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Chapter
9

GROUNDWATER

9.1 INTRODUCTION

In the previous chapters various aspects of surface water hydrology that deal with surface runoff have been discussed. Study of subsurface flow is equally important since about 30% of the world's fresh water resources exist in the form of groundwater. Further, the subsurface water forms a critical input for the sustenance of life and vegetation in arid zones. Due to its importance as a significant source of water supply, various aspects of groundwater dealing with the exploration, development and utilization have been extensively studied by workers from different disciplines, such as geology, geophysics, geochemistry, agricultural engineering, fluid mechanics and civil engineering and excellent treatises are available, (Ref. 1, 2 and 4 through 10). This chapter confines itself to only an elementary treatment of the subject of groundwater as a part of engineering hydrology.

9.2 FORMS OF SUBSURFACE WATER

Water in the soil mantle is called *subsurface water* and is considered in two zones (Fig. 9.1):

1. Saturated zone, and
2. Aeration zone.

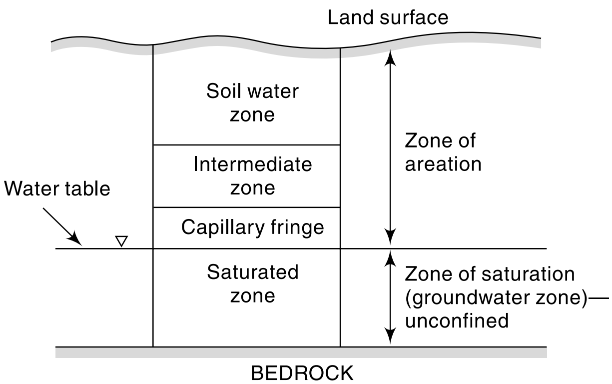

Fig. 9.1 Classification of Subsurface Water

SATURATED ZONE

This zone, also known as *groundwater zone,* is the space in which all the pores of the soil are filled with water. The water table forms its upper limit and marks a free surface, i.e. a surface having atmospheric pressure.

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. The zone of aeration has three subzones.

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SOIL WATER ZONE This 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.

CAPILLARY FRINGE In this the water is held by capillary action. This zone extends from the water table upwards to the limit of the capillary rise.

INTERMEDIATE ZONE This lies between the soil water zone and the capillary fringe.

The thickness of the zone of aeration and its constituent subzones depend upon the soil texture and moisture content and vary from region to region. The soil moisture in the zone of aeration is of importance in agricultural practice and irrigation engineering. The present chapter is concerned only with the saturated zone.

SATURATED FORMATION

All earth materials, from soils to rocks have pore spaces. Although these pores are completely saturated with water below the water table, from the groundwater utilization aspect only such material through which water moves easily and hence can be extracted with ease are significant. On this basis the saturated formations are classified into four categories:

1. Aquifer, 2. aquitard, 3. aquiclude, and 4. aquifuge.

AQUIFER An *aquifer* is a saturated formation of earth material which not only stores water but yields it in sufficient quantity. Thus an aquifer transmits water relatively easily due to its high permeability. Unconsolidated deposits of sand and gravel form good aquifers.

AQUITARD It is a formation through which only seepage is possible and thus the yield is insignificant compared to an aquifer. It is partly permeable. A sandy clay unit is an example of aquitard. Through an aquitard appreciable quantities of water may leak to an aquifer below it.

AQUICLUDE It 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 amounts of water due to its high porosity. Clay is an example of an aquiclude.

AQUIFUGE It is a geological formation which is neither porous nor permeable. There are no interconnected openings and hence it cannot transmit water. Massive compact rock without any fractures is an aquifuge.

The definitions of aquifer, aquitard and aquiclude as above are relative. A formation which may be considered as an aquifer at a place where water is at a premium (e.g. arid zones) may be classified as an aquitard or even aquiclude in an area where plenty of water is available.

The availability of groundwater from an aquifer at a place depends upon the rates of withdrawal and replenishment *(recharge)*. Aquifers play the roles of both a transmission conduit and a storage. Aquifers are classified as unconfined aquifers and confined aquifers on the basis of their occurrence and field situation. An *unconfined aquifer* (also known as *water table aquifer)* is one in which a free water surface, i.e. a water table exists (Fig. 9.2). Only the saturated zone of this aquifer is of importance in groundwater studies. Recharge of this aquifer takes place through infiltration of precipitation from the ground surface. A well driven into an unconfined aquifer will indicate a static water level corresponding to the water table level at that location.

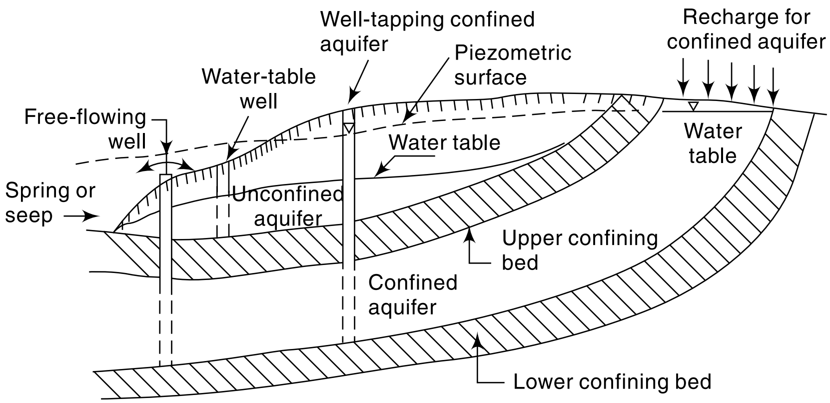

Fig. 9.2 Confined and Unconfined Acquifers

A *confined aquifer,* also known as *artesian aquifer,* is an aquifer which is confined between two impervious beds such as aquicludes or aquifuges (Fig. 9.2). Recharge of this aquifer takes place only in the area where it is exposed at the ground surface. The water in the confined aquifer will be under pressure and hence the piezometric level will be much higher than the top level of the aquifer. At some locations: the piezometric level can attain a level higher than the land surface and a well driven into the aquifer at such a location will flow freely without the aid of any pump. In fact, the term *artesian* is derived from the fact that a large number of such freeflow wells were found in Artois, a former province in north France. Instances of free-flowing wells having as much as a 50-m head at the ground surface are reported.

A confined aquifer is called a *leaky aquifer* if either or both of its confining beds are aquitards.

Water Table

A water table is the free water surface in an unconfined aquifer. The static level of a well penetrating an unconfmed aquifer indicates the level of the water table at that point. The water table is constantly in motion adjusting its surface to achieve a balance between the recharge and outflow from the subsurface storage. Fluctuations in the water level in a dug well during various seasons of the year, lowering of the groundwater table in a region due to heavy pumping of the wells and the rise in the water table of an irrigated area with poor drainage, are some common examples of the fluctuation of the water table. In a general sense, the water table follows the topographic features of the surface. If the water table intersects the land surface the groundwater comes out to the surface in the form of *springs* or *seepage.*

Sometimes a lens or localised patch of impervious stratum can occur inside an unconfined aquifer in such a way that it retains a water table above the general water table (Fig. 9.3). Such a water table retained around the impervious material is known as *perched water table.* Usually the perched water table is of limited extent and the yield from such a situation is very small. In groundwater exploration a perched water table is quite often confused with a general water table.

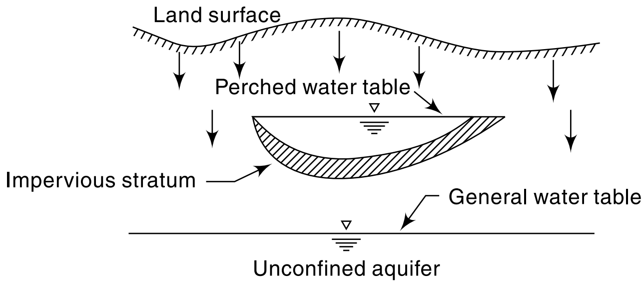

Fig. 9.3 Perched Water Table

The position of the water table relative to the water level in a stream determines whether the stream contributes water to the groundwater storage or the other way about. If the bed of the stream is below the groundwater table, during periods of low flows in the stream, the water surface may go down below the general water table elevation and the groundwater contributes to the flow in the stream. Such streams which receive groundwater flow are called *effluent streams* (Fig. 9.4 (a)). Perennial rivers and streams are of this kind. If, however, the water table is below the bed of the stream, the stream-water percolates to the groundwater storage and a hump is formed in the groundwater table (Fig. 9.4 (b)). Such streams which contribute to the groundwater are known as *influent streams*. Intermittent rivers and streams which go dry during long periods of dry spell (i.e. no rain periods) are of this kind.

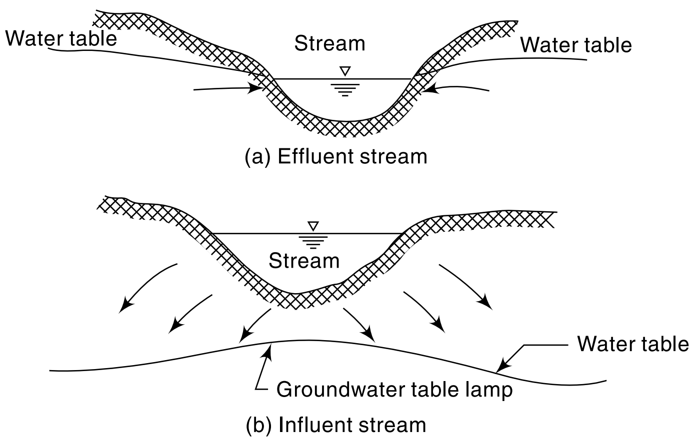

Fig. 9.4 Effluent and Influent Streams

9.3 AQUIFER PROPERTIES

The important properties of an aquifer are its capacity to release the water held in its pores and its ability to transmit the flow easily. These properties essentially depend upon the composition of the aquifer.

Porosity

The amount of pore space per unit volume of the aquifer material is called *porosity*. It is expressed as

$$n = \frac{V_v}{V_0} \tag{9.1}$$

where n = porosity, V_v = volume of voids and V_0 = volume of the porous medium. In an unconsolidated material the size distribution, packing and shape of particles determine the porosity. In hard rocks the porosity is dependent on the extent, spacing and the pattern of fracturing or on the nature of solution channels. In qualitative terms porosity greater than 20% is considered as large, between 5 and 20% as medium and less than 5% as small.

SPECIFIC YIELD While porosity gives a measure of the water-storage capability of a formation, not all the water held in the pores is available for extraction by pumping or draining by gravity. The pores hold back some water by molecular attraction and surface tension. The actual volume of water that can be extracted by the force of gravity from a unit volume of aquifer material is known as the *specific yield*, S_y. The fraction of water held back in the aquifer is known as *specific retention,* S_r. Thus porosity

$$n = S_y + S_r \tag{9.2}$$

The representative values of porosity and specific yield of some common earth materials are given in Table 9.1.

Table 9.1 Porosity and Specific Yield of Selected Formations

Formation	Porosity, %	Specific yield, %
Clay	45–55	1–10
Sand	35–40	10–30
Gravel	30–40	15–30
Sand stone	10–20	5–15
Shale	1–10	0.5–5
Lime stone	1–10	0.5–5

It is seen from Table 9.1 that although both clay and sand have high porosity the specific yield of clay is very small compared to that of sand.

DARCY'S LAW

In 1856 Henry Darcy, a French hydraulic engineer, on the basis of his experimental findings proposed a law relating the velocity of flow in a porous medium. This law, known as *Darcy's law,* can be expressed as

$$V = Ki \tag{9.3}$$

where V = Apparent velocity of seepage = Q/A in which Q = discharge and A = cross-sectional area of the porous medium. V is sometimes also known as discharge velocity. $i = -\frac{dh}{ds}$ = hydraulic gradient, in which h = piezometric head and s = distance measured in the general flow direction; the negative sign emphasizes that the piezometric head drops in the direction of flow. K = a coefficient, called *coefficient of permeability,* having the units of velocity.

The discharge Q can be expressed as

$$Q = K\,i\,A \tag{9.3a}$$
$$= K\,A\left(-\frac{\Delta H}{\Delta s}\right)$$

where ($-\Delta H$) is the drop in the hydraulic grade line in a length Δs of the porous medium.

Darcy's law is a particular case of the general viscous fluid flow. It has been shown valid for laminar flows only. For practical purposes, the limit of the validity of Darcy's law can be taken as Reynolds number of value unity, i.e.

$$\mathbf{Re} = \frac{V d_a}{\nu} = 1 \tag{9.4}$$

where **Re** = Reynolds number

d_a = representative particle size, usually $d_a = d_{10}$ where d_{10} represents a size such that 10% of the aquifer material is of smaller size.

ν = kinematic viscosity of water

Except for flow in fissures and caverns, to a large extent groundwater flow in nature obeys Darcy's law. Further, there is no known lower limit for the applicability of Darcy's law.

It may be noted that the *apparent velocity* V used in Darcy's law is not the actual velocity of flow through the pores. Owing to irregular pore geometry the actual velocity of flow varies from point to point and the *bulk pore velocity* (v_a) which represents the actual speed of travel of water in the porous media is expressed as

$$v_a = \frac{V}{n} \tag{9.5}$$

where n = porosity. The bulk pore velocity v_a is the velocity that is obtained by tracking a tracer added to the groundwater.

Coefficient of Permeability

The coefficient of permeability, also designated as *hydraulic conductivity* reflects the combined effects of the porous medium and fluid properties. From an analogy of laminar flow through a conduit (*Hagen-Poiseuille flow*) the coefficient of permeability K can be expressed as

$$K = C d_m^2 \frac{\gamma}{\mu} \tag{9.6}$$

where d_m = mean particle size of the porous medium, $\gamma = \rho g$ = unit weight of fluid, ρ = density of the fluid, g = acceleration due to gravity, μ = dynamic viscosity of the fluid and C = a shape factor which depends on the porosity, packing, shape of grains and grain-size distribution of the porous medium. Thus for a given porous material

$$K \propto \frac{1}{\nu}$$

where ν = kinematic viscosity = μ/ρ = f(temperature). The laboratory or *standard value* of the coefficient of permeability (K_s) is taken as that for pure water at a standard temperature of 20° C. The value of K_t, the coefficient of permeability at any temperature t can be converted to K_s by the relation

$$K_s = K_t(\nu_t/\nu_s) \tag{9.7}$$

where ν_s and ν_t represent the kinematic viscosity values at 20° C and t°C respectively.

The coefficient of permeability is often considered in two components, one reflecting the properties of the medium only and the other incorporating the fluid properties. Thus, referring to Eq. (9.6), a term K_0 is defined as

$$K = K_0 \frac{\gamma}{\mu} = K_0 \frac{g}{\nu} \tag{9.8}$$

where $K_0 = C\,d_m^2$. The parameter K_0 is called specific or *intrinsic permeability* which is a function of the medium only. Note that K_0 has dimensions of $[L^2]$. It is expressed in units of cm^2 or m^2 or in darcys where 1 darcy = 9.87×10^{-13} m^2. Where more than one fluid is involved in porous media flow or when there is considerable temperature variation, the coefficient K_0 is useful. However, in groundwater flow problems, the temperature variations are rather small and as such the coefficient of permeability K is more convenient to use. The common units of K are m/day or cm/s. The conversion factor for these two are

1 m/day = 0.0011574 cm/s

or 1 cm/s = 864.0 m/day

Some typical values of coefficient of permeability of some porous media are given in Table 9.2.

Table 9.2 Representative Values of the Permeability Coefficient

No. Material	K (cm/s)	K_0 (darcys)
A. Granular material		
1. Clean gravel	1–100	10^3–10^5
2. Clean coarse sand	0.010–1.00	10–10^3
3. Mixed sand	0.005–0.01	5–10
4. Fine sand	0.001–0.05	1–50
5. Silty sand	$1 \times 10^{-4} - 2 \times 10^{-3}$	0.1–2
6. Silt	$1 \times 10^{-5} - 5 \times 10^{-4}$	0.01–0.5
7. Clay	$< 10^{-6}$	$< 10^{-3}$
B. Consolidated material		
1. Sandstone	$10^{-6} - 10^{-3}$	$10^{-3} - 1.0$
2. Carbonate rock with secondary porosity	$10^{-5} - 10^{-3}$	$10^{-2} - 1.0$
3. Shale	10^{-10}	10^{-7}
4. Fractured and weathered rock (aquifers)	$10^{-6} - 10^{-3}$	$10^{-3} - 1.0$

At 20° C, for water, $\nu = 0.01$ cm^2/s and substituting in Eq. (9.8)

K_0 [darcys] = 10^3 K [cm/s] at 20°C

Consider an aquifer of unit width and thickness B, (i.e. depth of a fully saturated zone). The discharge through this aquifer under a unit hydraulic gradient is

$$T = KB \tag{9.9}$$

This discharge is termed *transmissibility, T* and has the dimensions of $[L^2/T]$. Its units are m^2/s or litres per day/metre width (l pd/m). Typical values of T lie in the range 1×10^6 l pd/m to 1×10^4 l pd/m. A well with a value of $T = 1 \times 10^5$ l pd/m is considered satisfactory for irrigation purposes.

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The coefficient of permeability is determined in the laboratory by a *permeameter.* For coarse-grained soils a *constant-head permeameter* is used. In this the discharge of water percolating under a constant head difference (ΔH) through a sample of porous material of cross-sectional area A and length L is determined. The coefficient of permeability at the temperature of the experiment is found as

$$K = \frac{Q}{A}\frac{1}{(\Delta H/L)}$$

For fine grained soils, a *falling-head permeameter* is used. Details of permeameters and their use is available in any good textbook in Soil Mechanics, e.g. Ref. 8. It should be noted that laboratory samples are disturbed samples and a permeameter cannot simulate the field conditions exactly. Hence considerable care in the preparation of the samples and in conducting the tests are needed to obtain meaningful results.

Under field conditions, permeability of an aquifer is determined by conducting pumping tests in a well. One of the many tests available for this purpose consists of pumping out water from a well at a uniform rate till steady state is reached. Knowing the steady-state drawdown and the discharge-rate, transmissibility can be calculated. Information about the thickness of the saturation zone leads one to calculate the permeability. Injection of a tracer, such as a dye and finding its velocity of travel is another way of determining the permeability under field conditions.

Stratification

Sometimes the aquifers may be stratified, with different permeabilities in each strata. Two kinds of flow situations are possible in such a case.

(i) When the flow is parallel to the stratification as in Fig. 9.5(a) equivalent permeability K_e of the entire aquifer of thickness $B = \sum_1^n B_i$ is

$$K_e = \frac{\sum_1^n K_i B_i}{\sum_1^n B_i} \tag{9.10}$$

The transmissivity of the formation is

$$T = K_e \Sigma B_i = \sum_1^n K_i B_i$$

(ii) When the flow is normal to the stratification as in Fig. 9.5(b) the equivalent permeability K_e of the aquifer of length

$$L = \sum_1^n L_i \text{ is}$$

$$K_e = \frac{\sum_1^n L_i}{\sum_1^n (L_i/K_i)} \tag{9.11}$$

(Note that in this case L is the length of seepage and the thickness B of the aquifer does not come into picture in calculating the equivalent permeability)

The transmissivity of the aquifer is $T = K_e \cdot B$

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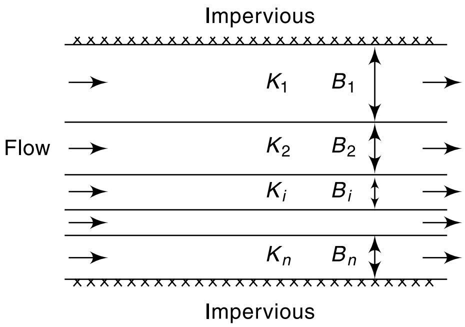

Fig. 9.5(a) Flow Parallel to Stratification

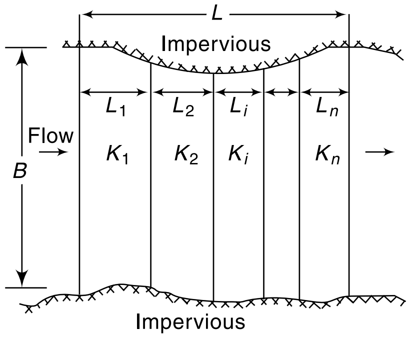

Fig. 9.5(b) Flow Normal to Stratification

EXAMPLE 9.1 *At a certain point in an unconfined aquifer of 3 km² area, the water table was at an elevation of 102.00 m. Due to natural recharge in a wet season, its level rose to 103.20 m. A volume of 1.5 Mm³ of water was then pumped out of the aquifer causing the water table to reach a level of 101.20 m. Assuming the water table in the entire aquifer to respond in a similar way, estimate (a) the specific yield of the aquifer and (b) the volume of recharge during the wet season.*

SOLUTION:

(a) Volume pumped out = area × drop in water table × specified yield S_y

$$1.5 \times 10^6 = 3 \times 10^6 \times (103.20 - 101.20) \times S_y$$

$$S_y = 0.25$$

(b) Recharge volume $= 0.25 \times (103.20 - 102.00) \times 3 \times 10^6 = 0.9$ Mm³

EXAMPLE 9.2 *A field test for permeability consists in observing the time required for a tracer to travel between two observation wells. A tracer was found to take 10 h to travel between two wells 50 m apart when the difference in the water-surface elevation in them was 0.5 m. The mean particle size of the aquifer was 2 mm and the porosity of the medium 0.3. If ν = 0.01 cm²/s estimate (a) the coefficient of permeability and intrinsic permeability of the aquifer and (b) the Reynolds number of the flow.*

SOLUTION:

(a) The tracer records the actual velocity of water

$$V_a = \frac{50 \times 100}{10 \times 60 \times 60} = 0.139 \text{ cm/s}$$

Discharge velocity $V = n\,V_a = 0.3 \times 0.139 = 0.0417$cm/s

Hydraulic gradient $i = \dfrac{0.50}{50} = 1 \times 10^{-2}$

Coefficient of permeability $K = \dfrac{4.17 \times 10^{-2}}{1 \times 10^{-2}} = 4.17$ cm/s

Intrinsic permeability, $K_0 = \dfrac{K\nu}{g} = \dfrac{4.17 \times 0.01}{981} = 4.25 \times 10^{-5}$ cm²

Since 9.87×10^{-9} cm² = 1 darcy

$$K_0 = 4307 \text{ darcys}$$

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(b) Reynolds number $\mathbf{Re} = \dfrac{Vd_a}{\nu}$

Taking d_a = mean particle size = 2 mm

$$\mathbf{Re} = \frac{0.0417 \times 2}{10} \times \frac{1}{0.01} = 0.834.$$

EXAMPLE 9.3 *Three wells A, B and C tap the same horizontal aquifer. The distances AB = 1200 m and BC = 1000 m. The well B is exactly south of well A and the well C lies to the west of well B. The following are the ground surface elevation and depth of water below the ground surface in the three wells.*

Well	Surface Elevation (metres above datum)	Depth of water table (m)
A	200.00	11.00
B	197.00	7.00
C	202.00	14.00

Determine the direction of groundwater flow in the aquifer in the area *ABC* of the wells.

SOLUTION: Let H = elevation of water table.

$$H_A = 200.00 - 11.00 = 189.00$$
$$H_B = 197.00 - 7.00 = 190.00$$
$$H_C = 202.00 - 14.00 = 188.00$$

Let *BA* = North direction, designated as *Y* direction.

The West direction will be called *X* direction.

The layout of the wells is shown in Fig. 9.6

Along *BA*: $-\Delta H_y = H_B - H_A = 190.00 - 189.00 = 1.00$ m

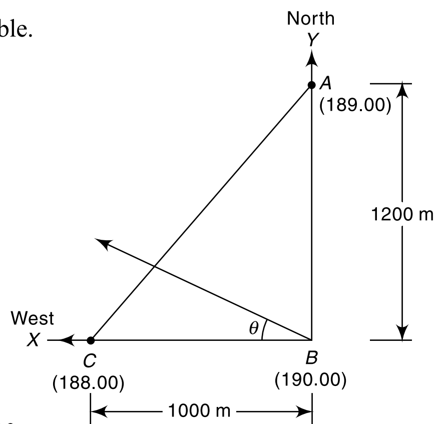

Fig. 9.6 Layout of Wells

$$i_y = -\frac{\Delta H_y}{L_{AB}} = \frac{1.00}{1200} = 1/1200.$$

$$V_y = K \cdot i_y = K/1200 \text{ m/s}$$

where K = coefficient of permeability.

Along *BC*, (*X* direction):

$$-\Delta H_x = H_B - H_C = 190.00 - 188.00 = 2.00 \text{ m}$$

$$i_x = -\frac{\Delta H_x}{L_{BC}} = \frac{2.00}{1000} = 1/500.$$

$$V_x = K \cdot i_x = K/500 \text{ m/s}$$

$$V = (V_x^2 + V_y^2)^{1/2} = \frac{K}{100}\left[\frac{1}{25} + \frac{1}{144}\right]^{1/2} = 2.167 \times 10^{-3} \text{ K m/s}$$

$$\tan\theta = \frac{V_y}{V_x} = \frac{K}{1200} \times \frac{500}{K} = \frac{1}{2.4}$$

$$\theta = 22.62° = 22° \, 37' \, 11.5''$$

where θ = inclination of V to X-axis (west direction). The groundwater flow will be in a direction which makes 22.62° with line BC and 67.38° with BA. Thus, the direction of groundwater flow is N 67° 27′ 48.5″ W.

9.4 GEOLOGIC FORMATIONS AS AQUIFERS

The identification of a geologic formation as a potential aquifer for groundwater development is a specialized job requiring the services of a trained hydrogeologist. In this section only a few general observations are made and for details the reader is referred to a standard treatise on hydrogeology such as Ref. 4.

The geologic formations of importance for possible use as an aquifer can be broadly classified as (i) unconsolidated deposits, and (ii) consolidated rocks. Unconsolidated deposits of sand and gravel form the most important aquifers. They occur as fluvial alluvial deposits, abandoned channel sediments, coastal alluvium and as lake and glacial deposits. The yield is generally good and may be of the order of 50–100 m^3/h. In India, the Gangetic alluvium and the coastal alluvium in the states of Tamil Nadu and Andhra Pradesh are examples of good aquifers of this kind.

Among consolidated rocks, those with primary porosity such as sandstones are generally good aquifers. Weathering of rocks and occurrence of secondary openings such as joints and fractures enhance the yield. Normally, the yield from these aquifers is less than that of alluvial deposits and typically may have a value of 20–50 m^3/h. Sandstones of Kathiawar and Kutch areas of Gujarat and of Lathi region of Rajasthan are some examples.

Limestones contain numerous secondary openings in the form of cavities formed by the solution action of flowing subsurface water. Often these form highly productive aquifers. In Jodhpur district of Rajasthan, cavernous limestones of the Vindhyan system are providing very valuable groundwater for use in this arid zone.

The volcanic rock basalt has permeable zones in the form of vesicles, joints and fractures. Basaltic aquifers are reported to occur in confined as well as under unconfined conditions. In the Satpura range some aquifers of this kind give yields of about 20 m^3/h.

Igneous and metamorphic rocks with considerable weathered and fractured horizons offer good potentialities as aquifers. Since weathered and fractured horizons are restricted in their thickness these aquifers have limited thickness. Also, the average permeability of these rocks decreases with depth. The yield is fairly low, being of the order of 5–10 m^3/h. Aquifers of this kind are found in the hard rock areas of Karnataka, Tamil Nadu, Andhra Pradesh and Bihar.

9.5 COMPRESSIBILITY OF AQUIFERS

In confined aquifers the total pressure at any point due to overburden is borne by the combined action of the pore pressure and intergranular pressure. The compressibility of the aquifer and also that of the pore water causes a readjustment of these pressures whenever there is a change in storage and thus have an important bearing on the storage characteristics of the aquifer. In this section a relation is developed between a defined storage coefficient and the various compressibility parameters.

Consider an elemental volume $\Delta V = (\Delta x \Delta y)\, \Delta Z = \Delta A\, \Delta Z$ of a compressible aquifer as shown in Fig. 9.7. A cartesian coordinate system with the Z-axis pointing vertically upwards is adopted. Further the following three compressible aquifer assumptions are made:

- The elemental volume is constrained in lateral directions and undergoes change of length in the z-direction only, i.e. ΔA is constant.
- The pore water is compressible
- The solid grains of the aquifer are incompressible but the pore structure is compressible.

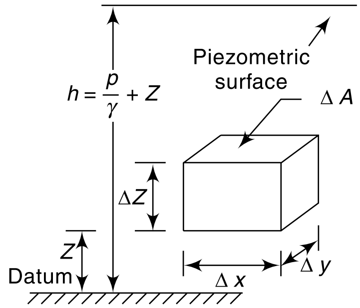

Fig. 9.7 Volume element of a compressible aquifer

By defining the reciprocal of the bulk modulus of elasticity of water as *compressibility of water* β, it is written as

$$\beta = -\frac{(d\,\Delta V_w)}{\Delta V_w}/dp \qquad (9.12)$$

where ΔV_w = volume of water in the chosen element of aquifer, and p = pressure. By conservation of mass

$$\rho \cdot \Delta V_w = \text{constant, where } \rho = \text{density of water.}$$

Thus $\rho\, d(\Delta V_w) + \Delta V_w\, d\rho = 0$

Substituting this relationship in Eq. (9.12),

$$\beta = d\rho/(\rho\, dp) \qquad (9.13)$$

or $d\rho = \rho\beta dp$ (9.13a)

Similarly by considering the reciprocal of the bulk modulus of elasticity of the pore-space skeleton as the *compressibility of the pores,* α, it is expressed as

$$\alpha = \frac{d(\Delta V)/\Delta V}{d\sigma_z} \qquad (9.14)$$

in which σ_z = intergranular pressure. Since $\Delta V = \Delta A \cdot \Delta Z$ with ΔA = constant,

$$\alpha = -\frac{d(\Delta Z)/\Delta Z}{d\sigma_z} \qquad (9.15)$$

The total overburden pressure $w = p + \sigma_z$ = constant.

Thus $dp = -d\,\sigma_z$, which when substituted in Eq. (9.15) gives

$$d\,(\Delta Z) = \alpha\,(\Delta Z)\,dp \qquad (9.16)$$

As the volume of solids ΔV_s in the elemental volume is constant,

$$\Delta V_s = (1 - n)\,\Delta A \cdot \Delta Z = \text{constant}$$

$$d(\Delta V_s) = (1 - n)\,d\,(\Delta Z) - \Delta Z \cdot dn = 0$$

where n = porosity of the aquifer. Using this relationship in Eq. (9.16),

$$dn = \alpha\,(1 - n)\,dp \qquad (9.17)$$

Now, the mass of water in the element of volume ΔV, is

$$\Delta M = \rho n\,\Delta A\,\Delta Z$$

or $$d(\Delta M) = \Delta V\left[n\,d\rho + \rho\,dn + \rho n\frac{d(\Delta Z)}{\Delta Z}\right]$$

i.e. $$\frac{d\,(\Delta M)}{\rho\,\Delta V} = n\frac{d\rho}{\rho} + dn + n\frac{d(\Delta Z)}{\Delta Z}$$

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Substituting from Eqs. (9.13), (9.17) and (9.15) for the terms in the right-hand side respectively

$$\frac{d(\Delta M)}{\rho \Delta V} = n\beta\, dp + \alpha(1-n)\, dp + n\,\alpha\, dp = (n\beta + \alpha)\, dp$$

$$= \gamma(n\beta + \alpha)\, dh = S_s dh \tag{9.18}$$

where $S_s = \gamma(n\,\beta + \alpha)$ and h = piezometric head $= z + \dfrac{\rho}{\gamma}$ and $\gamma = \rho g$ = weight of unit volume of water.

The term S_s is called *specific storage.* It has the dimensions of $[L^{-1}]$ and represents the volume of water released from storage from a unit volume of aquifer due to a unit decrease in the piezometric head. The numerical value of S_s is very small being of the order of 1×10^{-4} m^{-1}.

By integration of Eq. (9.18) for a confined aquifer of thickness B, a dimensionless *storage coefficient* S can be expressed as

$$S = \gamma(n\beta + \alpha)\, B \tag{9.19}$$

The storage coefficient S (also known as *Storativity)* represents the volume of water released by a column of a confined aquifer of unit cross-sectional area under a unit decrease in the piezometric head. The storage coefficient S and the transmissibility coefficient T are known as the *formation constants* of an aquifer and play very important role in the unsteady flow through the porous media. Typical values of S in confined aquifers lie in the range 5×10^{-5} to 5×10^{-3}. Values of α for some formation material and β for various temperatures are given in Tables 9.3 and 9.4 respectively.

Table 9.3 Range of α for Some Formation Materials

Material	Bulk modulus of elasticity, E_s (N/cm^2)	Compressibility $\alpha = 1/E_s$ (cm^2/N)
Loose clay	$10^2 - 5 \times 10^2$	$10^{-2} - 2 \times 10^{-3}$
Stiff clay	$10^3 - 10^4$	$10^{-3} - 10^{-4}$
Loose sand	$10^3 - 2 \times 10^3$	$10^{-3} - 5 \times 10^{-4}$
Dense sand	$5 \times 10^3 - 8 \times 10^3$	$2 \times 10^{-4} - 1.25 \times 10^{-4}$
Dense sandy gravel	$10^4 - 2 \times 10^4$	$10^{-4} - 5 \times 10^{-5}$
Fissured and jointed rock	$1.5 \times 10^4 - 3 \times 10^5$	$6.7 \times 10^5 - 3.3 \times 10^{-6}$

Table 9.4 Values of β for Water at Various Temperatures

Temperature (°C)	Bulk modulus of elasticity, E_w (N/cm^2)	Compressibility $\beta = 1/E_w$ (cm^2/N)
0	2.04×10^5	4.90×10^{-6}
10	2.11×10^5	4.74×10^{-6}
15	2.14×10^5	4.67×10^{-6}
20	2.20×10^5	4.55×10^{-6}
25	2.22×10^5	4.50×10^{-6}
30	2.23×10^5	4.48×10^{-6}
35	2.24×10^5	4.46×10^{-6}

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For an unconfined aquifer, the coefficient of storage is given by

$$S = S_y + \gamma(\alpha + n\beta)\, B_s \tag{9.20}$$

where B_s = saturated thickness of the aquifer. However, the second term on the right-hand side is so small relative to S_y that for practical purposes S is considered equal to S_y, i.e. the coefficient of storage is assumed to have the same value as the specific yield for unconfined aquifers.

The elasticity of the aquifer is reflected dramatically in the response of the water levels in the wells drilled in confined aquifers to changes in the atmospheric pressure. Increase in the atmospheric pressure causes an increase in the loading of the aquifer. The change in the pressure is balanced by a partial compression of the water and partial compression of the pore skeleton. An increase in the atmospheric pressure causes a decrease in the water level in the well. Converse is the case with the decrease in pressure. The ratio of the water level change to pressure head change is called *barometric efficiency* (BE) and is given in terms of the compressibility parameters as

$$BE = -\left(\frac{n\beta}{\alpha + n\beta}\right) \tag{9.21}$$

The negative sign indicates the opposite nature of the changes in pressure head and water level. Using in Eq. (9.21) (Eq. 9.19), $BE = -n\beta/\gamma S\, B$ and this affords a means of finding S. The barometric efficiency can be expected to be in the range 10–75%. It is apparent that unconfined aquifers have practically no barometric efficiency.

A few other examples of compressibility effects causing water level changes in artesian wells include (i) tidal action in coastal aquifers, (ii) earthquake or underground explosions, and (iii) passing of heavy railway trains.

9.6 EQUATION OF MOTION

Confined Groundwater Flow

If the velocities of flow in the cartesian coordinate directions *x, y, z* of the aquifer element, ΔV, are u, v and w respectively, the equation of continuity for the fluid flow is

$$\frac{\partial(\Delta M)}{\partial t} = -\left[\frac{\partial(\rho u)}{\partial x} + \frac{\partial(\rho v)}{\partial y} + \frac{\partial(\rho w)}{\partial z}\right] \tag{9.22}$$

From Eq. (9.18) considering the differentials with respect to time and taking the limit as ΔV approaches zero

$$\frac{\partial(\Delta M)}{\partial t} = S_s \rho \frac{dh}{dt} \tag{9.18a}$$

Further the aquifer is assumed to be isotropic with permeability coefficient K, so that the Darcy's equation for x, y and z directions can be written as

$$u = -K\frac{\partial h}{\partial x},\ v = -K\frac{\partial h}{\partial y} \text{ and } w = -K\frac{\partial h}{\partial z} \tag{9.23}$$

Using Eqs. (9.23) and (9.13) and noting that $h = \dfrac{p}{\gamma} + z$, the various terms of the right-hand side of Eq. (9.22) are written as

$$\frac{\partial(\rho u)}{\partial x} = \rho\frac{\partial u}{\partial x} + u\frac{\partial \rho}{\partial x} = -K\rho\frac{\partial^2 h}{\partial x^2} - K\rho^2 \beta g\left(\frac{\partial h}{\partial x}\right)^2$$

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$$\frac{\partial(\rho v)}{\partial y} = \rho\frac{\partial v}{\partial y} + v\frac{\partial \rho}{\partial y} = -K\rho\frac{\partial^2 h}{\partial y^2} - K\rho^2\beta g\left(\frac{\partial h}{\partial y}\right)^2$$

$$\frac{\partial(\rho w)}{\partial z} = \rho\frac{\partial w}{\partial z} + w\frac{\partial \rho}{\partial z} = -K\frac{\partial^2 h}{\partial z^2} - K\rho^2\beta g\left[\left(\frac{\partial h}{\partial z}\right)^2 - \frac{\partial h}{\partial z}\right]$$

Assembling these, Eq. (9.22) can be written as

$$K\rho\left[\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2}\right] + K\rho^2\beta g\left[\left(\frac{\partial h}{\partial x}\right)^2 + \left(\frac{\partial h}{\partial y}\right)^2 + \left(\frac{\partial h}{\partial z}\right)^2\right] - \frac{\partial h}{\partial z} = \rho S_s\frac{\partial h}{\partial t} \tag{9.24}$$

The second term on the left-hand side is neglected as very small, especially for $\partial h/\partial x \ll 1$, and Eq. (9.24) is rearranged to yield

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \frac{S_s}{K}\frac{\partial h}{\partial t} \tag{9.25}$$

Defining $S_sB = S$, $K\,B = T$, and $\nabla^2 h = \left(\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2}\right)$, Eq. (9.25) reads as

$$\nabla^2 h = \frac{S}{T}\frac{\partial h}{\partial t} \tag{9.26}$$

This is the basic differential equation governing unsteady groundwater flow in a homogeneous isotropic *confined aquifer.* This form of the equation is known as *diffusion equation.*

If the flow is steady, the $\partial h/\partial t$ term does not exist, leading to

$$\nabla^2 h = 0 \tag{9.27}$$

This equation is known as *Laplace equation* and is the fundamental equation of all potential flow problems. Being linear, the method of superposition is applicable in its solutions.

Equation (9.26) or (9.27) can be solved for suitable boundary conditions by analytical, numerical or analog methods to yield solutions to a variety of groundwater flow problems. The details of solution of the basic differential equation of groundwater are available in standard literature (Refs. 1, 3, 5, 6 and 7).

As an application of the Laplace equation (Eq. 9.27) a simple situation of steady state one-dimensional confined porous media flow is given below.

Confined Groundwater Flow between Two Water Bodies

Figure 9.8 shows a very wide confined aquifer of depth B connecting to 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 the head is h.

As the flow is in x direction only, Eq. (9.27) becomes

$$\frac{\partial^2 h}{\partial x^2} = 0 \tag{9.28}$$

On integrating twice $h = C_1 x + C_2$

On substitution of the boundary condition $h = h_0$ at $x = 0$

$$h = C_1 x + h_0$$

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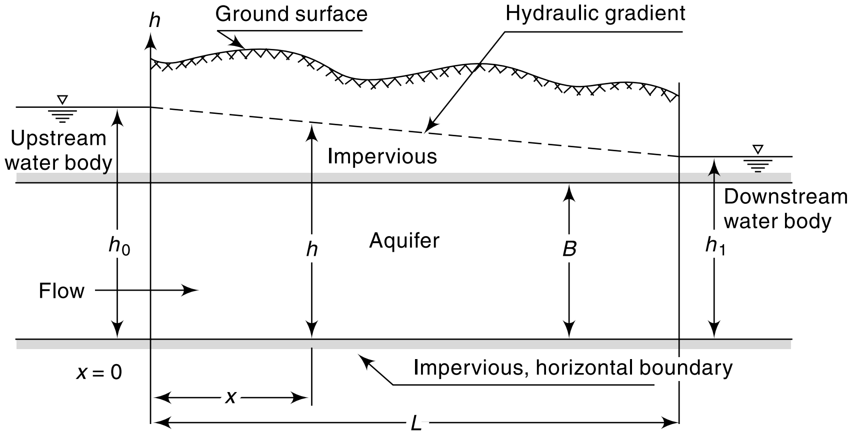

Fig. 9.8 Confined Groundwater Flow between Two Water Bodies

Also at $x = L$, $h = h_1$ and hence $C_1 = -\left(\frac{h_0 - h_1}{L}\right)$

Thus
$$h = h_0 - \left(\frac{h_0 - h_1}{L}\right) x \tag{9.29}$$

This is the equation of the hydraulic grade line, which is shown to vary linearly from h_0 to h_1.

By Darcy law, the discharge per unit width of the aquifer is

$$q = -KB\frac{dh}{dx} = -KB\,(-(h_0 - h_1)/L)$$

$$q = \frac{(h_0 - h_1)}{L} K\,B \tag{9.30}$$

Unconfined Flow by Dupit's Assumption

While Eq. (9.26) is specifically for confined aquifers, Eq. (9.27) which is the Laplace equation in h is applicable to steady flow of both confined and unconfined aquifers. However, 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. Also, in a section the line representing the water table, is also a streamline. These boundary conditions cause considerable difficulties in analytical solutions of steady unconfined flow problems by using the Laplace equation in h.

A simplified approach based on the assumptions suggested by Dupit (1863) which gives reasonably good results is described below. The basic assumptions of Dupit are:

- The curvature of the free surface is very small so that the streamlines can be assumed to be horizontal at all sections.
- The hydraulic grade line is equal to the free surface slope and does not vary with depth.

Consider an elementary prism of fluid bounded by the water table shown in Fig. 9.9(a).

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Let V_x = gross velocity of groundwater entering the element in x direction

V_y = gross velocity of groundwater entering the element in y direction

Assume a horizontal impervious base and no vertical inflow from top due to recharge. By Dupit's assumptions, $\partial V_x/\partial z = 0$ and $\partial V_y/\partial z = 0$. Considering the X direction:

The mass flux entering the element $M_{x1} = \rho\, V_x\, h\, \Delta\, y$

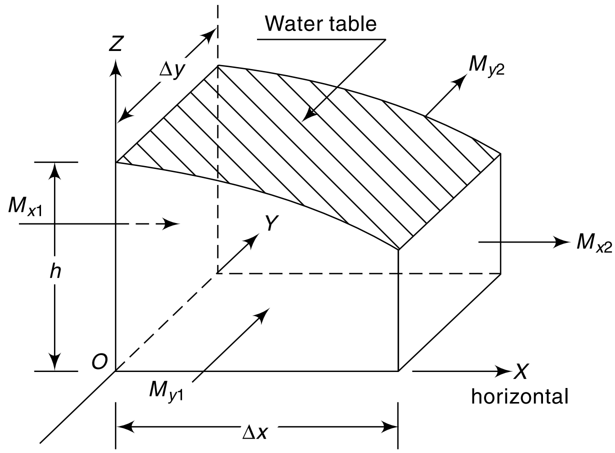

Fig. 9.9(a) Definition Sketch—Unconfined Groundwater Flow without Recharge

The mass flux leaving the element $M_{x2} = \rho V_x h\, \Delta y + \dfrac{\partial}{\partial x}(\rho V_x\, h\, \Delta y)\, \Delta x$

The net mass efflux from the element in x direction, by considering the flow entering the element as positive and outflow as negative, is

$$M_{x1} - M_{x2} = \Delta\, M_x = -\frac{\partial}{\partial x}(\rho V_x h\, \Delta y)\, \Delta x$$

Similarly the net mass efflux in y direction

$$M_{y1} - M_{y2} = \Delta\, M_y = -\frac{\partial}{\partial y}(\rho V_y h\, \Delta x)\, \Delta y$$

Further, there is neither inflow or outflow in the Z direction. Thus for steady, incompressible flow, by continuity

$$\Delta\, M_x + \Delta\, M_y = 0 \qquad (9.31)$$

Substituting for $\Delta\, M_x$ and $\Delta\, M_y$ and simplifying

$$\frac{\partial}{\partial x}(V_x h) + \frac{\partial}{\partial y}(V_y h) = 0 \qquad (9.32)$$

By Darcy law $V_x = -K\dfrac{\partial h}{\partial x}$ and $V_y = -K\dfrac{\partial h}{\partial y}$

Hence Eq. (9.32) becomes

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

or

$$\frac{\partial^2 h^2}{\partial x^2} + \frac{\partial^2 h^2}{\partial y^2} = 0$$

i.e.

$$\nabla^2 h^2 = 0 \qquad (9.33)$$

Thus the steady unconfined groundwater flow with Dupit's assumptions is governed by Laplace equation in h^2.

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Unconfined Flow with Recharge If there is a recharge, i.e. infiltration of water from the top ground surface into the aquifer, at a rate of R (m^3/s per m^2 of horizontal area) as in Fig. 9.9(b), the continuity equation Eq. (9.31) is to be modified to take into account the recharge. Consider the element of an uncon-fined aquifer as in Fig. 9.9(b) situated on a horizontal impervious bed. Here, in addition to ΔM_x and ΔM_y there will be a net inflow into the element in the Z direction given by

$$\Delta M_z = \rho R \Delta x \Delta y$$

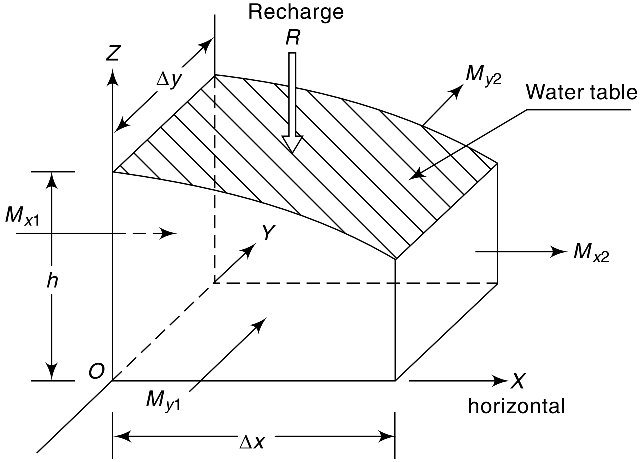

Fig. 9.9(b) Definition Sketch – Unconfined flow with Recharge

For steady, incompressible flow the continuity relationship for the element is

$$\Delta M_x + \Delta M_y + \Delta M_z = 0$$

i.e.
$$-\frac{\partial}{\partial x}(\rho V_x h \Delta x \Delta y) - \frac{\partial}{\partial y}(\rho V_y h \Delta x \Delta y) + \rho R \Delta x \Delta y = 0$$

Substituting $V_x = -K\frac{\partial h}{\partial x}$ and $V_y = -K\frac{\partial h}{\partial y}$ and simplifying

$$\frac{\partial^2 h^2}{\partial x^2} + \frac{\partial^2 h^2}{\partial y^2} = -\frac{2R}{K} \tag{9.34}$$

Equation (9.34) is the basic differential equation under Dupit's assumption for unconfined groundwater flow with recharge. Note that Eq. (9.33) is a special case of Eq. (9.34) with $R = 0$.

Use of Eq. (9.34) finds considerable practical application in finding the water table profiles in unconfined aquifers. A few examples are: (i) an unconfined aquifer separating two water bodies such as a canal and a river, (ii) various recharge situations, (iii) drainage problems, and (iv) infiltration galleries. To illustrate the use of Eq. (9.34) a situation of steady flow in an unconfined aquifer bounded by two water bodies and subjected to recharge from top is given below.

One Dimensional Dupit's Flow with Recharge

(1) The general case Consider an unconfined aquifer on a horizontal impervious base situated between two water bodies with a difference in surface elevation, as shown in Fig. 9.10. Further, there is a recharge at a constant rate of R m^3/s per unit horizontal area due to infiltration from the top of the aquifer. The aquifer is of infinite length and

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hence one dimensional method of analysis is adopted. A unit width of aquifer is considered for analysis.

From Eq. (9.34) $$\frac{\partial^2 h^2}{\partial x^2} = -\frac{2R}{K} \tag{9.35}$$

On integration with respect to x twice,

$$h^2 = -\frac{R}{K}x^2 + C_1 x + C_2 \tag{9.36}$$

where C_1 and C_2 are constants of integration

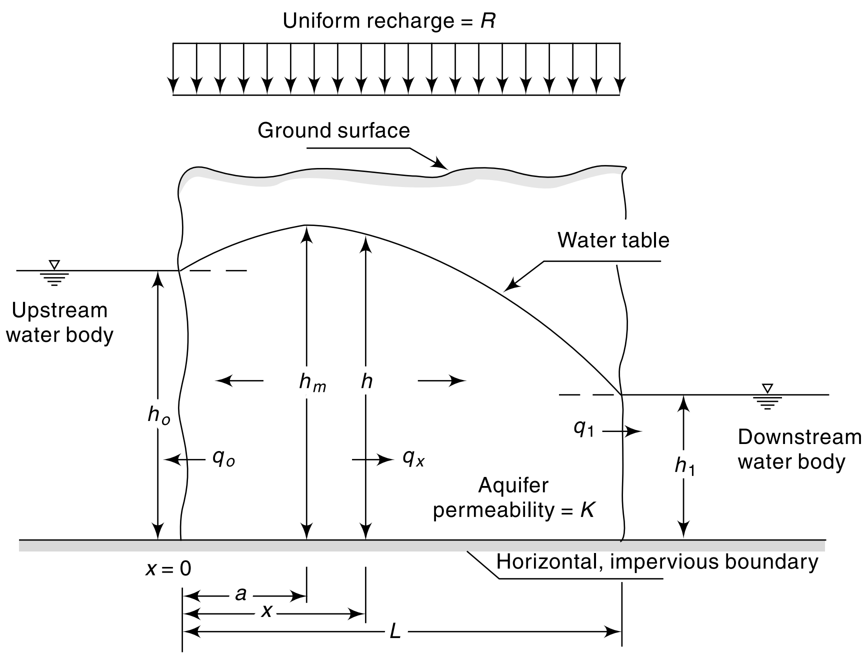

Fig. 9.10 One Dimensional Dupit Flow with Recharge

The boundary conditions are:

(i) at $x = 0$, $h = h_0$ hence, $C_2 = h_0^2$

(ii) at $x = L$, $h = h_1$ hence, $h_1^2 - h_0^2 = -\frac{R}{K}L^2 + C_1 L$

or $$C_1 = -\frac{\left(h_0^2 - h_1^2 - \frac{RL^2}{K}\right)}{L}$$

Thus Eq. (9.36) becomes

$$h^2 = -\frac{Rx^2}{K} - \frac{\left(h_0^2 - h_1^2 - \frac{RL^2}{K}\right)}{L}x + h_0^2 \tag{9.37}$$

The water table is thus an ellipse represented by Eq. (9.37). The value of h will in general rise above h_0, reaches a maximum at $x = a$ and falls back to h_1 at $x = L$ as

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shown in Fig. 9.10. The value of a is obtained by equating $\frac{dh}{dx} = 0$, and is given by

$$a = \frac{L}{2} - \frac{K}{R}\left(\frac{h_0^2 - h_1^2}{2L}\right) \tag{9.38}$$

The location $x = a$ is called the *water divide.* Figure 9.10 shows 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.

The discharge per unit width of aquifer at any location x is

$$q_x = -K\,h\frac{dh}{dx} = -K\left[-\frac{Rx}{K} - \frac{\left(h_0^2 - h_1^2 - \frac{RL^2}{K}\right)}{2L}\right]$$

$$q_x = R\left(x - \frac{L}{2}\right) + \frac{K}{2L}(h_0^2 - h_1^2) \tag{9.39}$$

It is obvious the discharge q_x varies with x. At the upstream water body, $x = 0$ and

Discharge $$q_0 = q_{x=0} = -\frac{RL}{2} + \frac{K}{2L}(h_0^2 - h_1^2) \tag{9.40}$$

At the downstream water body, $x = L$ and

$$q_1 = q_{x=L} = \frac{RL}{2} + \frac{K}{2L}(h_0^2 - h_1^2) = RL + q_0 \tag{9.40a}$$

(2) Flow without recharge When there is no recharge, $R = 0$ and the flow simplifies to that of steady one-dimensional flow in an unconfined aquifer as in Fig. 9.11.

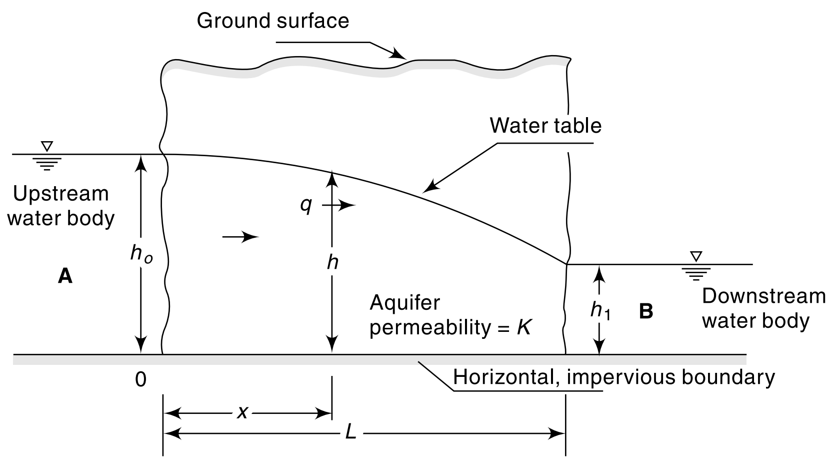

Fig. 9.11 One Dimensional Unconfined Flow without Recharge

By putting $R = 0$ in Eq. (9.37), the equation of the water table is given by

$$(h^2 - h_0^2) = \left(\frac{h_1^2 - h_0^2}{L}\right)x \tag{9.41}$$

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This represents a parabola (known as *Dupit's parabola*) joining h_0 and h_1 on either side of the aquifer.

Differentiating Eq. (9.41) with respect to x

$$2\,h\frac{dh}{dx} = \frac{(h_1^2 - h_0^2)}{L}$$

The discharge q per unit width of the aquifer is

$$q = -K\,h\frac{dh}{dx} = \frac{(h_0^2 - h_1^2)}{2L}K \qquad (9.42)$$

(3) Tile drain problem The provision of drains system is one of the most widely used method of draining waterlogged areas, the object being to reduce the level of the water table. Figure 9.12 shows a set of porous tile drains maintaining a constant recharge rate of R at the top ground surface.

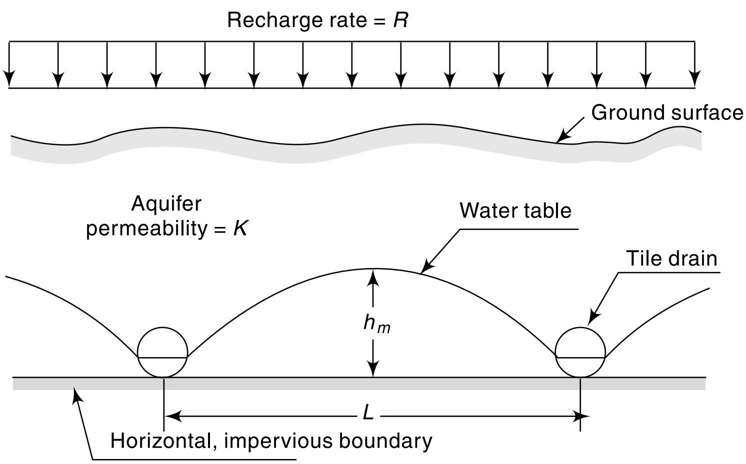

Fig. 9.12 Tile Drains under a Constant Recharge Rate

An approximate expression to the water table profile can be obtained by Eq. (9.37) by neglecting the depth of water in the drains, i.e. $h_0 = h_1 = 0$. The water table profile will then be

$$h^2 = \frac{R}{K}(L - x)\,x \qquad (9.43)$$

The maximum height of the water table occurs at $a = L/2$ and is of magnitude

$$h_m = \frac{L}{2}\sqrt{R/K} \qquad (9.44)$$

Considering a set of drains, since the flow is steady, the discharge entering a drain per unit length of the drain is

$$q = 2\left(R\frac{L}{2}\right) = RL \qquad (9.45)$$

Example 9.4 *Two parallel rivers A and B are separated by a land mass as shown in Fig. 9.13. Estimate the seepage discharge from River A to River B per unit length of the rivers.*

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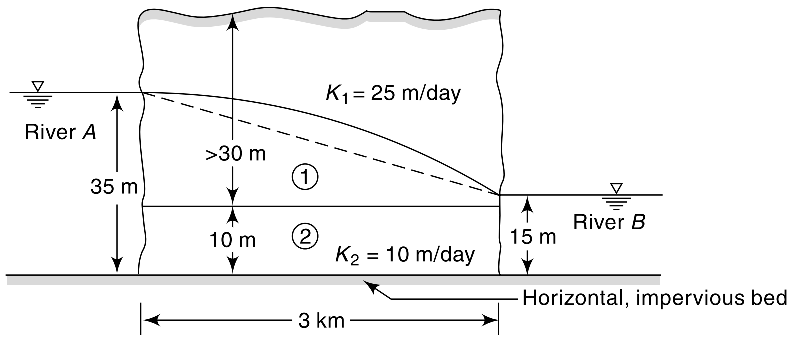

Fig. 9.13 Schematic Layout of Example 9.4

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 = 25 m/day

Consider a unit width of the aquifers.

For the confined aquifer 2:

From Eq. (9.30) $q_2 = \frac{(h_0 - h_1)}{L} K B$

Here h_0 = 35.0 m, h_1 = 15 m,
L = 3000 m, K_2 = 10 m/day and B = 10 m

$$q_2 = \frac{(35-15)}{3000} \times 10 \times 10 = 0.667 \text{ m}^3\text{/day per metre width}$$

For the unconfined aquifer 1:

From Eq. (9.42), $q_1 = \frac{(h_0^2 - h_1^2)}{2L} K$

Here h_0 = (35 – 10) = 25 m, h_1 = (15 – 10) = 5 m
L = 3000 m, K_1 = 25 m/day

$$q_2 = \frac{(25)^2 - (5)^2}{2 \times 3000} \times 25 = 2.5 \text{ m}^3\text{/day per metre width}$$

Total discharge from river A to river $B = q = q_1 + q_2$
$= 0.667 + 2.500 = 3.167$ m^3/day per unit length of the rivers

EXAMPLE 9.5 *An unconfined aquifer (K = 5 m/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 0.002 m^3/day per m^2 of horizontal area occurs on the ground surface, estimate*

(i) the water table profile

(ii) the location and elevation of the water table divide

(iii) the seepage discharges into the lakes and

(iv) the recharge rate at which the water table divide coincides with the upstream edge of the aquifer and the total seepage flow per unit width of the aquifer at this recharge rate.

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SOLUTION: Consider unit width of the aquifer

Referring to Fig. 9.14

h_0 = 10.0 m, h_1 = 8.0 m,

R = 0.002 m^3/day/m^2, L = 1200 m and K = 5 m/day.

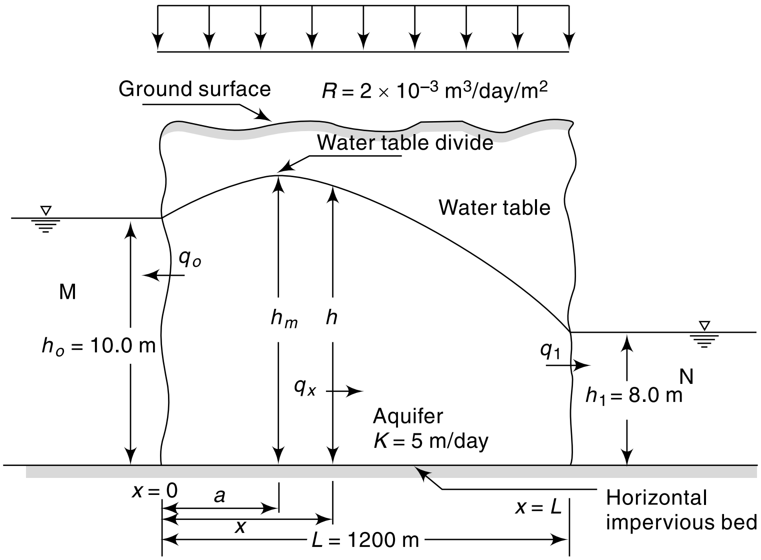

Fig. 9.14 Schematic Layout—Example 9.5

(i) The water table profile:

By Eq. (9.37), $$h^2 = -\frac{Rx^2}{K} - \frac{\left(h_0^2 - h_1^2 - \frac{RL^2}{K}\right)}{L}x + h_0^2$$

$$= -\left(\frac{0.002}{5}\right)x^2 - \frac{1}{1200}\left[(10)^2 - (8)^2 - \frac{0.002 \times (1200)^2}{5}\right]x + 10^2$$

$$h^2 = -0.0004\, x^2 + 0.45x + 100$$

(ii) Location of water table divide:

From Eq. (9.38) $$a = \frac{L}{2} - \frac{K}{R}\left(\frac{h_0^2 - h_1^2}{2L}\right)$$

$$a = \frac{1200}{2} - \left(\frac{5.0}{0.002}\right)\left(\frac{(10)^2 - (8)^2}{2 \times 1200}\right) = 562.5 \text{ m}$$

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

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

and $$h_m = \sqrt{226.56} = 15.05 \text{ m}$$

(iii) Discharge per unit width of the aquifer:

From Eq. (9.39) $$q_x = R\left(x - \frac{L}{2}\right) + \frac{K}{2L}(h_0^2 - h_1^2)$$

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At $x = 0$, $$q_0 = -R\frac{L}{2} + \frac{K}{2L}(h_0^2 - h_1^2)$$

$$= \frac{-(0.002 \times 1200)}{2} + \frac{5}{2 \times 1200}[(10)^2 - (8)^2] = -1.20 + 0.075$$

$$q_0 = -1.125 \text{ m}^3\text{/day per metre width}$$

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

At $x = L$, $q_1 = q_L$ and from Eq. (9.40a) $q_L = RL + q_0$

Hence q_1 = discharge into water body *N*

$= 0.002 \times 1200 + (-1.125) = 1.275$ m^3/day/m width.

(vi) when the distance of the water table divide $a = 0$:

From Eq. (9.36), $$a = \frac{L}{2} - \frac{K}{R}\left(\frac{h_0^2 - h_1^2}{2L}\right) = 0$$

$$\frac{L}{2} = \frac{K}{R}\left(\frac{h_0^2 - h_1^2}{2L}\right)$$

$$R = \frac{K}{L^2}(h_0^2 - h_1^2) = \frac{5.0}{(1200)^2}[(10)^2 - (8)^2]$$

$$= 1.25 \times 10^{-4} \text{ m}^3\text{/day/m}^2$$

Since $a = 0$, $q_0 = 0$ and by Eq. (9.40a)

$$q_1 = q_L = RL$$

$$= 1.25 \times 10^{-4} \times 1200 = 0.15 \text{ m}^3\text{/day/m width.}$$

9.7 WELLS

Wells form the most important mode of groundwater extraction from an aquifer. While wells are used in a number of different applications, they find extensive use in water supply and irrigation engineering practice.

Consider the water in an unconfined aquifer being pumped at a constant rate from a well. Prior to the pumping, the water level in the well indicates the static water table. A lowering of this water level takes place on pumping. If the aquifer is homogeneous and isotropic and the water table horizontal initially, due to the radial flow into the well through the aquifer the water table assumes a conical shape called *cone of depression.* The drop in the water table elevation at any point from its previous static level is called *drawdown.* The areal extent of the cone of depression is called *area of influence* and its radial extent *radius of influence* (Fig. 9.15). At constant rate of pumping, the drawdown curve develops gradually with time due to the withdrawal of water from storage. This phase is called an *unsteady flow* as the water table elevation at a given location near the well changes with time. On prolonged pumping, an equilibrium state is reached between the rate of pumping and the rate of inflow of groundwater from the outer edges of the zone of influence. The drawdown surface attains a constant position with respect to time when the well is known to operate under *steady-flow* conditions. As soon as the pumping is stopped, the depleted storage in the cone of depression is made good by groundwater inflow into the zone of influence. There is a gradual accumulation of storage till the original (static) level is reached. This stage

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is called *recuperation* or *recovery* and is an unsteady phenomenon. Recuperation time depends upon the aquifer characteristics.

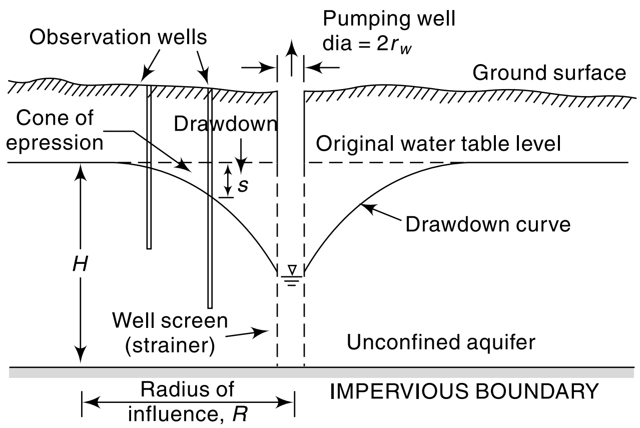

Fig. 9.15 Well Operating in an Unconfined Aquifer, (definition sketch)

Changes similar to the above take place to a pumping well in a confined aquifer also but with the difference that it is the piezometric surface instead of the water table that undergoes drawdown with the development of the cone of depression. In confined aquifers with considerable piezometric head, the recovery into the well takes place at a very rapid rate.

9.8 STEADY FLOW INTO A WELL

Steady state groundwater problems are relatively simpler. Expressions for the steady state radial flow into a well under both confined and unconfined aquifer conditions are presented below.

Confined Flow

Figure 9.16 shows a well completely penetrating a horizontal confined aquifer of thickness B. Consider the well to be discharging a steady flow, Q. The original piezometric head (static head) was H and the drawdown due to

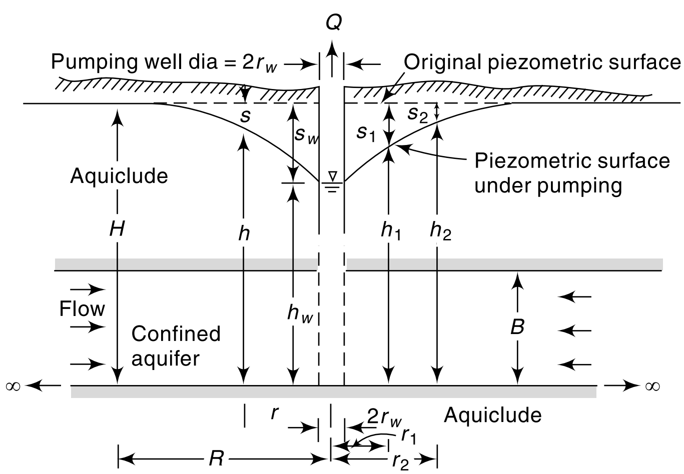

Fig. 9.16 Well Operating in a Confined Aquifer

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pumping is indicated in Fig. 9.16. The piezometric head at the pumping well is h_w and the drawdown s_w.

At a radial distance r from the well, if h is the piezometric head, the velocity of flow by Darcy's law is

$$V_r = K\frac{dh}{dr}$$

The cylindrical surface through which this velocity occurs is $2\pi r B$. Hence by equating the discharge entering this surface to the well discharge,

$$Q = (2\pi r B)\left(K\frac{dh}{dr}\right) \qquad \frac{Q}{2\pi KB}\frac{dr}{r} = dh$$

Integrating between limits r_1 and r_2 with the corresponding piezometric heads being h_1 and h_2 respectively,

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

or

$$Q = \frac{2\pi KB(h_2 - h_1)}{\ln\frac{r_2}{r_1}} \tag{9.46}$$

This is the equilibrium equation for the steady flow in a confined aquifer. This equation is popularly known as *Thiem's equation.*

If the drawdown s_1 and s_2 at the observation wells are known, then by noting that

$$s_1 = H - h_1,\ s_2 = H - h_2 \text{ and } KB = T$$

Equation (9.46) will read as

$$Q = \frac{2\pi T(s_1 - s_2)}{\ln\frac{r_2}{r_1}} \tag{9.47}$$

Further, at the edge of the zone of influence, $s = 0$, $r_2 = R$ and $h_2 = H$; at the well wall $r_1 = r_w$, $h_1 = h_w$ and $s_1 = s_w$. Equation (9.47) would then be

$$Q = \frac{2\pi T s_w}{\ln R/r_w} \tag{9.48}$$

Equation (9.47) or (9.48) can be used to estimate T, and hence K, from pumping tests. For the use of the equilibrium equation, Eq. (9.46) or its alternative forms, it is necessary that the assumption of complete penetration of the well into the aquifer and steady state of flow are satisfied.

EXAMPLE 9.6 *A 30-cm diameter well completely penetrates a confined aquifer of permeability 45 m/day. The length of the strainer is 20 m. Under steady state of pumping the drawdown at the well was found to be 3.0 m and the radius of influence was 300 m. Calculate the discharge.*

SOLUTION: In this problem, referring to Fig. 9.16,

$r_w = 0.15$ m $\qquad R = 300$ m $\qquad s_w = 3.0$ m $\qquad B = 20$ m

$K = 45/(60 \times 60 \times 24) = 5.208 \times 10^{-4}$ m/s

$T = KB = 10.416 \times 10^{-3}$ m^2/s

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By Eq. (9.48)

$$Q = \frac{2\pi T s_w}{\ln R/r_w} = \frac{2\pi \times 10.416 \times 10^{-3} \times 3}{\ln \frac{300}{0.15}} = 0.02583 \text{ m}^3\text{/s} = 25.83 \text{ lps} = 1550 \text{ lpm}$$

Example 9.7 *For the well in the previous example, calculate the discharge (a) if the well diameter is 45 cm and all other data remain the same as in Example 9.6(b) if the drawdown is increased to 4.5 m and all other data remain unchanged as in Example 9.6.*

Solution:

(a) $Q = \frac{2\pi T s_w}{\ln R/r_w}$

As T and s_w are constants, $\frac{Q_1}{Q_2} = \frac{\ln R/r_{w_2}}{\ln R/r_{w_1}}$

Putting R = 300 m Q_1 = 1550 lpm, r_{w_1} = 0.15 m and r_{w_2} = 0.225 m.

$$Q_2 = 1550 \frac{\ln 300/0.15}{\ln 300/0.225} = 1637 \text{ lpm}$$

[Note that the discharge has increased by about 6% for 50% increase in the well diameter.]

(b) $Q = \frac{2\pi T s_w}{\ln R/r_w}$

$Q \,\alpha\, s_w$ for constant T, R and r_w. Thus

$$\frac{Q_1}{Q_2} = \frac{s_{w_1}}{s_{w_2}}$$

$$Q_2 = 1550 \times \frac{4.5}{3.0} = 2325 \text{ lpm}$$

[Note that the discharge increases linearly with the drawdown when other factors remain constant.]

Unconfined Flow

Consider a steady flow from a well completely penetrating an unconfined aquifer. In this case because of the presence of a curved free surface, the streamlines are not strictly radial straight lines. While a streamline at the free surface will be curved, the one at the bottom of the aquifer will be a horizontal line, both converging to the well. To obtain a simple solution *Dupit's assumptions* as indicated in Sec. 9.6 are made. In the present case these are:

- For small inclinations of the free surface, the streamlines can be assumed to be horizontal and the equipotentials are thus vertical.
- The hydraulic gradient is equal to the slope of the free surface and does not vary with depth. This assumption is satisfactory in most of the flow regions except in the immediate neighbourhood of the well.

Consider the well of radius r_w penetrating completely an extensive unconfined horizontal aquifer as shown in Fig. 9.17. The well is pumping a discharge Q. At any radial distance r, the velocity of radial flow into the well is

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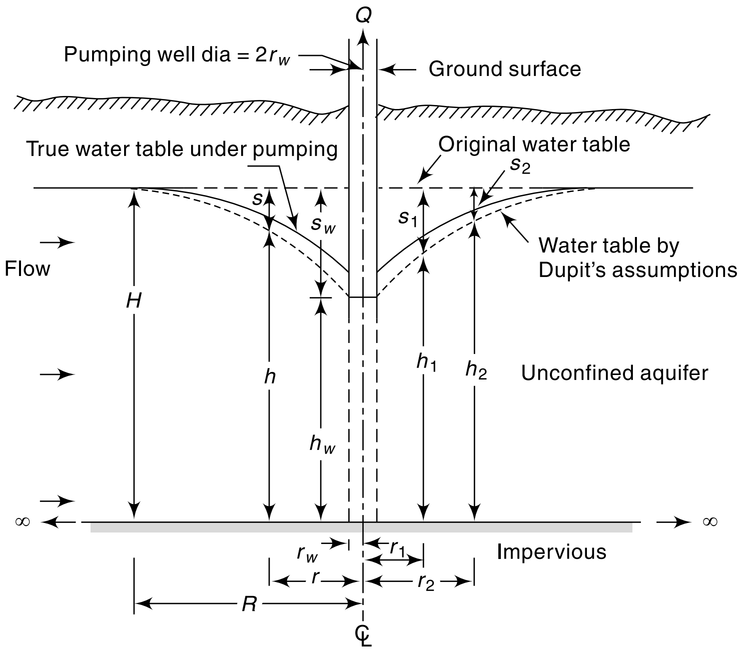

Fig. 9.17 Radial Flow to a Well in an Unconfined Aquifer

$$V_r = K\frac{dh}{dr}$$

where h is the height of the water table above the aquifer bed at that location. For steady flow, by continuity

$$Q = (2\,\pi\,r\,h)V_r = 2\,\pi\,r\,K\,h\,\frac{dh}{dr}$$

or $$\frac{Q}{2\,\pi K}\frac{dr}{r} = h\,dh$$

Integrating between limits r_1 and r_2 where the water-table depths are h_1 and h_2 respectively and on rearranging

$$Q = \frac{\pi K\,(h_2^2 - h_1^2)}{\ln\frac{r_2}{r_1}} \qquad (9.49)$$

This is the equilibrium equation for a well in an unconfined aquifer. As at the edge of the zone of influence of radius R, H = saturated thickness of the aquifer, Eq. (9.49) can be written as

$$Q = \frac{\pi K\,(H^2 - h_w^2)}{\ln\frac{R}{r_w}} \qquad (9.50)$$

where h_w = depth of water in the pumping well of radius r_w.

Equations (9.49) and (9.50) can be used to estimate satisfactorily the discharge and permeability of the aquifer by using field data. Calculations of the water-table profile by Eq. (9.49), however, will not be accurate near the well because of Dupit's

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assumptions. The water-table surface calculated by Eq. (9.49) which involved Dupit's assumption will be lower than the actual surface. The departure will be appreciable in the immediate neighbourhood of the well (Fig. 9.17). In general, values of R in the range 300 to 500 m can be assumed depending on the type of aquifer and operating conditions of a well. As the logarithm of R is used in the calculation of discharge, a small error in R will not seriously affect the estimation of Q. It should be noted that it takes a relatively long time of pumping to achieve a steady state in a well in an unconfined aquifer. The recovery after the cessation of pumping is also slow compared to the response of an artesian well which is relatively fast.

APPROXIMATE EQUATIONS If the drawdown at the pumping well $s_w = (H - h_w)$ is small relative to H, then

$$H^2 - h_w^2 = (H + h_w)\,(H - h_w) \approx 2\,H\,s_w$$

Noting that $T = KH$, Eq. (9.50) can be written as

$$Q = \frac{2\pi T\,s_w}{\ln \dfrac{R}{r_w}} \qquad (9.50a)$$

which is the same as Eq. (9.48). Similarly Eq. (9.49) can be written in terms of $s_1 = (H - h_1)$ and $s_2 = (H - h_2)$ as

$$Q = \frac{2\pi T\,(s_1 - s_2)}{\ln \dfrac{r_2}{r_1}} \qquad (9.49a)$$

Equations (949a) and (9.50a) are approximate equations to be used only when Eq. (9.49) or (9.50) cannot be used for lack of data. Equation (9.50a) over estimates the discharge by $[1/2\ (H/s_w - 1)]$ % when compared to Eq. (9.50).

EXAMPLE 9.8 *A 30-cm well completely penetrates an unconfined aquifer of saturated depth 40 m. After a long period of pumping at a steady rate of 1500 lpm, the drawdown in two observation wells 25 and 75 m from the pumping well were found to be 3.5 and 2.0 m respectively. Determine the transmissivity of the aquifer. What is the drawdown at the pumping well?*

SOLUTION:

(a) $$Q = \frac{1500 \times 10^{-3}}{60} = 0.025\ \text{m}^3/\text{s}$$

$h_2 = 40.0 - 2.0 = 38.0$ $\qquad r_2 = 75$ m

$h_1 = 40.0 - 3.5 = 36.5$ m $\qquad r_1 = 25$ m

From Eq. (9.49),

$$Q = \frac{\pi K\,(h_2^2 - h_1^2)}{\ln \dfrac{r_2}{r_1}}$$

$$0.025 = \frac{\pi K\,[(38)^2 - (36.5)^2]}{\ln \dfrac{75}{25}}$$

$$K = 7.823 \times 10^{-5}\ \text{m/s}$$

$$T = KH = 7.823 \times 10^{-5} \times 40 = 3.13 \times 10^{-3}\ \text{m}^2/\text{s}$$

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(b) At the pumping well, $r_w = 0.15$ m

$$Q = \frac{\pi K (H_1^2 - h_w^2)}{\ln \frac{r_1}{r_w}}$$

$$0.025 = \frac{\pi \times 7.823 \times 10^{-5} [(36.5)^2 - h_w^2]}{\ln \frac{25}{0.15}}$$

$$h_w^2 = 811.84 \quad \text{and} \quad h_w = 28.49 \text{ m}$$

Drawdown at the well, $s_w = 11.51$m

9.9 OPEN WELLS

Open wells (also known as *dug wells*) are extensively used for drinking water supply in rural communities and in small farming operations. They are best suited for shallow and low yielding aquifers. In hard rocks the cross sections are circular or rectangular in shape. They are generally sunk to a depth of about 10 m and are lined wherever loose over burden is encountered. The flow into the well is through joints, fissures and such other openings and is usually at the bottom/lower portions of the well. In unconsolidated formations (e.g. alluvial soils) the wells are usually dug to a depth of about 10 m below water table, circular in cross section and lined. The water entry into these wells is from the bottom. These wells tap water in unconfined aquifers.

When the water in an open well is pumped out, the water level inside the well is lowered. The difference in the water table elevation and the water level inside the well is known as *depression head*. The flow discharge into the well (Q) is proportional to the depression head (H), and is expressed as

$$Q = K_0 H \tag{9.51}$$

where the proportionality constant K_0 depends on the characteristic of the aquifer and the area of the well. Also, since K_0 represents discharge per unit drawdown it is called as *specific capacity* of the well. There is a *critical depression head* for a well beyond which any higher depression head would cause dislodging of soil particles by the high flow velocities. The discharge corresponding to the critical head is called as *critical or maximum yield*. Allowing a factor of safety (normally 2.5 to 3.0) a *working head* is specified and the corresponding yield from the well is known as *safe yield*.

Recuperation Test The specific capacity K_0 of a well is determined from the recuperation test described below.

Let the well be pumped at a constant rate Q till a drawdown H_1 is obtained. The pump is now stopped and the well is allowed to recuperate. The water depth in the well is measured at various time intervals t starting from the stopping of the well. Referring to Fig. 9.18,

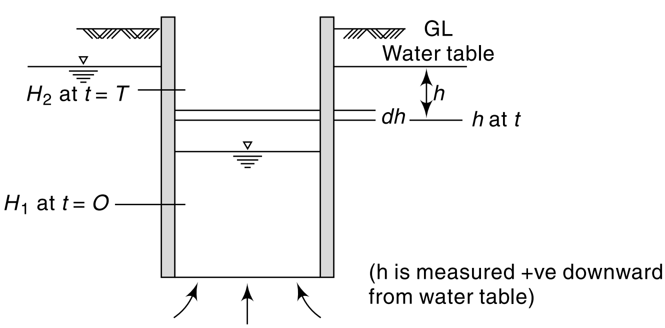

Fig. 9.18 Recuperation Test for Open well

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H_1 = drawdown at the start of recuperation, $t = 0$

H_2 = drawdown at a time, $t = T_r$

h = drawdown at any time t

Δh = decrease in drawdown on time Δt

At any time t, the flow into the well $Q = K_0 h$

In a time interval Δt causing a small change Δh in the water level,

$$Q \cdot \Delta t = K_0 h \cdot \Delta t = -A \cdot \Delta h$$

where A is the area of the well. In differential form

$$dt = -\frac{A}{K_0}\frac{dh}{h}$$

Integrating for a time interval T_r

$$\int_0^{T_r} dt = -\frac{A}{K_0}\int_{H_1}^{H_2}\frac{dh}{h}$$

$$T_r = \frac{A}{K_0}\ln\frac{H_1}{H_2} \tag{9.52}$$

or
$$\frac{K_0}{A} = \frac{1}{T_r}\ln\frac{H_1}{H_2} \tag{9.52a}$$

The term $\frac{K_0}{A} = K_s$ represents *specific capacity per unit well area* of the aquifer and is essentially a property of the aquifer. Knowing H_1, H_2 and the recuperation time T_r for reaching H_2 from H_1, and the specific capacity per unit well area is calculated by Eq. (9.52a).

Usually the K_s of an aquifer, determined by recuperation tests on one or more wells, is used in designing further dug wells in that aquifer. However, when such information is not available the following approximate values of K_s, given by Marriot, are often used.

Type of sub-soil	**Value of K_s in units of h^{-1}**
Clay	0.25
Fine sand	0.50
Coarse sand	1.00

The yield Q from an open well under a depression head H is obtained as

$$Q = K_s\, AH \tag{9.5a}$$

For dug wells with masonry sidewalls, it is usual to assume the flow is entirely from the bottom and as such A in Eq. (9.51a) represents the bottom area of the well.

Example 9.9 *During the recuperation test of a 4.0 m open well a recuperation of the depression head from 2.5 m to 1.25 m was found to take place in 90 minutes. Determine the (i) specific capacity per unit well area and (ii) yield of the well for a safe drawdown of 2.5 m (iii) What would be the yield from a well of 5.0 m diameter for a drawdown of 2.25 m?*

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SOLUTION: $A = \frac{\pi}{4} \times (4.0)^2 = 12.566 \text{ m}^2$

From Eq. (9.52a), $\frac{K_0}{A} = \frac{1}{T_r} \ln \frac{H_1}{H_2}$

Here T_r = 90 min = 1.50 h, H_1 = 2.5 m, and H_2 = 1.25 m

(i) $K_s = \frac{K_0}{A} = \frac{1}{1.5} \ln \frac{2.5}{1.25} = 0.462\ h^{-1}$

(ii) $Q = K_s \cdot A \cdot H = 0.462 \times 12.566 \times 2.5 = 14.52 \text{ m}^3/\text{h}$

(iii) $A_2 = \frac{\pi}{4} \times (5.0)^2 = 19.635$

$Q = K_s \times A_2 \times H_2 = 0.462 \times 19.635 \times 2.25 = 20.415 \text{ m}^3/\text{h}$

9.10 UNSTEADY FLOW IN A CONFINED AQUIFER

When a well in a confined aquifer starts discharging, the water from the aquifer is released resulting in the formation of a cone of depression of the piezometric surface. This cone gradually expands with time till an equilibrium is attained. The flow configuration from the start of pumping till the attainment of equilibrium is in unsteady regime and is described by Eq. (9.26).

In polar coordinates, Eq. (9.26), to represent the radial flow into a well, takes the form

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} = \frac{S}{T}\frac{\partial h}{\partial t} \tag{9.53}$$

Making the same assumptions as used in the derivation of the equilibrium formula (Eq. 9.46), Thies (1935) obtained the solution of this equation as

$$s = (H - h) = \frac{Q}{4\pi T}\int_u^\infty \frac{e^{-u}}{u}\,du \tag{9.54}$$

where $s = H - h$ = drawdown at a point distance r from the pumping well, H = initial constant piezometric head, Q = constant rate of discharge, T = transmissibility of the aquifer, u = a parameter = $r^2 S/4Tt$, S = storage coefficient and t = time from start of pumping. The integral on the right hand side is called the *well function, W (u)*, and is given by

$$W(u) = \int_u^\infty \frac{e^{-u}}{u}\,du = -0.577216 - \ln u + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!}\ldots \tag{9.55}$$

Table of $W(u)$ are available in literature (e.g. Refs. 1, 9 and 10). Values of $W(u)$ can be easily calculated by the series (Eq. 9.55) to the required number of significant digits which rarely exceed 4. For small values of $u(u \le 0.01)$, only the first two terms of the series are adequate.

The solution of Eq. (9.54) to find the drawdown s for a given S, T, r, t and Q can be obtained in a straightforward manner. However, the estimation of the aquifer constants S and T from the drawdown *vs* time data of a pumping well, which involve trial-and-error procedures, can be done either by a digital computer or by semigraphical methods such as the use of *Type curve*[1, 8, 9] or by *Chow's method* described in literature[1].

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For small values of u ($u \le 0.01$), Jacob (1946, 1950) showed that the calculations can be considerably simplified by considering only the first two terms of the series of $W(u)$, (Eq. 9.55). This assumption leads Eq. (9.54) to be expressed as

$$s = \frac{Q}{4\pi T}\left[-0.5772 - \ln\frac{r^2 S}{4Tt}\right]$$

i.e.
$$s = \frac{Q}{4\pi T}\ln\left[\frac{2.2\,Tt}{r^2 S}\right] \qquad (9.56)$$

If s_1 and s_2 are drawdowns at times t_1 and t_2,

$$(s_2 - s_1) = \frac{Q}{4\pi T}\ln\frac{t_2}{t_1} \qquad (9.57)$$

If the drawdown s is plotted against time t on a semi-log paper, the plot will be a straight line for large values of time. The slope of this line enables the storage coefficient S to be determined. From Eq. (9.54), when $s = 0$,

$$\frac{2.25\,T\,t_0}{r^2 S} = 1$$

or
$$S = \frac{2.25\,T\,t_0}{r^2} \qquad (9.58)$$

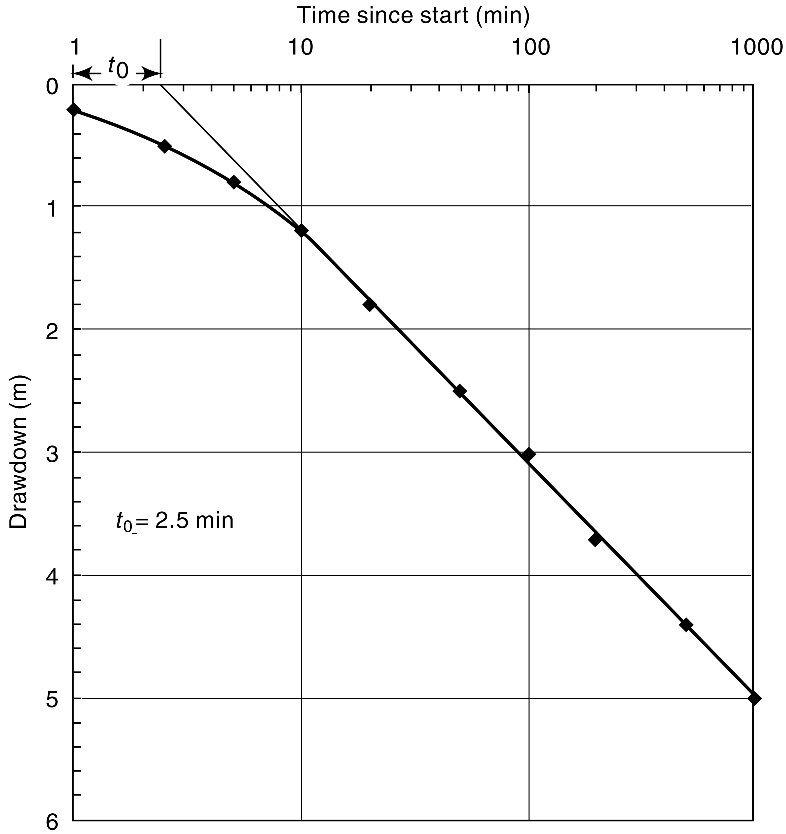

Fig. 9.19 Time – Drawdown Plot—Example 9.10

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in which t_0 = time corresponding to "zero" drawdown obtained by extrapolating the straight-line portion of the semi-log curve of s *vs* t (Fig. 9.19). It is important to remember that the above approximate method proposed by Jacob assumes u to be very small.

DRAWDOWN TEST Equations (9.56 and 9.57) relating drawdown s with time t and aquifer properties is used to evaluate formation constants S and T through pumping test. The method is known as drawdown test.
Procedure: An observation well at a distance r from the production well is selected. The pumping is started and the discharge is maintained at a constant value (Q) throughout the test. Values of the drawdown s are read at the observation well at various times, t. The time intervals between successive readings could progressively increase to cut down on the number of observations. The pumping is continued till nearly steady state conditions are reached. This may take about 12 to 36 hours depending on the aquifer characteristics. The best values of S and T are obtained from Eqs. 9.56 and 9.57 through semi-log plot of s against time t.

EXAMPLE 9.10 *A 30-cm well penetrating a confined aquifer is pumped at a rate of a 1200 lpm. The drawdown at an observation well at a radial distance of 30 m is as follows:*

Time from start (min)	1.0	2.5	5	10	20	50	100	200	500	1000
Drawdown (m)	0.2	0.5	0.8	1.2	1.8	2.5	3.0	3.7	4.4	5.0

Calculate the aquifer parameters S and T.

SOLUTION: The drawdown is plotted against time on a semilog plot (Fig. 9.19). It is seen that for $t > 10$ min. the drawdown values describe a straight line. A best-fitting straight line is drawn for data points with $t > 10$ min. From this line,

when $s = 0,\ t = t_0 = 2.5$ min = 150 s

$s_1 = 3.1$ m at $t_1 = 100$ min

$s_2 = 5.0$ m at $t_2 = 1000$ min

Also, $Q = 1200$ lpm $= 0.02\ \text{m}^2/\text{s}$

From Eq. (9.57)

$$s_2 - s_1 = \frac{Q}{4\pi T}\ln\frac{t_2}{t_1}$$

$$(5.0 - 3.1) = \frac{0.02}{4\times\pi\times T}\ln\frac{1000}{100}$$

$$T = \frac{0.02}{4\pi\times 1.9}\ln 10 = 1.929\times 10^{-3}\ \text{m}^3/\text{s/m} = 1.67\times 10^5\ \text{lpd/m}$$

From Eq. (9.58),

$$S = \frac{2.25\,T\,t_0}{r^2} = \frac{2.25\times 1.929\times 10^{-3}\times 150}{(30)^2}$$

i.e. $S = 7.23\times 10^{-4}$

EXAMPLE 9.11 *A well is located in a 25 m confined aquifer of permeability 30 m/day and storage coefficient 0.005. If the well is being pumped at the rate of 1750 lpm, calculate the drawdown at a distance of (a) 100 m and (b) 50 m from the well after 20 h of pumping.*

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SOLUTION:

(a) $T = K\,B = \dfrac{30}{86400} \times 25 = 8.68 \times 10^{-3}\ \text{m}^2/\text{s}$

$$u = \frac{r^2 S}{4Tt} = \frac{(100)^2 \times (0.005)}{4 \times (8.68 \times 10^{-3}) \times (20 \times 60 \times 60)} = 0.02$$

Using Theis method and calculating $W(u)$ to four significant digits,

$$W(u) = -0.5772 - \ln(0.02) + (0.02) - \frac{(0.02)^2}{2.2!} + \frac{(0.02)^3}{3.3!}$$

$$= -0.5772 + 3.9120 + 0.02 - 0.0001 + 4.4 \times 10^{-7} = 3.3547$$

$$S_{100} = \frac{Q}{4\pi T} W(u)$$

$$= \left(\frac{1.750}{60}\right) \times \frac{1}{4\pi(8.68 \times 10^{-3})} \times 3.3547 = 0.897\ \text{m}$$

(b) $r = 50$ m, $u = \dfrac{(50)^2 \times (0.005)}{4 \times (8.68 \times 10^{-3}) \times (20 \times 60 \times 60)} = 0.005$

$$W(u) = -0.5772 - \ln 0.005 + 0.005 = 4.726$$

$$S_{50} = \left(\frac{1.750}{60}\right) \times \frac{1}{4\pi(8.68 \times 10^{-3})} \times 4.726 = 1.264\ \text{m}$$

(Note that for small values of u, i.e. $u < 0.01$, $W(u) \approx -0.5772 - \ln u$).

RECOVERY OF PIEZOMETRIC HEAD

Consider a well pumped at constant rate of Q. Let s_1 be the drawdown at an observation well near the well in time t_1. If the pumping is stopped at the instant when the time is t_1, the ground water flow into the cone of depression will continue at the same rate Q. Since there is not withdrawal now, the water level in the observation well will begin to rise and the drawdown will begin to decrease. This is known as the *recovery of the cone of depression*. The variation of the water level with time during pumping and in the recovery phase is shown in Fig. 9.20.

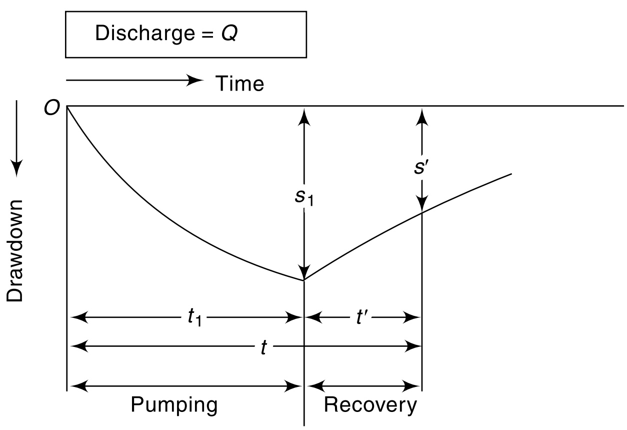

Fig. 9.20 Variation of Piezometric Head in Pumping and Recovery Head

The drawdown at the observation well at any time t' after the cessation of the pumping is known as *residual drawdown* and can be calculated as

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$$s' = \frac{Q}{4\pi T}[W(u) - W(u')] \tag{9.59}$$

where $u = \frac{r^2 S}{4\pi T t}$ and $u' = \frac{r^2 S}{4\pi T t'}$

$t = t_1 + t'$ = time from start of pump and t' = time since stoppage of pumping (start of recovery).

For small values of r and large values of t' Eq. (9.59) can be approximated as

$$s' = \frac{Q}{4\pi T} \cdot \ln\frac{t}{t'} = \frac{2.302}{4\pi} \cdot \frac{Q}{T} \cdot \log\frac{t}{t'} \tag{9.60}$$

The plot of residual drawdown s' vs (t/t') on semi-log paper represents a straight line with its slope as $\left[\frac{2.302\,Q}{4\pi T}\right]$.

Recovery Test The relationship of the residual recovery given by Eq. (9.60) is used as a method of assessing the transmissibility T of the aquifer. The procedure is known as *Recovery test*. In this test, the pump is run at constant discharge rate for a sufficiently long time t_1, and then stopped. The value of t_1 depends on the type of aquifer and aquifer characteristics and may range from 12 to 24 hours. The recovery of water level s' in an observation well situated at a distance r from the production well is noted down at various times (t'). In view of the logarithmic nature of the variation of residual drawdown with the time ratio (t/t'), the time intervals between successive readings could progressively increase, When observation wells are not available, the recovery water levels can be observed in the production well itself and this is a positive advantage of this test.

The value of *transmissibility* T is calculated from plot of s' against (t/t') on semi-log axes.

It is to be noted that the recovery test data does not enable the determination of the storage coefficient S.

Example 9.12 *Recovery test on a well in a confined aquifer yielded the following data:*
Pumping was at a uniform rate of 1200 m^2/day and was stopped after 210 minutes of pumping. Recovery data was as shown below:

Time since stoppage of pump (min)	2	5	10	20	40	90	150	210
Residual drawdown (m)	0.70	0.55	0.45	0.30	0.25	0.19	0.15	0.10

Solution: Here since t_1 = 210 min, $t = t_1 + t' = 210 + t'$
The time ratio t/t' is calculated (as shown in the table below) and a semi-log plot of s' *vs* t/t' is plotted (Fig. 9.21).

t'	2	5	10	20	40	90	150	210
t	212	215	220	240	250	300	360	420
t/t'	106	43	22	11.5	6.25	3.33	2.40	2.0
s'	0.70	0.55	0.45	0.30	0.25	0.19	0.15	0.10

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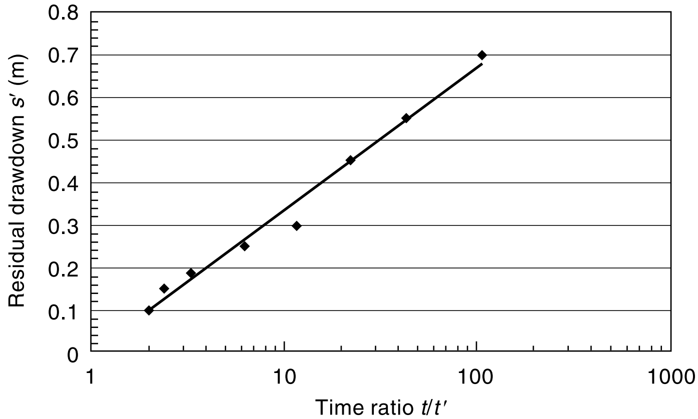

Fig. 9.21 Plot of residual drawdown against time ratio (t/t')—Example 9.12

A best fitting straight line through the plotted points is given by the equation $s' = 0.1461 \ln (t/t') - 0.0027$

By Eq. (9.60), Slope of best fit line $= 0.1461 = \dfrac{Q}{4\pi T}$

$$T = \frac{1200}{0.1461 \times 4\pi} = 654 \text{ m}^2/\text{day}$$

9.11 WELL LOSS

In a pumping artesian well, the total drawdown at the well s_w, can be considered to be made up of three parts:

1. Head drop required to cause laminar porous media flow, called *formation loss*, s_{wL} (Fig. 9.22);
2. drop of piezometric head required to sustain turbulent flow in the region nearest to the well where the Reynolds number may be larger than unity, s_{wt}; and
3. head loss through the well screen and casing, s_{wc}.

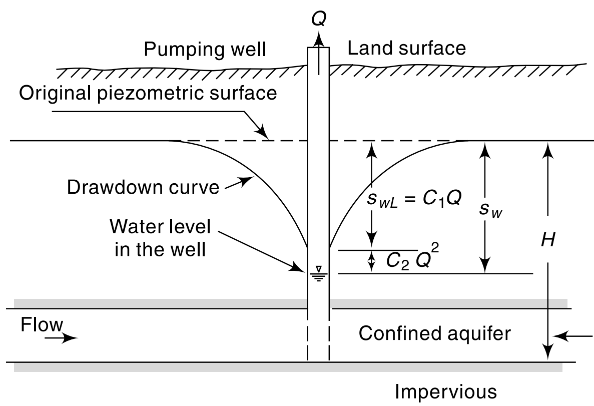

Fig. 9.22 Definition Sketch for Well Loss

Of these three,

$$s_{wL} \propto Q \quad \text{and} \quad (s_{wt} \text{ and } s_{wc}) \propto Q^2$$

thus
$$s_w = C_1Q + C_2Q^2 \tag{9.61}$$

where C_1 and C_2 are constants for the given well (Fig. 9.21). While the first term C_1Q is the formation loss the second terms C_2Q^2 is termed *well loss.*

The magnitude of a well loss has an important bearing on the pump efficiency. Abnormally high value of well loss indicates clogging of well screens, etc. and requires

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immediate remedial action. The coefficients C_1 and C_2 are determined by pump test data of drawdown for various discharges.

9.12 SPECIFIC CAPACITY

The discharge per unit drawdown at the well (Q/s_w) is known as *specific capacity* of a well and is a measure of the performance of the well. For a well in a confined aquifer under equilibrium conditions and neglecting well losses, by Eq. (9.48).

$$\frac{Q}{s_w} = \frac{2\pi}{\ln R/r_w} T \qquad \text{i.e.} \qquad Q/s_w \propto T$$

However, for common case of a well discharging at a constant rate Q under unsteady drawdown conditions, the specific capacity is given by

$$\frac{Q}{s_w} = \frac{1}{\frac{1}{4\pi T}\ln\frac{2.25\,T\,t}{r_w^2 \cdot S} + C_2 Q} \qquad (9.62)$$

where t = time after the start of pumping. The term C_2Q is to account for well loss. It can be seen that the specific capacity depends upon T, s, t, r_w and Q. Further, for a given well it is not a constant but decreases with increases in Q and t.

9.13 RECHARGE

Addition of surface water to zone of saturation is known as *recharge*. Recharge taking place naturally as a part of hydrologic cycle is called *natural recharge* while the process of increasing infiltration of surface water to groundwater systems by altering natural conditions is known as *artificial recharge*.

Natural Recharge

The amount of precipitation that infiltrates into the soil and reaches the zone of saturation is an important component of natural recharge. Seepage from irrigated lands is another important component of recharge of groundwater. In this process the infiltration phase is natural while the supply of water to the irrigated lands is through artificial means and as such it is sometimes called as *incidental recharge*. Other means of natural recharge are seepage from reservoirs; rivers, streams and canals; and other water bodies. Estimation of recharge rates of aquifers is an important component of groundwater resource estimation and in proper utilization of groundwater.

Artificial Recharge

The process of artificially enhancing the amount of water recharging the aquifer in a given location is known as artificial recharge. In the face of present-day large demands for groundwater artificial recharge is resorted to

- Conserve runoff
- Improve quantity of available groundwater
- Reduce or correct saltwater intrusion.

Various recharging methods commonly adopted are

- Spreading [Flooding, Basin, Ditch, Pit & Channel]
- Through injection wells
- Induced recharge from surface water bodies

- Subsurface dykes
- Percolation tanks, Check dams, Nala bunds and other watershed treatment methods.

Estimation of Recharge

Groundwater Resource Estimation Committee[11] (GEC-97) recommends two approaches to assessment of recharge to groundwater. These are (i) Groundwater level fluctuation method, and (ii) Rainfall infiltration factor method.

(1) Groundwater Level Fluctuation and Specific Yield Method

In this method the groundwater level fluctuations over a period (usually a monsoon season) is used along with the specific yield to calculate the increase in storage in the water balance equation. Thus for a given area of extent A (usually a watershed), for a water level fluctuation of h during a monsoon season,

$$h\, S_y A = R_G - D_G - B + I_s + I \tag{9.63}$$

where S_y = specific yield
R_G = gross recharge due to rainfall and other sources
D_G = gross water draft
B = base flow into the stream from the area
I_s = recharge from the stream into the ground water body
I = net ground water flow into the area across the boundary (i.e. inflow – outflow)

Writing $R = R_G - B + I + I_s$

Eq. 9.63 would be $R = h\, S_y A - D_G$ (9.64)

where R = possible recharge, which is gross recharge minus the natural recharge of the area, and would consist of other recharge factors as

$$R = R_{rf} + R_{gw} + R_{wi} + R_t \tag{9.65}$$

where R_{rf} = recharge from rainfall
R_{gw} = recharge from irrigation in the area (includes both surface and ground water sources)
R_{wi} = recharge from water conservation structures
R_t = recharge from tanks and ponds

Computations, using Eq. (9.63) through (9.65) are usually based on the monsoon season rainfall and corresponding groundwater fluctuation covering a span of 30 to 50 years to obtain normal monsoon recharge due to rainfall. The recharge in non-monsoon months is taken as zero if rainfall in non-monsoon months is less than 10% of normal annual rainfall. The computation for calculating the total annual recharge is carried out for both monsoon months and non-monsoon months and the total annual recharge is obtained as a sum of these two.

The specific yield for various hydrogeologic conditions in the country is estimated through norms given in Table 9.5.

(2) Rainfall Infiltration Method In areas where ground water level monitoring is not adequate in space and time, rainfall infiltration method may be used. The recharge from rainfall in monsoon season is taken as a percentage of normal monsoon rainfall in the area.

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Thus $R_{rf} = f\,A\,P_{nm}$ (9.66)

where R_{rf} = recharge from rainfall in monsoon season

f = rainfall infiltration factor

P_{nm} = normal rainfall in monsoon season

A = area of computation for reacharge.

Table 9.5 Norms for Specific Yield (S_y in percentage)

No.	Description of the area	Recommended value	Minimum value	Maximum value
1	**Alluvial areas**			
	Sandy alluvium	16	12	20
	Silty alluvium	10	8	12
	Clayey alluvium	6	4	8
2	**Hard Rock Areas**			
	Weathered granite, gneiss and schist			
	• with low clay content	3.0	2.0	4.0
	• with significant clay content	1.5	1.0	2.0
	Weathered or vesicular, jointed basalt	2.0	1.0	3.0
	Laterite	2.5	2.0	3.0
	Sandstone	3.0	1.0	5.0
	Quartzite	1.5	1.0	2.0
	Limestone	2.0	1.0	3.0
	Karstified limestone	8.0	5.0	15.0
	Phyllites, Shales	1.5	1.0	2.0
	Massive, poorly fractured rock	0.3	0.2	0.5

The norms for the rainfall factor f for various hydrogeological situations in the country are given in the following table.

Table 9.6 Norms for Selection of Rainfall Factor f

No.	Area	Value of f in percentage		
		Recommended value	Minimum value	Maximum value
1	**Alluvial areas**			
	• Indo-Gangetic and inland areas	22	20	25
	• East coast	16	20	18
	• West coast	10	8	12
2	**Hard Rock Areas**			
	• Weathered granite, gneiss and schist with low clay content	11	10	12
	• Weathered granite, gneiss and schist with significant clay content	8	5	9
	• Granulite facies like charnokite etc.	5	4	6
	• Vesicular and jointed basalt	8	5	9

(*Contd.*)

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(*Contd.*)

• Weathered basalt	5	4	6
• Laterite	13	12	14
• Semi-consolidated sand stone	7	6	8
• Consolidated sand stone, quartzite, limestone (except cavernous limestone)	7	6	8
• Phyllites, Shales	12	10	14
• Massive poorly fractured rocks	6	5	7

The same factors are used for non-monsoon months also with the condition that the recharge is taken as zero if the normal rainfall in non-monsoon season is less than 10% of normal annual rainfall.

Recharge from sources other than rainfall are also estimated by using appropriate factors (e.g., Tables 9.7, 9.8 and 9.9). The total recharge is obtained as the sum of recharge from rainfall and recharge from other sources.

Recharge due to Seepage from Canals When actual specific values are not available, the following norms may be adopted:

Table 9.7 Recharge due to Seepage from Canals

1	Unlined canals in sandy soils with some silt content	• 1.8 to 2.5 cumec/million sq.m of wetted area *or* • 15–20 ha.m/day/million sq. m of wetted area
2	Unlined canals in normal soils with some silt content	• 3 to 3.5 cumec/million sq. m of wetted area *or* • 25–30 ha.m/day/million sq. m of wetted area
3	Lines canals and canals in hard rock areas	• 20% of above values for unlined canals

Recharge from Irrigation The recharge due to flow from irrigation may be estimated, based on the source of irrigation (ground water or surface water), the type of crop and the depth of water table below ground level through the use of normal given below:

Table 9.8 Recharge from Irrigation

Source of Irrigation	**Type of Crop**	**Recharge as percentage application**		
		Water table below Ground level		
		< 10 m	**10–25 m**	**> 25 m**
Ground water	Non-paddy	25	15	5
Surface water	Non-paddy	30	20	10
Ground water	Paddy	45	35	20
Surface water	Paddy	50	40	25

Recharge from Water Harvesting Structures The following norms are commonly followed in the estimation of recharge from water harvesting structures:

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Table 9.9 Recharge factors for Tanks and other Water Harvesting Structures

S. No.	Structure	Recharge Factor
1	Recharge from Storage Tanks/ Ponds	1.4 mm/day for the period in which the tank has water, based on the average area of water spread. If the data on average water spread is not available, 60% of the maximum water spread area may be used.
2	Recharge from Percolation Tanks	50% of gross storage, considering the number of fillings. Half this value of recharge is assumed to be occurring during monsoon season.
3	Recharge due to Check dams and Nala bunds.	50% of gross storage. Half the value of recharge is assumed to be occurring during monsoon season.

Detailed procedure for the estimation of Groundwater resources of an area under Indian conditions available in Ref. 11.

9.14 GROUNDWATER RESOURCE

The quantum of groundwater available in a basin is dependent on the inflows and discharges at various points. The interrelationship between inflows, outflows and accumulation is expressed by the water budget equation

$$\Sigma I \, \Delta t - \Sigma Q \, \Delta t = \Delta S \tag{9.67}$$

where $\Sigma I \, \Delta t$ represents all forms of recharge and includes contribution by precipitation; infiltration from lakes, streams and canals; and artificial recharge, if any, in the basin.

$\Sigma Q \, \Delta t$ represents the net discharge of groundwater from the basin and includes pumping, surface outflows, seepage into lakes and rivers and evapotrans-piration. ΔS indicates the change in the groundwater storage in the basin over a time Δt.

Considering a sufficiently long time interval, Δt of the order of a year, the capability of the groundwater storage to yield the desired demand and its consequences can be estimated. It is obvious that too large a withdrawal than what can be replenished naturally leads ultimately to the permanent lowering of the groundwater table. This in turn leads to problems such as drying up of open wells and surface storages like swamps and ponds and change in the characteristics of vegetation that can be supported by the basin. Similarly, too much of recharge and scanty withdrawal or drainage leads to waterlogging and consequent decrease in the productivity of lands.

The maximum rate at which the withdrawal of groundwater in a basin can be carried without producing undesirable results is termed *safe yield*. This is a general term whose implication depends on the desired objective. The "undesirable" results include (i) permanent lowering of the groundwater table or piezometric head, (ii) maximum drawdown exceeding a preset limit leading to inefficient operation of wells, and (iii) salt-water encroachment in a coastal aquifer. Depending upon what undesirable effect is to be avoided, a safe yield for a basin can be identified.

The permanent withdrawal of groundwater from storage is known as *mining* as it connotes a depletion of a resource in a manner similar to the exploitation of mineral resource.

The total groundwater resource of a region can be visualized as being made up of two components: *Dynamic resource* and *Static resource*. The dynamic resource represents the safe yield, which is essentially the annual recharge less the un-avoidable natural discharge. The static resource is the groundwater storage available in the pores of the aquifer and its exploitation by mining leads to permanent depletion. Generally the static resource is many times larger than the dynamic resource. However, mining is resorted to only in case of emergencies such as droughts etc. and in exceptional cases of planned water resources development. In essence, it is a resource to be used in emergency. As such, the utilisable groundwater resource of a region is the safe yield at a given state of development. It is often said, in a general sense, that the water resource is a replenishable resource. So far as the groundwater resource is concerned, only the dynamic component is replenishable and the static component is non-replenishable.

The annual utilisable groundwater resource of a region is computed by using the water budget method. The total annual recharge is made up of:

- rainfall recharge
- seepage from canals
- deep percolation from irrigated areas
- inflow from influent streams etc.
- recharge from tanks, lakes, submerged lands, and
- artificial recharge schemes, if any.

The groundwater losses from aquifers occur due to

- outflow to rivers
- transpiration by trees and other vegetation
- evaporation from the water table

The difference between the enumerated annual recharges and the losses as above is the annual groundwater resource which is available for irrigation, domestic and industrial uses. It should be noted that with the growth in the expansion of canal irrigation the groundwater resources also grow. Further, increase in the recharge of surface waters through artificial methods would also enhance the groundwater resources.

The National water policy (1987) stipulates:

- Exploitation of the groundwater resources should be so regulated as not to exceed the recharging possibilities, as also to ensure social equity. Groundwater recharge projects should be developed and implemented for augmenting the available supplies.
- There should be periodical reassessment on a scientific basis of the groundwater resources taking into consideration the quality of the water available and economic viability.

Categories of Groundwater Development

The groundwater development in areas are categorized as safe, semi-critical and critical based on the stage of groundwater development and long-term trend of pre and post-monsoon groundwater levels.

Category	**% of groundwater development**	**Long term decline of pre & post-monsoon groundwater levels**
Safe	< 70%	Not significant
Semi-critical	70% to 90%	Significant
Critical	90% to 100%	Significant
Over exploited	> 100%	Significant

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Groundwater Resources of India

The National Commission on Agriculture (1976) estimated the groundwater resources at 350 km^3, of which 260 km^3 was available for irrigation. The groundwater over exploitation committee (1979) estimated the groundwater potential as 467.9 km^3. The Groundwater estimation committee (1984) suggested a suitable methodology for estimation of groundwater. Using these norms CGWB (1955) has estimated the total replenishable groundwater potential of the country (dynamic) at 431.89 km^3. The statewise and basinwise estimates of dynamic groundwater (fresh) resource made by CGWB (1995) are given in Tables 9.10 and 9.11.

Table 9.10 State-wise Dynamic Fresh Groundwater Resource

S. No.	States	Total Replenishable Groundwater Resource from Normal Natural Recharge	Total Replenishable Groundwater Resource due to Recharge Augmentation from Canal Irrigation	Total Replenishable Groundwater Resource
		Km^3 per year	Km^3 per year	Km^3 per year
1	Andhra Pradesh	20.03	15.26	35.29
2	Arunachal Pradesh	1.44	0.00	1.44
3	Assam	24.23	0.49	24.72
4	Bihar	28.31	5.21	33.52
5	Goa	0.18	0.03	0.21
6	Gujarat	16.38	4.00	20.38
7	Haryana	4.73	3.80	8.53
8	Himachal Pradesh	0.29	0.28	.037
9	Jammu and Kashmir	2.43	2.00	4.43
10	Karnataka	14.18	2.01	16.19
11	Kerala	6.63	1.27	7.90
12	Madhya Pradesh	45.29	5.60	50.89
13	Maharashtra	33.40	4.47	37.87
14	Manipur	3.15	0.00	3.15
15	Meghalaya	0.54	0.00	0.54
16	Mizoram		Not Assessed	
17	Nagaland	0.72	0.00	0.72
18	Orissa	16.49	3.52	20.01
19	Punjab	9.47	9.19	18.66
20	Rajasthan	10.98	1.72	12.70
21	Sikkim		Not Assessed	
22	Tamil Nadu	18.91	7.48	26.39
23	Tripura	0.57	0.10	0.67
24	Uttar Pradesh	63.43	20.39	83.82
25	West Bengal	20.30	2.79	23.09
26	Union Territories	0.35	0.05	0.40
	Total	**342.43**	**89.46**	**431.89**

[*Source: Ref. 12*]

Table 9.11 Basinwise Dynamic Fresh Groundwater Resource[12] (Unit: km^3/year)

S. No.	River Basin	Total Replenishable Groundwater Resource from Normal Natural Recharge	Total Replenishable Groundwater Resource due to Recharge Augmentation from Canal Irrigation	Total Replenishable Groundwater Resource
1	Indus	14.29	12.21	26.50
2	Ganga-Brahmaputra-Meghna Basin			
	2a Ganga sub-basin	136.47	35.10	171.57
	2b Brahmaputra sub-basin and	25.72	0.83	26.55
	2c Meghna (Barak) sub-basin	8.52	0.00	8.52
3	Subarnarekha	1.68	0.12	1.80
4	Brahmani-Baitarani	3.35	0.70	4.05
5	Mahanadi	13.64	2.86	16.50
6	Godavari	33.48	7.12	40.60
7	Krishna	19.88	6.52	26.40
8	Pennar	4.04	0.89	4.93
9	Cauvery	8.79	3.51	12.30
10	Tapi	6.67	1.6	8.27
11	Narmada	9.38	1.42	10.80
12	Mahi	3.50	0.50	4.00
13	Sabarmati	2.90	0.30	3.20
14	West flowing rivers of Kutchch and Saurashtra	9.10	2.10	11.20
15	West flowing rivers south of Tapi	17.70	2.15	15.55
16	West flowing rivers between Mahanadi and Godavari			
17	East flowing rivers between Godavari and Krishna	[12.82]	[5.98]	[18.80]
18	East flowing rivers between Krishna and Pennar			
19	East flowing rivers between Pennar and Cauvery	[18.20]	[5.55]	[12.65]
20	East flowing rivers south of Cauvery			
21	Area North of Ladakh no draining into India		Not Assessed	
22	Rivers draining into Bangladesh		Not Assessed	
23	Rivers draining into Myanmar		Not Assessed	
24	Drainage areas of Andaman, Nicobar and Lakshadweep islands		Not Assessed	
	Total	**342.43**	**89.46**	**431.89**

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It is seen from Tables 9.10 and 9.11 though the rainfall is the principal source of recharge, the contribution of canal seepage and irrigation return flows has also been significant in some states. The contributions are more than 40% in states of Punjab, Haryana, J&K and Andhra Pradesh.

Groundwater extraction by individuals, organizations and local bodies has become a common phenomenon. Consequently, there is considerable development and utilization of available groundwater sources. Further, the level of extraction has reached critical or over-exploitation level in several pockets in many states. Table 9.12 gives a list of states with considerable exploitation groundwater resources.

Table 9.12 States with more than 40% Groundwater Development[12]

State	Percent area over-exploited	Level of Groundwater development
Punjab	52	94
Haryana	42	84
Rajasthan	19	51
Tamil Nadu	14	61
Gujarat	6	42

Utilizable Groundwater Resources

As seen from Table 9.10 or 9.11, the sum total of potential for natural recharge from rainfall and due to recharge augmentation from canal irrigation system in the country 431.9 km^3/year. The utilizable groundwater potential is calculated as 396 km^3/year as below:

1. Total replenishable groundwater potential = 432 km^3/year
2. Provision for domestic drinking water and other uses @ 15% of item 1 = 71 km^3/year
3. Utilizable groundwater resource for irrigation @ 90% of (item 1 – item 2) = 325 km^3/year
4. Total utilizable dynamic groundwater resource (Sum of items 2 and 3) = 396 km^3/year

9.15 GROUNDWATER MONITORING NETWORK IN INDIA

The Central Groundwater Board monitors the ground water levels from a network of about 15000 stations (mostly dug wells selected from existing dug wells evenly distributed throughout in the country). Dug wells are being replaced by piezometers for water level monitoring. Measurements of water levels are taken at these stations four times in a year in the months of January, April/May, August and November. The groundwater samples are also collected during April/May measurements for chemicals analyses every year.

The data so generated are used to prepare maps of groundwater level depths, water level contours and changes in water levels during different periods and years. Deeper groundwater level of over 50 metres is observed in Peidmont aquifer in Bhabar belt in foot hills of Himalayas. In Western Rajasthan ground water levels have depths ranging from 20 to 100 metres. In peninsular region, water levels range from 5 to 20 metres below land surface.

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References

1. Bauwer, H., *Groundwater Hydrology.* McGraw-Hill' Kogakusha, Tokyo, 1978.
2. Bear, J., *Hydraulics of Groundwater,* McGraw-Hill, New York, 1979.
3. Central Water Commission, *Water Resources of India,* CWC Pub. No. 30/88, April 1988, CWC, New Delhi, India.
4. Davis, S.N. and R.J. M. DeWiest, *Hydrogeology,* John Wiley and Sons; New York, 1966.
5. Halek, V. and J. Svec, *Groundwater Hydraulics,* Elsevier, Amsterdam, 1979.
6. Marino, M.A. and J.N. Luthin, *Seepage and Groundwater,* Elsevier; Amsterdam, 1982.
7. Polubarinova Kochina, P.Y., *Theory of Groundwater Movement,* English Trans. by R.J.M. Dewiest, Princeton Univ. Press, Princeton, 1962.
8. Taylor, D.W., *Fundamentals of Soil Mechanics,* John Wiley and Sons, New York, 1956.
9. Todd, D.K., *Groundwater Hydrology,* John Wiley and Sons, New York, 1958,
10. Walton, W.C., *Groundwater Resource Evaluation,* McGraw-Hill, Kogakusha, Tokyo, 1970.
11. Ministry of Water Resources, GOI, *Report of the Groundwater Resources Estimation Committee*, Groundwater Resource Estimation Methodology, New Delhi, June 1997.
12. Min. of Water Resources, GOI, *Report of the National Commission for Integrated Water Resources Development*, Vol. 1, New Delhi, Sept. 1999.

Revision Questions

9.1 Explain briefly the following terms as used in groundwater flow studies
(a) Specific yield (b) Storage coefficient
(c) Specific capacity (d) Barometric efficiency

9.2 Distinguish between
(a) Aquifer and aquitard (b) Unconfined aquifer and a leaky aquifer
(c) Influent and effluent streams (d) Water table and piezometric surface
(e) Specific capacity of a well and the specific yield of an aquifer

9.3 Explain the following
(a) Perched water table (b) Intrinsic permeability
(c) Bulk pore velocity (d) Well loss (e) Recharge

9.4 Discuss the geological formations in India which have potential as aquifers.

9.5 Explain the behaviour of water level in wells in confined aquifers due to changes in the atmospheric pressure.

9.6 Develop the equation relating the steady state discharge from a well in an unconfined aquifer and depths of water table at two known positions from the well. State clearly all the assumptions involved in your derivation.

9.7 What are Dupit's assumptions? Starting from an elementary prism of fluid bounded by a water table, show that for the steady one-dimensional unconfined groundwater flow with a recharge rate R, the basic differential equation is

$$\frac{\partial^2 h}{\partial x^2} = -\frac{2R}{K}$$

where K = permeability of the porous medium.

9.8 Sketch a typical infiltration gallery. Calculate the discharge per unit length of the infiltration gallery by making suitable assumptions. State clearly the assumptions made.

9.9 Derive the basic differential equation of unsteady groundwater flow in a confined aquifer. State clearly the assumptions involved.

9.10 Describe a procedure by using Jacob's method to calculate the aquifer parameters of a confined aquifer by using the well pumping test data.

9.11 Describe the recovery test to estimate the transmissivity of a confined aquifer.

9.12 The aquifer properties S and T of a confined aquifer in which a well is driven are known. Explain a procedure to calculate the drawdown at a location away from the well at any instant of time after the pump has started.

9.13 Explain briefly

(a) Safe yield of an aquifer (b) Mining of water

(c) Recharge estimation (d) Groundwater estimation

9.14 Discuss the principle of recuperation test of an open well.

9.15 What are the commonly used methods to assess the recharge of groundwater in an area? Explain briefly any one of the methods.

9.16 Describe the groundwater resources of india and its utilization.

PROBLEMS

9.1 In a laboratory test of an aquifer material a fully saturated sample of volume 5 litres was taken and its initial weight of 105 N was recorded. When allowed to drain completely it recorded a weight of 97 N. The sample was then crushed, dried and then weighed. A weight of 93 N was recorded at this stage. Calculate the specific yield and relative density of the solids. (Assume unit weight of water γ= 9.79 kN/m^3)

9.2 A confined aquifer is 25 m thick and 2 km wide. Two observation wells located 2 km apart in the direction of flow indicate heads of 45 and 39.5 m. If the coefficient of permeability of the aquifer is 30 m/day, calculate (a) the total daily flow through the aquifer and (b) the piezometric head at an observation well located 300 m from the upstream well.

9.3 In a field test a time of 6 h was required for a tracer to travel between two observation wells 42 m apart. If the difference in water-table elevations in these wells were 0.85 m and the porosity of the aquifer is 20% calculate the coefficient of permeability of the aquifer.

9.4 A confined aquifer has a thickness of 30 m and a porosity of 32%. If the bulk modulus of elasticity of water and the formation material are 2.2×10^5 and 7800 N/cm^2 respectively, calcuate (a) the storage coefficient, and (b) the barometric efficiency of the aquifer.

9.5 An extensive aquifer is known to have a groundwater flow in N 30° E direction. Three wells *A*, *B* and *C* are drilled to tap this aquifer. The well *B* is to East of *A* and the well *C* is to North of *A*. The following are the data regarding these wells:

Distance (m)	Well	Ground surface elevation (m above datum)	Water table elevation (m above datum)
	A	160.00	157.00
AB = 800 m	*B*	159.00	156.50
AC = 2000 m	*C*	158.00	?

Estimate the elevation of water table at well *C* when the wells are not pumping.

9.6 A confined stratified aquifer has a total thickness of 12 m and is made up of three layers. The bottom layer has a coefficient of permeability of 30 m/day and a thickness of 5.0 m. The middle and top layers have permeability of 20 m/day and 45 m/day respectively and are of equal thickness. Calculate the transmissivity of the confined aquifer and the equivalent permeability, if the flow is along the stratification.

9.7 A pipe of 1.2 m diameter was provided in a reservoir to act as an outlet. Due to disuse, it was buried and completely clogged up for some length by sediment. Measurements indicated the presence of fine sand (K_1 = 10 m/day) deposit for a length of 100 m, at the upstream end and of coarse sand (K_2 = 50 m/day) at the downstream end for a length of 50 m. In between these two layers the presence of silty sand (K_3 = 0.10 m/day) for some length is identified. For a head difference of 20 m on either side of the clogged length the seepage discharge is found to be 0.8 m^3/day. Estimate the length of the pipe filled up by silty sand.

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9.8 A confined horizontal aquifer of thickness 15 m and permeability $K = 20$ m/day, connects two reservoirs M and N situated 1.5 km apart. The elevations of the water surface in reservoirs M and N measured from the top of the aquifer, are 30.00 m and 10.00 m respectively. If the reservoir M is polluted by a contaminant suddenly, how long will it take the contaminant to reach the reservoir N? Assume the porosity of the aquifer $n = 0.30$.

9.9 An Infiltration gallery taps an unconfined aquifer ($K = 50$ m/day) situated over a horizontal impervious bed (Fig. 9.23). For the flow conditions shown, estimate the discharge collected per unit length of the gallery.

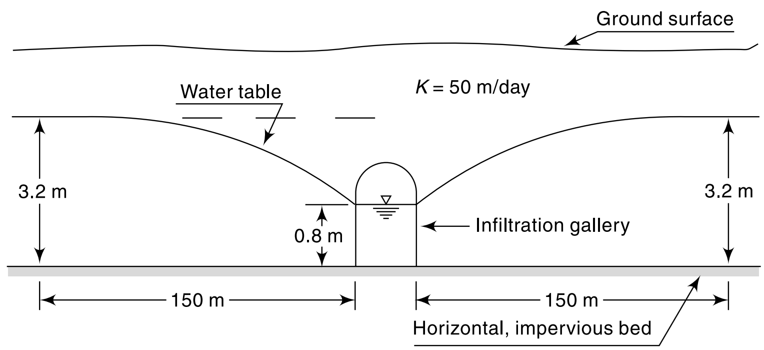

Fig. 9.23 Schematic Layout of Problem 9.9

9.10 An unconfined aquifer has infiltration of irrigation water at a uniform rate R at the ground surface. Two open ditches, as shown in Fig. 9.24, keep the water table in equilibrium. Show that the spacing L of the drains is related as

$$L^2 = \frac{4K}{R}[h_m^2 - h_0^2 + 2D(h_m - h_0)]$$

where K = coefficient of permeability of the aquifer. Neglect the width of drains. [*Note:* This relation is known as *Hooghoudt's* equation for either open ditch or subsurface drains. When $h_0 = 0$ and $D = 0$, this equation reduces to Eq. (9.44).]

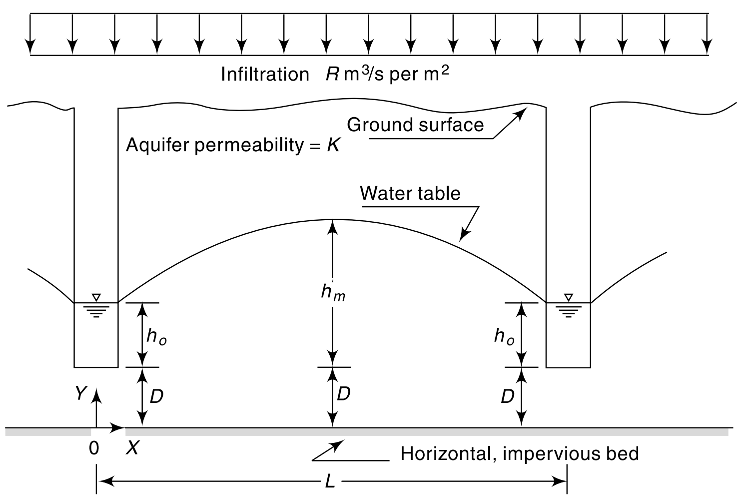

Fig. 9.24 Seepage to open ditches—Problem 9.10

9.11 A canal and a stream run parallel to each other at a separation distance of 400 m. Both of them completely penetrate an unconfined aquifer (K = 3.0 m/day) located above a horizontal impervious bed. The aquifer forms the separation land mass between the two water bodies. The water surface elevations in the canal and the stream are 5.0 m and 3.0 m, the datum being the top of the horizontal impervious layer. Estimate
(a) the uniform infiltration rate that will create a water table divide at a distance of 100 m from the canal.
(b) the elevation of the water table divide and
(c) the seepage discharges into the two water bodies.

9.12 Two rivers A and B run parallel to each other and fully penetrate the unconfined aquifer situated on a horizontal impervious base. The rivers are 4.0 km apart and the aquifer has a permeability of 1.5 m/day. In an year, the average water surface elevations of the rivers A and B, measured above the horizontal impermeable bed, are 12.00 m and 9.00 m respectively. If the region between the rivers received an annual net infiltration of 20 cm in that year, estimate
(a) the location of the groundwater table divide and
(b) the average daily groundwater discharge into the rivers A and B from the aquifer between them.

9.13 A 30-cm well completely penetrates an artesian aquifer. The length of the strainer is 25 m. Determine the discharge from the well when the drawdown at the pumping well is 4.0 m. The coefficient of permeability of the aquifer is 45 m/day. Assume the radius of influence of the well as 350 m.

9.14 A 20-cm dia tubewell taps an artesian aquifer. Find the yield for a drawdown of 3.0 m at the well. The length of the strainer is 30 m and the coefficient of permeability of the aquifer is 35 m/day. Assume the radius of influence as 300 m.
If all other conditions remain same, find the percentage change in yield under the following cases:
(a) The diameter of the well is 40 cm; (b) the drawdown is 6.0 m;
(c) the permeability is 17.5 m/day.

9.15 The discharge from a fully penetrating well operating under steady state in a confined aquifer of 35 m thickness is 2000 lpm. Values of drawdown at two observation wells 12 and 120 m away from the well are 3.0 and 0.30 m respectively. Determine the permeability of the aquifer.

9.16 A confined aquifer of thickness B has a fully penetrating well of radius of r_0, pumping a discharge Q at a steady rate. An observation well M is located at a distance R from the pumping well. Show that the travel time for water to travel from well M to the pumping well is

$$t = \frac{\pi B \eta}{Q}(R^2 - r_0^2) \quad \text{where } \eta = \text{Porosity of the aquifer.}$$

9.17 A 45-cm well penetrates an unconfined aquifer of saturated thickness 30 m completely. Under a steady pumping rate for a long time the drawdowns at two observation wells 15 and 30 m from the well are 5.0 and 4.2 m respectively. If the permeability of the aquifer is 20 m/day, determine the discharge and the drawdown at the pumping well.

9.18 A 30-cm well fully penetrates an unconfined aquifer of saturated depth 25 m. When a discharge of 2100 lpm was being pumped for a long time, observation wells at radial distances of 30 and 90 m indicated drawdown of 5 and 4 m respectively. Estimate the coefficient of permeability and transmissibility of the aquifer. What is the drawdown at the pumping well?

9.19 For conducting tests on a 30 cm diameter well in an unconfined aquifer two observation wells A and B are bored at distances 25 m and 40 m respectively from the centre of the pumping well. When water is pumped at a rate of 10 litres/s the water depth in the pumping well is 10.0 m above the horizontal impervious layer up to which the well is driven. The median grain size of the aquifer is 2.5 mm and the permeability is known to be 0.1 cm/s. Calculate (a) the depth of water above the impervious layer in the observation wells *A* and *B*; (b) the Reynolds number of flow at the pumping well and observation wells *A* and *B*. [Assume kinematic viscosity of water = 0.01 cm^2/s].

9.20 A 45-cm well in an unconfined aquifer of saturated thickness of 45 m yields 600 lpm under a drawdown of 3.0 m at the pumping well, (a) What will be the discharge under a drawdown of 6.0 m? (b) What will be the discharge in a 30-cm well under a drawdown of 3.0 m? Assume the radius of influence to remain constant at 500 m in both cases.

9.21 For conducting permeability tests in a well penetrating an unconfined aquifer, two observation wells *A* and *B* are located at distances 15 and 30 m respectively from the centre of the well. When the well is pumped at a rate of 5 lps, it is observed that the elevations of the water table above the impervious layer, up to which the well extends are 12.0 and 12.5 m respectively at *A* and *B*. Calculate the permeability of the aquifer in m/day.

9.22 Calculate the discharge in m^3/day from a tubewell under the following conditions:

Diameter of the well	=	45 cm
Drawdown at the well	=	12 m
Length of strainer	=	30 m
Radius of influence of the well	=	200 m
Coefficient of permeability	=	0.01 cm/s
Aquifer	=	unconfined

9.23 A fully penerating well of 30-cm diameter in an unconfined aquifer of saturated thickness 50 m was found to give the following drawdown-discharge relations under equilibrium condition.

Drawdown at the pumping well (m)	Discharge (lpm)
3.0	600
11.7	1800

If the radius of influence of the well can be assumed to be proportional to the discharge through the well, estimate the flow rate when the drawdown at the well is 6.0 m.

9.24 A 45-cm well in an unconfined aquifer was pumped at a constant rate of 1500 lpm. At the equilibrium stage the following drawdown values at two observation wells were noted:

Observation	Radial distance from pumping well (m)	Drawdown (m)
A	10	5.0
B	30	2.0

The saturated thickness of the aquifer is 45 m. Assuming the radius of influence to be proportional to the discharge in the pumping well, calculate:
(a) Drawdown at the pumping well; (b) transmissibility of the aquifer;
(c) drawdown at the pumping well for a discharge of 2000 lpm; and
(d) radius of influence for discharges of 1500 and 2000 lpm.

9.25 A 4.5 m diameter open well has a discharge of 30.0 m^3/h with a drawdown of 2.0 m. Estimate the (i) specific capacity per unit well area of the aquifer and (ii) discharge from a 5.0 m open well in this aquifer under a depression head of 2.5 m.

9.26 In a recuperation test of a 3.0 m diameter open well the water level changed from Elevation 114.60 m to 115.70 m in 120 minutes. If the water table elevation is 117.00 m, diameter (i) the specific capacity per unit well area of the aquifer and (ii) discharge in the well under a safe drawdown of 2.75 m.

9.27 During a recuperation test, the water in an open well as depressed by pumping by 2.5 m it recuperated 1.8 m in 80 minutes. Calculate the yield from a well of 4.0 m diamater under a depression heat of 3.0 m.

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9.28 The drawdown time data recorded at an observation well situated at a distance of 50 m from the pumping well is given below:

Time (min)	1.5	3	4.5	6	10	20	40	100
Drawdown (m)	0.15	0.6	1.0	1.4	2.4	3.7	5.1	6.9

If the well discharge is 1800 lpm, calculate the transmissibility and storage coefficients of the aquifer.

9.29 Estimate the discharge of a well pumping water from a confined aquifer of thickness 20 m with the following data:
Distance of observation well from the pumping well = 100 m
Drawdown at the observation well after 4 hours of pumping = 1.5 m
Drawdown at the observation well after 16 hours of pumping = 2.0 m
Storage coefficient, $S = 0.0003$

9.30 A fully penetrating well in a confined aquifer is being pumped at a constant rate of 2000 Lpm. The aquifer is known to have a storage coefficient of 0.005 and transmissibility of 480 m^2/day. Find the drawdown at a distance of 3.0 m from the production well after (i) one hour and (ii) 8 hours after pumping.

9.31 A fully penetrating well in a confined aquifer is pumped at the rate of 60 m^3/h from an aquifer of storage coefficient and transmissibility 4×10^{-4} and 15 m^2/h respectively. Estimate the drawdown at a distance of 100 m after 8 hours of pumping.

9.32 A fully penetrating confined aquifer is pumped at a constant rate of 100 m^3/h. At an observation well located at 100 m from the pumping well the drawdown where observed to be 0.65 m and 0.80 m after one and two hours of pumping respectively. Estimate the formation constants of the aquifer.

9.33 A well in a confined aquifer was pumping a discharge of 40 m^3/hour at uniform rate. The pump was stopped after 300 minutes of running and the recovery of drawdown was measured. The recovery data is shown below. Estimate the transmissibility of the aquifer.

Time since stopping of the pump (min)	1	3	5	10	20	40	90	150	200
Residual drawdown (m)	1.35	1.18	1.11	1.05	0.98	0.88	0.80	0.70	0.68

Objective Questions

9.1 A geological formation which is essentially impermeable for flow of water even though it may contain water in its pores is called
(a) aquifer (b) aquifuge (c) aquitard (d) aquiclude

9.2 An aquifer confined at the bottom but not at the top is called
(a) Semiconfined aquifer (b) unconfined aquifer
(c) confined aquifer (d) perched aquifer

9.3 A stream that provides water to the water table is termed
(a) affluent (b) influent (c) ephemeral (d) effluent

9.4 The surface joining the static water levels in several wells penertating a confined aquifer represents
(a) water-table surface (b) capillary fringe
(c) piezometric surface of the aquifer (d) cone of depression.

9.5 Flowing artesian wells are expected in areas where
(a) the water table is very close to the land surface (b) the aquifer is confined
(c) the elevation of the piezometric head line is above the elevation of the ground surface
(d) the rainfall is intense

9.6 Water present in artesian aquifers is usually
(a) at sub atmospheric pressure (b) at atmospheric pressure
(c) at 0.5 times the atmospheric pressure (d) above atmospheric pressure

9.7 The volume of water that can be extracted by force of gravity from a unit volume of aquifer material is called
(a) specific retention (b) specific yield
(c) specific storage (d) specific capacity

9.8 Which of the pairs of terms used in groundwater hydrology are not synonymous?
(a) Permeability and hydraulic conductivity (b) Storage coefficient and storativity
(c) Actual velocity of flow and discharge velocity
(d) Water table aquifer and unconfined aquifer

9.9 The permeability of a soil sample at the standard temperature of 20°C was 0.01 cm/s. The permeability of the same material at a flow temperature of 10° C is in cm/s
(a) < 0.01 (b) > 0.01
(c) = 0.01 (d) depends upon the porous material

9.10 A soil has a coefficient of permeability of 0.51 cm/s. If the kinematic viscosity of water is 0.009 cm^2/s, the intrinsic permeability in darcys is about
(a) 5.3×10^4 (b) 474 (c) 4.7×10^7 (d) 4000

9.11 Darcy's law is valid in a porous media flow if the Reynolds number is less than unity. This Reynolds number is defined as
(a) (discharge velocity × maximum grain size)/μ
(b) (actual velocity × average grain size)/ν
(c) (discharge velocity × average grain size)/ν
(d) (discharge velocity × pore size)/ν

9.12 Two observation wells penetrating into a confined aquifer are located 1.5 km apart in the direction of flow. Heads of 45 m and 20 m are indicated at these two observation wells. If the coefficient of permeability of the aquifer is 30 m/day and the porosity is 0.25, the time of travel of an inert tracer from one well to another is about
(a) 417 days (b) 500 days (c) 750 days (d) 3000 days

9.13 A sand sample was found to have a porosity of 40%. For an aquifer of this material, the specific yield is
(a) = 40% (b) > 40% (c) < 40% (d) dependent on the clay fraction

9.14 An unconfined aquifer of porosity 35%, permeability 35 m/day and specific yield of 0.15 has an area of 100 km^2. The water table falls by 0.20 m during a drought. The volume of water lost from storage in Mm^3 is
(a) 7.0 (b) 3.0 (c) 4.0 (d) 18.0

9.15 The unit of intrinsic permeability is
(a) cm/day (b) m/day (c) darcy/day (d) cm^2

9.16 The dimensions of the storage coefficient S are
(a) L^3 (b) LT^{-1} (c) L^3/T (d) dimensionless

9.17 The dimensions of the coefficient of transmissibility T are
(a) L^2/T (b) L^3T^2 (c) L/T^2 (d) dimensionless

9.18 The coefficient of permeability of a sample of aquifer material is found to be 5 m/day in a laboratory test conducted with water at 10°C. If the kinematic viscosity of water at various temperatures is as below:

Temp in °C	10	20	30
$\nu(m^2/s)$	1.30×10^6	1.00×10^6	0.80×10^6

the standard value of the coefficient of permeability of the material, in m/day, is about
(a) 4.0 (b) 5.0 (c) 6.5 (d) 9.0

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9.19 A stratified unconfined aquifer has three horizontal layers as below

Layer	Coefficient of permeability (m/day)	Depth (m)
1	6	2.0
2	16	4.0
3	24	3.0

The effective vertical coefficient of permeability of this aquifer, in m/day, is about
(a) 13 (b) 15 (c) 24 (d) 16

9.20 An aquifer confined at top and bottom by impervious layers is stratified into three layers as follows:

Layer	Thickness (m)	Permeability (m/day)
Top layer	3.0	30
Middle layer	2.0	10
Bottom layer	5.0	20

The transmissibility of the aquifer in m^2/day is
(a) 6000 (b) 18.2 (c) 20 (d) 210

9.21 The specific storage is
(a) storage coefficient/aquifer depth (b) specific yield per unit area
(c) specific capacity per unit depth of aquifer (d) porosity-specific detention

9.22 When there is an increase in the atmospheric pressure, the water level in a well penetrating a confined aquifer
(a) decreases (b) increases
(c) does not undergo any change
(d) decreases or increases depending on the elevation of the ground.

9.23 The specific capacity of a well is the
(a) volume of water that can be extracted by the force of gravity from unit volume of aquifer
(b) discharge per unit drawdown at the well
(c) drawdown per unit discharge of the well
(d) rate of flow through a unit width and entire thickness of the aquifer

9.24 In one-dimensional flow in an unconfined aquifer between two water bodies, when there is a recharge, the water table profile is
(a) a parabola (b) part of an ellipse
(c) a straight line (d) an arc of a circle

9.25 In one-dimensional flow in a confined aquifer between two water bodies the piezometric head line is
(a) a straight line (b) a part of an ellipse
(c) a parabola (d) an arc of a circle

9.26 For one-dimensional flow without recharge in an unconfined aquifer between two water bodies the steady water table profile is
(a) a straight line (b) a parabola (c) an ellipse (d) an arc of a circle

9.27 The discharge per unit drawdown at a well is known as
(a) specific yield (b) specific storage
(c) safe yield (d) specific capacity.

9.28 The specific capacity of a well in confined aquifer under equilibrium conditions and within the working limits of drawdown
(a) can be taken as constant (b) decreases as the drawdown increases
(c) increases as the drawdown increases
(d) increases or decreases depending upon the size of the well

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Chapter
10

Erosion and Reservoir Sedimentation

10.1 INTRODUCTION

Erosion, transportation and deposition of sediment in a watershed are natural processes which are intimately connected with the hydrologic processes. Soil and water conservation in watershed and reservoir sedimentation are important parameters affecting the success and economy of many water resources development activities in a basin. This chapter briefly deals with erosion, sediment yield and reservoir sedimentation aspects of the erosion phenomenon. This chapter is only a brief introduction to the topic. For details excellent treatises are available and Refs 2 and 5 contain some valuable source material on this topic.

Erosion is the wearing away of land. Natural agents such as water, wind and gravity are eroding the land surface since geologic times. Out of many erosion causing agents the role of water in detachment, transportation and deposition is indeed very significant. Since recent past, human activities like agricultural practice, mining, building activities, railway and road construction are contributing significantly to erosion of land surface. Water storage structures like reservoirs, tanks and ponds act as receptacles for deposition of eroded material.

10.2 EROSION PROCESSES

Processes

Erosion takes place in the entire watershed including the channels. During a rainfall event, when rain drops impact on a soil surface, the kinetic energy of the drops breaks the soil aggregates and detaches the particles in the impact area. The detached particles are transported by surface run off. Depending upon the flow conditions, topography and geometry of the channel etc. there may be some deposition of the eroded material enroute. Erosion takes place in various modes, which can be classified as follows:

Inter-Rill Erosion In this the detached particles due to raindrop impact are transported over small distances in surface flow of shallow depth without formation of elementary channels called *rills*. The mode of transport is essentially *sheet flow* and the inter-rill erosion from this mode is known as *sheet erosion.* Sheet erosion removes a thin covering of soil from large areas, often from the entire fields, more or less, uniformly during every rain which produces a run-off. The existence of sheet erosion is reflected in the muddy colour of the run-off from the fields.