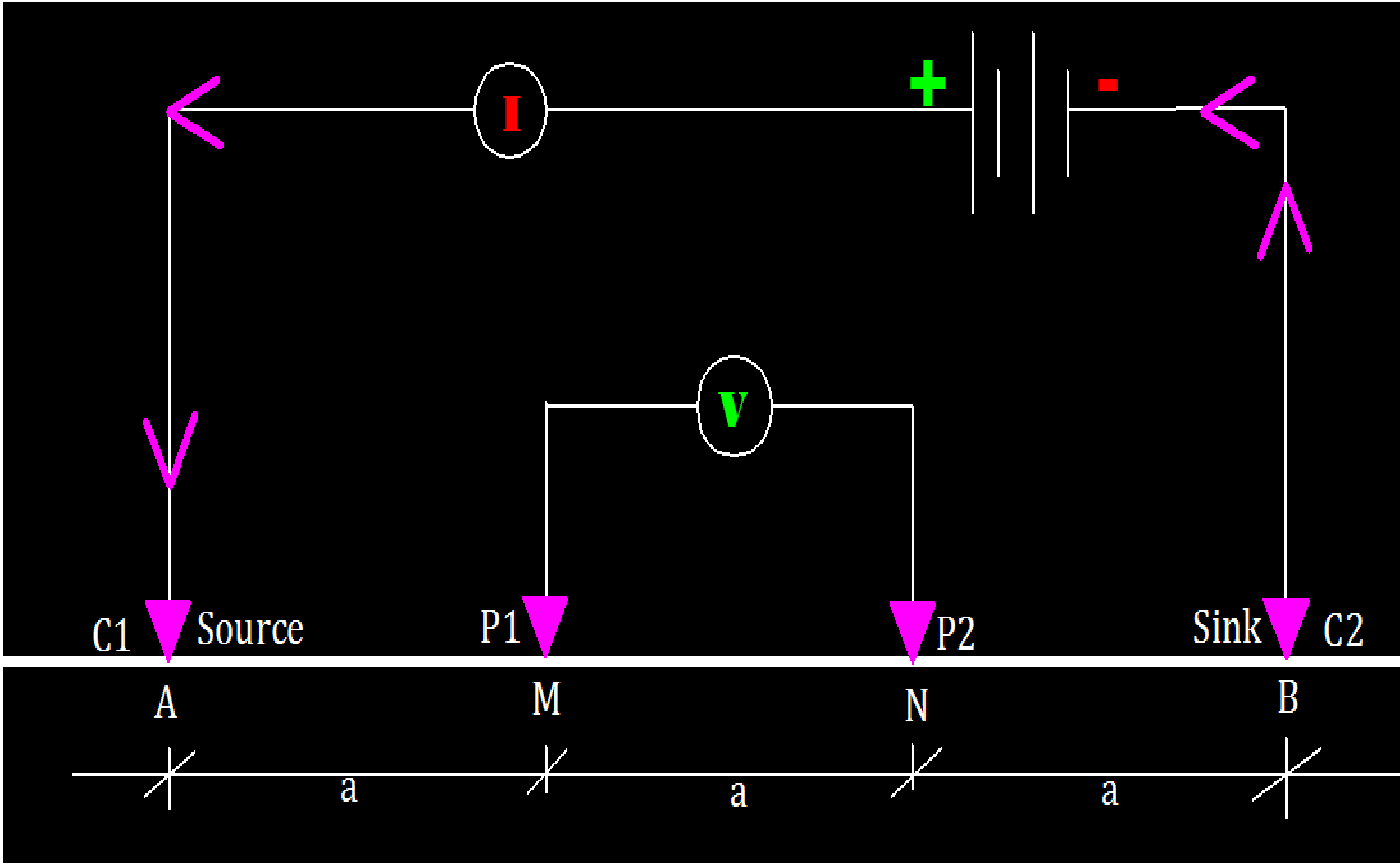
Electrode Configurations/Setups

Many different arrangements of electrode can be used to measure resistivity, i.e., there are several electrode configuration in common usage. Some of these are:

- Wenner system
- Schlumberger system
- Lee system
- Dipole-Dipole system

The basic relationships in all methods are:

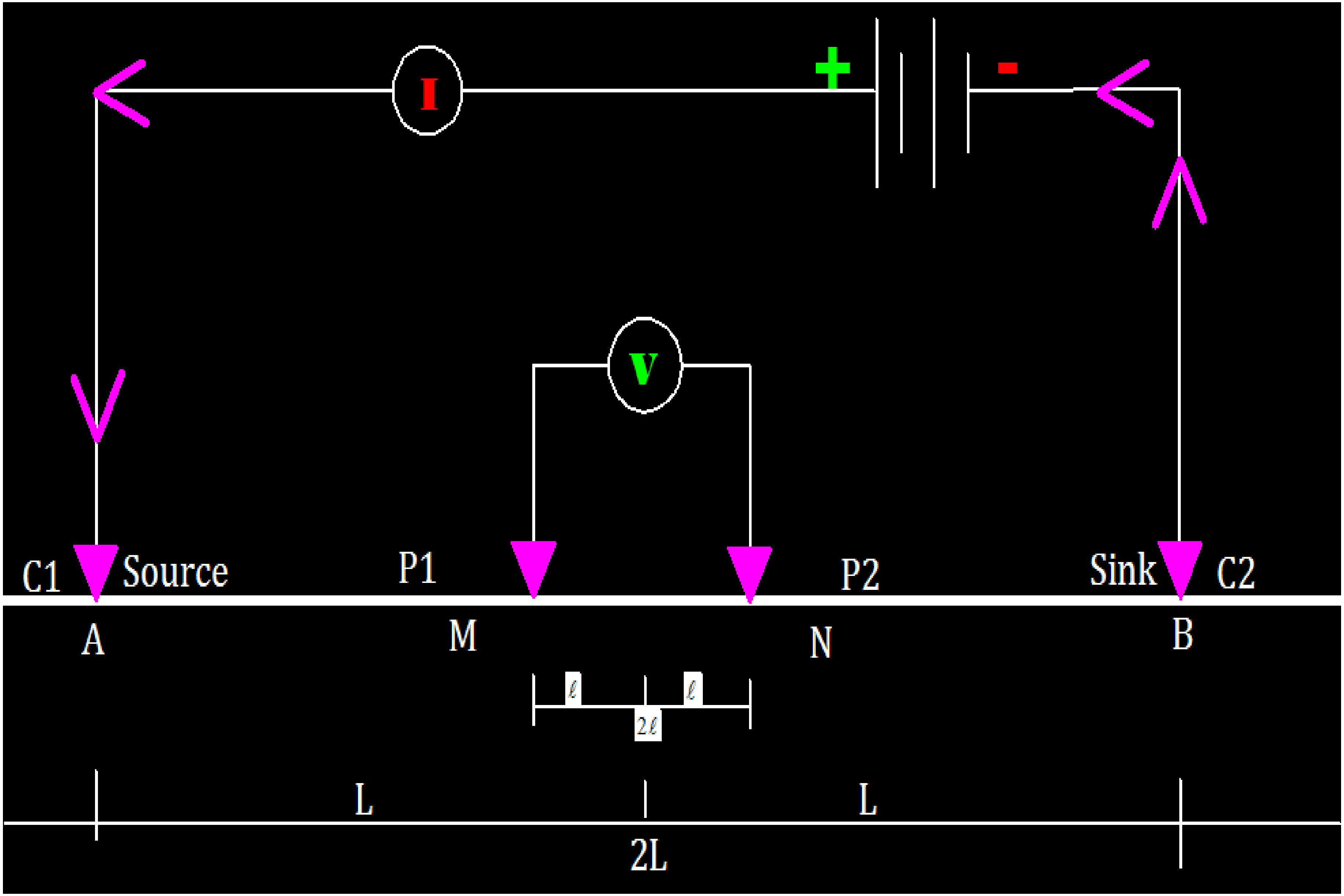
- Electrodes (two current & two potential electrodes)
- Linear system.
- The commonly used systems of electrode arrangements are the Wenner and Schelumberger



Wenner Electrode Configuration

$\rho a = 2\pi Ra$; APPARENT RESISTIVITY FOR WENNER ELECTRODE CONFIGURATION

ℓ



Schelumberger Electrode Configuration

Apparent Resistivity for Schelumberger Electrode Configuration

$$\boldsymbol{\rho}_a = \boldsymbol{\pi} R\left(\frac{L^2 - \ell^2}{2\ell}\right) \qquad if \;\; 2L >> 2\ell$$

$$\boldsymbol{\rho}_a = \boldsymbol{\pi} R\left(\frac{L^2}{2\ell}\right) \qquad if \;\; 2L >> 5(2\ell)$$

Types of Instruments

There are basically two types of instruments to conduct the electrical resistivity survey:

NGRI resistivity meter,

a d.c. type meter manufactured by the National Geophysical Research Institute, Hyderadad (South India).In this instrument V and I are separately measured to obtain the resistance R (=V/I). Generally battery packs with different voltage of 15,30,45 and 90 volts are employed

Terrameter

An a.c. type of instrument manufactured by Atlas Copco ABEM AB, Sweden.The output is 6 Watts at 100, 200 or 400 volts using low frequency(1-4 Hz) square waves. This instrument directly gives the resistance, R, in ohms.It is a good instrument for conducting rapid electrical resistivity surveys for location sites for drilling bore wells.

Types of Resistivity Surveys

- Regardless of the specific electrode spread employed, there are two basic procedures in resistivity work.
- The particular procedure to be used depends on whether one is interested in resistivity variations with depth or with lateral extent.
- Electric drilling (or electrical depth sounding): for detecting vertical changes.
- Electrical mapping, or trenching or horizontal profiling: for detecting subsurface changes in horizontal direction or the lateral spread.

Electric Drilling (or electrical sounding , Es)

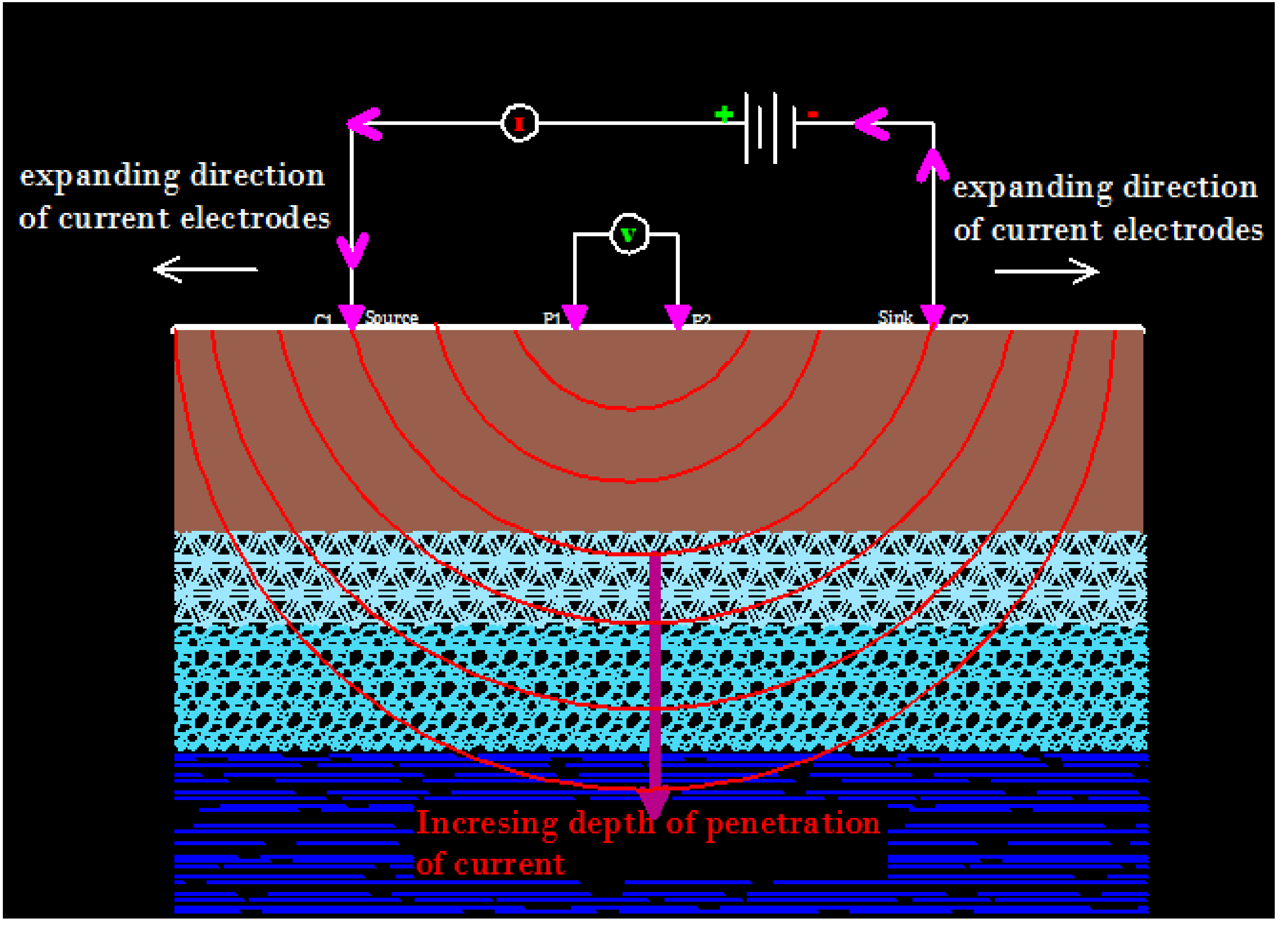
It is produced by taking a series of measurements at a point, but moving the current electrodes further apart for each measurement. The depth of current penetration is then increased

The fraction of total current which flows at depth varies with the current electrode separation, the field procedure is to use a fixed center with an expanding spread. Both then Wenner and Schlumberger layout are particularly suited to this technique. But Schlumberger system is superior to the Wenner array since voltage electrodes do not have to be moved each time.

A curve of the variation of apparent resistivity with depth AB/2 = L can drawn on log-log paper form the results. It reveals the variations of ρa with depth.

The presence of horizontal or gently dipping beds of different resistivites is best detected by the expanding spread. Hence the method is useful in determining depths of aquifers, bed rock, fractured or weathered zones in rock and fresh water salt interfaces.

Electric Depth Sounding



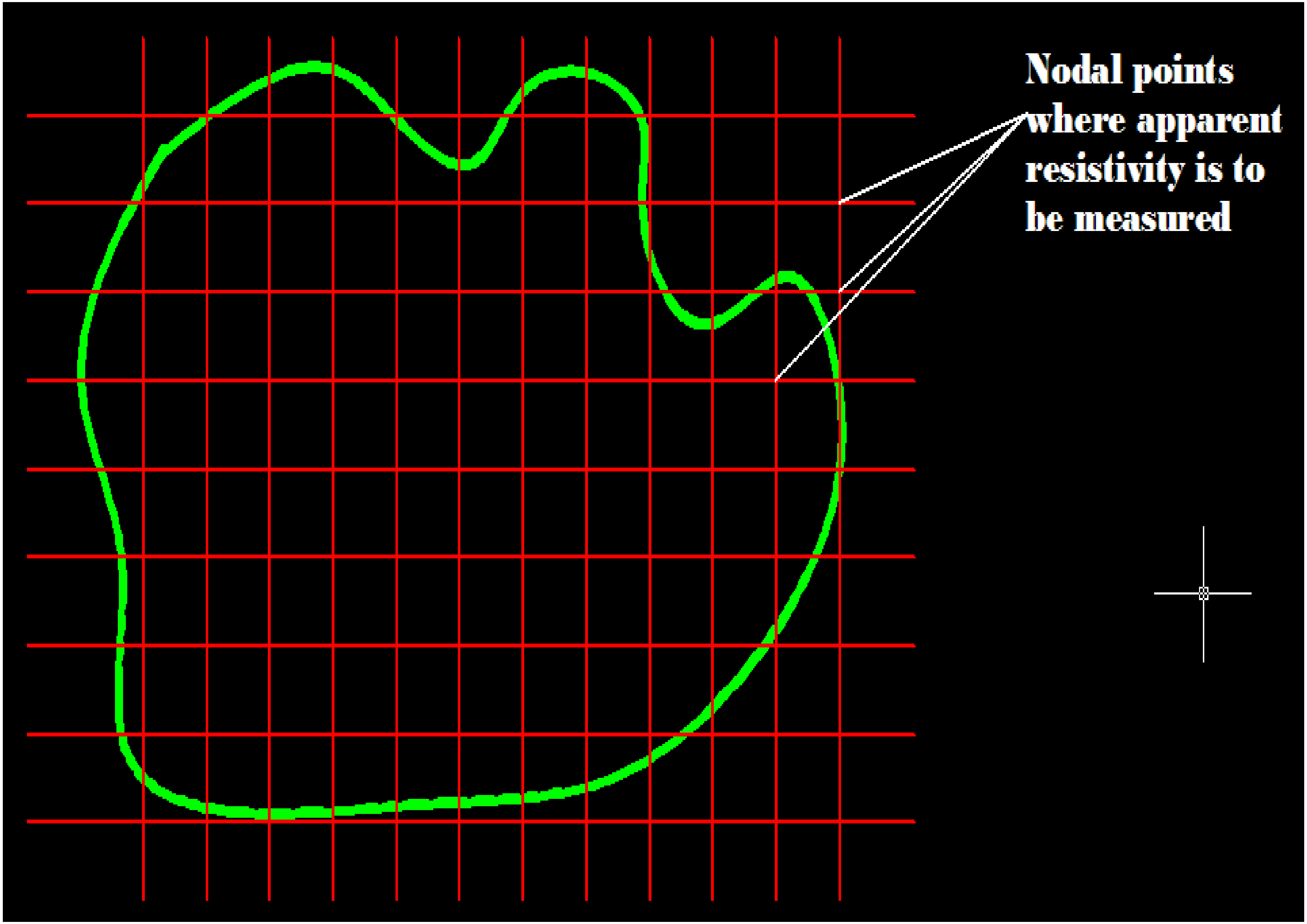
Electric Mapping

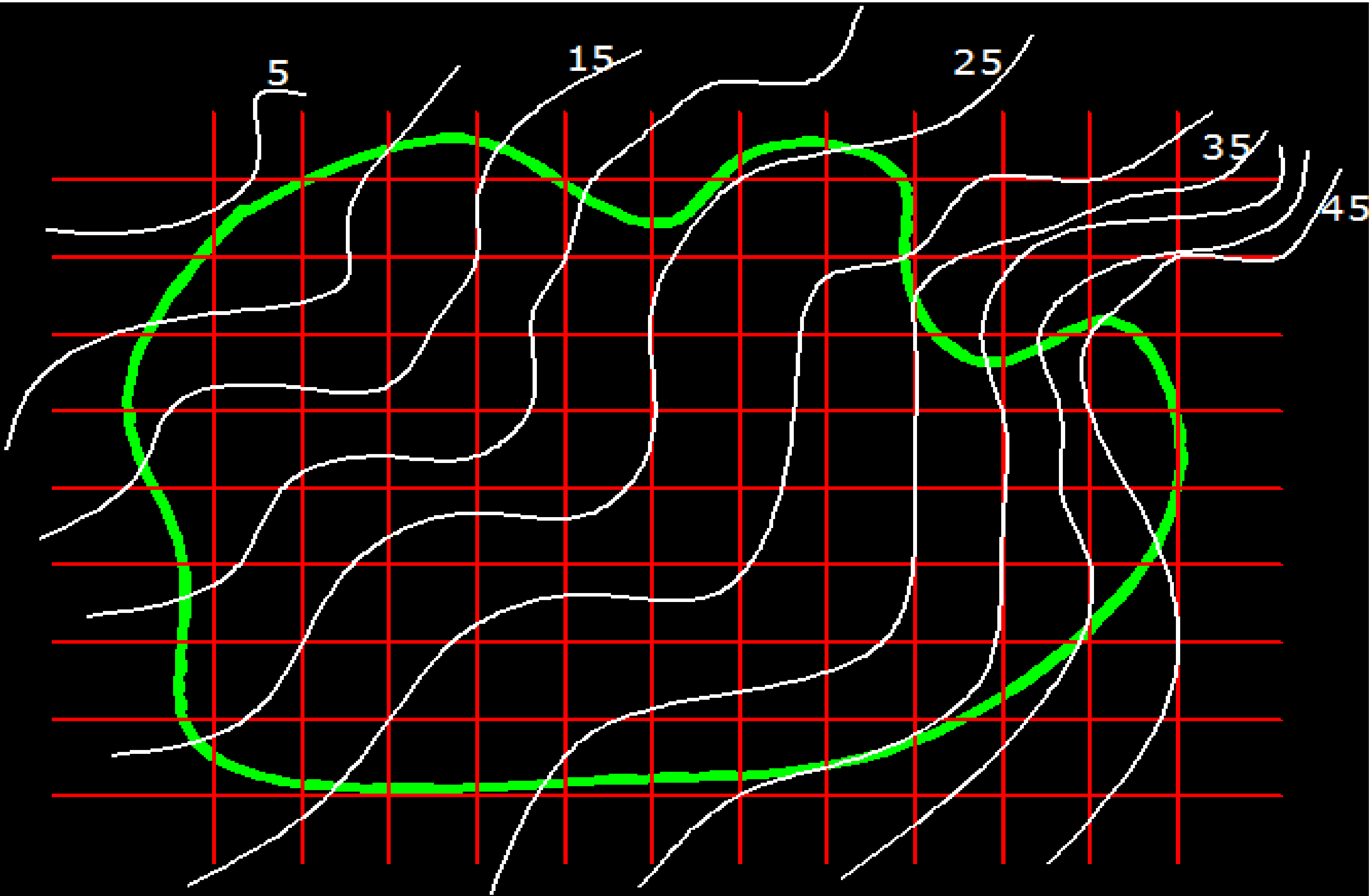
- This method is useful particularly in mineral exploration, where the detection of isolated bodies of anomalous resistivity is required.
- In traversing or profiling method the electrode separation is kept constant for two or three values (say a= 10m, 15m, or 20m) and the center of the electrode spread is moved from one station to another station (grid points) to have the same constant electrode separations.

Profiling can be carried out along a series of parallel lines and a resistivity contour map of the area showing iso-resistivity lines can be prepared.

This will indicate areas of high resistivity and will be useful in identifying aquifer formations.

Such maps are useful for detecting changes in bedrock or aquifer depth (for example, in tracing buried valleys), vertical discontinuities such as faults and fractured zone, changes in groundwater quality (including travel of contaminated water), and changes in the depth of fresh water-salt water interfaces (especially in costal areas).


Nodal points where apparent resistivity is to be measured



Resistivity contour map of a typical area
(Mapped using Wenner electrde set-up)

General Field Procedure

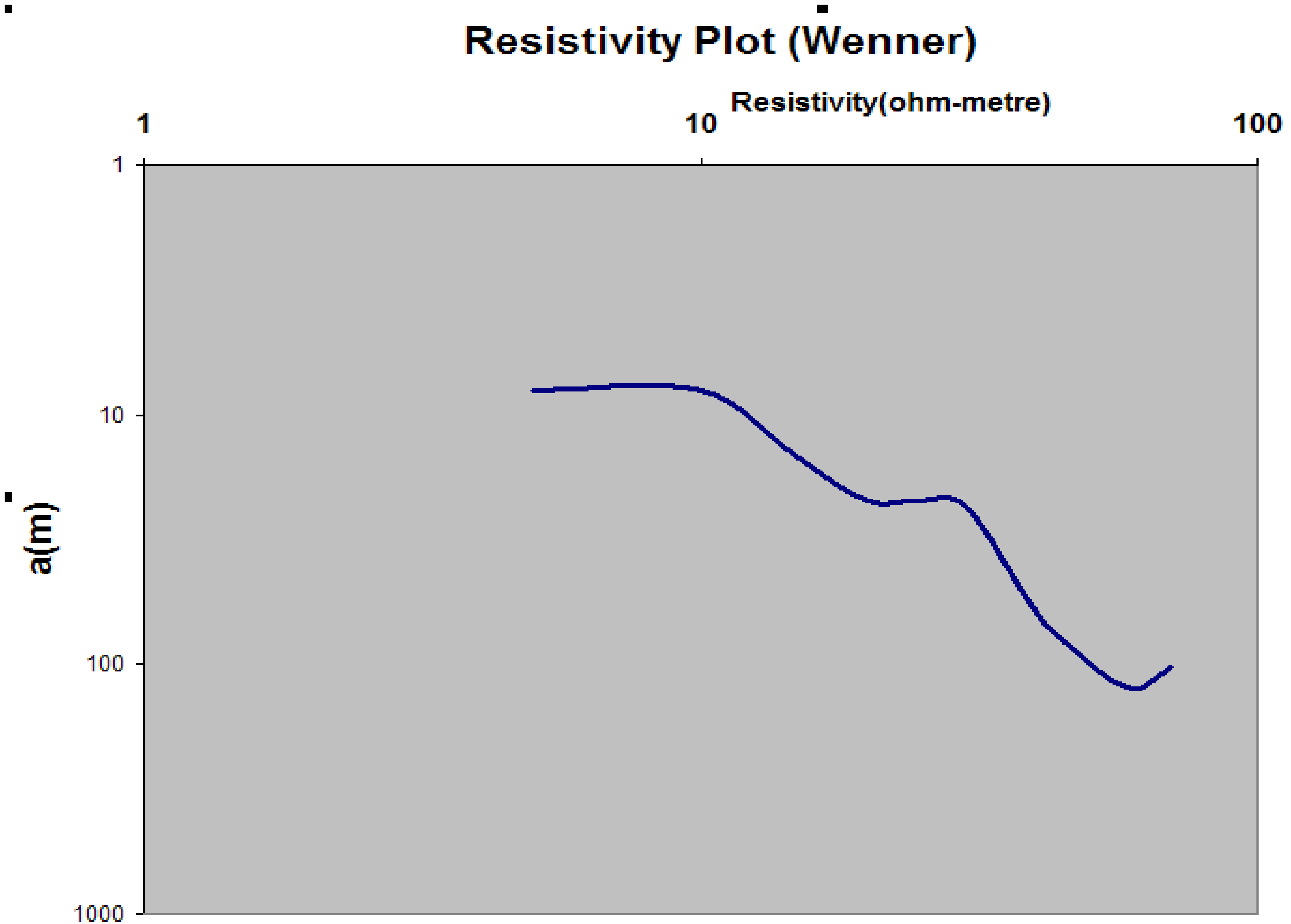
1. Site Investigation (Reconnaissance)-Preliminary preparation to collect the data

- Have some idea of general geology (aquifer, aquicludes, GWT). That means, before starting the survey understand the geology, structure & topography of the area under investigation.
- Also ensure that no man made structures like buried pipe lines, high tension electric wires, exist.
- Start with depth sounding (Schlumberger method) to determine the thickness & estimated depth of water table.
- From the trend of the formation (aquifer) conduct additional depth sounding (Schlumberger) at sites following the trench of the aquifer.

- If there is a control point (bore hole/well) in the area, the results could be correlated & calibrated accordingly.
- Finally, keep the electrode spacing constant (Wenner) to map the aquifer, i.e. conduct electrical mapping.

2. Collection of the data
3. Analysis/Synthesis of the data – calculation of the resistivity (depth wise + in lateral extent)
4. Presentation of the Results

Resistivty Plot (Schelumberger)
1
10
100
1000
1
10
100
1000
AB/2
Resistivty (ohm-metre)


Resistivity Plot (Wenner)
Resistivity(ohm-metre)
1
10
100
1
10
100
1000
a(m)

Interpretation of Results

The results of can be interpreted:

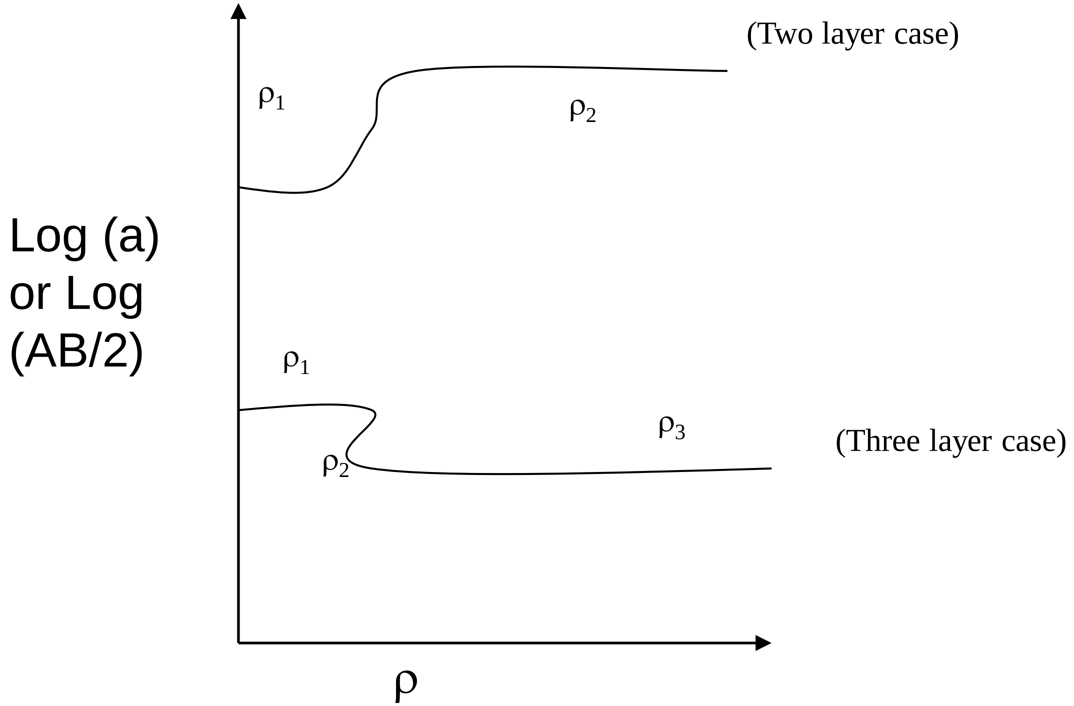
- Qualitatively
- Quantitatively

Example:

Qualitative

One inflection point -------- 2 layer case

Two inflection points ----- 3 layer case



- The shapes of curves in this case give rise to a number of layers encountered in the subsurface i.e., the layers can be 2,3 up to 10.
- When $\rho 2 > \rho 1$. This means the upper layer is more conductive compared to lower layer.
  When $\rho 2 > \rho 1$.
- This means the upper layer is less conductive compared to lower layer.
- The qualitative interpretation of the three layer case has four types of curves

Quantitative Interpretations

- Automatic interpretation by computer programs is
- available nowadays.
- Curve matching on log-log graph paper on the same modulus as the “standard curves”. Example: (Master curve by Orellanar and Mooney, 1996)
- Interpretation using Tag curves
- Interpretation using Direct slope method

Direct Slope Method

- It consists of plotting the field resistivity data for determination of absolute resistivity and thickness of layers.
- The field data are processed to obtain the cumulative resistivity ($\sum\rho a$) for plotting curve (ρa vs a), tangents are drawn to the curve and the values of ‘a’ at which the slope of the curve changes give the depths to the top of each layer.

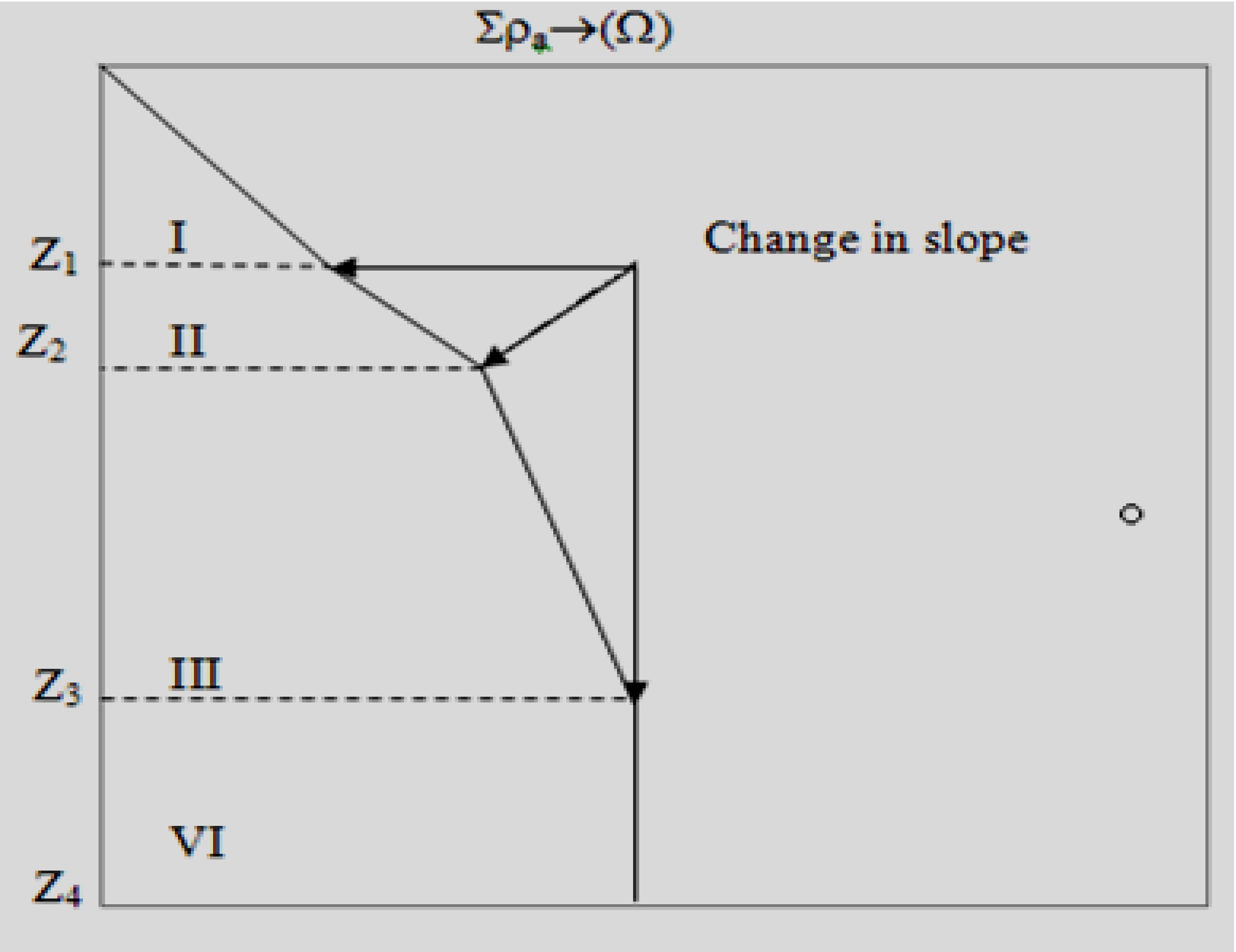


Fig Cumulative Resistivity curve

Use of electrical resistivity method:

- Correlating lithology and drawing geophysical section
- Bed rock profile for subsurface studies
- Fresh water-salt water interface separation profiling
- Water quality in shallow aquifers & groundwater pollution as in oil field brine pollution, pollution by irrigation waters and pollution by sea water intrusion, which cause change in electrical conductivity.

Limitations of the RESISTIVITY Method:-

- The method can not be used whenever potential disturbances exist: underground cables, pipelines or metal fences are in contact with the soil or near power line (high tension wires) with transformers.
- Depth is up to 500m
- In rugged topography measurements are strongly affected. In areas where rock formation is dipping greater than 15 the measurements are seriously affected.

Example

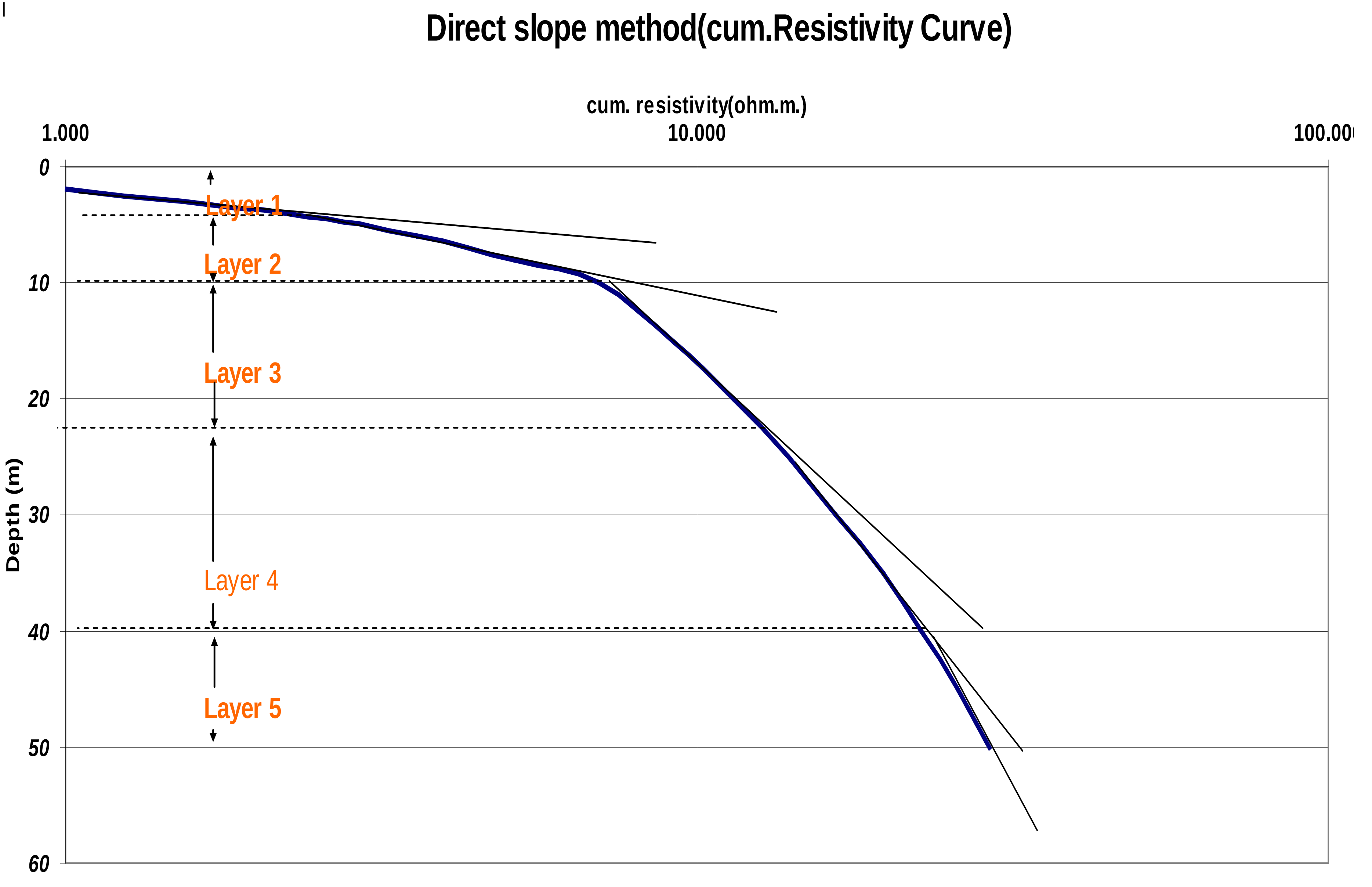
Results of resistivity depth sounding are given below using a Wenner Electrode Configuration:

a(m)	2	4	6	8	10	15	20	25	30
R(=V/I)(Ω)	0.0800	0.049	0.036	0.031	0.029	0.023	0.018	0.0165	0.0140

a(m)	35	40	45	50
R(Ω)	0.014	0.012	0.011	0.010

Interpret the data using direct slope method

a(m)	R(=V/I)(W)	ρ = 2π.Ra	cum Res.(Σρ)
2	0.08	1.006	1.006
4	0.049	1.232	2.238
6	0.036	1.358	3.595
8	0.031	1.559	5.154
10	0.029	1.823	6.977
15	0.023	2.169	9.146
20	0.018	2.263	11.409
25	0.0165	2.593	14.001
30	0.014	2.640	16.641
35	0.014	3.080	19.721
40	0.012	3.017	22.739
45	0.011	3.111	25.850
50	0.01	3.143	28.993



Direct slope method(cum.Resistivity Curve)
cum. resistivity(ohm.m.)
1.000
10.000
100.000
0
10
20
30
40
50
60
Depth (m)
Layer 1
Layer 2
Layer 3
Layer 4
Layer 5

Seismic Refraction Method

- Seismic refraction method involves the determination of elastic wave velocities through geologic formations.
- The method is based on the fact that elastic waves travel through different materials at different velocities.
- The waves are produced by a small dynamite explosion, sledge hammer etc. and picked up at various points on the ground surface by set of receivers called geophones and recorded.

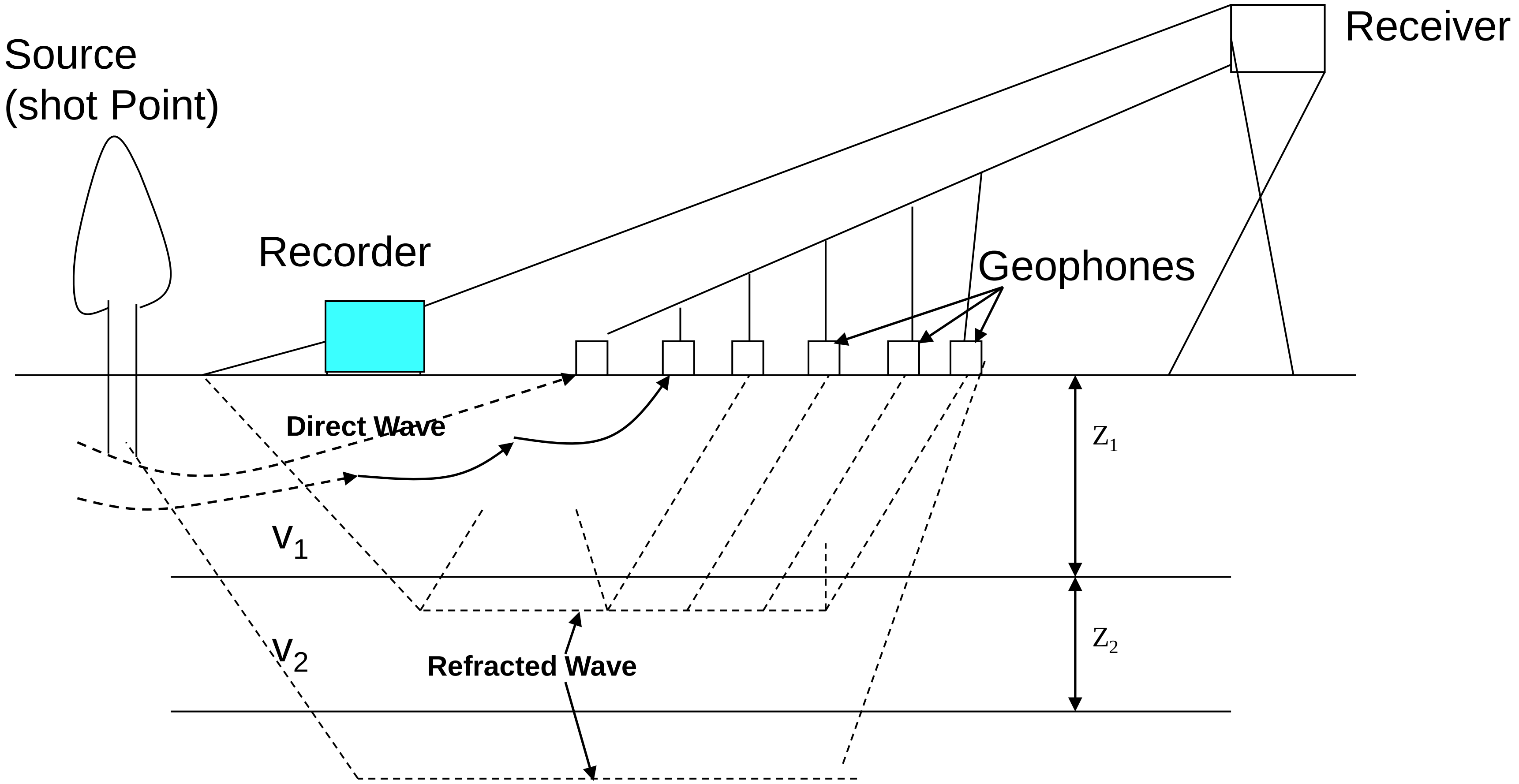


Fig Seismic Refraction Field set up

- With refraction seismic surveys, the geophones are uniformly spaced on a straight line from the shot point to record the arrival time of the first shock waves.
- That is, these waves may have traveled straight from the shot point to the geophones, or they may have been refracted and reflected in the deeper layers.
- By knowing the arrival time of different waves at different distances from the energy source, the velocity of propagation of the wave through each rock layer is calculated.
- The method is more accurate.

Rock Formation	Range of Velocities (m/s)
Dry sand and loose soil	150-400
Alluvium	500-1500
Wet sand	600-1800
Clays	900-3000
Sand stone	2000-4300
Shale	2100-4000
Limestone	3000-6000
Igneous and metamorphic rocks	4500-6500

- Seismic waves follow the same laws of propagation as light & may be reflected or refracted at any interface where a velocity change occurs.
- Refracted and reflected shock waves will reach the more remote geophones sooner than the straight-traveling waves if the velocity of sound in the deeper layers is much greater than that in the surface materials.

- Plotting the arrival time of the first shock wave at each geophone against distance of geophone from shot point yields a curve which for a layered profile consists of a succession of straight-line sections.
- The first section represents the firs layer (or top layer) of the profile, the 2nd section the second layer etc…
- A time-travel curve (time versus distance from source to geophone) is drawn and by knowing the distance X1 to the first point on the curve where a change in slope is indicated, the depth to the rock layer can be computed from the equation:

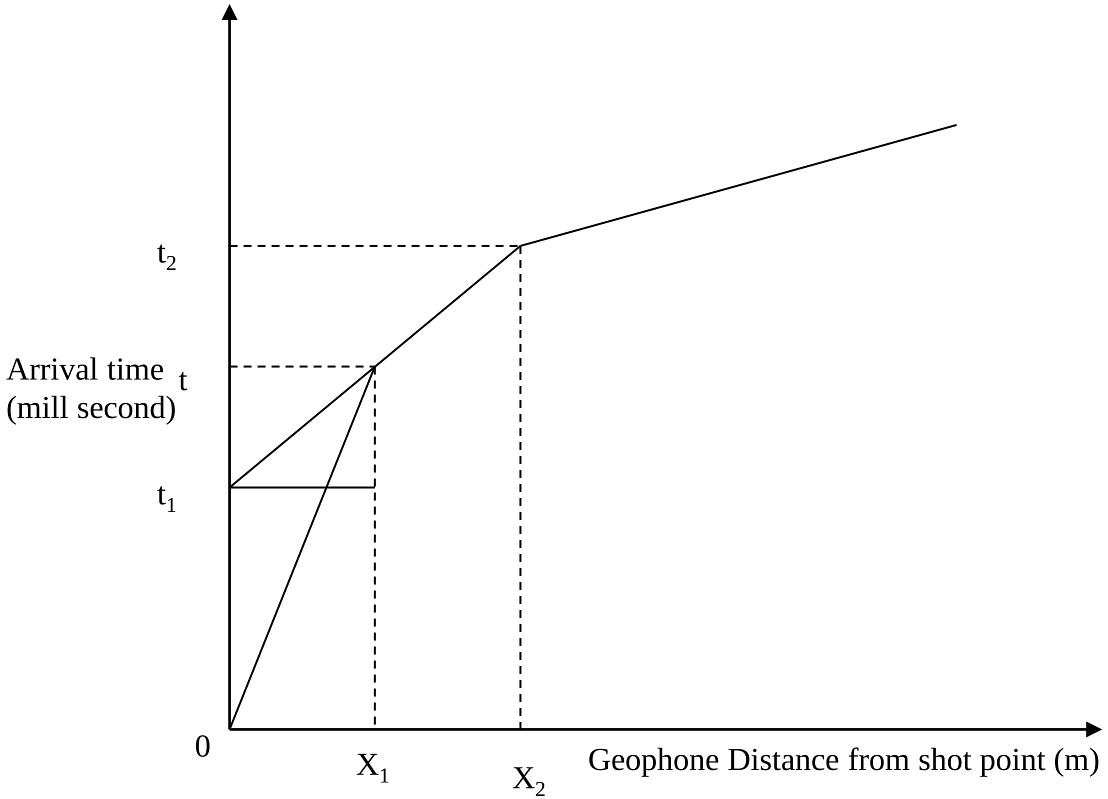


Fig Time-distance graph

$$Z_1 = \frac{X_1}{2}\sqrt{\frac{V_2 - V_1}{V_1 + V_2}}$$

Or from Time intercept formula,

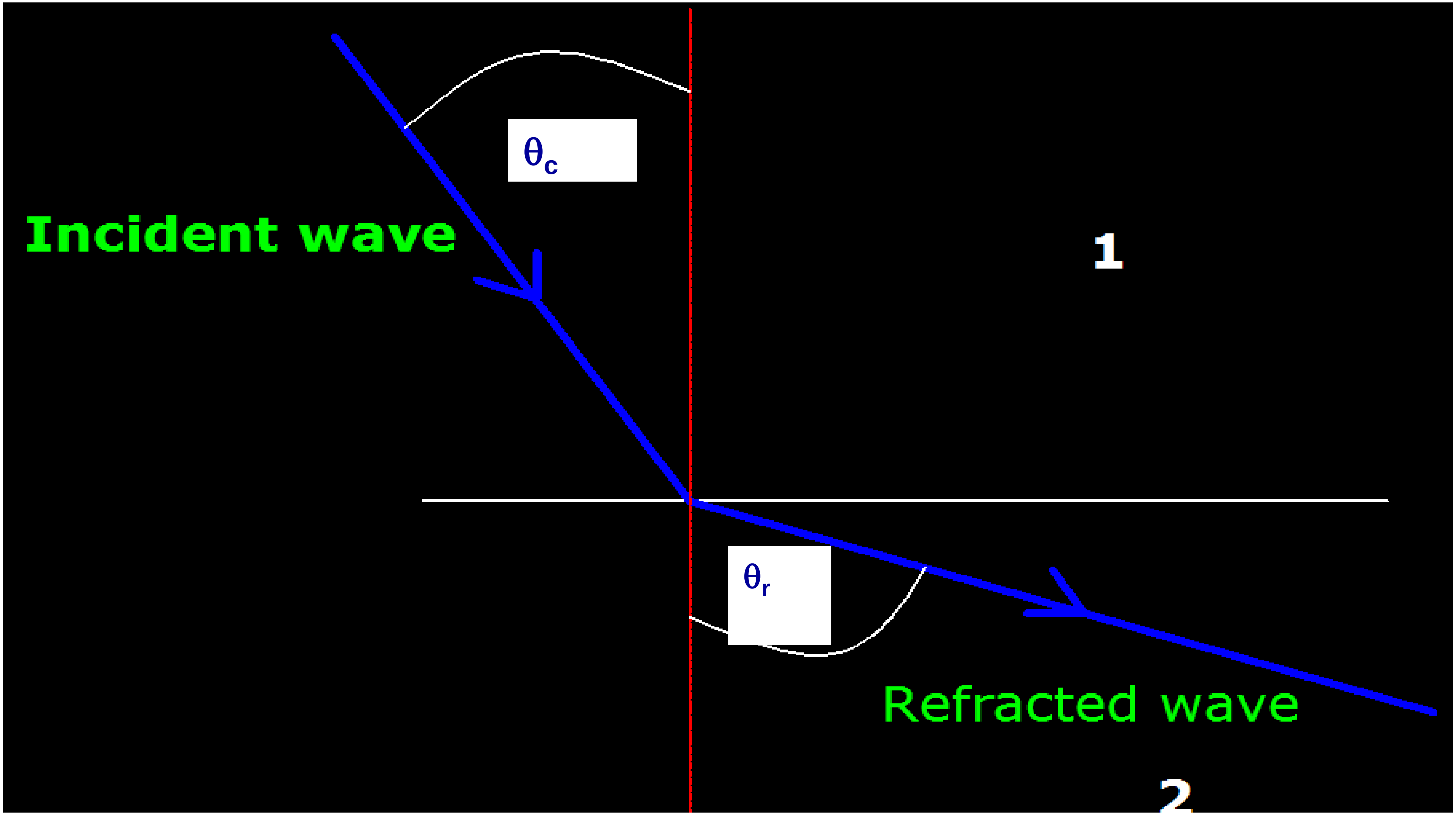
$$Z_1 = \frac{t_1}{2}\frac{V_1 V_2}{\sqrt{V_2{}^2 - V_1{}^2}}$$

The depth Z_2 of the second layer is given by:

$$Z_2 = \left(\frac{t_2}{2} - Z_1 \sqrt{\frac{V_3^2 - V_1^2}{V_3 V_1}} \right) \frac{V_3 V_2}{\sqrt{V_3^2 - V_2^2}}$$

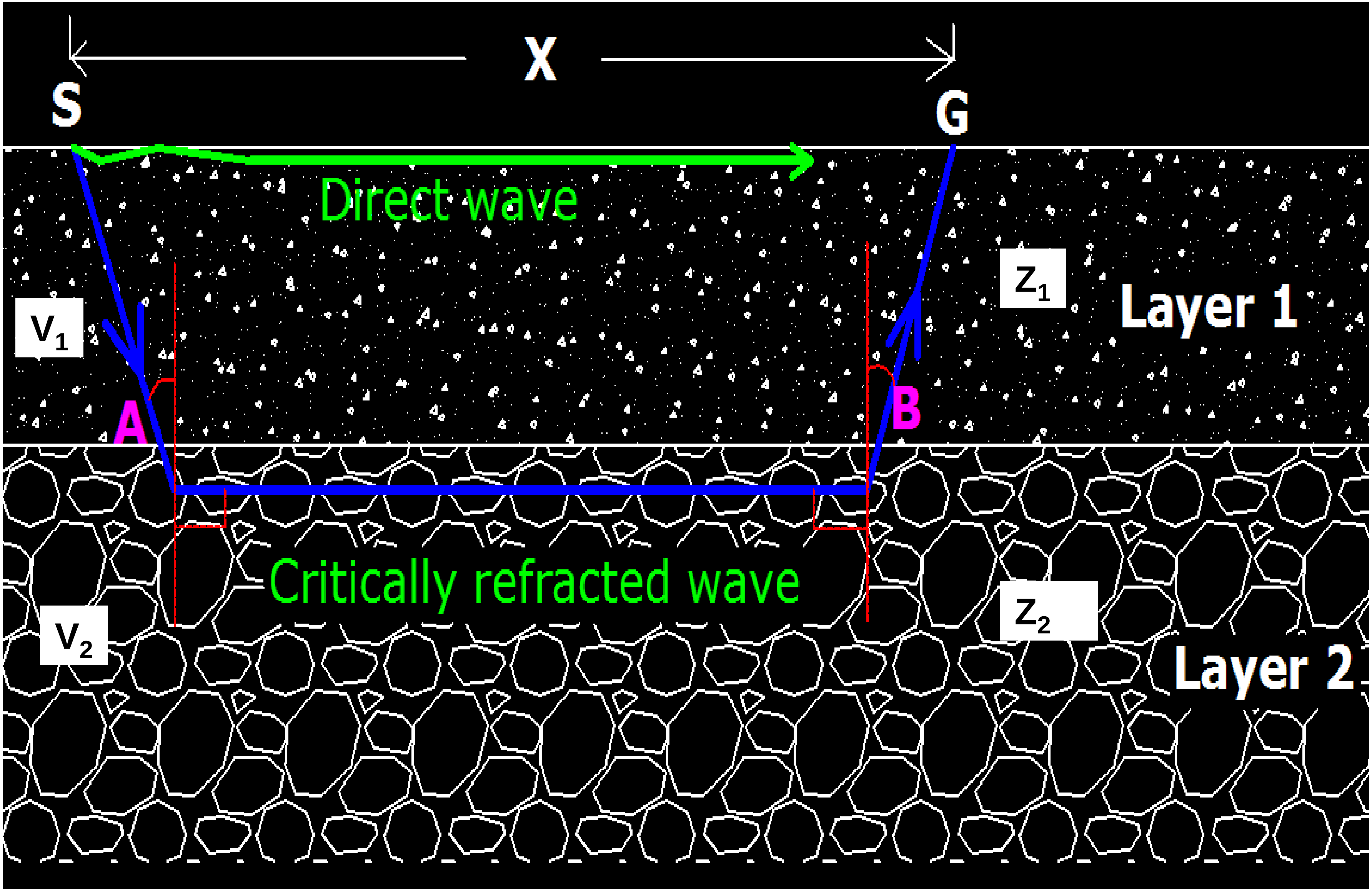
An approximate equation for Z2 presented by Geophysical Specialties Company (1960) is:

$$Z_2 = \frac{X_2}{2} \sqrt{\frac{V_3 - V_2}{V_3 + V_2}} - \frac{Z_1}{6}$$


θc
Incident wave
1
θr
Refracted wave
2

Snell’s Law:

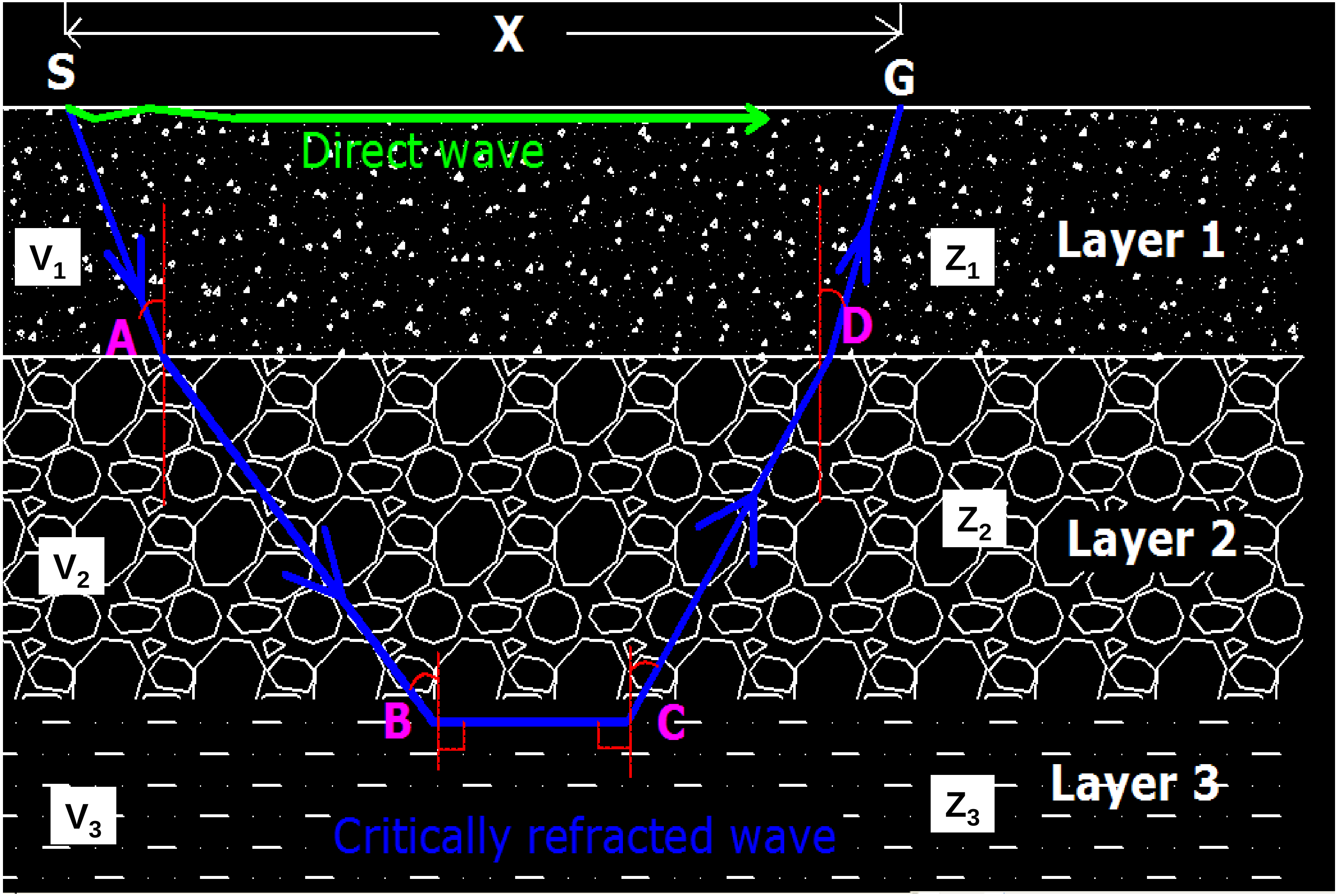
- $\mathrm{Sin}\theta_c/V_1 = \mathrm{Sin}\theta_r/V_2$ (for Two layer Case)
- $\mathrm{Sin}\theta_c = V_1/V_2$ (For Critically Refracted Wave)
- $\mathrm{Sin}\theta_{c1}/V_1 = \mathrm{Sin}\theta_{c2}/V_2 = \mathrm{Sin}\theta_{r2}/V_3$
- $\mathrm{Sin}\theta_{c1} = V_1/V_3$; $\mathrm{Sin}\theta_{C2} = V_2/V_3$ (For a wave critically refracted in the 3rd Layer)
- $\mathrm{Sin}^2\theta + \mathrm{Cos}^2\theta = 1$
- $\mathrm{Tan}(\theta) = \sin(\theta) / \cos(\theta)$


X
S
G
Direct wave
Z_1
Layer 1
V_1
A
B
Critically refracted wave
V_2
Z_2
Layer 2

$V_2 > V_1$

- At critical distance, the direct wave and the refracted (head wave) arrive at the same time.
- Thus TIME traveled by direct wave = Xc/V1.
- Distance traveled by head wave =

$$\frac{X_c}{V_2} + 2Z_1 \frac{\sqrt{V_2^{\,2} - V_1^{\,2}}}{V_1 V_2}$$



X
S
G
Direct wave
V1
Z1
Layer 1
A
D
Z2
Layer 2
V2
B
C
Layer 3
V3
Z3
Critically refracted wave

Example

Example 3: In refraction shooting, nine geophones were placed along a straight line at distance of 40, 60, 80,100,140,180,220,260 and 320 meters from the shoot point. The seismic second gave the following data.

Geophones	G_1	G_2	G_3	G_5	G_5	G_6	G_7	G_8	G_9
Distance from shot point (m)	**40**	**60**	**80**	**100**	**140**	**180**	**220**	**260**	**320**
Time of first arrival (milli seconds)	**75**	**110**	**150**	**160**	**180**	**200**	**205**	**215**	**225**

Draw the time-distance graph and determine the velocity of the shock wave and thickness of each layer.

6.2 Subsurface Investigation of Ground water

- Detailed and comprehensive study of groundwater and conditions under which it occurs can only be made by subsurface investigation.
- Test drilling furnishes information on substrata in a vertical line from the surface.
- Geophysical logging techniques provide information on physical properties of geologic formations, water quality, and well construction.

- In such methods of investigation, in-situ information about the sub-surface is obtained by test drilling, examining the geologic log, geologic time log or placing the instrument/device at the required position to collect the data.

Some of the most commonly used subsurface investigation methods are:

- Test Drilling
- Preparing and examining the geologic log
- Geologic time log
- Electric logging
- Electric resistivity logging
- Spontaneous (self) potential logging
- Radio active logging

- Natural gamma logging (or Gamma ray logs)
- Gamma-gamma logging
- Neutron logging
- Induction logging
- Sonic logging
- Fluid logging
- Temperature logging
- Flow meter & Tracer logging

The End

Thank you

Chapter 7
Water wells and Construction

Water well is a hole or shaft, in most cases vertical, excavated in the earth, or sunk in to the ground intercepting one or more water bearing strata, for bringing groundwater to the surface.

The objectives of water well

- To provide water with a good quality
- To provide a sufficient quantity of water
- To provide water for a long time
- To provide water at low cost

Classification of water wells

Wells can be classified based on

- Methods of construction as: (dug wells and tube wells),
- Their depth (shallow wells and deep wells); and
- (Vertical wells and Horizontal wells).

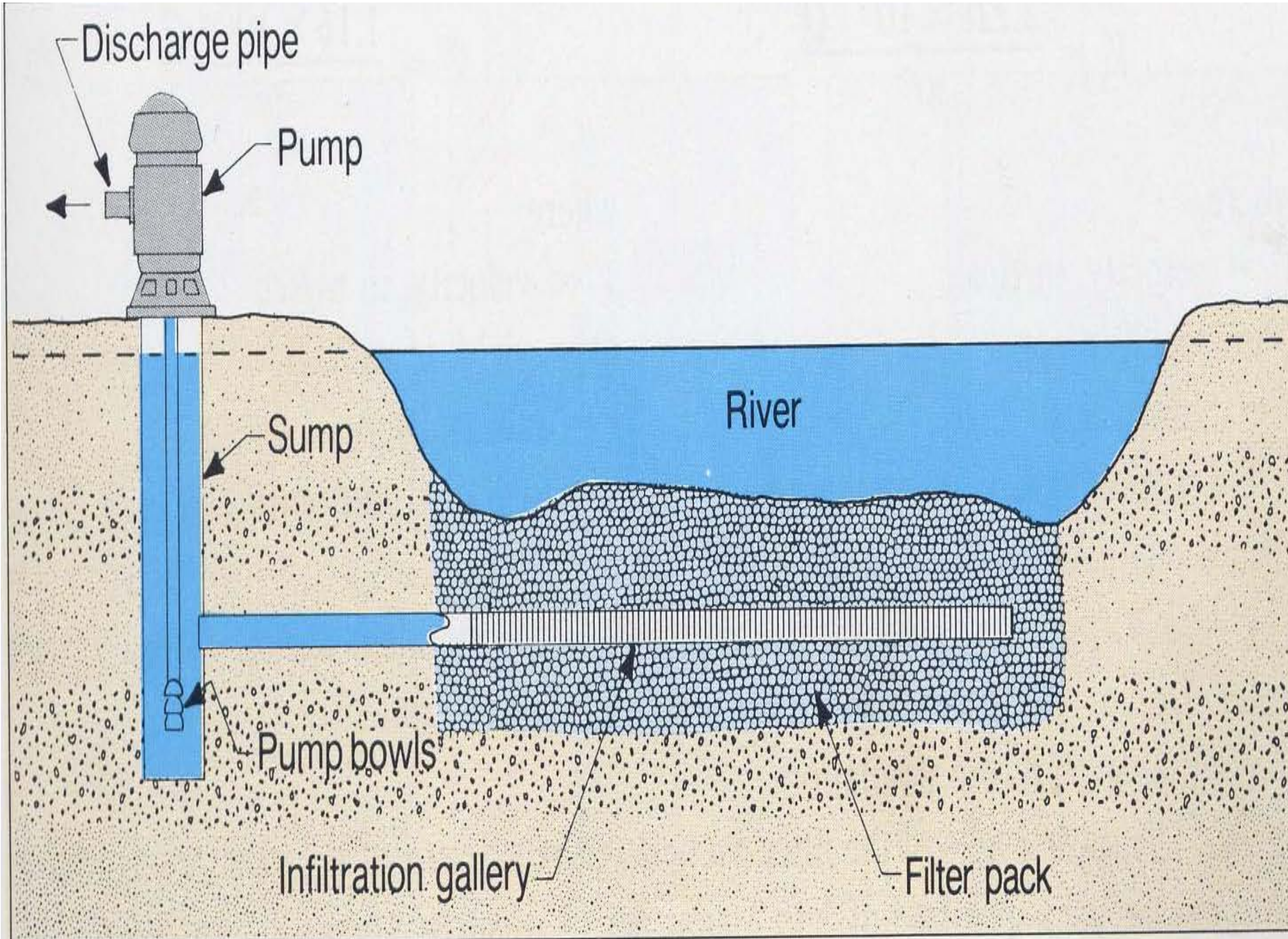


Figure 22.23. Cross section of pump placed in sump of infiltration gallery.

Advantages of Tube wells

A. Do not require much space.
B. Can be constructed quickly.
C. yield of water can be obtained even in years of scanty rainfall.
D. Economical when deep-seated aquifers are encountered
E. Generally good quality of water is tapped.

Disadvantages of Tube wells

a. Requires costly and complicated drilling equipment and machinery.
b. Requires skilled workers and great care to drill and complete the tube wells.
c. Installation of costly submersible pumps is required.
d. Possibility of missing fractures and cuttings may result drying in many holes.

Methods for Shallow Wells Construction

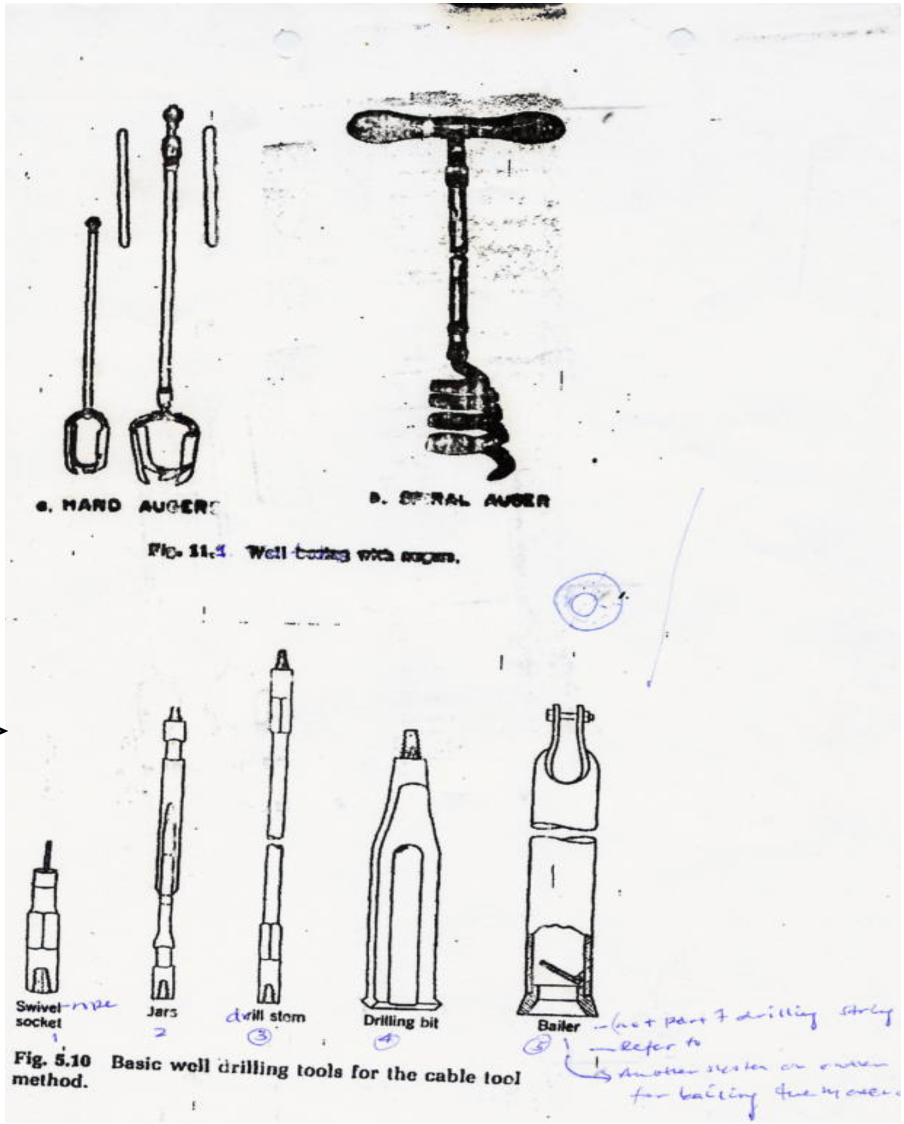
Boring Method

- In this method the hole is constructed by the use hand or power driven anger which is turned to bore the hole to the designed depth.
- Cuttings are removed by pulling and emptying the anger. It can drill to 30m or more in soft sands that are free of rocks.

Boring Method

Construction is by either Hand or Power Driven Auger.

Hand Driven Augers



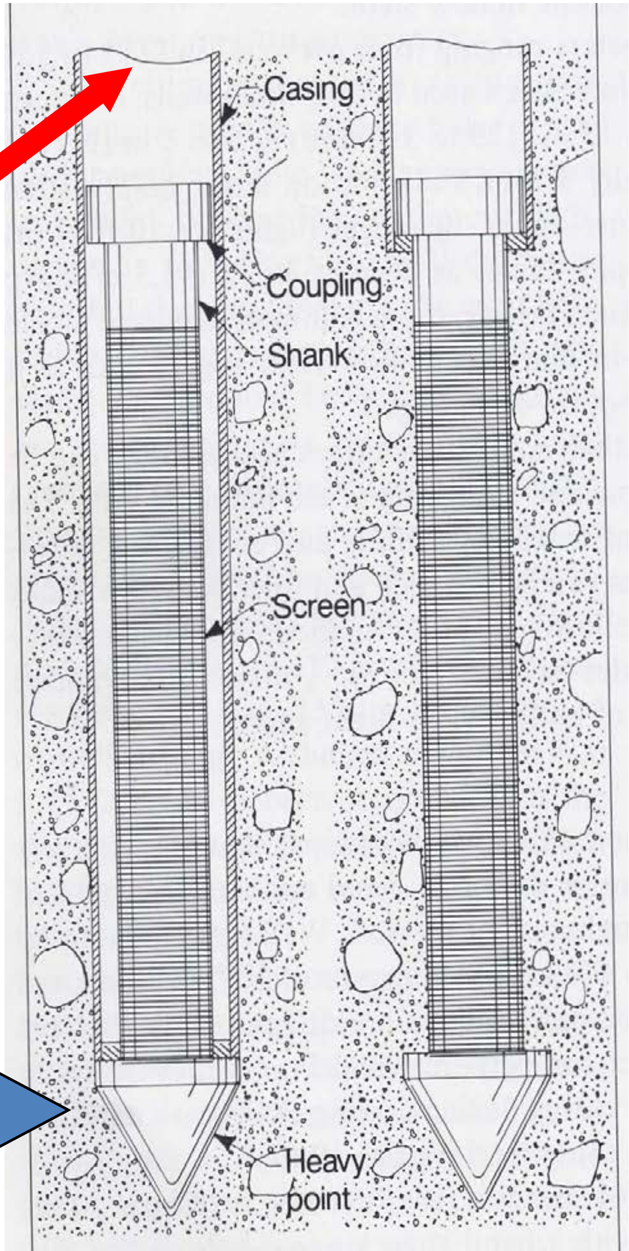
Driving Method

- In this method the hole is constructed by forcing a casing (well pipe) equipped with a drive (well) point into the ground by a series of blows either manually or machine delivered on the top of the casing
- Driven wells should be installed only in soft formations that are relatively free of cobbles or boulders(fragment of rocks)

Power Driven Bucket Type Auger

Driving Method

Hammering is applied at the top



Well Pipe or Drive

Jetting method

- A jetted well is a well which is constructed by means of boring equipment using water jetted under high pressure to facilitate rapid boring.
- Jetting is pumping water down the pipe and out through the well point where the force of the water losing the surround soil materials.

Tube Wells Drilling Methods

1. Cable – Tool Drilling Method
2. Conventional fluid Rotary or Direct Circulation Method
3. Reverse Circulation Rotary Drilling method Cable – Tool Drilling Method
4. Down the hole Hammer Drilling method

Cable – Tool Drilling Method

- The cable- tool method, also known as the standard method, is used to construct wells by alternately lifting and dropping a set of drilling tools suspended on a wire rope .
- The repeated action of the percussion drill permits bit penetration of the underground formations.

Conventional fluid Rotary Drilling Method

- In this method drilling is accomplished by rotating a drill pipe and bit by means of a power drive.
- The drill bit cuts and breaks up the rock material as it penetrates the formation.
- Drilling fluid is pumped down through the rotating drill pipe and holes in the bit.

Con,t…

- This fluid swirls in the bottom of the hole, picking up material broken by the bit, and then flows upwards in the well bore, carrying the cuttings to the surface.
- The drill pipe and bit move progressively downward, deepening the hole as the operation proceeds.

Reverse Circulation Rotary Drilling method

- In this system, the drilling fluid with cutting return inside the drill string & is discharged into a settling tank or pit. Downward flow is in the annulus between the drill string & borehole.

- The system components are similar to those of the direct rotary except for rotation.

Down the hole Hammer Drilling method

- This method is called down-the –hole-hammer drilling and is commonly used to bore crystalline rocks.
- The action is rotary percussive and does not rely on heavy down pressure.
- In hard formation the DTH hammer is most effective but becomes less so as the rock strength reduces.

Well completion

- The drilling of the borehole alone does not complete the construction of an efficient well. Well completion involves:-

A. Placement of Casing

B. Cementing of Casing

C. Placement of Well Screens

D. Gravel Packing

Well Casing

Reasons for using casing in water well:

- To prevent the collapse of the walls of the borehole (i.e. structural support against caving in) serving as a lining.
- To exclude, along with grouting, pollutants either from surface or subsurface from entering the water sources
- To provide a channel for conveying the water to the surface.

Con,t…

- To provide a channel for conveying the water into the well for injection purpose.
- To provide a housing for the pump mechanism
- To provide a channel for conveying a cement grout in the well for cementation purpose
- Serving as a reservoir for a gravel pack

Surface Casing

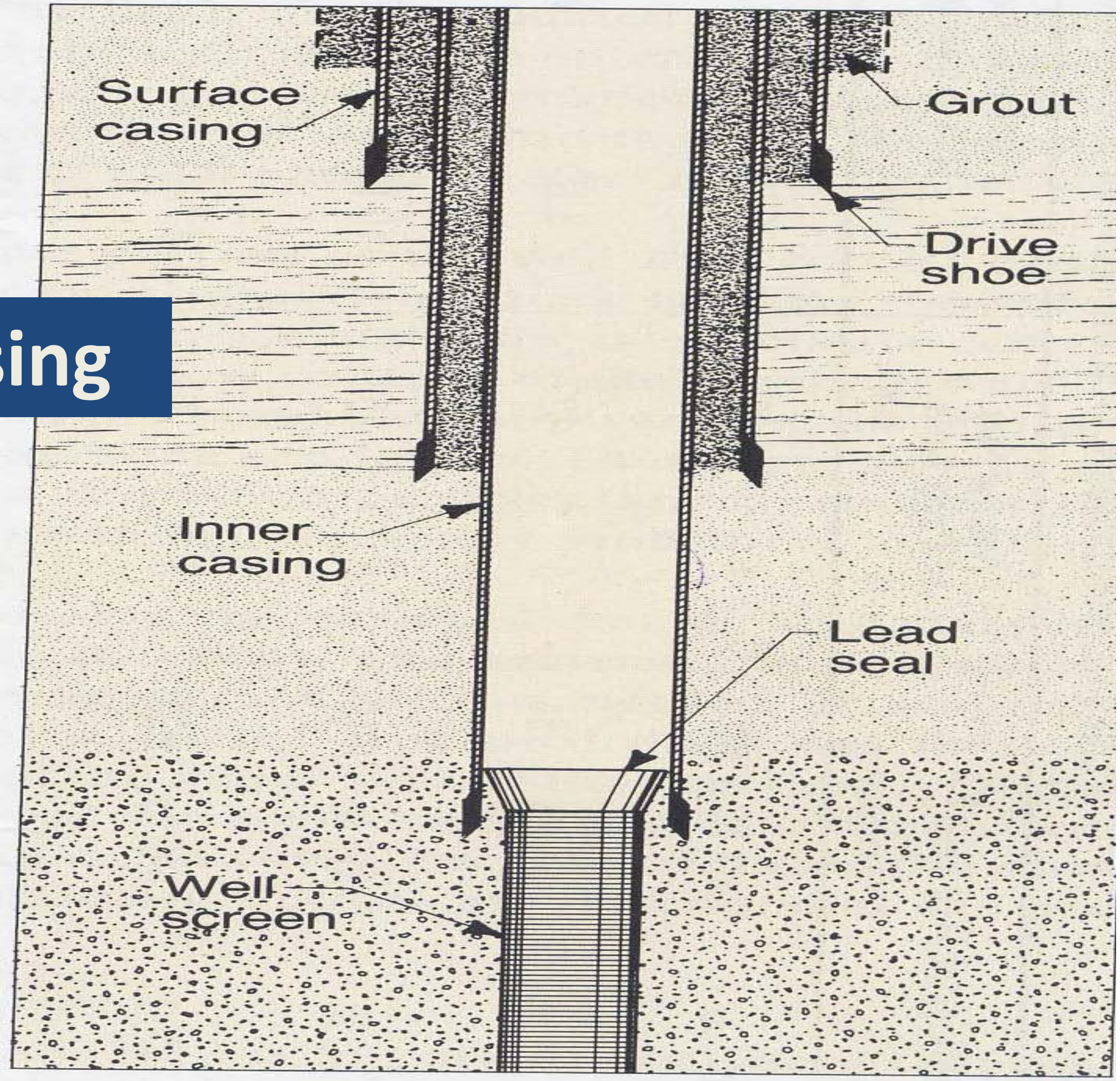


Figure 13.1. A deep well constructed by the cable tool method using successively smaller diameter casing at greater depth. In some installations, the inner and working casings are cut off, after allowing for sufficient overlap.

Well Grouting

Well grouting involves filling the space around the pipe or casing with a suitable an impervious material.

Reasons for well grouting are:

- to protect an aquifer, or aquifers, from entry of contaminating fluids flowing into it.
- to prevent undesirable water movement from one aquifer to another for the purposes of maintaining quality
- Protecting the well against the entry of unwanted water from the surface or a subsurface zone.

Con,t…

- to protect the casing against exterior corrosive and also to assure structural integrity of casing against external pressure and buckling.
- To make the casing stay tight in the drilled hole.

Con,t…

❖Prior to grouting the annular space should be flushed to assure that the space is open and ready to receive the sealing material. Grouting should be done in one continuous operation in which the annular space is filled.

Materials mostly used in well grouting are:-

- concrete
- Sand cement
- Neat cement
- Bentonite clay

Water well Screens

- A water well screen is usually a pipe with slots or openings along its wall. Wells that obtain water from sand and gravel formations require the use of well screens for proper completion.
- Well screen serves as the intake component of a well and support and stabilizes the aquifer and filter zone .

Con,t…

- Well screen permits water to enter the well from the saturated aquifer, allows a maximum amount of water to enter the well with a minimum hydraulic resistance.
- Prevents sand movement into the well
- Stabilizes the sides of the hole
- Holes in hard rock formations with large fissures would not generally require screening.

Basic Requirements of A Well Screen

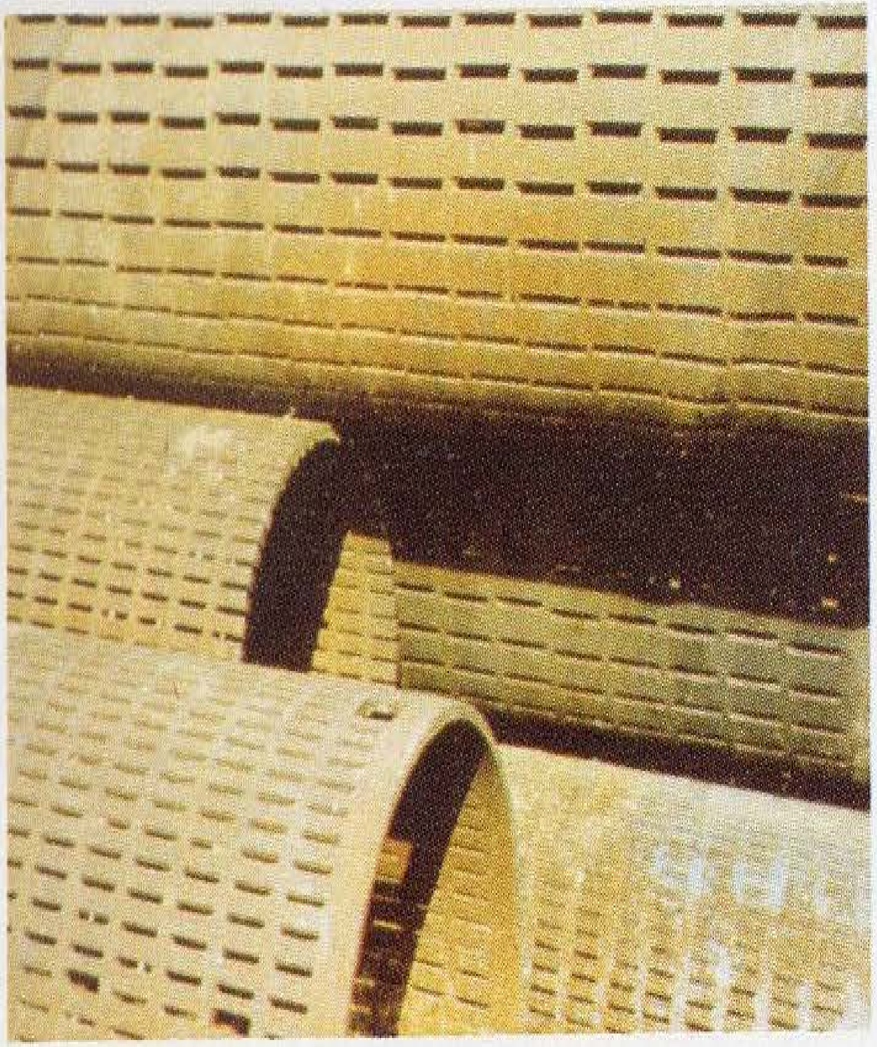
- it should be resistant to corrosion and deterioration,
- it should be strong enough to prevent collapse of a hole,
- it should offer minimum resistance to the flow of water, and

size and shape of each slot

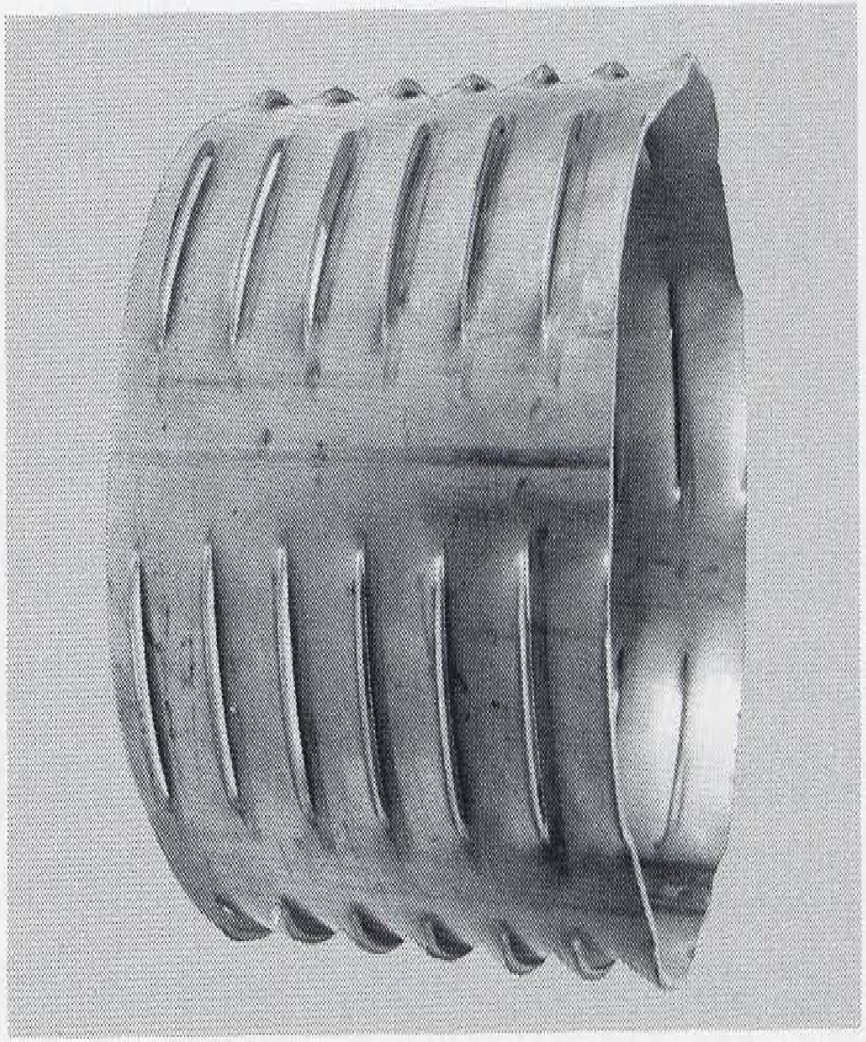
Types of well screens

1. Perforated pipe
2. Punched and slotted pipe
3. Reinforced wire wrapped punch pipe
4. Louvered pipe
5. Continuous slot wire wound screen

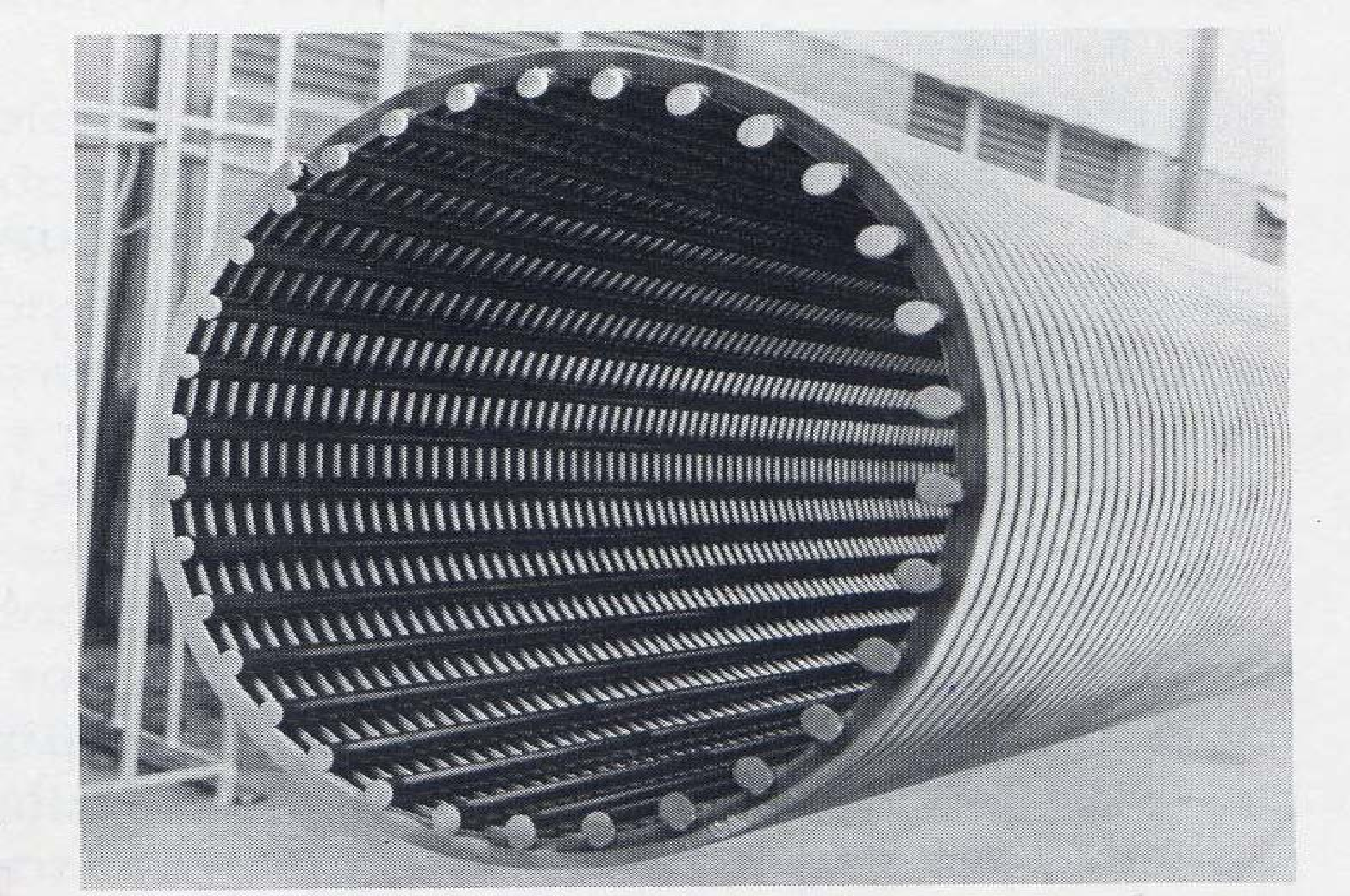
Punched



Louvered



Continuous Slot Wire Wound Screen



Artificial Gravel Pack

- A gravel pack or filter pack consists of clean sand or gravel of selected grain size and gradation which is installed in the annular space between the screen and the wall of the well bore.
- The pack has a larger average grain size and usually a smaller coefficient of uniformity than the aquifer material.

Con,t…

The Importance of Gravel pack

- It prevents or minimizes greatly the flow of sand from the aquifer into the screen thus improving the quality of water, and reducing the wear and tear on pumps.
- It permits use of a larger screen slot size and consequently larger open area so that entrance velocity is lowered and head losses to the well are reduced.
- It increases the effective diameter of the well to reduce the possibility of excess sand production.
- It fills the space between the borehole wall and lining pipe, and thus prevents formation slumping
- To stabilize fine grained, and to avoid sand pumping.

Water Well Development

A tube well is not completely ready for use just after construction. The tube well can function successfully only after proper development.

Water well development is a process whereby the mud cake or compacted borehole wall, resulting from drilling activity, is broken down and removed by pumping.

Con,t…

- Tube wells are developed to increases their specific capacity, prevent sanding and obtain maximum economic well life.
- Development work is necessary step in completing all types of wells.
- Most wells will not perform at maximum efficiency if they are not properly developed

The main objectives of well development

- To remove mud or clay particles which may have blocked the water movement from the aquifer into the well.
- To increase the porosity and improve the permeability of the water bearing formation in the vicinity of the well.
- to stabilize the sand formation (gravel pack) around a screened well and the formation.
- To reduce drawdown in the well during production or pumping

Methods of well development

The methods commonly employed for well development are

1. Over Pumping,
2. Backwashing,
3. Use Of Compressed Air,
4. Hydro Fracturing, Jetting And
5. Use Of Dispersing Agents (Chemicals)

Over pumping

- Loose sand and material are removed by pumping from the well at a higher rate than the well will be pumped when put into service.
- Over pumping has the advantage that much of the fine material brought into the borehole is pumped out immediately

Backwashing

- sand and fine materials are loosened by reversing the direction of flow through the screen.
- By changing the flow respectively the loose material will be moved through the screen into the well.

Air development

- The surging action is created by lifting the water near to the surface by injecting air into the well and then shut off the air to allow the water to flow back through the well and formation.
- Pumping water with air lift can be used for cleaning a well from sand and fine material

Water Jetting

- High velocity water jetting can be used to loosen sand and fine material from the filter zone and the screen.
- Maximum development efficiency is achieved if water jetting is combined with simultaneous pumping with air lift, as the loosened material is not allowed to settle again.

Hydro fracturing

- High pressure pumps are used to overcome the pressure of overlying rock and inject fluids into newly opened fractures.
- Pressure in the production zone usually causes small, tight breaks in the rock to open up and spread radically.

Dispersing Agents

- Sometimes it is necessary to add chemical agents to disperse the clay particles in the mud cake to speed up the development process.
- Development of the well shall be continued until water pumped from the well at the maximum pumping rate is clear and free of sand.

Well Testing for performance

- Water well may be tested for either of two main purposes:
- The usual objective is to obtain information about the performance and efficiency of the well being pumped.
- to provide data for the principal factors of aquifer performance, transmisivity and storage coefficient, can be calculated.

Con,t…

In general, the data obtained from pumping test is necessary to determine:-

- capacity of the well
- Aquifer characteristics
- Well efficiency
- Pumping rates
- Pump installation depth settings
- operation and maintenance of the well
- Well design and construction equipment.

Thank you

Chapter 8
Ground water Balance and Management

Water resources management means intervention in matters concerning water.

Such matters may be the planning, design or operation of hydraulic works, but they may also be factors that are related indirectly to water (e.g. control of land use as a factor affecting ground water quality; or water pricing to influence water demand).

Objectives of Water Resources Management

- Conserve and control the water resources zone so as to prevent or minimize excessive or deficiencies in quantity or quality.
- Provide or maintain water in such places and times, and according to the many single quantity and quality requirements;
- Minimize expenditures involved in accomplishing all of the above

Functions of groundwater management

A. Regulation of water consumption

B. Augmentation of water supply

C. Aquifer restoration

A. Regulation of water consumption

- Water consumption can be regulated either directly by allocated or indirectly by a fee or tax on consumption.
- The objective of this function is to maintain the aquifer yield at a satisfactory level and to prevent the mining of the aquifer when water withdrawals through a specified period of time exceed the aquifer recharge during the same period.

B)Augmentation of water supply

Several methods are used to increase the water supply, such as

- artificial recharge,
- relocation of wells, or
- importing water

C) Aquifer restoration

- certain measures should be taken to restore the integrity of the aquifer against pollution and excessive withdrawal.
- The latter effect would deplete the groundwater levels or piezometric heads, which might require deepening the wells or increasing pump lifts.

Purpose of Ground water Management

The ideal purpose of groundwater management in a basin is

- to develop the maximum possible groundwater
- to satisfy the requirements of all users within the basin and
- to meet specific pre determined conditions, such as the level of water quality, the cases of development and operation and certain legal, social, and political constraints.

The Need for Groundwater management

A. The continuous increase in population and the recent increase in water demands due to the continuously rising standard of living created

B. Aquifers can no longer be looked on as everlasting sources of abundant water of good quality. Increases in urban wastes and expansion of industry and agriculture lead to deterioration in the quality of groundwater

All these problems and others dictated the urgent need for groundwater management.

Groundwater Balance

The intervention takes the form of modifications imposed on the various components of water balance.

Water and pollutants carried with it may enter an aquifer, or a considered portion of one, in the following ways:

1. Groundwater inflow through aquifer boundaries and leakage from overlying or underlying aquifers.
2. Natural replenishment (infiltration) from precipitation over the area.

Con,t…

3. Return flow from irrigation and septic tanks (or similar structures, including faulty water supply or sewage networks)

4. Artificial recharge.

5. Seepage from influent streams and lakes

Con,t…

Water and pollutants carried with it may leave an aquifer in the following ways

1. Groundwater outflow through boundaries and leakage out of the considered aquifer into underlying or overlying strata.
2. Pumping and drainage
3. Seepage into effluent streams and lakes
4. Spring discharge
5. Evapo transpiration

Change in Storage

The difference between total inflow and total outflow of water and of pollutants during any period is stored in the aquifer, causing a rise in water levels and in the concentration of pollutants, respectively.

Regional Groundwater Balance

We can now summaries the regional groundwater balance by the following equation

[Ground water inflow]-[Groundwater outflow]+[Natural replenishment]+[Return flow]+[Artificial recharge]+[Inflow from streams and lakes]-[spring discharge]-[Evapo-transpiration]-[Pump age and drainage]

= [Increase in volume of water stored in an aquifer]

Where all the terms are expressed as volume of water during the balance period

Groundwater Resources Management Issues

Groundwater Resources Management Issues can be grouped roughly under three different headings:

A. Groundwater **quantity** management,

B. groundwater **quality** management and

C. Groundwater-related **environmental protection.**

The End

Thank you

Chapter 9
Artificial Recharge

1. Introduction

- Artificial recharge is the process by which the groundwater is augmented at a rate exceeding natural replenishment.
- Any man-made scheme or facility with the objective of adding water to an aquifer may be considered to be an artificial recharge system
- Recharge refers to addition of water to the aquifer storage.
- It can be artificial recharge, natural recharge or induced recharge

Con,t…

It includes water from Irrigation, cesspools, septic tanks, water mains, sewers, landfills, waste-disposal facilities, canals, and reservoirs.

Natural recharge refers to the recharge of an aquifer mainly by precipitation

Natural recharge is usually produced under one or more of the following conditions

- Deep infiltration of precipitation
- Seepage from surface water (stream & lakes)
- Under flow from another basin (if hydraulically interrelated

2. Objectives of an artificial recharge

Major objectives of an artificial recharge program are:

- Conservation of water resources.
- Better use of ground water reservoirs by recharging close to points of demand.
- Elimination of evaporation loss and other undesirable effects associated with aboveground reservoirs.
- Increasing the ground water supply.

con,t…

Conditions favorable for recharge

- Aquifer boundaries,
- Surface and ground water inflow and outflow,
- Storage capacity, porosity,
- hydraulic conductivity, and
- Available water sources.

4. Methods of Artificial Recharge

Artificial recharge methods can be classified in two broad groups:

- Direct methods and
- Indirect methods

Direct methods are subdivided into

- Surface spreading techniques and
- Sub-surface techniques

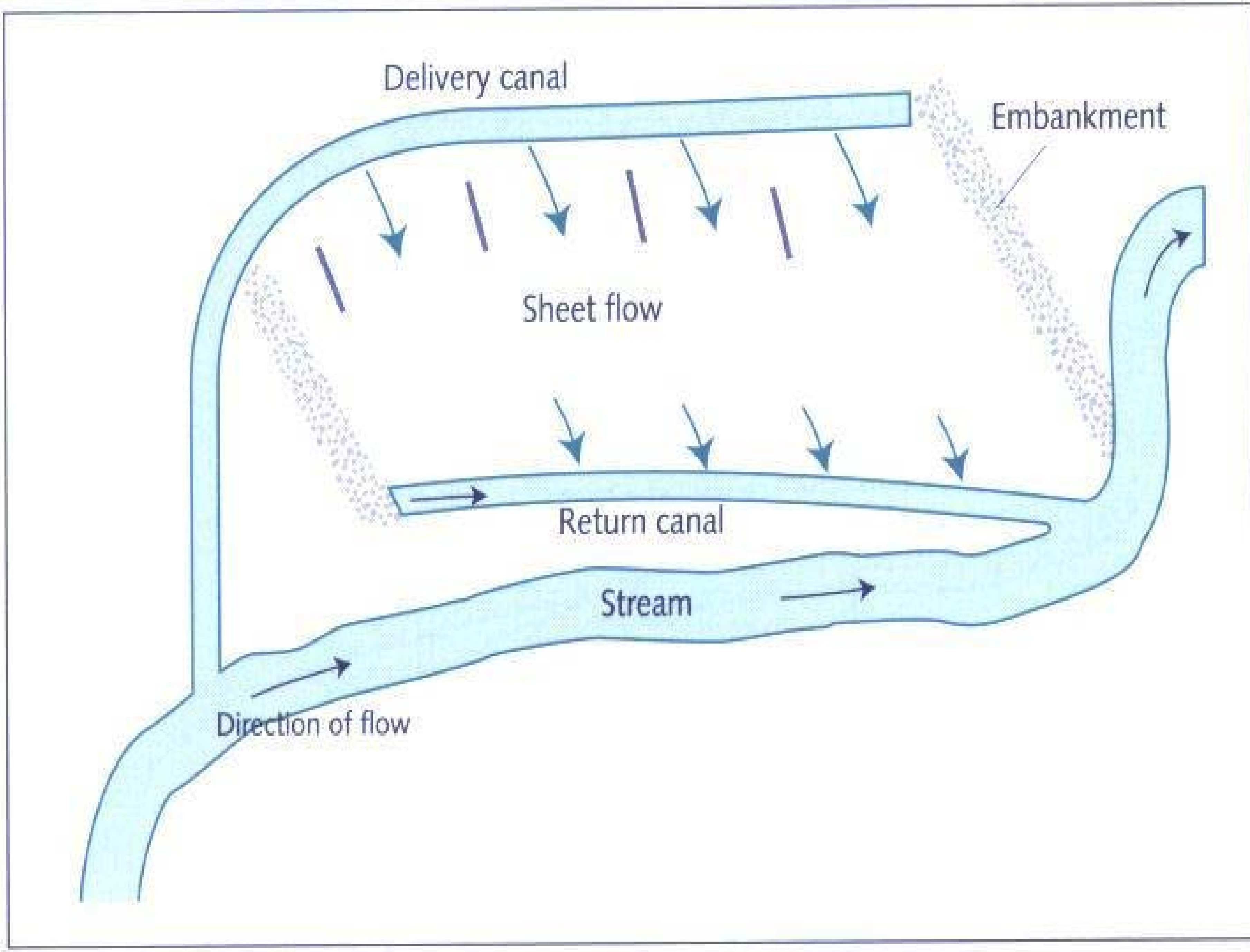
Con,t…

Surface spreading techniques

- Flooding,
- Ditch and furrow,
- Recharge basin,
- Runoff conservation structures,
- Stream channel modification and
- Surface irrigation,

1. Flooding

Flooding techniques are very useful in selected areas where the hydrogeology favors recharging the unconfined aquifer by spreading surplus surface water from canals or streams over large areas for a sufficient length of time to recharge the groundwater body.


Delivery canal
Embankment
Sheet flow
Return canal
Stream
Direction of flow

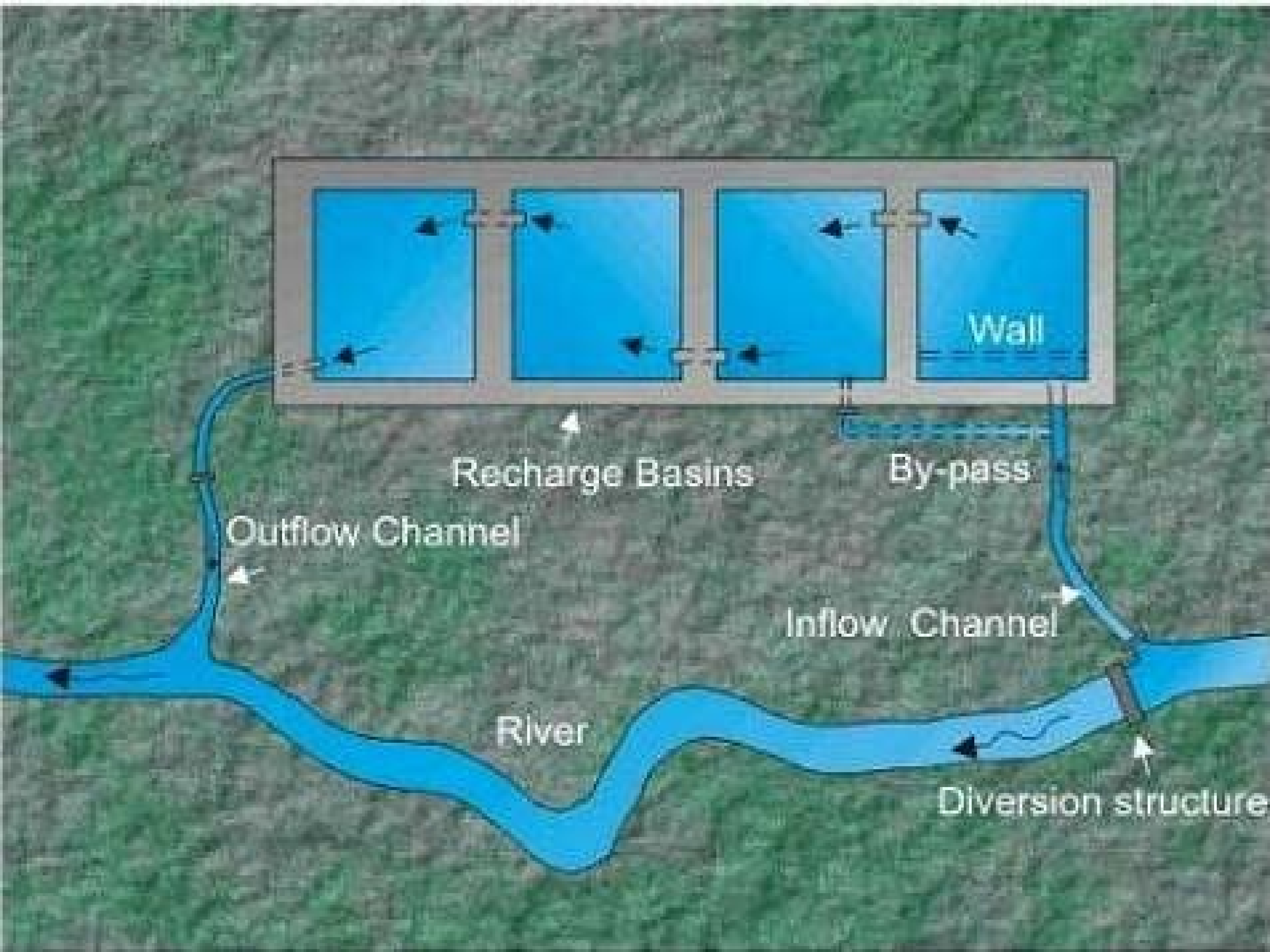
2. Ditch and furrow method

Widely used In areas with

- Irregular topography,
- Shallow, flat-bottomed and closely spaced ditches or furrows provide maximum water contact area for recharge water from the source stream or canal.
- This technique requires less soil preparation than recharge basins and is less sensitive to silting.

3. Recharge basin

- Artificial recharge basins are either excavated or are enclosed by dykes.
- They are commonly built paralle1 to ephemeral or intermittent stream channels.
- They can also be constructed at other locations where a canal or any other water source provides the water.
- In alluvia areas multiple recharge basins are generally constructed parallel to the stream.



Wall
Recharge Basins
By-pass
Outflow Channel
Inflow Channel
River
Diversion structure

4. Run off conservation structures

- Rainfall is a major source of water but it is not evenly distributed throughout the year.
- During the monsoon period, surplus water is wasted in the form of surface run-off.
- Water resources planning should address this phenomenon by making efforts to harvest rainwater, especially during rainy seasons.
- The main aim of the rainwater harvesting is to conserve the generated surface run-off by collecting it in reservoirs, both surface and sub-surface.

Con,t…

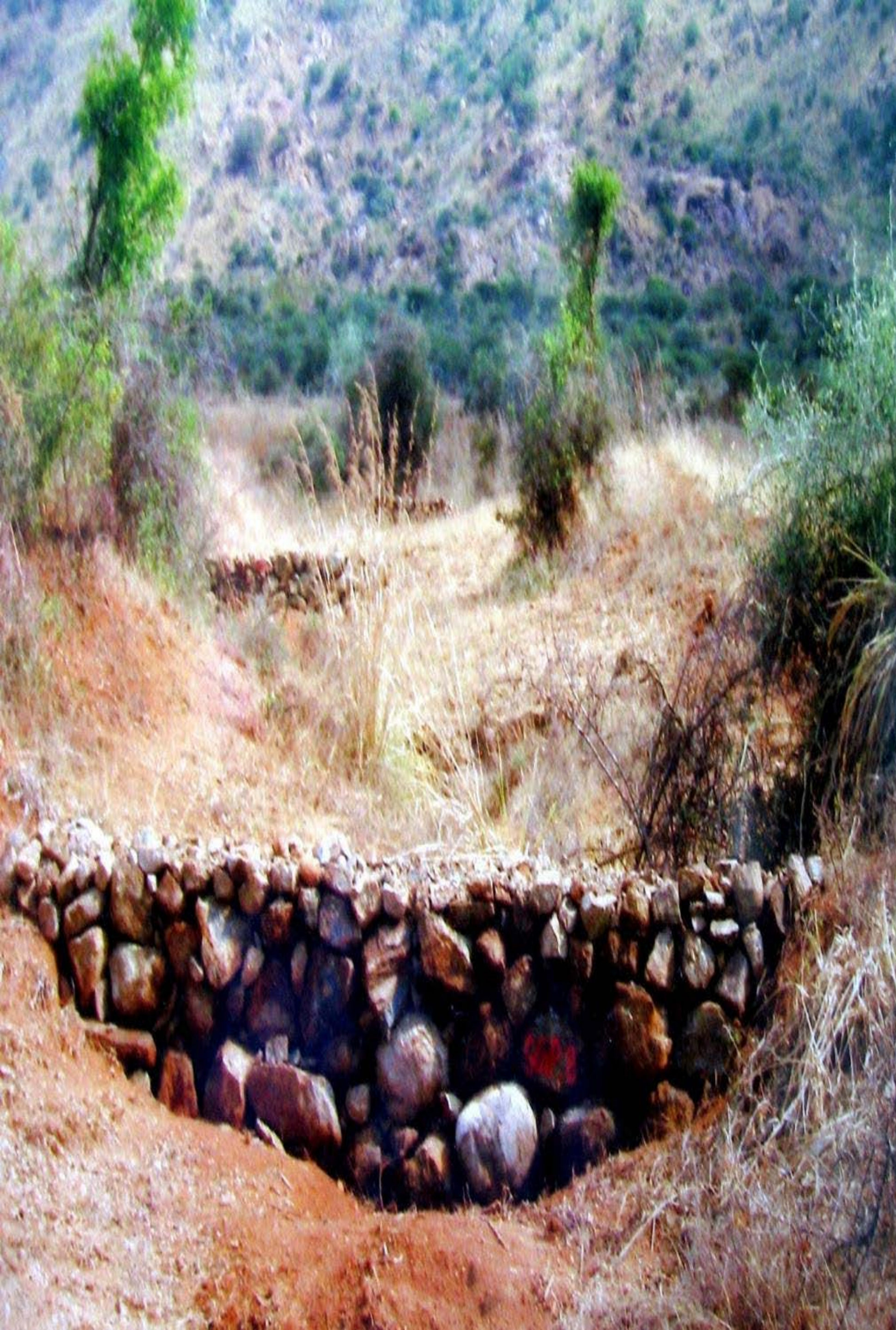
- Multi-purpose measures are desirable, that is mutually complimentary and conducive to soil and water conservation, afforestation and increased agricultural productivity.

The structures widely used are

- Gully plug,
- Bench terracing,
- Contour bunding,
- Recharge trench,
- Masonry check dams,
- Small weirs,
- Percolation tank.

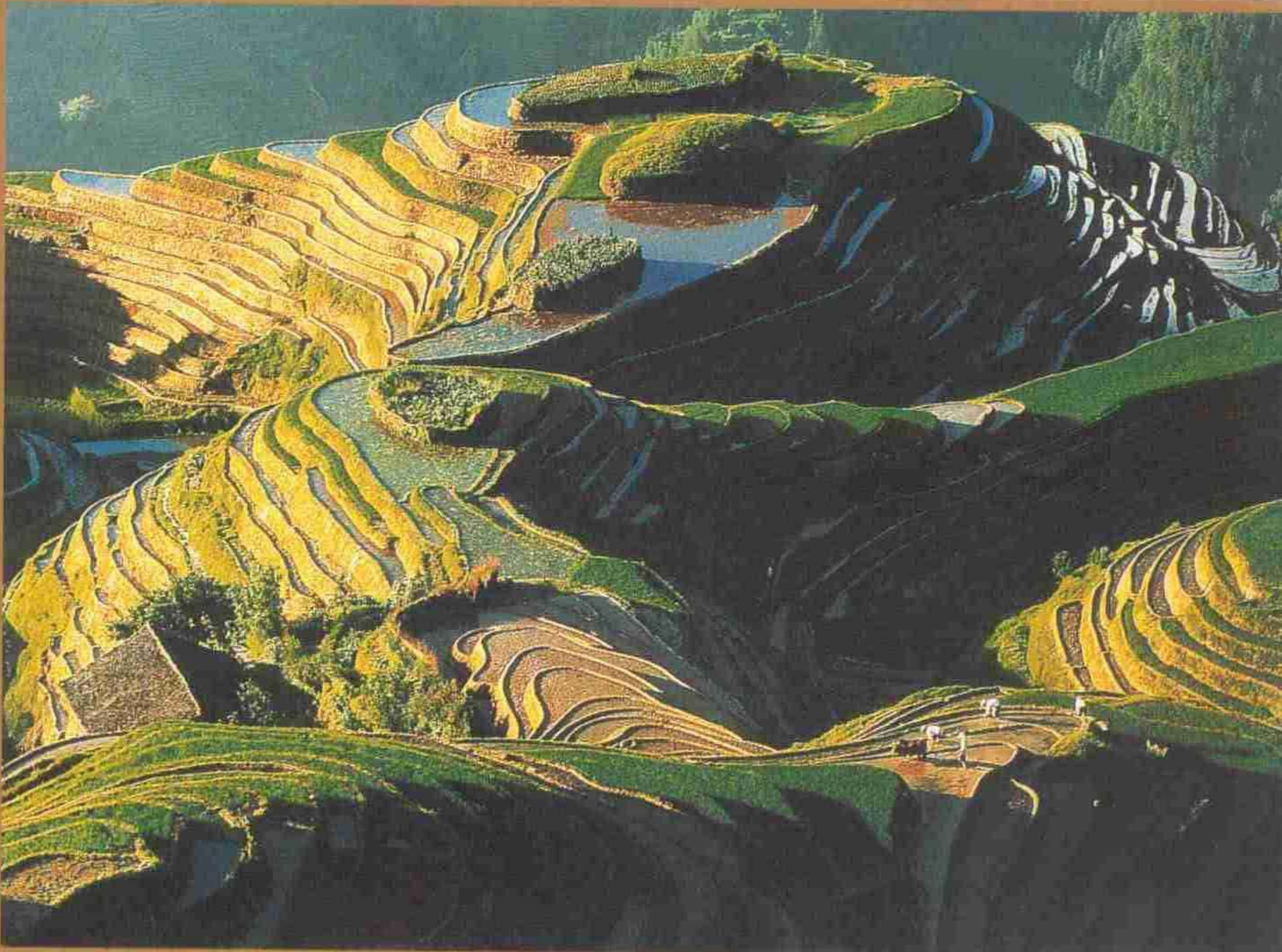
1. Gully plug

- Gully plugs are the smallest run-off conservation structures built across small gullies and streams rushing down the hill slopes.
- they carry run-off from tiny catchments during rainy seasons.
- Usually the bund is constructed by using local stones.




2. Bench Terracing

- Sloping lands with surface gradients up to 8% and with adequate soil cover can be leveled by bench terracing to bring them under cultivation.
- Bench terracing helps in soil conservation and in holding run-off water on the terraced areas for longer,
- resulting in increased infiltration recharge.



3. Contour bunding

- Contour bounding is a watershed management practice to build up soil moisture storage and recharge the groundwater.
- This technique is adopted generally in low rainfall areas.

Land bunding / soil bunding

4. Recharge trench

- Recharge trenches are constructed if permeable strata of adequate thickness are available at shallow depth.
- The shallow trench is filled with pebbles and boulders.
- Trenches are constructed across the land slope.

5. Small weirs

- A series of small weirs are made across selected stream or irrigation canal sections. So that the stream flow is hindered and water is retained on pervious soil rock surfaces for longer.
- As compared to gully plugs, weirs are constructed across bigger irrigation canals or second order streams with gentler slopes.
- The reservoir behind the weir acts like a mini percolation tank.

6. Percolation tanks

- Percolation tanks are artificially created surface water bodies.
- submerging a highly permeable land area so that surface run-off is made to percolate and recharge the groundwater storage.

7. Stream channel modification

- A natural drainage channel can be modified so as to increase infiltration by detaining stream flow and increasing the stream bed area in contact with water.
- The channel is so modified that the flow gets spread over a wider area, increasing Contact with the percolating river bed.

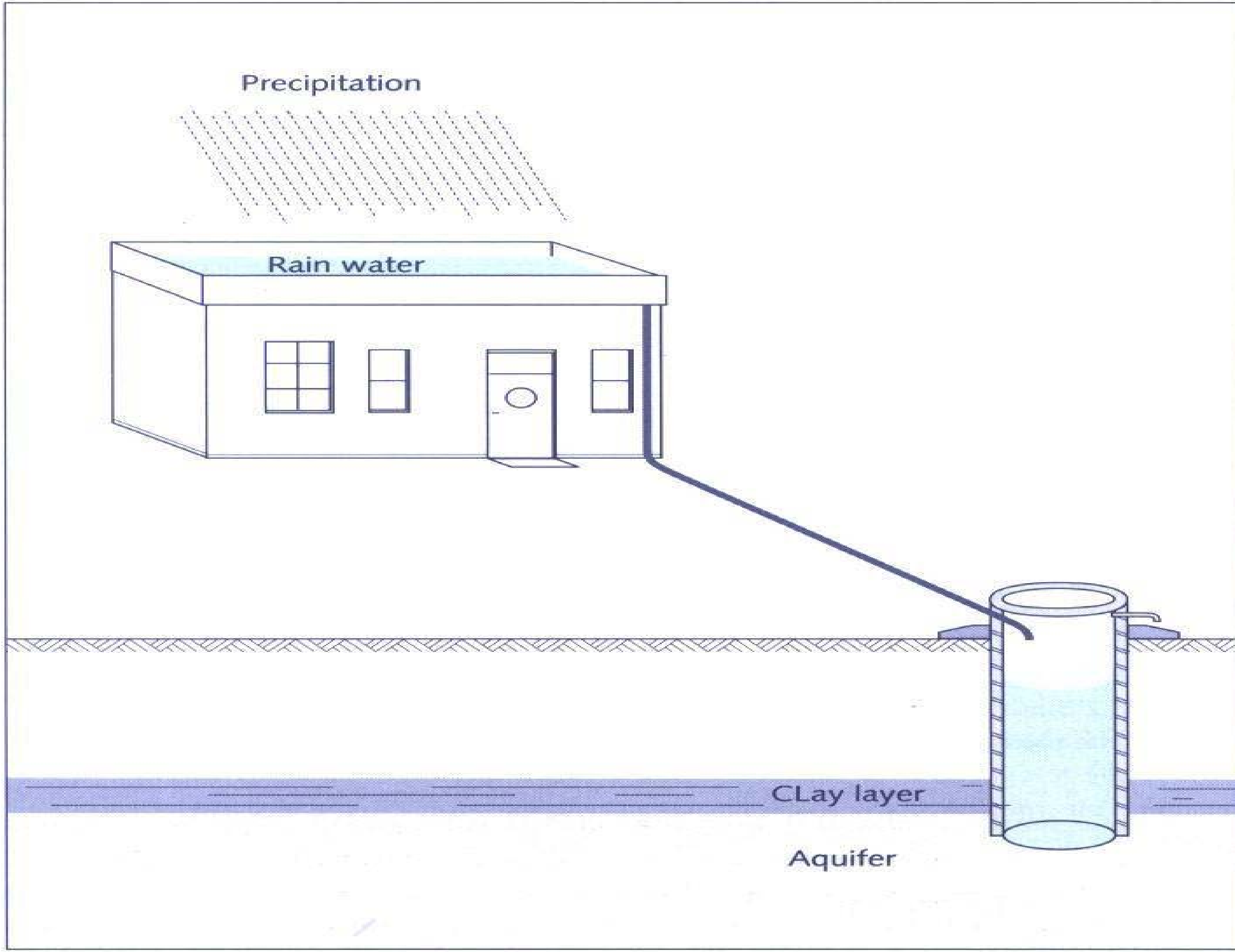
8. Surface irrigation

- In irrigation practices the farmers tend to use excessive amounts of water by flooding the fields whenever water is available.
- The large number of unlined irrigation canals also contributes significantly to the groundwater recharge.
- Irrigation of land with poor drainage facilities may lead to the development of water logging and Stalinization of large areas.

Sub surface techniques

1. Abandoned dug well

- A dry/unused dug well can be used as a recharge structure.
- The recharge water is guided through a pipe to the bottom of the well or at least below the standing water level, to avoid scouring the bottom and trapping air bubbles in the aquifer.


Precipitation
Rain water
CLay layer
Aquifer

2. Injection wells

- Injection wells are structures similar to tube well but with the purpose of augmenting the ground water storage of a confined aquifer by pumping in treated surface water under pressure.

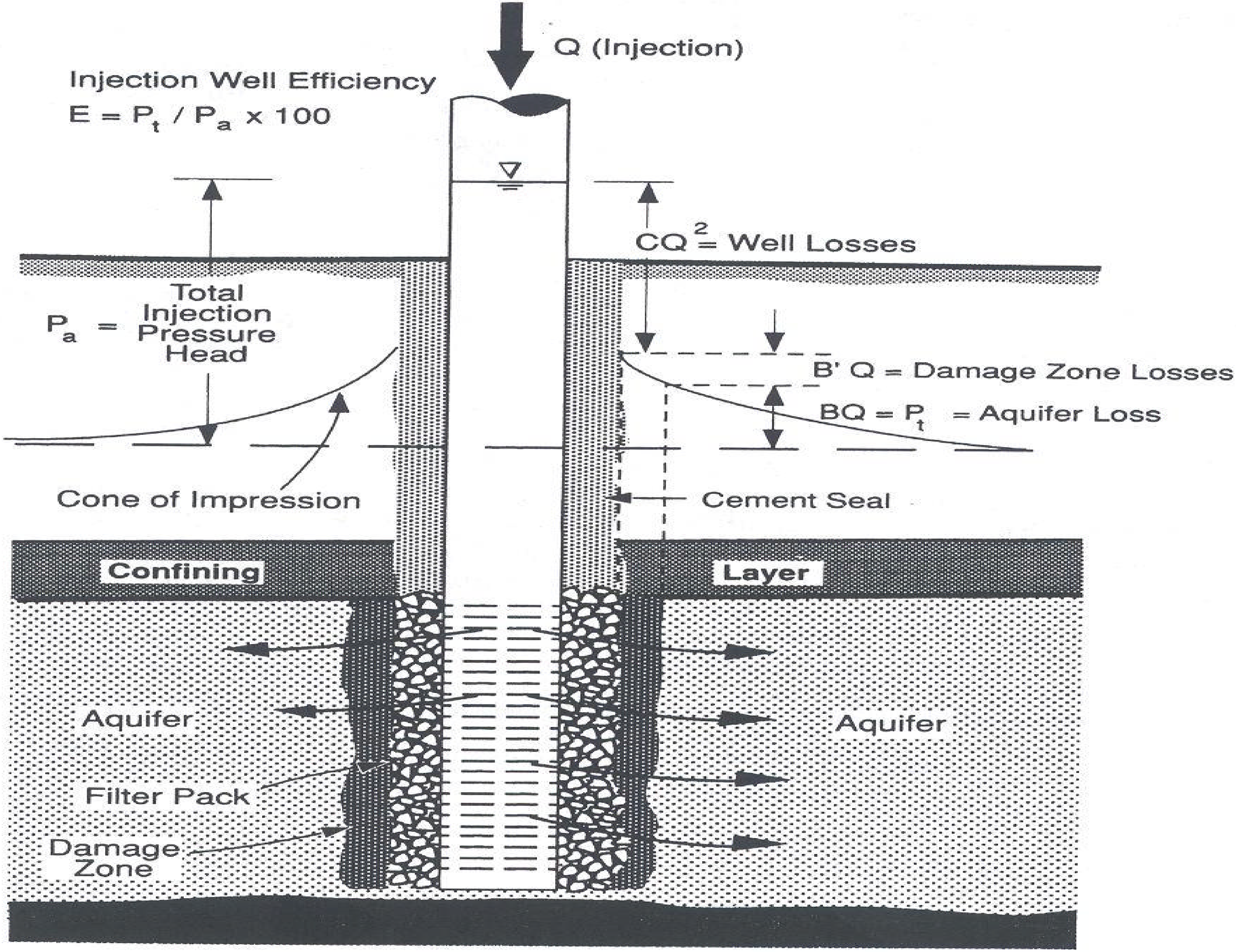
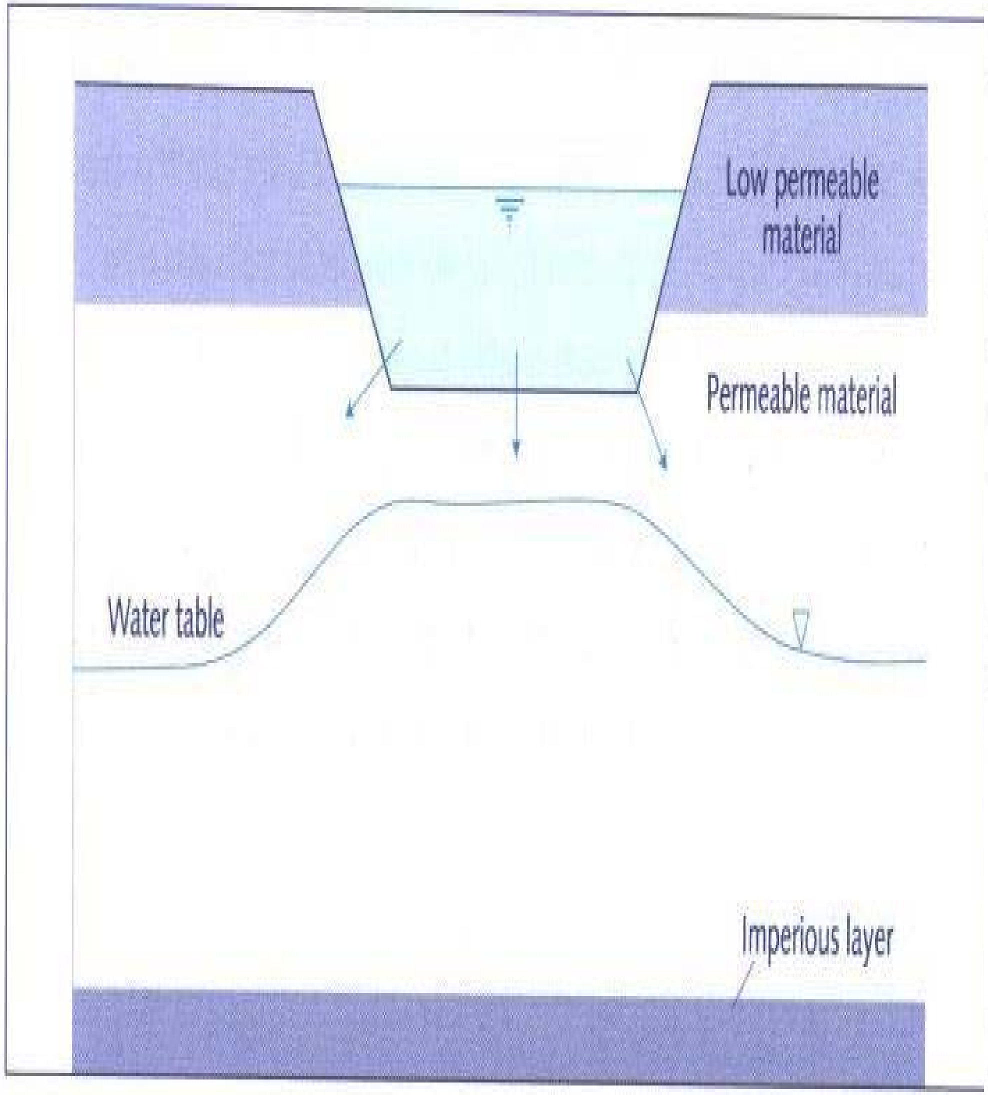


Figure 19.7. Head-Loss components in an injection well.

3. Recharge pits

- Recharge pits are excavations of variable dimensions that are sufficiently deep to penetrate less permeable strata.
- Recharge pits differ from gravity head recharge wells as the latter do not necessarily reach the unconfined aquifer and the recharging water has to infiltrate through the vadose zone.



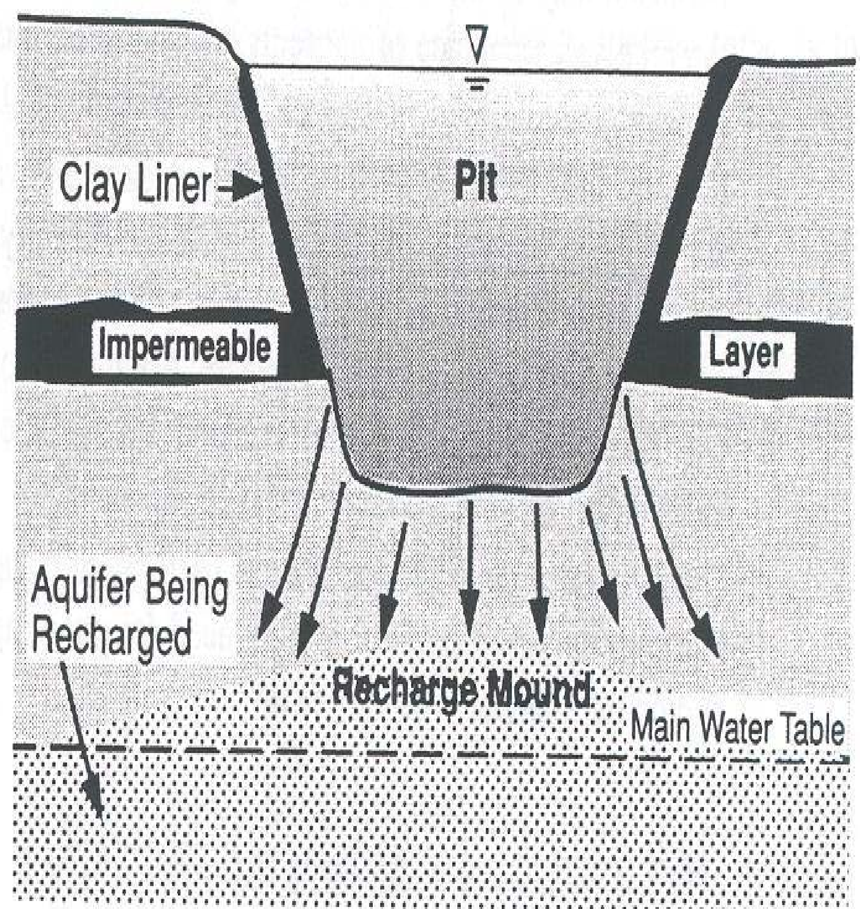
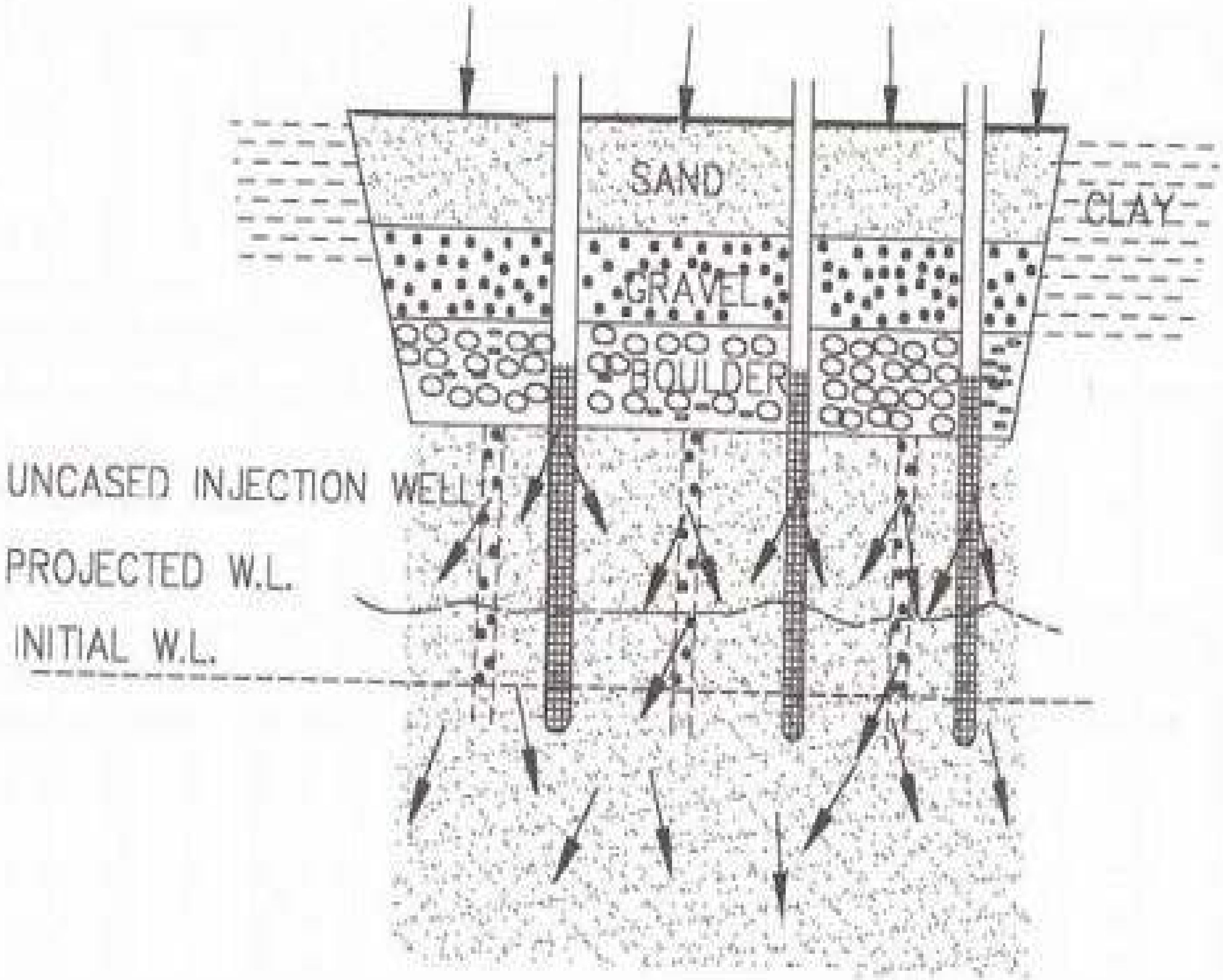


Figure 19.4. Use of a pit to recharge an aquifer overlain by an impermeable layer.

4. Recharge shaft

- In cases where an aquifer is located deep below the ground surface and overlain by poorly permeable strata, a shaft is used for artificial recharge.
- A recharge shaft is similar to a recharge pit but much smaller in cross section
- A recharge shaft may be dug manually if the stratum is non-caving.


SAND
CLAY
GRAVEL
BOULDER
UNCASED INJECTION WELL
PROJECTED W.L.
INITIAL W.L.

Indirect methods

Indirect methods of artificial recharge adopt the technique of induced recharge by means of

- Enhanced Streambed Infiltration (Induced infiltration)
- Groundwater Conservation Structures
- Rooftop Rainwater harvesting

THE END
10 Q