Hydrology course

Forth year geology

Lecturer

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Lesson 1 Introduction to Groundwater

1.1What is Groundwater?

Groundwater is broadly defined as the water present in the zone of saturation below the ground. A precise and practical definition of groundwater is given in Lesson 2. The zone of saturation is technically called ‘aquifer’. Aquifers are significantly porous and permeable to supply water to wells and springs. On the other hand, water stored in ponds, lakes, rivers, streams, seas/oceans and other surface reservoirs is called surface water.

The term Hydrogeology or Subsurface Hydrology (popularly known as Groundwater Hydrology) is defined as the study of the occurrence, distribution, movement, and geological interaction of water in the earth’s crust, especially groundwater. A similar term ‘Geohydrology’ is sometimes used as a synonym for hydrogeology, although it more properly describes an engineering field dealing with subsurface fluid hydrology.

A ‘groundwater basin’ is defined as a hydrogeologic unit comprising one large aquifer or several connected and interrelated aquifers. It may or may not coincide with a physiographic unit. As we know that watershed/catchment or drainage basin is the basic hydrologic unit for managing surface water resources. Similarly, ‘groundwater basin’ is the basic unit for groundwater management. The modern concept of water management emphasizes that surface water and groundwater should be treated as a single resource and unlike traditional approach, both surface water and groundwater should be managed in an integrated manner at a basin or sub-basin scale.

1.2 Groundwater and the Water Cycle

Water perpetually circulates on the earth from the oceans to the atmosphere to land and back to the oceans; this is called water cycle or hydrologic cycle. Note that the term ‘hydrologic cycle’ literally means “Water-Science Cycle”, and hence the correct term to describe this cyclic movement of water in nature is water cycle, which should be used instead of widely-used term ‘hydrologic cycle’. The major pathways in the water cycle are schematically shown in Fig. 1.1. Thus, the water cycle describes how water moves into and out of various domains viz., atmosphere, land surface, subsurface (underground) and oceans. The main components of water cycle are precipitation, evaporation, transpiration, infiltration, surface runoff (overland flow and streamflow), and subsurface runoff (interflow, vadose-water flow and groundwater flow).

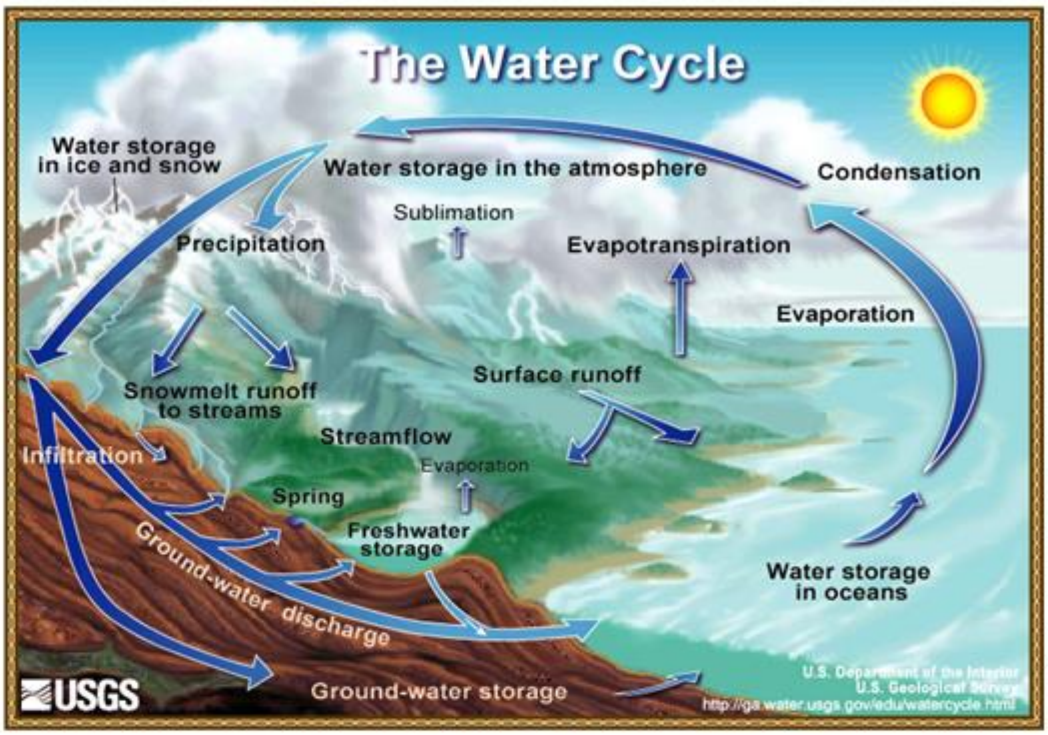

Fig. 1.1. Schematic diagram of the water cycle.

(Source: http://ga.water.usgs.gov)

Of the water falling on the land, a proportion quickly evaporates, some flows into streams or lakes as overland flow, and some infiltrates into the subsurface. Of the water entering the subsurface, some is transpired back into the atmosphere by plants, some is retained in the vadose zone, some reaches saturated zone (aquifer) as groundwater recharge, and the remaining water follows a subsurface pathway back to the land surface and oceans (Fig. 1.1). Note that water moving in the water cycle is neither gained nor lost, i.e., it is conserved (Input – Output = Change in Storage). Thus, the water cycle follows the principle of continuity.

Groundwater is found in aquifers (water-bearing geologic formations), which act as conduits for water transmission and as underground reservoirs for water storage. Practically, all groundwater originates as surface water. Water enters aquifers from the land surface or from surface water bodies through the vadose zone, and then it travels slowly within the aquifer for varying distances until it finally returns to the land surface by natural flow, plants, or humans (Fig. 1.2). The residence time of groundwater in the subsurface can vary from days to thousands of years (centuries or millennia) depending on the length of the flow path and the transmissivity of porous media.

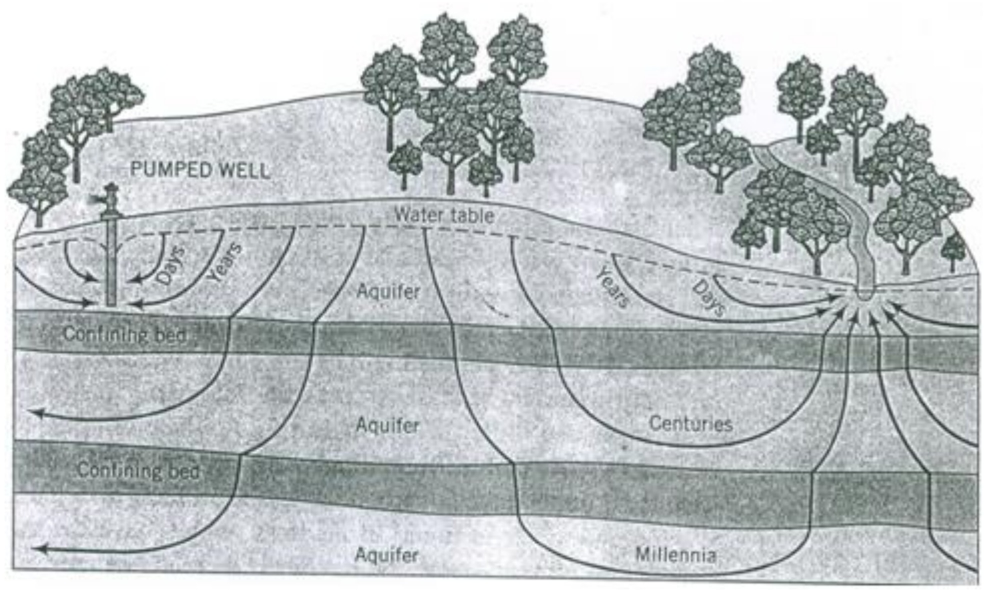

Fig. 1.2. Schematic of groundwater flow paths and residence time in multi-layered aquifer systems. (Source: Winter et al., 1998)

Principal sources of natural groundwater recharge are precipitation, streamflow, lakes and reservoirs, while the artificial sources of recharge are seepage from canals, return flow from irrigation, recharge from storage and percolation tanks, recharge due to check dams, and the water purposely applied to augment groundwater. The discharge of groundwater occurs when water emerges from underground (subsurface) as flow into streams, lakes or oceans (called ‘baseflow’), or as springs. Very shallow groundwater may return directly to the atmosphere by evapotranspiration. Pumping of wells constitutes the major artificial discharge of groundwater.

1.3 Importance of Groundwater

The study of groundwater is essential because of several reasons. Of the freshwater readily available for human use (approximately 1% of the liquid freshwater available on the earth), about 98% is groundwater and the remaining is surface water. Hence, groundwater serves as a major source of water supply to life (humans, animals and ecosystems) throughout the world. Because of its physical and chemical quality, groundwater provides a reliable source of water supply in both humid and arid/semi-arid regions of the world and during emergencies (e.g., droughts, earthquakes, etc.) as well as it sustains flow in rivers/streams and lakes during dry periods. Thus, groundwater is one of the most valuable natural resources of the earth, which supports human health, human livelihoods, socio-economic development, and ecological diversity.

Besides the above-mentioned vital roles, groundwater also influences the design and construction of engineering facilities such as dams, open-pit mines, tunnels, deep foundations, and geologic storage of nuclear wastes or carbon sequestration. Groundwater is also important due to its geologic role by supporting various geological processes such as the formation of soils and their alternation, the development of landslides, rock falls, channel networks and karst landscapes, oil formation and valuable mineral deposits. Thus, groundwater plays a variety of roles on a global scale, which make this resource so vital for human beings. However, the water resource and engineering aspects of groundwater hydrology are the major focus of practice, though the groundwater hydrology field has a rich relationship with other earth sciences.

Lesson 2 Occurrence of Groundwater

In order to understand the occurrence of groundwater and its vertical distribution, let's first consider the hydrological zones present below the ground (Fig. 2.1). The zone between the ground surface and the top of capillary fringe is called unsaturated zone (or, zone of aeration) which consists of voids (pores or interstices) partially filled with water and partially with air. Water is held at a pressure less than the atmospheric pressure in the unsaturated zone. The zone between bottom of the unsaturated zone and top of the water table is called capillary zone, wherein most voids are filled with water but the water is held at a pressure less than the atmospheric pressure. Finally, the zone extending from the water table to an impermeable layer is called saturated zone (or, zone of saturation), wherein all voids are completely filled with water. In this zone, water is held at a pressure greater than the atmospheric pressure, and hence it moves in a direction based on the contiguous hydraulic situation.

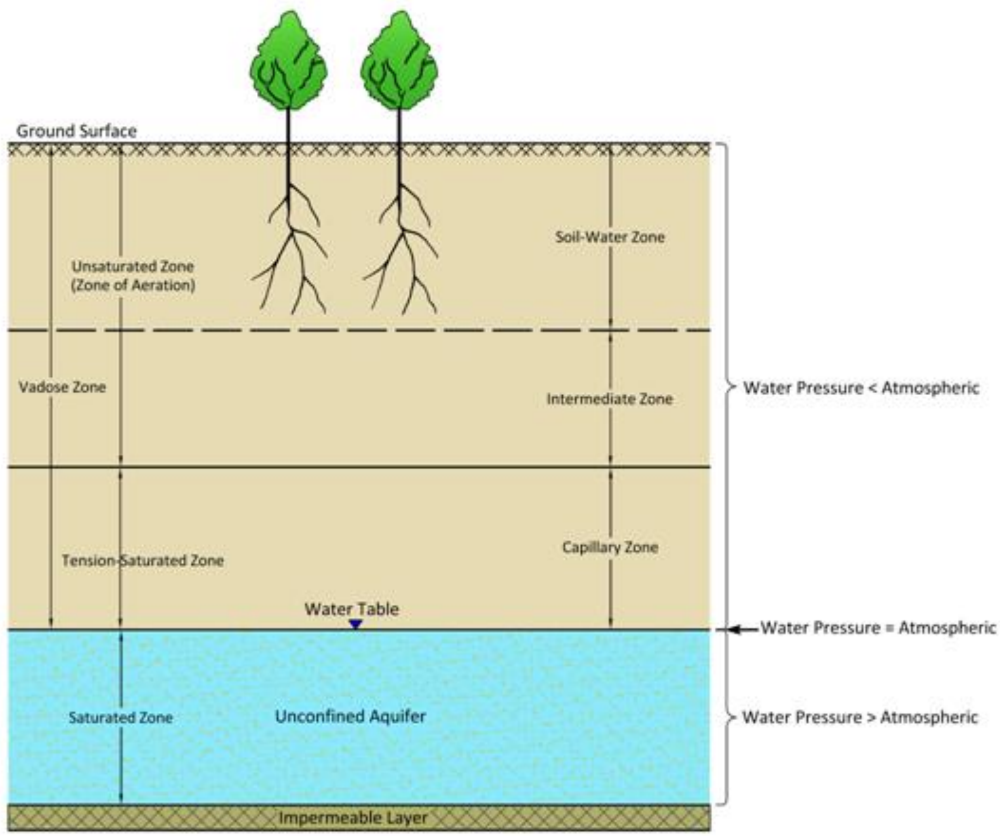

Fig. 2.1. Hydrologic zones below the ground.

(Source: Modified from Sanders, 1998)

The unsaturated zone can be further sub-divided into 'soil-water zone' and 'intermediate zone' (Todd, 1980). The zone between the ground surface and the top of water table is known as the vadose (shallow) zone. Thus, the vadose zone consists of unsaturated zone and capillary zone (also known as 'capillary fringe'). The water present in the vadose zone is called vadose water which is held at a pressure less than the atmospheric pressure. Hence, while this water is still able to move within the vadose zone due to matric potential and gravity, it cannot move out of the zone into wells, pits, or other water collection systems that are exposed to the atmospheric pressure. Note that the term vadose zone is technically more appropriate than the conventional term unsaturated zone. This is because portions of the vadose zone may actually be saturated, even though the pressure of water is below the atmospheric pressure. Hence, the term vadose zone has become popular and is widely used these days in the fields of groundwater hydrology and soil physics.

Broadly speaking, the water stored in the zone of saturation is called groundwater. Not all underground water is groundwater, rather only free water or gravitational water (the water that moves freely under the force of gravity into wells) constitutes the groundwater. Therefore, a precise and practical definition of groundwater is (Bouwer, 1978): "Groundwater is that portion of the water beneath the earth's surface, which can be collected through wells, tunnels, or drainage galleries, or which flows naturally to the earth's surface via seeps or springs". Depths to groundwater may range from 1 m or less to 1000 m or more. There are also places where groundwater does not exist at all.

2.2 Types of Subsurface Formations

In subsurface hydrology, the material present below the ground is normally called subsurface formation or subsurface deposit. Subsurface formations can be divided into two broad groups: (a) soil, and (b) geologic formations. As we know that the soil is formed by weathering of rocks. However, geologic formations can be consolidated, semi-consolidated and unconsolidated formations. Consolidated geologic formations are the rocks formed by cementation, compaction and recrystallisation, and their grains are tightly held together. They are also known as 'hard rocks'. Examples of consolidated geologic formations are igneous and metamorphic rocks such as granite, basalt and schist, and indurated sedimentary rocks such as sandstone, shale and limestone. Semi-consolidated geologic formations are sedimentary rocks wherein the induration process is incomplete and the primary porosity (intergranular porosity) is preserved to a varying degree. Among the semi-consolidated sedimentary rocks, sandstone is considered most productive because in an early stage of cementation, its primary porosity is very high like sand. On the other hand, unconsolidated geologic formations are comprised of non-indurated colluvial, alluvial, aeolian (wind-borne sediments), lacustrine, marine (coastal) and glacial deposits. These formations/deposits consist of sand, silt, clay, gravel and pebbles.

2.3 Aquifer and Confining Layers

From the groundwater viewpoint, geologic formations can be sub-divided into: (i) aquifer layers (or aquifers), and (ii) confining layers. Literally, aquifer means 'water-bearing formation'. In subsurface hydrology (hydrogeology), an aquifer is defined as "a single geologic formation or a group of geologic formations that can transmit and yield water in usable quantities". Aquifers are the target for all groundwater exploration and development programs. The intrinsic permeability (k) of aquifers is greater than 10-2 darcy.

Many types of geologic formations serve as aquifers. Unconsolidated geologic formations (e.g., alluvial deposits) and semi-consolidated geologic formations serve as aquifers because of primary porosity (i.e., intergranular porosity), while consolidated geologic formations (i.e., hard rocks) serve as aquifers primarily due to secondary porosity caused by fractures, fissures, solution cavities/channels, lava tubes, shrinkage cracks, etc. The geologic formations having potential for aquifers are: alluvial deposits, limestone, volcanic rocks, sandstone, and weathered igneous and metamorphic rocks. On the other hand, conglomerates, and solid igneous and metamorphic rocks mostly serve as bedrocks or confining layers. Clay, silt and coarser particles mixed with clay and/or silt are usually porous, but their pores are so small that they are practically regarded as semi-permeable or impermeable in most cases. They better serve as leaky or non-leaky confining layers. Note that the lithology, stratigraphy and structure of rock formations control the horizontal and vertical extent and the nature of aquifers or confining layers.

In hydrogeology, the geologic formations which are not aquifers are termed confining layers. A confining layer is defined as (Fetter, 2000): "A geologic formation having little or no intrinsic permeability". Confining layers could be grouped as 'leaky confining layers' or 'non-leaky confining layers' (Fetter, 2000) depending on whether they can contribute significant leakage through them or not. Confining layers have an intrinsic permeability (k) of less than 10-2 darcy; it is an arbitrary limit and depends on local conditions (Fetter, 2000). For example, in areas of clay (k = 10-4 darcy), a layer of silt of 10-2 darcy might be used to supply water to a small well. On the other hand, the same silty layer might be considered a confining layer, if it were found in an area of coarse gravels with k = 100 darcys. In fact, groundwater moves through most confining layers, but the rate of movement is very slow. Traditionally, confining layers are classified as aquitards, aquicludes and aquifuges. However, the terms ‘non-leaky confining layer’ and ‘leaky confining layer’ are becoming popular in subsurface hydrology in order to distinguish whether a confining layer is non-leaky or leaky in nature.

Aquiclude is defined as a geologic formation that can store significant amount of water but does not have the capability to transmit a significant amount of water. Clay is an ideal example of aquiclude.

Aquitard is defined as a geologic formation that can store some water as well as can transmit water at a relatively low rate compared to aquifers. Although an aquitard may not yield water economically, it can hold appreciable amounts of water. Sandy clay is an ideal example of aquitard.

On the other hand**, aquifuge** is defined as a geologic formation that can neither store nor transmit water. Solid granite is an ideal example of aquifuge.

Thus, aquifuge is essentially a non-leaky confining layer, whereas aquitards and aquicludes are essentially leaky confining layers. In practice, however, aquiclude is often considered as a non-leaky confining layer because leakage through aquicludes is generally very small which can be considered practically insignificant.

2.4 Origin and Age of Groundwater

Almost all groundwater can be considered as a part of the water cycle. Relatively small amount of groundwater may enter the water cycle from other origins, which is often called generic types of groundwater namely magmatic water, connate water, juvenile water and metamorphic water (Todd, 1980).

The water originated from magma is called magmatic water. The magmatic water available at relatively shallow depths (probably 3 to 5 km) is known as volcanic water and when it is available at deeper depths (>3 or 5 km), it is called plutonic water. Connate water is the water present in the interstices of fossils (remains of buried animals or that have hardened into rocks) and has been out of contact with the atmosphere for considerable time of a geologic period. This water might have been derived from oceanic or freshwater sources and is highly mineralized. Juvenile water is the water that has not earlier been a part of the hydrosphere. It is derived from magma or atmosphere. Finally, metamorphic water is the water associated with rocks during metamorphism process.

The water presently withdrawn from an aquifer (e.g., deep and extensive aquifers) might have entered the aquifer thousands of years ago. For instance, an analysis of the groundwater samples from deep wells in deserts of the United Arab Republic and Saudi Arabia indicated groundwater ages of 20,000 to 30,000 years (Todd, 1980). Investigation of the residence time of water under the ground (i.e., age of groundwater) is known as 'age-dating'. The radioisotopes tritium (H-3) and carbon-14 (C-14) have been found to be very useful for estimating the age of groundwater. Tritium is applicable for estimating groundwater residence times of up to 50 years, whereas carbon-14 is applicable for estimating groundwater residence times of several hundred years to about 50,000 years (Todd, 1980).

Lesson 3 Aquifer and Its Properties

3.1 Introduction

As mentioned in Lesson 2, the geologic formation that can store and yield water in usable quantities is called an aquifer. As groundwater is the most reliable source of water for domestic, industrial and agricultural sectors, the goal of all the groundwater exploration and development programs is to find out aquifers in a particular locality to meet the local water demand.

We have learned in Lesson 2 that most common aquifer materials are unconsolidated sands and gravels, which occur in alluvial valleys, old stream beds covered by fine deposits (buried valleys), coastal plains, dunes, and glacial deposits. Sandstones are also good aquifer materials. Cavernous limestones with sufficient solution channels, caves, underground streams, and other karst developments can also be high-yielding aquifers. Basalts, lavas, and other materials of volcanic origin can make excellent aquifers if they are sufficiently porous or fractured and if the vesicles are interconnected (in case of lava). However, other sedimentary rocks such as shales, solid limestones, etc. generally don't serve as good aquifers. Small water yields may be possible where these rocks are highly fractured. The same is true for granite, gneiss, and other crystalline or metamorphic rocks

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3.2 Types of Aquifers

Aquifer can be basically classified into three types: (i) unconfined aquifer, (ii) confined aquifer, and (iii) leaky aquifer. Sometimes, fourth type of the aquifer is known as 'perched aquifer', but it is not the focus of any groundwater exploration. Fig. 3.1 illustrates the types of aquifers available below the ground.

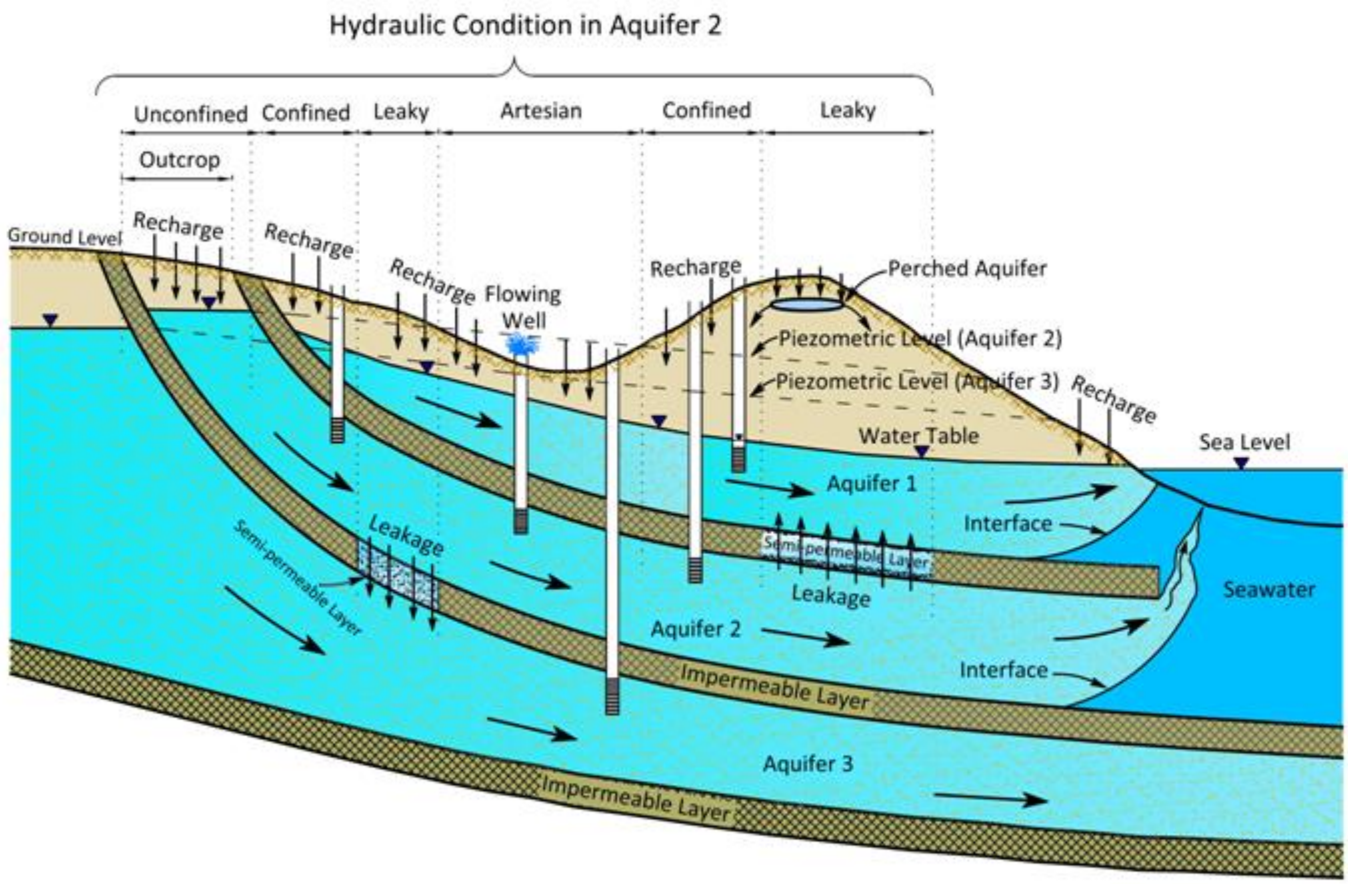

Fig. 3.1. Types of aquifers for groundwater development.

(Source: Bear, 1979)

3.2.1 Unconfined Aquifers

Aquifers which are bounded by a free surface (known as 'water table') at the upper boundary and a confining layer at the lower boundary are called unconfined aquifers (Aquifer 1 in Fig. 3.1). At the water table, water is at the atmospheric pressure, and hence unconfined aquifers are also called 'water-table aquifers' or 'phreatic aquifers'.

Unconfined aquifers receive recharge directly from the overlying surface through rainfall infiltration or percolation from surface water bodies. They usually exhibit a shallow water level. A typical indicator of an unconfined aquifer is that the water level in a well tapping this aquifer is equal to the water table position at that location of the aquifer. In other words, water level doesn't rise above the water table.

3.2.2 Confined Aquifers

Aquifers which are bounded both above and below by impervious or semi-pervious layers are called confined aquifers and the water present in these aquifers are under pressure (Aquifers 2 and 3 in Fig. 3.1). Confined aquifers are sometimes also called 'pressure aquifers' or 'artesian aquifers'; the latter term is gradually becoming obsolete. Since the water present in a confined aquifer is at a pressure greater than the atmospheric pressure, the water level in a borewell penetrating a confined aquifer will always rise above the top confining layer of the aquifer. The term 'piezometric level' is used to denote this water level. Thus, 'Piezometric level' is an imaginary position to which the water level will rise in a borewell tapping a confined aquifer. Piezometric level in two dimensions is called 'piezometric surface'.

Unlike unconfined aquifers, confined aquifers don’t receive significant amounts of recharge from the overlying surface. The groundwater within a confined aquifer is under a pressure equal to the sum of the weight of the atmosphere and the overburden. As mentioned above, the groundwater level in a well penetrating a confined aquifer is usually above the upper boundary of the confined aquifer. However, there may be cases where the piezometric level of a confined aquifer is above the ground surface. The well tapping such a confined aquifer yields water like a spring, and hence it is called a ‘flowing well’ and such a confined aquifer is known as an ‘artesian aquifer’. Note that the word ‘artesian’ comes from the name of a place in France where flowing wells were seen for the first time in the world. Hence, this word is now widely understood to refer only to the hydraulic condition in a confined aquifer due to which flowing wells exist (i.e., groundwater flows naturally beyond the ground surface). Unfortunately, some books on groundwater still use the term ‘artesian aquifer’ synonymous with ‘confined aquifer’.

Moreover, most confined aquifers are unconfined at their exposed edges in the upstream portion of the aquifer, which is called ‘outcrop’ (Fig. 3.1). They receive significant recharge through the outcrop by direct rainfall infiltration into this unconfined portion. Confined aquifers also receive recharge through their upper and lower leaky confining layers under natural conditions or when pressure changes are artificially induced by pumping or injection. Groundwater flux to and from an aquifer through a confining layer is termed ‘leakage’ (Fig. 3.1) and the confining layer is called a ‘leaky confining layer’ or ‘aquitard’.

3.2.3 Leaky Aquifers

If an aquifer (confined aquifer or unconfined aquifer) loses or gains water through adjacent semi-permeable layers, it is called a ‘leaky aquifer’ (Fig. 3.1). Therefore, the terms ‘leaky confined aquifer’ and ‘leaky unconfined aquifer’ are widely used depending on whether the leaky aquifer is confined or unconfined. However, the case of ‘leaky confined aquifer’ has been mostly dealt with by the groundwater experts. This is why, the term ‘semi-confined aquifer’ is sometimes used to denote a ‘leaky aquifer’. The term ‘nonleaky’ is also used to describe the status of a confined or unconfined aquifer, such as ‘nonleaky unconfined aquifers’ and ‘nonleaky confined aquifers’. In reality, ideal confined aquifers or ideal unconfined aquifers occur less frequently than do leaky aquifers.

3.2.4 Perched Aquifers

A perched aquifer is a special type of an unconfined aquifer, in which water exists under water-table conditions. Therefore, the upper boundary of this aquifer is called ‘perched water table’ (Fig. 3.1). Perched aquifer always exists in the vadose zone above an unconfined aquifer or a confined aquifer when a low-permeability layer impedes the downward movement of water above it. Perched aquifers have generally very limited areal extent and they may not have sufficient storage to support significant well production. Therefore, perched aquifers are not the target of a groundwater exploration. However, perched aquifers can support shallow dug wells, thereby can provide water supply to a small community for a limited time period.

It should be noted that hydraulically single aquifers seldom exist in nature. An aquifer is generally part of a system of two or more aquifers, which is more complex. Aquifer thickness, hydraulic head, Darcy velocity, seepage velocity, hydraulic conductivity, transmissivity, intrinsic permeability, storage coefficient (specific storage), specific yield, and specific retention are the important hydraulic and hydrogeologic parameters which are used to characterize an aquifer system. Some of these parameters are discussed in the subsequent section, while others will be discussed in Lesson 4.

3.3 Properties of Aquifers

The hydrogeologic factors which govern the storage and fluid-transmitting characteristics of an aquifer system are called ‘aquifer properties’ or ‘aquifer parameters’. Storage-related aquifer properties (or storage parameters) are: porosity, effective porosity, specific retention, specific yield, storage coefficient, and specific storage. On the other hand, fluid-transmission-related aquifer properties (or yield parameters) are: intrinsic permeability, hydraulic conductivity, and transmissivity. These properties are defined below.

3.3.1 Porosity

'Porosity' (n) of a porous medium (soil or subsurface formation) is defined as the ratio of the volume of voids (V_v) in a porous medium to the total volume of the porous medium (V). That is,

$$n = \frac{V_v}{V} \qquad (3.1)$$

Porosity is a dimensionless parameter and is computed as n = (1- r_b/r_p), where r_b = bulk density of the soil/subsurface formation, and r_p = particle density of the soil/subsurface formation.

In general, rocks (consolidated subsurface formations) have lower porosities than soils or unconsolidated subsurface formations. Gravels, sands and silts, which are made up of angular and rounded particles, have lower porosities than the soil rich in platy clay minerals. Also, poorly-sorted deposits/subsurface formations have lower porosities than well-sorted deposits/subsurface formations.

3.3.2 Effective Porosity

Because of the presence of isolated (not interconnected) pores, dead-end pores, micropores (i.e., extremely small-size pores), and adhesion forces in a porous medium, only a very small fraction of the total porosity is effective (i.e., permeable). 'Effective Porosity' is defined as the portion of void space in a porous material through which fluid (liquid or gas) can flow. In other words, it is the fraction of total porosity which is available for fluid flow. It is also called 'kinematic porosity'.

Like the definition of porosity (total porosity) of a porous medium, effective porosity (n_e) can be defined as follows:

$$n_e = \frac{\textit{Volume of water able to circulate in the porous medium}}{\textit{Total volume of the porous medium (soil or aquifer material)}} \qquad (3.2)$$

Note that the definition of effective porosity is linked to the concept of fluid (water or gas) circulation and not to the percentage of the volume occupied by a fluid.

3.3.3 Specific Retention

'Specific Retention' (S_r) of a soil or aquifer material is defined as the ratio of the volume of water retained after saturation against gravity to its own volume. That is,

$$S_r = \frac{V_r}{V} \qquad (3.3)$$

Where, V_r = volume of retained water, and V = total volume of the soil or aquifer material.

It should be noted that S_r increases with decreasing grain size. For example, clay may have a porosity of 50% with a specific retention of 48%. Moreover, the terms 'field capacity' and 'retained water' refer to the same water content, but differ by the zone in which they occur; the former occurs in the unsaturated zone and the latter in the zone of saturation.

3.3.4 Specific Yield

'Specific Yield' (S_y) or 'Drainable Porosity' of a soil or aquifer material is defined as the ratio of the volume of water that, after saturation, can be drained by gravity to its own volume. That is,

$$S_y = \frac{V_d}{V} \qquad (3.4)$$

Where, V_d = volume of water drained by gravity (i.e., drainable volume), and V = total volume of the porous medium (soil or aquifer material).

As the volume of water drained (V_d) and the volume of water retained (V_r) constitute the total water volume in a saturated porous material, the sum of the two is equal to the total porosity (n) of a porous material. That is,

$$n = S_y + S_r \qquad (3.5a)$$

Or,

$$S_y = n - S_r \qquad (3.5b)$$

The above basic definition of specific yield is very common in Vadose Zone Hydrology and Groundwater Hydrology and is applied when the specific yield is determined in the laboratory. However, if the specific yield is determined in the field, it is defined as the volume of water released from or taken into storage per unit area of an unconfined aquifer per unit change in water table position. This definition is widely used to estimate seasonal/annual groundwater storage in an area or a basin due to rise in the water table during recharge period as well as to estimate groundwater withdrawal/discharge from an area due to lowering of the water table during the period of groundwater pumping or recession.

Specific yield is a dimensionless parameter of the aquifer. The values of specific yield or drainable porosity depend on the grain size, shape and distribution of pores, compaction of the subsurface formation, and duration of drainage. It should be noted that fine-grained materials yield little water, whereas coarse-grained materials permit a considerable release of water, and hence serve as aquifers. Further, the value of S_y decreases with depth due to compaction. The values of S_y generally range from about 0.01 to 0.30 (Freeze and Cherry, 1979) depending on the type of porous materials present in the saturated zone/aquifer or vadose zone. Table 3.1 shows the typical values of porosity and specific yield (effective porosity) for selected unconsolidated and consolidated formations.

Table 3.1. Typical values of porosity and specific yield for different geological materials (Brassington, 1998)

Sl. No.	Type of Geological Material	Porosity (%)	Specific Yield (%)
1	Coarse Gravel	28	23
2	Medium Gravel	32	24
3	Fine Gravel	34	25
4	Coarse Sand	39	27
5	Medium Sand	39	28
6	Fine Sand	43	23
7	Silt	46	8
8	Clay	42	3
9	Fine-grained Sandstone	33	21
10	Medium-grained Sandstone	37	27
11	Limestone	30	14
12	Dune Sand	45	38

13	Loess	49	18
14	Peat	92	44
15	Schist	38	26
16	Siltstone	35	12
17	Till (mainly Sand)	31	16
18	Till (mainly Silt)	34	6
19	Tuff	41	21

3.3.5 Storage Coefficient and Specific Storage

‘Storage Coefficient’ or ‘Storativity’ (S) of an aquifer is defined as the volume of water released from or taken into storage per unit area of an aquifer per unit change in hydraulic head. Here, the hydraulic head denotes ‘piezometric level’ for confined aquifers and ‘water table’ for unconfined aquifers. Figure 3.2 illustrates the concept of storage coefficient (storativity). Thus, the term ‘storage coefficient’ or ‘storativity’ applies to both confined and unconfined aquifers.

Storage coefficient is a dimensionless aquifer parameter. Mathematically, it is expressed as follows:

$$S = S_s \times b \qquad (3.6)$$

Where S_s = specific storage of the aquifer material, and b = thickness of the aquifer.

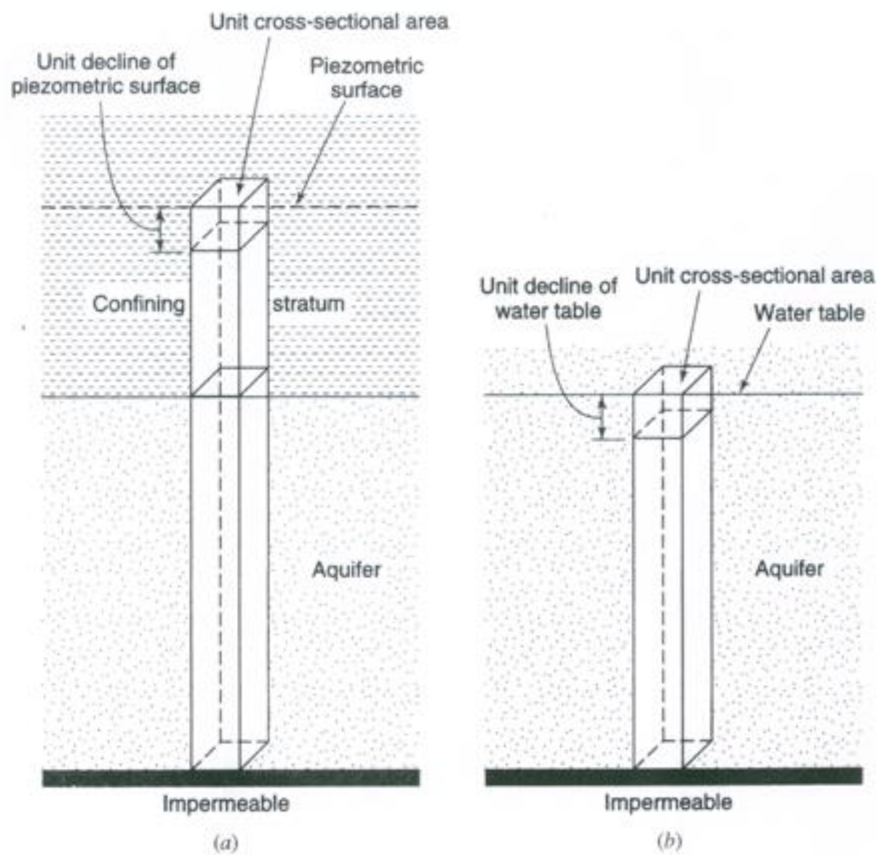

Fig. 3.2. Schematic representation of storage coefficient for: (a) Confined aquifers; (b) Unconfined aquifers. (Source: Todd, 1980)

The 'Specific Storage' (S_s) of an aquifer is defined as the volume of water released from or taken into storage per unit volume of an aquifer per unit change in hydraulic head. The specific storage has the dimension of [L^{-1}]. It is mathematically expressed as follows:

$$S_s = \rho_w g(\alpha + n\beta) = \gamma_w(\alpha + n\beta) \quad (3.7)$$

Where, ρ_w = density of water; α = compressibility of the aquifer material (a is equal to $\frac{1}{E_s}$, wherein E_s is bulk modulus of elasticity of aquifer skeleton); β = compressibility of water (β is equal to $1/K_w$, wherein K_w is bulk modulus of elasticity of water); n = porosity of the aquifer material; and γ_w = unit weight of water. Thus, the specific storage is a more fundamental aquifer parameter, which depends on the type of aquifer material, water present in the aquifer, and the overburden stress.

The value of K_w (bulk modulus of elasticity of water) is $2.1X10^9$ N/m^2, whereas the values of E_s (bulk modulus of elasticity of aquifer skeleton) for some geological materials are given in Table 3.2 (Raghunath, 2007).

Table 3.2. Values of E_s for selected geological materials (Raghunath, 2007)

Sl. No.	Geological Material	Value of E_s (N/m^2 ´ 10^5)
1	Plastic clay	5-40
2	Stiff clay	40-80
3	Medium-hard clay	80-150
4	Loose sand	100-200
5	Dense sand	500-800
6	Dense sandy gravel	1,000-2,000
7	Fissured or Jointed Rock	1,500-30,000
8	Sound Rock	>30,000

If we substitute Eqn. (3.7) in Eqn. (3.6), the expanded form of the equation for storage coefficient (storativity) would be:

$$S = \gamma_w(\alpha + n\beta) \times b \qquad (3.8)$$

It is obvious from Eqn. (3.8) that besides the aquifer compressibility (α) and water compressibility (β), the storage coefficient (S) of an aquifer is a function of aquifer thickness (i.e., aquifer geometry) which is a location-specific quantity, and hence it varies from one location to another over a basin or sub-basin. Note that the expression of storage coefficient [Eqn. (3.8)] consists of two parts: (i) the amount of water released from an aquifer or taken into aquifer storage due to the compressibility of the aquifer material/skeleton (S_c), which is expressed as, and (ii) the amount of water released from an aquifer or taken into aquifer storage due to the compressibility of water (S_w), which is expressed as.

It is worth mentioning that the phenomena of aquifer compression and water expansion due to drop in the groundwater level also occur in unconfined aquifers. However, their contribution to the volume of water released by an unconfined aquifer is negligible in most cases as compared to the volume of water derived from the gravity drainage of pores. Therefore, for practical purposes, the storage coefficient (S) is equal to the specific yield (S_y) for unconfined aquifers and the concept of specific storage is almost exclusively used for the analysis of confined aquifers $S_w = \gamma_w \times n \times \beta \times b$.

The values of storage coefficient (S) of confined aquifers are relatively small and they often range from 0.005 to 0.00005 (Freeze and Cherry, 1979). It indicates that large pressure changes over extensive areas are required to produce substantial water yields from a confined aquifer. Considering the range of values of specific yield (S_y) mentioned above, it is clear that the specific yield of unconfined aquifers is considerably higher than the storage coefficient of confined aquifers.

3.3.6 Intrinsic Permeability

'Intrinsic permeability' or 'permeability' of porous media (soils or subsurface formations) is defined as their ability to transmit a fluid (liquid or gas) through them. It is a property of the medium only and is independent of the fluid properties. Intrinsic permeability (k) is mathematically expressed as follows:

$$k = Cd^2 \qquad (3.9)$$

Where, C = dimensionless proportionality constant commonly known as 'shape factor' (accounting for the shape of pore space which is a function of mean grain diameter, sphericity and roundness of grains, distribution of grain sizes, and nature of their packing), and d = diameter of the pore space (also known as 'characteristic grain diameter' or 'characteristic length').

The dimension of k is $[L^2]$ and its SI unit is m^2. Because of very small values of k, it is also expressed in square micrometers $[(mm)^2]$, i.e., 10^{-12} m^2. However, in the petroleum industry, the value of k is measured in a unit termed 'darcy', which is defined as (Todd, 1980):

$$1\,darcy = \frac{\dfrac{(1\,centipoise)\times(1\,cm^3/s)}{1\,cm^2}}{1\,atmosphere/cm}$$

Note that 1 darcy = 0.987 $(mm)^2$ = 0.987×10^{-8} cm^2.

3.4 Water Yielding Mechanisms of Aquifers

Unconfined aquifers yield water to wells or other water collection facilities due to actual drainage (dewatering) of pores. Air replaces the water initially present in the dewatered zone as the water table drops from a higher elevation to a lower elevation. Thus, the water released from an unconfined aquifer is mainly derived from the dewatering of pores (voids). Therefore, the storage coefficient (S) for an unconfined aquifer corresponds to its specific yield (S_y).

Note that if the water table lowers at a rapid rate due to large pumping rate, the drainage of pores may not take place sufficiently fast to deliver the full specific yield. In this case, continued drainage of pores will occur for some time even if the water table has already receded to lower levels. Thus, the specific yield of an aquifer is dependent on the rate of fall of water table, and varies with time and distance from a pumping well.

On the other hand, confined aquifers remain completely saturated, and hence it does not release water due to drainage of pores. In confined aquifers, water is basically released from or taken into storage as a result of changes in pore volume due to aquifer compressibility and water compressibility (changes in water density associated with a change in pore-water pressure). Thus, confined aquifer releases water due to the compression of aquifer material and the expansion of water (decrease in water density owing to the decrease in pore-water pressure). Therefore, the capacity of confined aquifers to release water from storage is remarkably different from that of the unconfined aquifers. In other words, the amount of water yielded by these mechanisms per unit drop in piezometric level is considerably less than that yielded by the drainage of pores per unit drop in water table. Besides these two primary water-yielding mechanisms, the confined aquifers also yield water by other two mechanisms (Bouwer, 1978): (i) leakage from adjoining aquifers through aquitards (e.g., an overlying unconfined aquifer or an underlying confined aquifer), and (ii) direct draining of pores at the outcrop of a confined aquifer or if the confined aquifer hydraulically becomes an unconfined aquifer when the piezometric level in a confined aquifer drops to or below the bottom of its top confining layer.

3.5 Example Problems

3.5.1 Example Problem 1

In an unconfined aquifer extending over 4 km^2, the water table was initially at 26 m below the ground surface. Sometime after an irrigation of 20 cm (full irrigation), the water table rises to a depth of 25.5 m below the ground surface. Afterward 1.5 ´ 10^6 m^3 of groundwater was withdrawn from this aquifer, which lowered the water table to 27.5 m below the ground surface. Determine: (i) specific yield of the aquifer, and (ii) soil moisture deficit (SMD) before irrigation.

Solution:

(i) Volume of groundwater withdrawn from the unconfined aquifer = Area of the aquifer ´ Drop in the water table ´ Specific yield

Substituting the values, we have,

$$1.5\times10^6 = 4\times10^6\times(27.5-25.5)\times S_y = 4\times10^6\times2.0\times S_y$$

$$\therefore \ S_y = \frac{1.5\times10^6}{4\times10^6\times2.0} = 0.19, \textbf{ Ans.}$$

(ii) Volume of water recharged due to irrigation (V_R) = Area of the aquifer influenced by irrigation ´ Rise in the water table ´ S_y

Let us consider the aquifer area influenced by irrigation to be 140 m^2, then the volume of water recharged (V_R) will be:

V_R = 140´X(26.0-25.5)´0.19 = 13.3 m^3

Volume of irrigation water (V_I) = 140 × 0.20 = 28.0 m^3

Now, Soil moisture deficit (SMD) before irrigation = V_I - V_R = 28.0-13.3 = 14.7 m^3.

Or, SMD = $\frac{14.7}{140}$ = 0.105 m = 10.5 cm, Ans.

3.5.2 Example Problem 2

In an area of 200 ha, the water table declines by 3.5 m. If the porosity of the aquifer material is 30% and the specific retention is 15%, determine: (i) specific yield of the aquifer, and (ii) change in groundwater storage.

Solution:

(i) We know, Porosity = Specific yield (S_y) + Specific retention (S_r)

$\Rightarrow$ $0.30 = S_y + 0.15$

$\therefore$ $S_y = 0.30 – 0.15 = 0.15$ or 15%, **Ans.**

(ii) Change in groundwater storage = Area of the aquifer ´ Drop in the water table ´ Specific yield

= $(200X10^4)X3.5X0.15$

= $105X10^4$ m^3, Ans.

3.5.3 Example Problem 3

The average thickness of a confined aquifer extending over an area of 500 km^2 is 25 m. The piezometric level of this aquifer fluctuates annually from 10 m to 22 m above the top of the aquifer. Assuming a storage coefficient of the aquifer as 0.0006, estimate annual groundwater storage in the aquifer.

Solution:

Annual groundwater storage (GWS) in the confined aquifer is given as:

GWS = Area of the aquifer ´ Rise in the piezometric level ´ Storage coefficient

= $(500X10^6)$´$(22-10)X0.0006$

= $3.6X10^6$ m^3, Ans.

esson 4 Principles of Groundwater Flow

4.1 Concept of Fluid Potential and Hydraulic Head

4.1.1 What is Fluid Potential?

Fluid potential is defined as “a physical quantity, capable of being measured at every point in a flow system, whose properties are such that flow always occurs from regions in which the quantity has higher values to those in which it is lower, regardless of direction in space”.

Fluid flow through porous media is a mechanical process. The forces driving the fluid forward must overcome the frictional forces set up between moving fluid and the grains of the porous medium. The flow is therefore accompanied by an irreversible transformation of mechanical energy to thermal energy through the mechanism of frictional resistance. The direction of flow must therefore be away from regions in which the mechanical energy per unit mass of fluid is higher and towards regions in which it is lower. Thus, the mechanical energy per unit mass at any point in the flow system can be defined as the work required to move a unit mass of fluid from an arbitrary chosen standard state to the point in question. The fluid potential for flow through porous media is therefore the mechanical energy per unit mass of fluid. Now, in order to relate the fluid potential to elevation and pressure terms, let us consider the following example.

We wish to calculate the work done in lifting unit mass of fluid from an arbitrary standard state (Point A) to some point (Point B) in a subsurface flow system (Fig. 4.1).

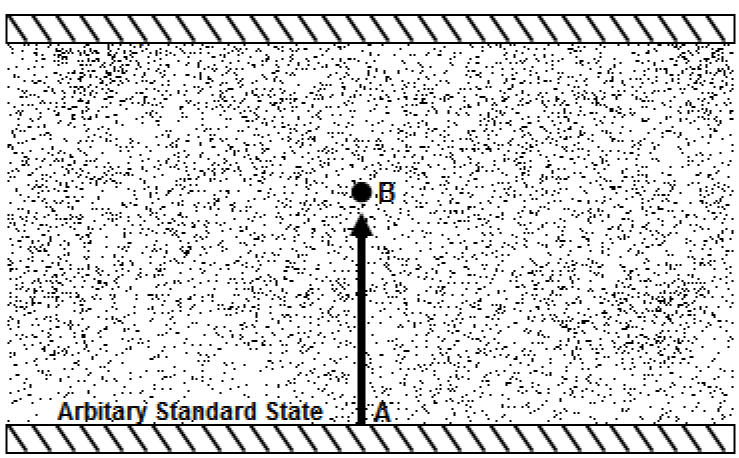

Fig. 1. Example of a subsurface flow system.

At Point A, we have:

Elevation (z) = 0,

Pressure (P) = P_o (atmospheric pressure),

Velocity (v) = 0,

Density (r) = r_o, and

Volume of unit mass = $\frac{1}{\rho_o}$ = V_o.

Similarly, at Point B, we have:

Elevation = z,

Pressure = P,

Velocity = v,

Density = r, and

Volume of unit mass = $\frac{1}{\rho}$ = V.

Now, three components to the work calculation are as follows:

(a) Work done in lifting the mass of the fluid (m) from elevation z = 0 to elevation z:

$$W_1 = mgz \tag{4.1}$$

(b) Work done in accelerating the fluid of mass ‘m’ from velocity v = 0 to velocity v:

$$W_2 = \frac{1}{2} mv^2 \tag{4.2}$$

(c) Work done on the fluid to raise the fluid pressure from P_o (atmospheric pressure) to P:

$$W_3 = m\int_{P_o}^{P} \frac{V}{m} dP = m\int_{P_o}^{P} \frac{dP}{\rho} \qquad (4.3)$$

If the fluid is to flow from point B to point A at the standard state, then Eqn. (4.1) is the loss in potential energy, Eqn. (4.2) is the loss in kinetic energy, and Eqn. (4.3) is the loss in fluid-pressure energy (elastic energy).

Fluid Potential (f) is the sum of W_1, W_2, and W_3 for unit mass of fluid (i.e., m = 1 in the above equations) as given below:

$$\phi = gz + \frac{1}{2}v^2 + \int_{P_o}^{P} \frac{dP}{\rho} \qquad (4.4)$$

For incompressible fluid, is constant. Thus, Eqn. (4.4) becomes

$$\phi = gz + \frac{1}{2}v^2 + \frac{P - P_o}{\rho} \qquad (4.5a)$$

It is common in groundwater hydrology to set P_o equal to zero and work in gauge pressure (i.e., pressure above atmospheric). That is, Eqn. (4.5a) can also be written as:

$$\phi = gz + \frac{1}{2}v^2 + \frac{P}{\rho} \qquad (4.5b)$$

The right hand terms of Eqns. (4.5a and 4.5b) indicate the total mechanical energy per unit mass of the fluid, i.e., fluid potential.

4.1.2 Relation between Fluid Potential and Hydraulic Head

After dividing Eqn. (4.5b) both sides by g (acceleration due to gravity), we have:

$$\phi / g = z + \frac{1}{2g}v^2 + \frac{P}{\rho g} \qquad (4.6)$$

Eqn. (4.6) indicates the total mechanical energy per unit weight of the fluid, which is known as hydraulic head (h). Hence, the right hand terms of Eqn. (4.6) can be replaced by h, and then Eqn. (4.6) reduces to:

$$\phi = gh \qquad (4.7)$$

That is, Fluid potential = Hydraulic Head × Acceleration due to gravity.

Note that Eqn. (4.6) expresses all the terms in units of energy per unit weight, which has the advantage of having all units in length dimensions.

For the flow through porous media, the flow velocity is very low, and hence the second term of Eqn. (4.6) is usually neglected. Then, Eqn. (4.6) can be written as follows:

$$\phi / g = z + \frac{P}{\rho g} \qquad (4.8a)$$

Or, $$h = z + h_p \qquad (4.8b)$$

Where, h = hydraulic head, z = elevation head, and h_p = pressure head.

Thus, hydraulic head (h) at any point in a porous medium is the sum of ‘elevation head’ and ‘pressure head’. Since pressure head is a function of space and time, and hence hydraulic head is also a function of space (x, y, z) and time (t). The ‘pressure head’ at a point in the saturated porous medium is measured by installing a tube or a small diameter pipe at that point. In the laboratory, the tube is called a ‘monometer’ and for the field, small diameter pipe is called a ‘piezometer'. The ‘elevation head’ is measured with respect to a datum (reference line), which is usually ‘mean sea level’ (MSL).

4.2 Darcy’s Law

Based on a laboratory experiment on a sand column, Henry Darcy in 1856 observed that the rate of flow (Q) through the sand column is directly proportional to the head loss over the column length (dh) and cross-sectional area (A), and is indirectly proportional to the column length (dl). That is,

$$Q \propto A\frac{dh}{dl} \quad or \quad Q = -KA\frac{dh}{dl} \qquad (4.9)$$

In Eqn. (4.9), the constant of proportionally (K) is known as the hydraulic conductivity of the porous medium. Sometimes K is also called saturated hydraulic conductivity of the porous medium because of the fact that the Darcy's law is strictly valid for flow through a saturated porous medium. Negative sign in Eqn. (4.9) indicates that the flow occurs in the direction of decreasing head. The term $\frac{dh}{dl}$ is the head loss per unit length of flow and is called hydraulic gradient (i). Equation (4.9) can also be written as follows:

$$\frac{Q}{A} = -K\frac{dh}{dl} \quad or \; q = -\; K\frac{dh}{dl} \qquad (4.10)$$

Where, q is called Darcy velocity (Darcy flux) or specific discharge. Darcy velocity is a volume flux defined as the discharge per unit bulk area (including both pore space and solids) of a porous medium. Thus, Darcy velocity (q) is a macroscopic flux and in no case, it is equal to the displacement of fluid elements per unit time (usual meaning of velocity). Therefore, better terms like 'Darcy flux' or 'specific discharge' have been suggested by hydrogeologists and soil physicists instead of Darcy velocity.

Darcy's law is widely used to quantity flow in aquifer systems. For example, if the Darcy flux in an aquifer is 0.1 m/day and the aquifer normal to the flow direction is 10 m thick and 1000 m wide, then the groundwater flow rate in the aquifer is: q×A = 0.1×10×1000 = 1000 m^3/day. Groundwater discharge (flow rate) can also be calculated if the values of hydraulic conductivity (K), hydraulic gradient (i) and the area of an aquifer (A) are known.

4.3 Validity of Darcy's Law

The Darcy law is valid for the groundwater flow condition when head loss is directly proportional to the velocity of flow. Such a flow condition exists when the groundwater flow is laminar. That is, the Darcy law is valid for laminar flow only.

To check the validity of the Darcy law, a non-dimensional number called Reynolds Number is used. Reynolds Number (R_e) is given as:

$$R_e = \frac{\rho V d}{\mu} = \frac{\text{Inertial Force}}{\text{Viscous Force}} \qquad (4.11)$$

Where, ρ = density of the fluid, V = flow velocity, d = characteristic length, and m = dynamic viscosity of the fluid.

For flow through porous media, V is usually taken as the Darcy velocity and d is better represented by d_{10} (effective grain diameter/size of the porous media), though d is also taken as d_{50} (mean or median grain diameter/size of the porous media) by some hydrogeologists.

Experiments have shown that the Darcy's law is strictly valid for $R_e<1$, but it doesn't depart seriously up to $R_e = 10$ (Ahmed and Sunada, 1969). Hence, in practice, the Darcy's law may be applied to flow conditions that exist when $R_e \leq 10$. A range of values rather than a unique limit must be stated because as inertial forces in the tortuous paths of porous-media flow increase, turbulence occurs gradually (Todd, 1980). The irregular flow paths of eddies and swirls associated with the turbulence first occur in larger pores; with increasing velocity they spread to smaller pores. For the fully developed turbulence, the head loss (ΔH) varies approximately with the second power of the velocity (i.e., $\Delta H \ \mu V^2$) rather than linearly as in the case of laminar flow.

Fortunately, most natural groundwater flow occurs with $R_e \leq 1$, and hence the Darcy's law is generally applicable. Deviations from the Darcy's law can occur in some special situations where steep hydraulic gradients exist. For example, groundwater flow in the vicinity of pumping wells as well as turbulent flow in rocks having fractures, fissures and solution cavities/channels such as basalt and limestone that contain large underground openings.

Example Problem: A sand aquifer has an effective grain diameter of 0.2 mm. The density of groundwater is 1.003×10^3 kg/m^3 and its dynamic viscosity is 1.15×10^{-3} N s/m^2 (kg/s m). If the rate of groundwater flow is 0.0016 m/s, check the validity of the Darcy's law for this flow system.

Solution: Reynolds number (R_e) is given as:

$$R_e = \frac{\rho v d}{\mu}$$

Here, r = 1.003×10^3 kg/m^3, v = 0.0016 m/s, d = 0.2 mm = 0.0002 m, and m = 1.15×10^{-3} N s/m^2. Substituting these values in the above equation, we have

$$R_e = \frac{1.003\times10^3\times0.0016\times0.0002}{1.15\times10^{-3}}$$

=0.2791 , which is less than 1.

Therefore, Darcy's law is valid for the given flow system.

4.4 Seepage Velocity

In reality, groundwater movement occurs through the conductive pores and cracks of the aquifer material only, and hence the actual velocity of groundwater is greater than the Darcy flux or specific discharge. The velocity of groundwater through the effective pores of a porous medium is called seepage velocity or groundwater velocity (sometimes also called “average linear velocity”), which is actual velocity of groundwater flow. Thus, actual groundwater discharge (Q) can be computed as follows:

$$Q = n_e \times V_s \times A \qquad (4.12)$$

Where, n_e = effective porosity of the aquifer, V_s = seepage velocity or actual groundwater velocity, and the remaining symbols have the same meaning as defined earlier.

Equation (4.12) can also be written as:

$$V_s = \frac{Q}{A \times n_e} \qquad (4.13a)$$

Or,

$$V_s = \frac{q}{n_e} \qquad (4.13b)$$

Equation (4.13b) gives the relationship between the Darcy flux (specific discharge) and the seepage velocity. Obviously, the seepage velocity (actual groundwater velocity) can be computed by dividing the Darcy flux (q) with effective porosity (n_e). For example, if the Darcy flux in an aquifer is 0.1 m/day and the effective porosity of the aquifer (n_e) is 12%, then the actual groundwater velocity will be: = $V_s = \frac{q}{n_e} = \frac{0.1}{0.12}$. = 0.83 m/day, which is about eight times the Darcy velocity. Thus, the seepage velocity or actual groundwater velocity is always greater than the Darcy velocity.

4.5 Hydraulic Conductivity and Transmissivity

4.5.1 Hydraulic Conductivity

Hydraulic conductivity (K) of a saturated porous medium can be defined from Darcy's law as:

$$K = \frac{Q}{A \times i} \qquad \text{Or,} \quad K = \frac{q}{i} \tag{4.14}$$

Where, K = hydraulic conductivity of the aquifer, Q = groundwater discharge (rate of groundwater flow), A = cross-sectional area of the aquifer, i = hydraulic gradient, and q = specific groundwater discharge or Darcy flux.

Thus, hydraulic conductivity (K) can be defined as 'the groundwater discharge per unit cross-sectional area of the aquifer under a unit hydraulic gradient'. Alternatively, it can also be defined as 'the specific groundwater discharge under a unit hydraulic gradient. K has the dimension of velocity (i.e., L/T).

The relationship between hydraulic conductivity (K) and intrinsic permeability (k) of a porous medium is given as follows (Todd, 1980):

$$K = \frac{k \rho g}{\mu} \tag{4.15a}$$

Or, $$K = \frac{kg}{\nu} \qquad (4.15b)$$

Where, μ = dynamic viscosity of the groundwater, ρ= density of the groundwater, g = acceleration due to gravity, and v = $\frac{\mu}{\rho}$ = kinematic viscosity of the groundwater.

It is clear from Eqn. (4.15a) that the hydraulic conductivity of a porous medium is dependent on the properties of both the porous medium and the fluid (liquid or gas) passing through it. In groundwater hydrology, K is usually expressed in m/day which gives an easy understanding of groundwater flow because the movement of groundwater is normally slow.

4.5.2 Transmissivity

Transmissivity (T) of an aquifer system describes how transmissive an aquifer is in moving water through its pore spaces. It is defined as the product of hydraulic conductivity (K) and the saturated thickness (b) of the aquifer. That is,

$$T = Kb \qquad (4.16)$$

Transmissivity of an aquifer system can also be defined from Darcy's law as follows:

According to the Darcy's law, we have:

$$Q = (b \times w) \times K \times i$$

$$\Rightarrow \quad Q = Kb \times w \times i$$

$$\therefore \quad Q = T \times w \times i$$

(4.17)

Where, Q = groundwater flow rate (discharge), K = hydraulic conductivity of the aquifer, b = saturated thickness of the aquifer, w = width of the aquifer, i = hydraulic gradient, and T = transmissivity of the aquifer.

Thus, transmissivity can be defined as 'the rate of groundwater flow through the entire saturated thickness of an aquifer of unit width under a unit hydraulic gradient'.

Note that the concept of transmissivity inherently assumes horizontal flow in aquifer systems. This concept can also be used for unconfined aquifers. In this case, b in Eqn. (4.16) is replaced with (average saturated thickness of the unconfined aquifer) because of varying saturated thickness caused by the presence of free water surface (i.e., water table).

4.6 Leakage Factor

Leakage factor is a characteristic length of leaky aquifers. It is mathematically expressed as:

$$B = \sqrt{Tb'/K'} \tag{4.18a}$$

Or,

$$B = \sqrt{TC} \tag{4.18b}$$

Where, T = transmissivity of the aquifer, b' = thickness of the leaky confining layer (aquitard), K' = vertical hydraulic conductivity of the leaky confining layer (aquitard), and c= $\frac{b'}{K'}$ hydraulic resistance of the aquitard.

The leakage factor (B) has the dimension of length, i.e., [L]. Moreover, Leakance or Leakage Coefficient is defined as $\frac{1}{C} = \frac{K'}{b'} = \frac{T}{B^2}$. The numerical values of leakance are usually given as a measure of leakage rates (i.e., ability of the aquitard to transmit vertical flow through it). Leakance has the dimension of time inverse (e.g., day^{-1})

esson 6 Analysis of Steady Groundwater Flow

Under steady-state flow conditions, the groundwater level (piezometric level in the confined aquifer or water table in the unconfined aquifer) remains constant with time. Therefore, the groundwater level is a function of space only under steady-state flow conditions. Steady groundwater flow occurs in an aquifer system when the rate of groundwater recharge is equal to the rate of groundwater discharge. In this lesson, the analysis of steady groundwater flow in homogeneous and isotropic confined and unconfined aquifer systems is discussed.

6.1 Steady Flow in Confined Aquifers

If there is a steady movement of groundwater in a confined aquifer, there will be a linear gradient or slope to the piezometric surface; i.e., its two-directional projection is a straight line. For this type of groundwater flow, Darcy's law can be directly applied. In Fig. 6.1, a portion of a homogeneous and isotropic confined aquifer of uniform thickness is shown wherein the hydraulic head has a linear gradient. Two observation wells/piezometers are installed L distance apart in the aquifer where the hydraulic head can be measured.

Using Darcy's law, the quantity of groundwater flow per unit width of the aquifer (q) can be determined as:

$$q = Kb\frac{dh}{dx} \tag{6.1}$$

Where, K = mean hydraulic conductivity of the confined aquifer, b = thickness of the confined aquifer, and $\frac{dh}{dx}$ = hydraulic gradient in the X-direction.

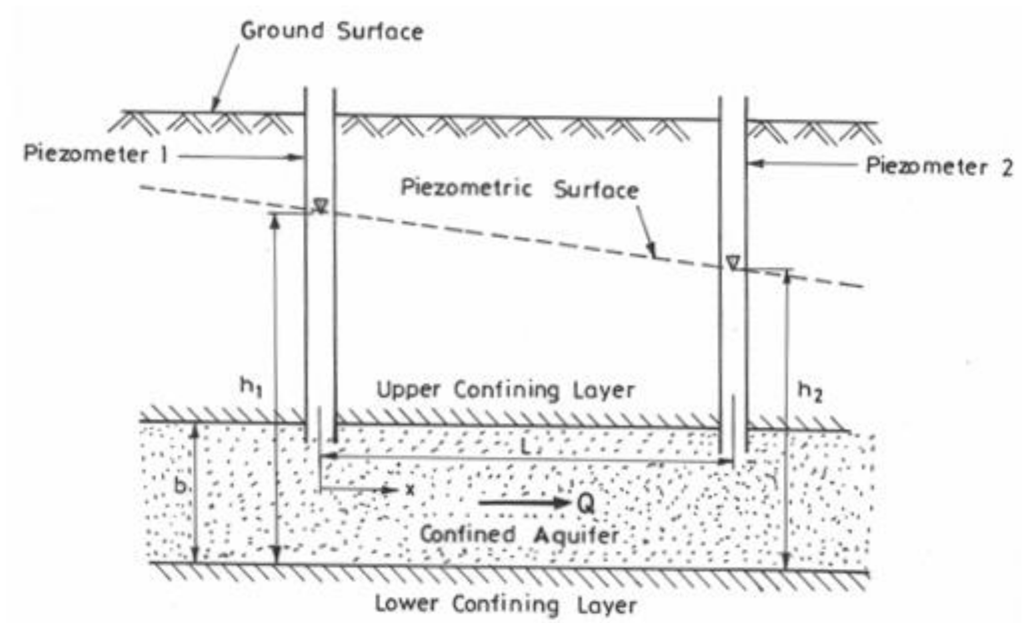

Fig. 6.1. Steady flow through a confined aquifer of uniform thickness. (Modified from Fetter, 1994)

One may be interested to know the hydraulic head (h) at some intermediate distance, x between Piezometer 1 having hydraulic head h_1 and Piezometer 2 having hydraulic head h_2. This can be determined from the following equation:

$$h(x) = h_1 - \frac{q}{Kb}x \tag{6.2}$$

Where, h(x) = hydraulic head at distance x, and x = distance from Piezometer 1.

6.2 Steady Flow in Unconfined Aquifers

In unconfined aquifers, as illustrated in Fig. 6.2, the fact that the water table constitutes the upper boundary of the groundwater flow region complicates flow determination. The shape of the water table determines the flow distribution, but at the same time the flow distribution governs the water-table shape. Therefore, a direct analytical solution of the Laplace equation is not possible in this case.

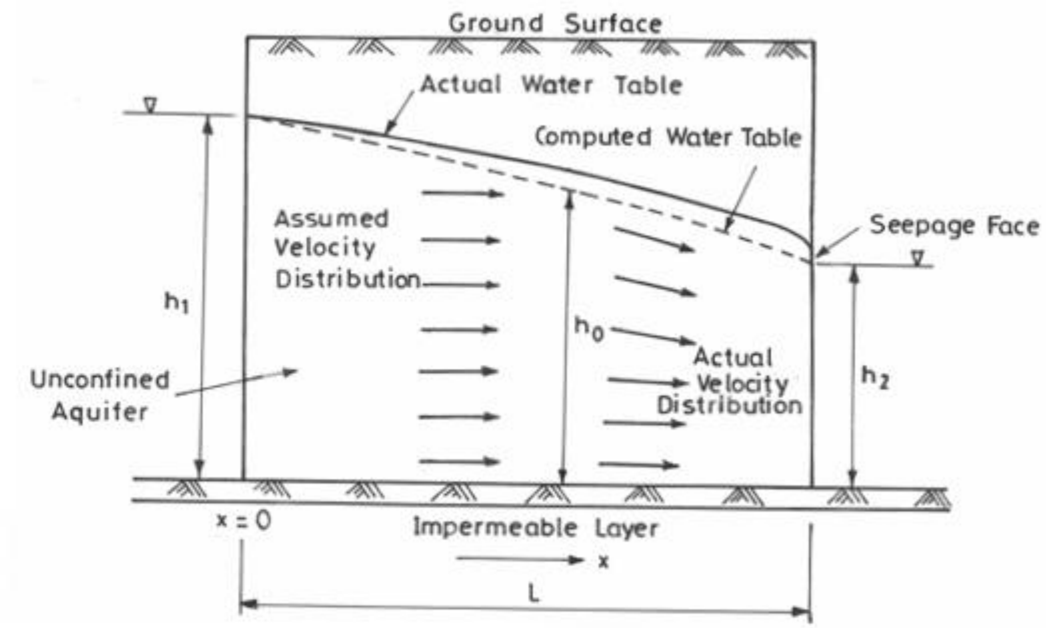

Fig. 6.2. Steady flow in an unconfined aquifer between two water bodies with vertical boundaries. (Modified from Todd, 1980)

Moreover, the saturated thickness of unconfined aquifers decreases in the direction of flow (Fig. 6.2). If there is no recharge or evaporation, the quantity of water flowing through the left side (upstream end) is equal to that flowing through the right side (downstream end). From Darcy’s law, it is obvious that since the cross-sectional area is smaller on the right side, the hydraulic gradient must be greater on this side. Thus, the water-table gradient in unconfined flow is not constant; rather it increases in the direction of flow.

The above problem was solved by Dupuit in 1863 by adopting certain simplifying assumptions, which are well-known as the Dupuit assumptions or Dupuit-Forchheimer assumptions. These assumptions are: (i) the hydraulic gradient in an unconfined flow system is equal to the slope of the water table, and (ii) for small water-table gradients, the streamlines are horizontal and the equipotential lines are vertical. Solutions based on these assumptions have proved to be very useful in many practical problems. However, the Dupuit assumptions do not allow for a seepage face above the outflow side (Fig. 6.2). Furthermore, since the slope of the parabolic water table increases in the direction of flow, the Dupuit assumptions become increasingly poor approximations to the actual flow; therefore, the actual water table deviates more and more from the computed water table in the direction of flow (Fig. 6.2). Thus, the actual water table always lies above the computed water table. The reason for this can be explained by the fact that the Dupuit flows are assumed to be horizontal, whereas the actual velocities of the same magnitude have a downward vertical component so that a greater saturated thickness (i.e., larger height of the water table from the aquifer base) is required for the same discharge.

6.2.1 Steady Unconfined Flow without Recharge or Evapotranspiration

For steady unconfined flow without recharge or evapotranspiration, given the Dupuit assumptions, Darcy's law can be applied to determine groundwater discharge per unit width of the aquifer (q) at any vertical section:

$$q = -Kh\frac{dh}{dx} \qquad (6.3)$$

Where, h = saturated thickness of the unconfined aquifer, K = mean hydraulic conductivity of the unconfined aquifer, and $\frac{dh}{dx}$ = hydraulic gradient in the X-direction.

Applying boundary conditions, i.e., at x = 0, h = h_1; at x = L, h = h_2 (Fig. 6.2), Eqn. (6.3) can be integrated with these boundary conditions as:

$$\int_o^L q\,dx = -K\int_{h_1}^{h_2} h\,dh$$

$$\Rightarrow \; q\left[x\right]_o^L = -K\left[\frac{h^2}{2}\right]_{h_1}^{h_2}$$

$$\Rightarrow \; qL = -K\left(\frac{h_2^{\,2}}{2} - \frac{h_1^{\,2}}{2}\right)$$

$$\Rightarrow \; qL = -K\left(\frac{h_2^{\,2}}{2} - \frac{h_1^{\,2}}{2}\right)$$

$$\therefore \; q = \frac{K}{2L}\left(h_1^{\,2} - h_2^{\,2}\right) \qquad (6.4)$$

Where, L = flow length, h_1 = head at the origin (at x = 0), and h_2 = head at a distance L (at x = L).

Equation (6.4) is known as the Dupuit equation, which indicates that the water table is parabolic in form. For flow between two fixed water bodies of constant heads h_1 and h_2 as shown in Fig. 6.2, the water-table slope at the upstream boundary of the aquifer (neglecting the capillary zone) can be given as:

$$\frac{dh}{dx} = -\frac{q}{Kh_1} \qquad (6.5)$$

However, the boundary $h = h_1$ is an equipotential line because of the constant fluid potential in the water body. Consequently, the water table must be horizontal at this section, which is inconsistent with Eqn. (6.5).

It should be noted that because of the Dupuit-Forchheimer assumptions, many discrepancies arise. Nevertheless, for flat water-table slopes, where the sine and tangent are nearly equal, the Dupuit equation [Eqn. (6.4)] closely predicts the water-table position except near the downstream portion. In general, the Dupuit equation accurately determines q or K for the given boundary heads.

6.2.2 Steady Unconfined Flow with Recharge or Evapotranspiration

For steady one-dimensional unconfined flow subject to recharge or evapotranspiration, as shown in Fig. 6.3, we can obtain the following equation for the water-table position:

$$h(x) = \left[h_1^2 - \frac{(h_1^2 - h_2^2)x}{L} + \frac{R}{K}(L - x)x \right]^{0.5}$$

(6.6)

Where, h (x) = hydraulic head (water-table height from the aquifer base) at a distance x from the origin (upstream end), x = distance from the origin, L = distance from the origin to the point where h_2 is measured, h_1 = head at the origin (upstream end), h_2 = head at the distance L (downstream end), K = mean hydraulic conductivity of the unconfined aquifer, and R = recharge rate.

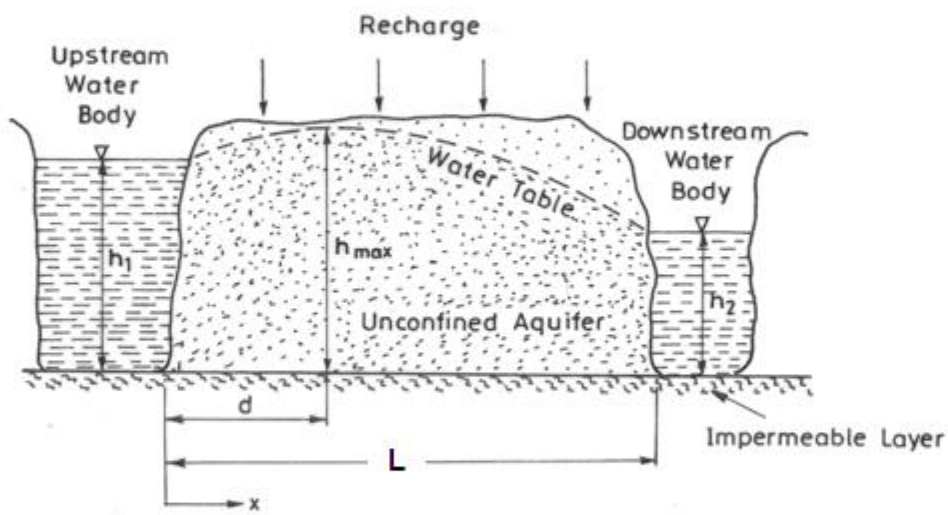

Fig. 6.3. Steady unconfined flow subject to recharge.

(Modified from Fetter, 1994)

Equation (6.6) can be used to find the height of the water table (from the aquifer base) anywhere between two points located L distance apart if the saturated thickness of the unconfined aquifer is known at the two end points (i.e., h_1 and h_2 are known). It should be noted that if significant evapotranspiration (ET) occurs instead of recharge (R), then in Eqn. (6.6), the term R will be replaced by ET with negative sign (i.e., - ET) .

In the absence of recharge or evapotranspiration, Eqn. (6.6) will reduce to:

$$h(x) = \left[h_1^2 - \frac{(h_1^2 - h_2^2)x}{L} \right]^{0.5} \qquad (6.7)$$

Equation (6.7) is called Dupuit parabola.

Now, by differentiating Eqn. (6.6) and considering $q_x = -Kh\frac{dh}{dx}$, it can be shown that the discharge per unit width at any section x distance from the origin $[q(x)]$ is given by:

$$q(x) = \frac{K(h_1^2 - h_2^2)}{2L} - R\left(\frac{L}{2} - x\right) \qquad (6.8)$$

If the water table is subject to recharge (R), there may be water divide with a crest in the water table (known as 'groundwater divide'). In this case, q(x) will be zero at the groundwater divide. If d is the distance from origin to groundwater divide, then substituting q(x) = 0 and x = d into Eqn. (6.8) yields:

$$d = \frac{L}{2} - \frac{K}{R} \times \frac{\left(h_1^2 - h_2^2\right)}{2L} \qquad (6.9)$$

Where, all the variables have the same meaning as defined earlier.

If the distance from the origin to the groundwater divide has been found, then the water-table height (from the aquifer base) at the groundwater divide (i.e., h_{max}) can be determined by replacing x with d in Eqn. (6.6). That is,

$$h_{max} = \left[h_1^2 - \frac{\left(h_1^2 - h_2^2\right)d}{L} + \frac{R}{K}\left(L - d\right)d \right]^{0.5} \qquad (6.10)$$

In Eqn. (6.10), all the variables have the same meaning as defined earlier.

6.3 Example Problems

6.3.1 Problem 1

A confined aquifer is 3.0 m thick. The piezometric level drops 0.15 m between two observation wells which are located 238 m apart. The hydraulic conductivity of the aquifer is 6.5 m/day and the effective porosity is 0.15. Determine the following: (a) Discharge of groundwater through a strip of the aquifer having 10 m width, and (b) Average linear velocity of groundwater.

Solution: From the question, we have:

Thickness of aquifer, $b = 3.0$ m

Difference in piezometric levels, $\Delta h = 0.15$ m

Distance between the observation wells, $\Delta L = 238$ m

Hydraulic conductivity of the aquifer, $K = 6.5$ m/day

Effective porosity, $n_e = 0.15$

Width of the aquifer strip, $W = 10$ m

(a) Groundwater discharge per unit width of the confined aquifer (q) is given as:

$$q = K \times b \times \frac{\Delta h}{\Delta L}$$

$$= 6.5 \times 3 \times \frac{0.15}{238} = 0.012 \text{ m}^2\text{/day}$$

Groundwater discharge through the 10 m aquifer strip = $W \times q$ = 10×0.012

= 0.12 m^3/day, Ans.

$$\frac{K \times i}{n_e} = \frac{K}{n_e} \times \frac{\Delta h}{\Delta L}$$

(b) Average linear velocity of groundwater = $= \frac{6.5}{0.15} \times \frac{0.15}{238}$

= 0.027 m/day, Ans.

6.3.2 Problem 2

An unconfined aquifer has a hydraulic conductivity of 1.2×10^{-2} cm/s. There are two fully penetrating observation wells installed in this aquifer, which are separated by a distance of 98.5 m from each other. In the upstream observation well, the water level is 7.5 m above the aquifer bottom, and in the downstream observation well, it is 6.0 m above the aquifer bottom.

(i) What is the groundwater discharge per 40 m-wide strip of the aquifer? Express your answer in cubic meters per day.

(ii) What is the water-table elevation at a point midway between the two observation wells?

Solution: From the question, we have:

Hydraulic conductivity of the aquifer, (K) = 1.2×10^{-2} cm/s = 1.2×10^{-4} m/s

Distance between the two observation wells, L = 98.5 m.

Considering bottom of the aquifer as a datum, hydraulic head at the upstream observation well (h_1) = 7.5 m, and hydraulic head at the downstream observation well (h_2) = 6.0 m.

Width of the aquifer strip, W = 40 m

(i) Groundwater discharge per unit width of the unconfined aquifer (q) is given as:

$$q = \frac{K \times \left(h_1^2 - h_2^2\right)}{2 \times L}$$

$$= \frac{\left(1.2 \times 10^{-4}\right) \times \left(7.5^2 - 6.0^2\right)}{2 \times 98.5} = \frac{24.3 \times 10^{-4}}{197}$$

$= 1.23 \times 10^{-5}$ m^2/s

Groundwater discharge per 40 m-wide aquifer strip = W × q

$= 40 \times (1.23 \times 10^{-5})$

$= 49.2 \times 10^{-5}$ m^3/s

$= 42.51$ m^3/day, Ans.

(ii) Distance of the point midway between the two observation wells (x) = 98.5/2 = 49.25 m. Water-table elevation at the point midway between two observation wells (h_{mid}) can be calculated from the following equation:

$$h_{mid} = \left[h_1^2 - \frac{\left(h_1^2 - h_2^2\right)x}{L} \right]^{0.5}$$

, where x = 49.25 m

$$= \left[7.5^2 - \frac{\left(7.5^2 - 6.0^2\right) \times 49.25}{98.5} \right]^{0.5}$$

$$= \sqrt{56.25 - 10.125} = \sqrt{46.125} = 6.79 \text{ m,}$$ Ans.

Lesson 8 Introduction to Water Wells

8.1Introduction

Water well is a hole or shaft, usually vertical, excavated into the earth for bringing groundwater to the surface (Todd, 1980). Wells also serve other purposes such as for observation/exploration, artificial recharge and disposal of wastewaters (very restricted these days due to environmental concern). Wells of horizontal extent (known as ‘horizontal wells’) are sometimes constructed because of special groundwater situations. Horizontal wells are advantageous in the situation where groundwater is to be derived primarily from infiltration of streamflow (e.g., collector wells) or in the situations where aquifers are thin, poorly permeable or underlain by permafrost or saline water (e.g., infiltration galleries, and small-diameter perforated pipes drilled into hillsides). Interested readers are referred to Todd (1980), Raghunath (2007), Michael et al. (2008) and Sarma (2009) for the details about different types of horizontal wells.

This lesson deal with various functions of wells, classification of water wells, advantages and disadvantages of open wells and tubewells/borewells, and the selection of sites for well drilling and type of well.

8.2 Functions of Wells

Wells are used for a variety of purposes, which are as follows:

(1) To supply water to meet domestic, municipal, industrial and agricultural requirements.

(2) To control seawater intrusion.

(3) To remove contaminated water from a polluted aquifer.

(4) To lower water table for construction projects.

(5) To relieve pressure under dams.

(6) To drain agricultural land or urban land.

(7) To inject surface water or once used groundwater into the ground for augmenting groundwater resources. That is, to artificially recharge aquifers at rates greater than the natural recharge.

(8) To dispose of wastewater or hazardous wastes into isolated aquifers. This function of wells is highly restricted these days due to its detrimental effects on environment.

8.3 Classification of Water Wells

There are many ways to classify water wells such as based on well depth, method of construction, type of aquifer, entry of water into wells, type of formation (unconsolidated and consolidated formations), etc. (Sarma, 2009; Michael et al., 2008). In this lesson, the classification which is somewhat generic and has greater practical significance has been adopted. Broadly, water wells can be classified into four groups according to their functions: (a) water supply wells, (b) recharge wells, (c) drainage wells, and (d) monitoring wells. Water supply, recharge and drainage wells can be further classified as open wells and tubewells depending their design and method of construction. Tubewells are classified as shallow tubewells and deep tubewells depending on the availability of aquifer layers and the quantity of desired water supply. Some special types of tubewells are known as borewells and cavity wells. Similarly, a special type of open well is known as a dug-cum-bore well. On the other hand, monitoring wells or observation wells are small-diameter (usually 1” to 2”) tubewells for monitoring groundwater levels and taking groundwater samples for exploring water quality. The major types of water wells are succinctly described in subsequent sections. More detailed discussion on the types of water wells can be found in Sarma (2009) and Michael et al. (2008).

8.3.1 Open Wells

Open wells, also known as dug wells, are popular since ancient times and are the most convenient and cost-effective way of harnessing groundwater present in shallow and low-yielding unconfined aquifers for small-scale water supply (e.g., domestic and small-scale irrigation purposes). They can be constructed both in consolidated formations (e.g., alluvial plains and river deltas) and in unconsolidated formations (e.g., weathered and fractured hard-rock formations). Open wells may be either circular or rectangular in shape. Generally, the circular shape is adopted for open wells in alluvial and other such formations because of its greater structural strength. Open wells are of large size with the diameter usually ranging from 2 to 5 m (Michael et al., 2008), though the diameter may be as large as 20 m (Sarma, 2009) under special circumstances. The open wells of larger size and rectangular in shape are preferred in hard-rock formations to facilitate larger amount of groundwater inflow into the well. The depth of open wells varies from a few meters to about 50 m (Sarma, 2009).

Open wells can be of four types (Michael et al., 2008): (a) unlined open wells, (b) open wells with pervious lining, (c) open wells with impervious lining, and (d) dug-cum-bore wells. They are briefly described in subsequent sub-sections.

8.3.1.1 Unlined and Lined Open Wells

Open wells dug for purely temporary purposes are normally not protected by lining of their walls (Fig. 8.1). The depth of unlined open wells is limited to about 6.5 m in order to ensure stability. However, open wells dug for permanent purposes in loose and unconsolidated formations require lining (steining) to prevent the collapse of side walls and are usually lined with dry bricks or stone masonry (Fig. 8.1). Pervious lining is suitable when the water-bearing formation consists of coarse sand and/or gravel. It is economical and more lasting where aquifer and subsoil conditions are favorable and when the rate of withdrawal is not excessive (Michael et al., 2008).

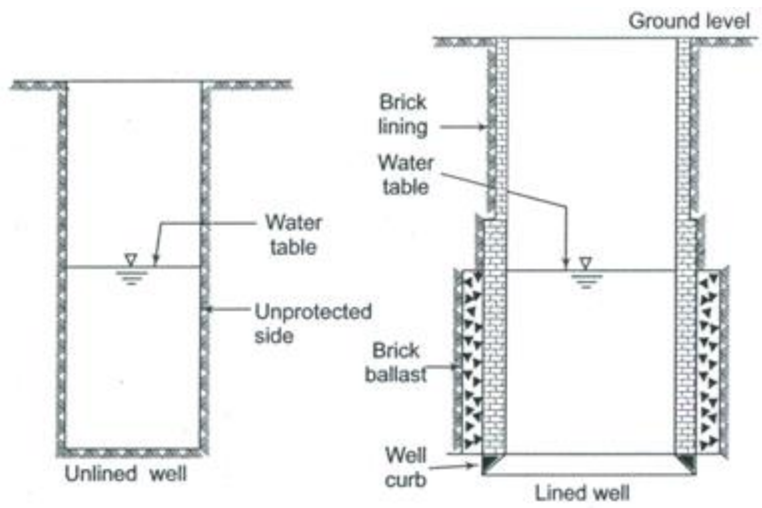

Fig. 8.1. Unlined open well and an open well lined with pervious lining.

(Source: Michael et al., 2008)

Impervious lining such as permanent masonry lining (laid in cement mortar) are normally used in the open wells constructed in alluvial formations (Fig. 8.2). The depth of open wells with impervious linings is generally larger than the two types described above, but the depth usually does not exceed 30 m because of excessive construction cost beyond the 30-m depth. Open wells with reinforced cement concrete (RCC) lining are also sometimes used, especially for greater depths. RCC collar wells (also called ‘ring wells’) are used in some shallow water-table regions mainly for domestic water supply.

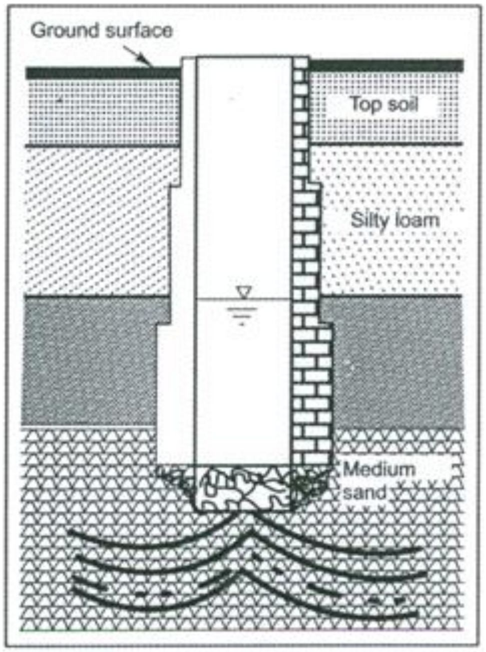

Fig. 8.2. Open well lined with permanent masonry lining.

(Source: Michael et al., 2008)

On the other hand, the open wells in hard-rock areas are excavated pits through the rock and are lined only a couple of meters from top (Fig. 8.3) because the rocky formation ensures the stability of side walls.

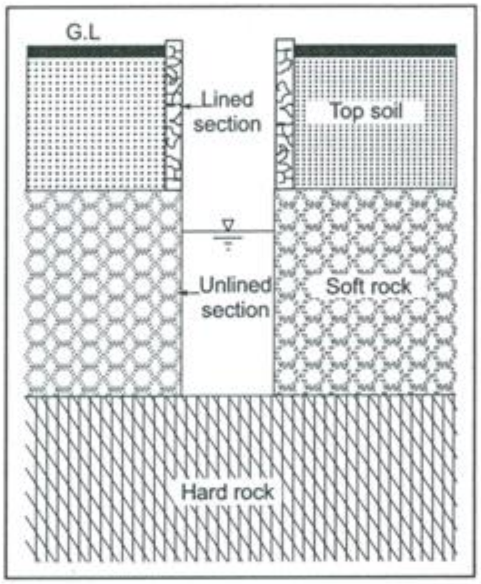

Fig. 8.3. Open well in a hard-rock formation. (Source: Michael et al., 2008)

8.3.1.2 Dug-cum-Bore Wells

Sometimes dug wells are provided with a vertical borehole to augment their yields; such open wells are called Dug-cum-bore wells (Fig. 8.4). The small borehole of size ranging from 4 to 15 cm in diameter is drilled through the bottom of the dug well up to the water-bearing formation lying below the well-bottom. Usually, only one bore is drilled at the center of the dug well constructed in unconsolidated formations. However, in hard-rock formations, the number of bores may range from 1 to 6 depending on the nature of the rock and the size of the dug well (Michael et al., 2008; Sarma, 2009). The vertical bore is provided with a strainer/screen against the aquifer layer and with blind pipes against the non-aquifer layers (Fig. 8.4). Note that dug-cum-bore wells are hydraulically superior to ordinary dug wells and provide higher yields compared to ordinary dug wells. However, their success depends on the availability of a good confined aquifer at a reasonable depth below the bottom of the dug well.

The details about the type of open wells and their design, construction, hydraulics and maintenance can be found in Michael et al. (2008) and Sarma (2009).

8.3.2 Tubewells

Tubewells are wells consisting of pipes ranging in size from 6 to 45 cm in diameter and sunk into an aquifer (Sarma, 2009). Tubewells are constructed by installing a pipe below the ground surface passing through different geological formations comprising water-bearing and non-water-bearing strata. Blind pipes (casing pipes) are placed in the non-water-bearing layers and well screens are placed in the water-bearing layers (Fig. 8.5). Several tubewells have been and are being installed worldwide for meeting water demands in domestic, agricultural and industrial sectors. The type of tubewell to be constructed

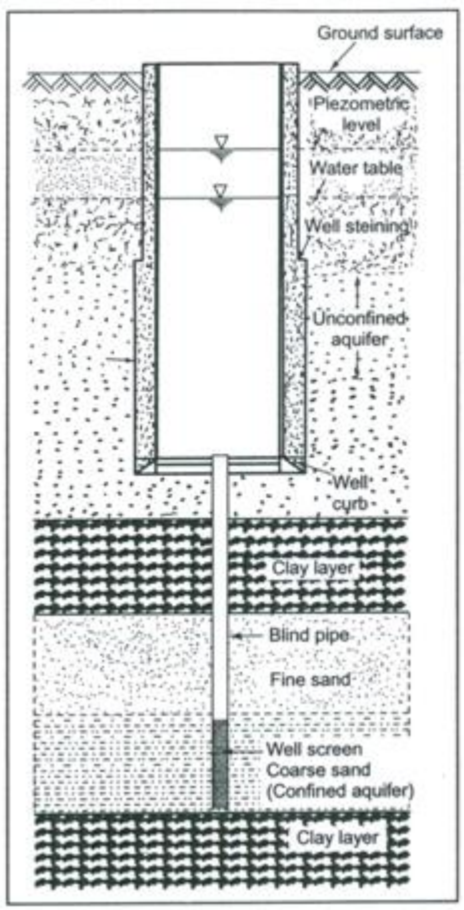

Fig. 8.4. Dug-cum-bore well with a screened vertical bore.

(Source: Michael et al., 2008)

depends on the type of geological formation, intended use of the well and the availability of fund. The design of tubewells is discussed in Lesson 16, while their construction, development and maintenance are discussed in Lessons 17 to 19.

Tubewells are also classified based on the depth, method of construction, entry of water into the wells and the type/nature of the aquifer (Michael et al., 2008; Sarma, 2009). As mentioned above, based on the depth of the well, tubewells

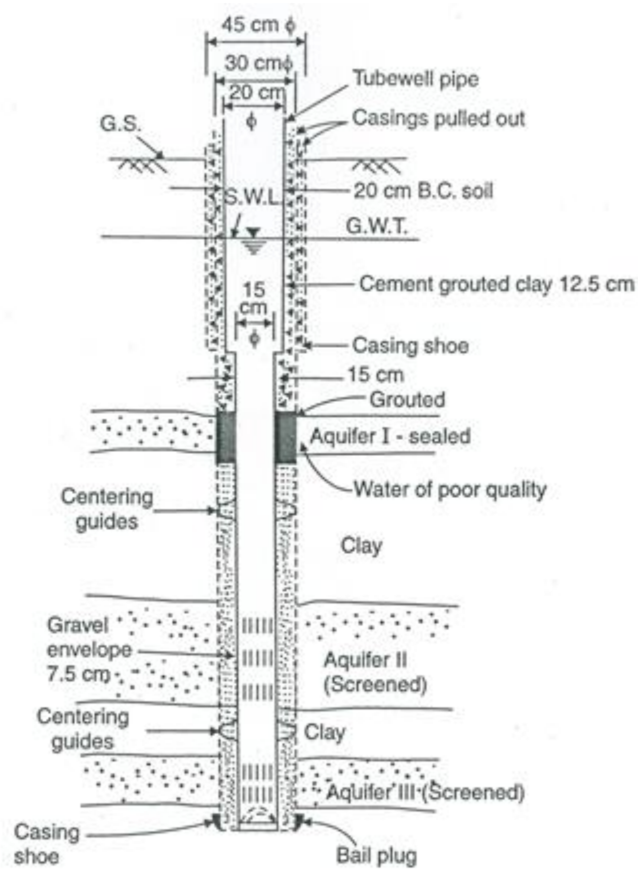

Fig. 8.5. A typical tubewell in an unconsolidated formation.

(Source: Raghunath, 2007)

are classified as shallow tubewells or deep tubewells. Shallow tubewells are of low capacity and their average depth is normally less than 35 m. They mostly tap one aquifer. Deep tubewells are of high capacity and their depth usually ranges from 60 to 300 m (Michael et al., 2008). They often tap two or more aquifers. Based on the method of construction, tubewells are classified as bored tubewells, drilled tubewells, driven tubewells and jetted tubewells; they are described in Lesson 17. Tubewells in unconsolidated formations generally consist of blind pipes, strainers and gravel pack (if necessary). However, tubewells in hard-rock formations are known as borewells, because the borehole remains stable for most of its depth and the tube is placed only in the upper weathered soil zone (Fig. 8.6). No strainer/screen or gravel pack is required for borewells.

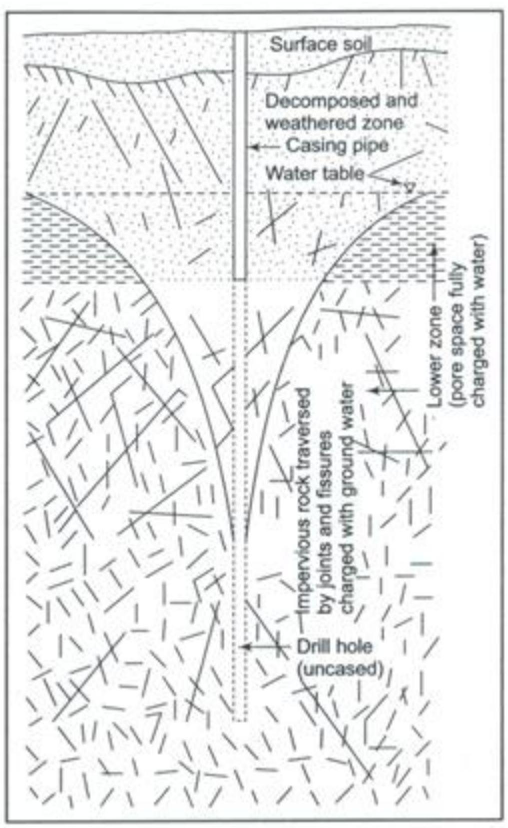

Fig. 8.6. Schematic of a borewell tapping fissured zone in a hard-rock area. (Source: Michael et al., 2008)

Moreover, tubewells are also classified as fully penetrating tubewells or partially penetrating tubewells depending on whether the well screen penetrates the saturated thickness of the aquifer fully or partially. In some special hydrogeologic situations, the drilled hole is terminated at the top of the confined aquifer without penetrating it, and hence no strainer is required; such wells are called cavity wells or non-penetrating wells which are described below. In coastal areas, partially penetrating wells with controlled rate of pumping are used expediently to ‘skim’ the upper layer of fresh water overlying the saline water. Such tubewells are popularly known as skimming wells (Michael et al., 2008; Sarma, 2009).

8.3.3 Cavity Wells

Cavity well is a shallow tubewell drilled in an alluvial formation. If a relatively thin impervious formation consisting of stiff clay, conglomerate or stone is encountered at a shallow depth underlain by an extensive thick sandy confined aquifer, then it is an excellent location for constructing a cavity well. A hole is drilled using the hand boring set, and casing pipe is lowered to rest firmly on the stiff clay layer as shown in Fig. 8.7. Water enters the cavity well through the bottom only and screens are not used in such wells. Thus, the cavity wells do not penetrate the aquifer, and hence they are also known as non-penetrating wells.

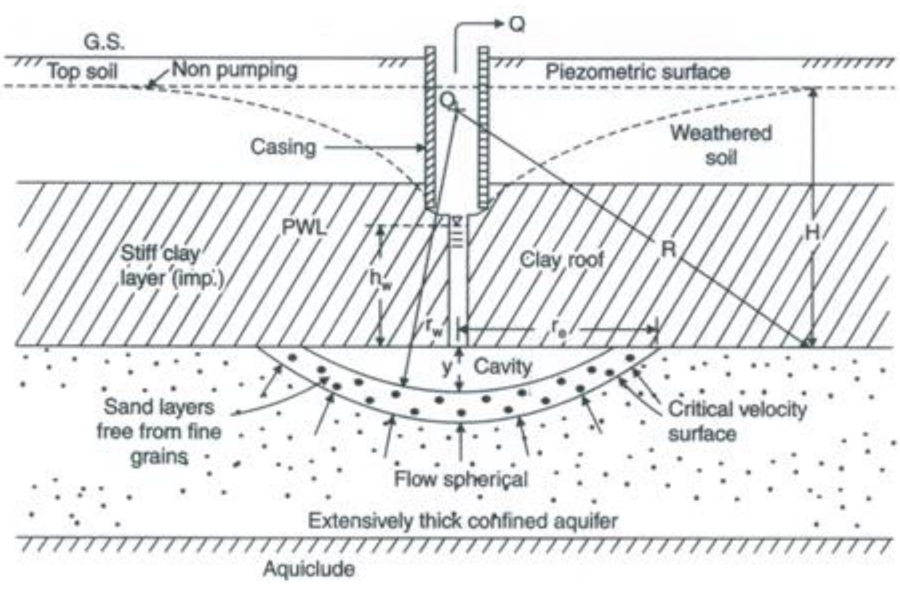

Fig. 8.7. Schematic of a cavity well. (Source: Raghunath, 2007)

A hole of small cross-sectional area is drilled into the sand formation and is developed into a big hollow cavity by pumping at a high rate. In the initial stage of pumping, fine sand comes along with water resulting in the formation of a cavity. During development, the size of the cavity increases till the velocity of groundwater flow at its perimeter becomes small enough to retain the aquifer material in place. With further pumping ultimately equilibrium condition is reached when clean water is discharged. The depth of the cavity at the centre varies from 15 to 30 cm with 6 to 8 m radius of the cavity (Raghunath, 2007).

The flow of water into the cavity is spherical and the yield is low. Cavity wells have usually a shorter lifespan and the failure is caused mainly due to the collapse of the clay roof. Therefore, an essential requirement for a cavity well is that it should have a strong and reliable roof. Since the depth of the cavity well is usually small, deepwell pumps are not necessary. Thus, the capital costs of construction, development and pumpset installation for a cavity well are low, and hence cavity wells are very economical compared to other tubewells.

8.3.4 Filter Points

In deltaic regions, where the aquifer formations mostly consist of coarse sand and gravel, the tubewells are shallow (<15 m deep) and consist of a well screen and a short casing pipe. Such tubewells are called filter points. They are small-diameter (<7.5 cm) tubewells from which water is mostly withdrawn manually (Sarma, 2009).

8.4 Advantages and Disadvantages of Open Wells and Tubewells

The advantages and disadvantages of the open wells and tubewells/borewells are described below (Raghunath, 2007).

8.4.1 Advantages of Open Wells

1. Storage capacity of water is available in the well itself.
2. They do not require sophisticated equipment and skilled persons for constructions.

3. They can be easily operated by installing an ordinary centrifugal pump or by using a manual water-lifting device.
4. They can be revitalized by deepening by vertical boring or by blasting at the bottom, or by creating horizontal or inclined bores on the sides of the well to intercept water-bearing formations.

8.4.2 Disadvantages of Open Wells

(1) Large land space is needed for open wells and for the excavated material.

(2) Construction of open wells is slow and laborious.

(3) They are subject to high seasonal fluctuations of water table.

(4) They are very susceptible to drying up in the years of drought or even during the later part of the dry season.

(5) They involve high cost of construction as their depth increases, especially in hard-rock regions.

(6) Deeper aquifers cannot be economically tapped by open wells.

(7) There is an uncertainty of getting good-quality groundwater.

(8) They are vulnerable to contamination unless they are provided with suitable sanitary protection and are sealed from surface water ingress.

8.4.3 Advantages of Tubewells

(1) They do not require much land space and can be constructed even in a limited open area.

(2) They can be constructed quickly due to the use of mechanized equipment.

(3) They can provide sustained supply of water even during drought years. In other words, tubewells provide the only source of water supply during emergencies (i.e., natural and anthropogenic calamities).

(4) They are economical and more reliable, especially when deep and extensive aquifers are encountered.

(5) They can also serve as flowing wells under special hydrogeologic conditions. In this situation, no water-lifting device and energy are required.

(6) They usually provide good-quality groundwater.

(7) They are relatively less vulnerable to contamination.

8.4.4 Disadvantages of Tubewells

(1) They often require costly and sophisticated drilling equipment.

(2) They need skilled personnel and great care for drilling, completion, and maintenance.

(3) Costly pumps are required for lifting groundwater from borewells.

(4) There is a possibility of missing fractures, fissures and joints in hard-rock regions, thereby resulting in many dry borewells.

(5) Rehabilitation of tubewells/borewells is generally very expensive and requires skilled manpower.

(6) Cost of pumping is normally higher than the open wells.

8.5 Selection of Well Site and Type of Well

The following factors should be carefully studied before selecting suitable sites for constructing wells (Raghunath, 2007):

(1) Topography;

(2) Climate;

(3) Vegetation;

(4) Geology;

(5) Porosity, permeability and alteration of rocks;

(6) Joints and faults in rocks;

(7) Folded strata;

(8) Outcrops in the area (if any);

(9) Proximity of surface water bodies (e.g., tanks, rivers, springs, lakes, unlined channels, reservoirs, etc.); and

(10) Depth and yield of the existing tubewells/open wells in the vicinity.

Apart from the above factors, satellite images and hydrogeological maps of the area are very helpful in making a rapid reconnaissance of the area, where a large-scale well construction program is to be implemented. Also, some well-known facts should be kept in mind while selecting well sites. They are: (i) wells located at the lowest level in valleys generally have a greater possibility of yielding large amount of water than the wells located on slopes or ridges; and (ii) the wells located close to rivers/streams, or within the influence of other surface water bodies like lakes, ponds/tanks and reservoirs will have better yields and will ensure reliable water supply.

Once the preliminary assessment of well sites has been made, and there is no constraint of money and time, geophysical methods of groundwater exploration are also employed, of which electrical resistivity method has been found to be quite helpful in the selection of well sites. In addition, subsurface exploration can be done by test drilling and logging techniques can be used to explore various rock formations at different depths and their water-bearing properties. However, the use of subsurface exploration techniques is essential and economically justified for large water supply projects only.

After determining the purpose and the quantity of water required, the type of well suitable for the purpose can be selected. Besides these two major factors, the following information is also helpful in identifying a suitable type of well: (i) availability of land space, (ii) stratigraphy and hydrogeologic characteristics of the subsurface formations; (iii) seasonal fluctuation of groundwater levels; (iv) cost of well construction and that of water-lifting devices; and (v) the economics of groundwater pumping, which can be ignored if there is no other reliable source of potable water in an area.

As to the quality of groundwater, the groundwater present in igneous rocks is generally acidic in nature and low in mineral contents. The groundwater is hard and brackish in basalts and shales as well as in the alluvium of deltaic areas close to the sea. However, groundwater of good-quality is generally expected from river alluviums and sandstones

Lesson 12 Determination of Aquifer Parameters

12.1 Introduction

Although hydraulic conductivity (K) in saturated zones can be determined by a variety of techniques, the commonly used techniques can be grouped into two major classes: (a) laboratory methods, and (b) field methods. In general, field methods are more reliable than the laboratory methods. Among the field methods, pumping test is the most reliable and standard method for determining K and other hydraulic parameters of aquifer systems. Laboratory methods include grain-size analysis (GSA) method and permeameter methods (‘constant-head permeameter method’ and ‘falling-head permeameter method’). Field methods include tracer test, auger-hole method, slug test, and pumping test. These laboratory and field methods for determining hydraulic conductivity of saturated porous media are succinctly discussed in this lesson.

12.2 Laboratory Methods

12.2.1 Grain-Size Analysis (GSA) Method

Hydraulic conductivity of the aquifer material is related to its grain/particle size. Grain-size analysis (GSA) method is based on predetermined relationships between an easily determined soil property (e.g., texture, pore-size distribution, grain-size distribution, etc.) and the hydraulic conductivity (K). In general, the permeability of porous subsurface formations appears to be proportional to some mean grain diameter squared, which reflects the size of a pore, along with the spread or distribution of grain/particle sizes. Determination of hydraulic conductivity from the grain-size analysis of geologic samples (aquifer or non-aquifer materials) is useful, especially during the initial stage of many groundwater studies such as designing aquifer tests or any preliminary studies when the field measured aquifer hydraulic conductivity is not available.

Grain-size analysis method involves the collection of geologic samples from the field during test drilling or well drilling and their sieve analysis in the laboratory. The collected geologic samples are subjected to sieve analysis by using a set of standard sieves and the results of sieve analysis are expressed as the weight percentage passing (or percentage finer than) the mesh size of each sieve. These data are used to construct a grain-size distribution curve (also known as ‘particle-size distribution curve’) for a given geologic sample. Grain-size distribution curve is constructed by plotting grain/particle sizes on the logarithmic scale on X-axis) and percentage finer by weight on the arithmetic scale on Y-axis as shown in Fig. 12.1. From this curve, one can obtain grain-size values at different values of percent finer; for example, the grain-size value at 10% (denoted by D_{10}) which is called ‘effective grain size’ or the grain-size value at 50% (denoted by D_{50}) which is called ‘mean grain size’.

Several formulae, varying from very simple to complex, based on analytic or experimental work have been developed for the estimation of K from the grain-size distribution data; for example, Hazen formula, Harleman formula, Shepherd formula, Kozeny-Carman formula, Alyamani and Sen formula, etc. (Freeze and Cherry, 1979; Batu, 1998). Of these formulae, the Hazen formula is a simple relationship between the hydraulic conductivity (K) and the effective grain size (or diameter), and it is often used in groundwater hydrology for the estimation of hydraulic conductivity from grain-size distribution data. It is given as (Freeze and Cherry, 1979):

$$K = A \times D_{10}^2 \qquad (12.1)$$

Where, K = hydraulic conductivity, (cm/s); D_{10} = effective grain diameter, (mm) which is determined from the grain-size distribution curve (Fig. 12.1); and A = constant, which is usually taken as 1.0 (Freeze and Cherry, 1979).

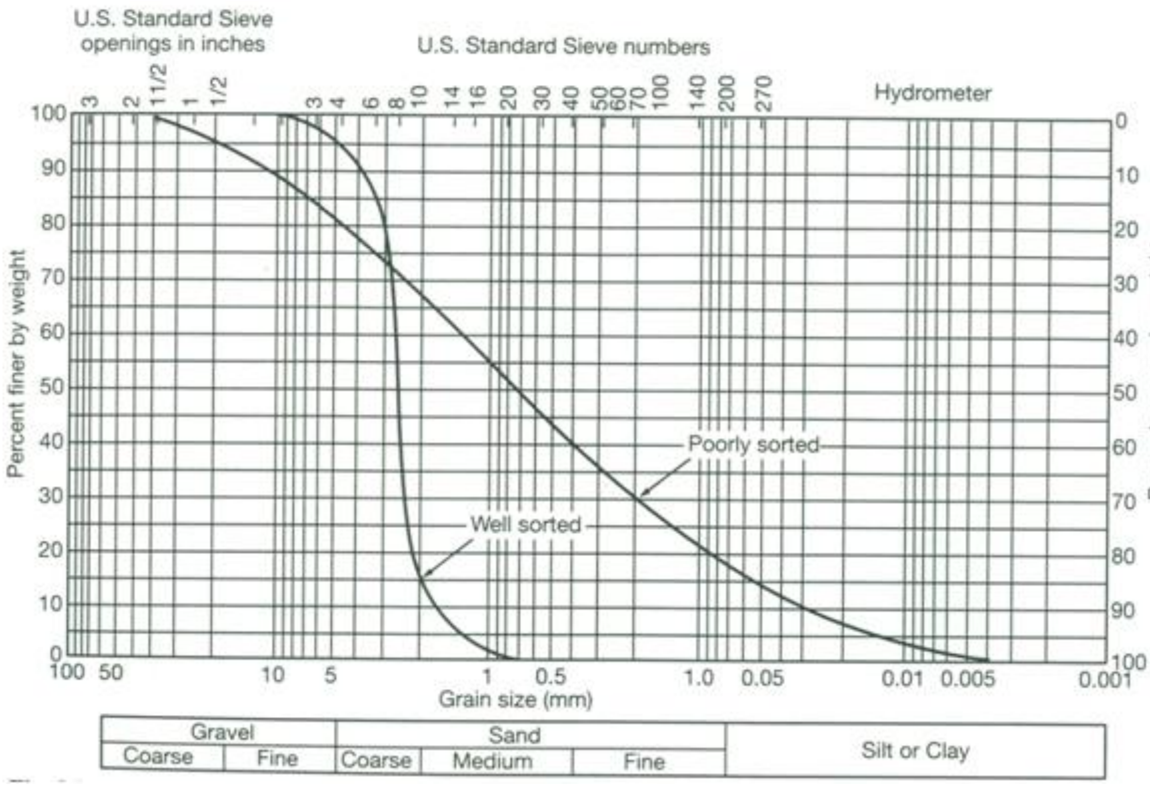

Fig. 12.1. Grain-size distribution curves for well sorted and poorly sorted samples. (Source: Brassington, 1998)

The advantage of the GSA method is that an estimate of the K value is often simpler and faster than its direct determination. However, the major drawback of the method is that the empirical relationship may not be accurate in all cases, and hence may be subject to random errors.

12.2.2 Permeameter Methods

In the laboratory, hydraulic conductivity of undisturbed geologic samples or soil samples can be determined in the laboratory by a permeameter. The permeameter methods essentially provide saturated hydraulic conductivity. If undisturbed geologic samples can be collected from shallow aquifers or confining layers using a core sampler, these samples can be used to determine the saturated hydraulic conductivity of aquifer or non-aquifer materials in the laboratory in the same way as undisturbed soil samples. In permeameters, flow is maintained through a small sample of material while the measurements of flow rate and head loss are made. The constant-head and falling-head types of permeameters (Fig. 12.2) are simple to operate and widely used.

The constant-head permeameter [Fig. 12.2(a)] can measure hydraulic conductivities of consolidated or unconsolidated formations under low heads.

Water enters the medium cylinder from the bottom and is collected as overflow after passing upward through the material. From the Darcy's law, the hydraulic conductivity (K) can be expressed as:

$$K = \frac{VL}{Ath} \qquad (12.2)$$

Where, V = flow volume collected during time t, A = cross-sectional area of the sample, L = length of the sample, and h = constant head applied to the sample.

It is important that the sample be thoroughly saturated to remove entrapped air. Several different heads in a series of tests provide a reliable measurement.

A second procedure utilizes the falling-head permeameter as shown in Fig. 12.2(b). In this case, water is added to the tall tube; it flows upward through the cylindrical sample and is collected as overflow. The test consists of measuring the rate of fall of the water level in the tube. The hydraulic conductivity (K) can be obtained by noting that the flow rate in the tube must equal that through the sample. Flow rate in the tube (Q) is given as:

$$Q = \pi r_t^2 \times \frac{dh}{dt} \qquad (12.3)$$

and the flow rate through the sample is given by Darcy's law as:

$$Q = \pi r_c^2 \times K \times \frac{h}{L} \qquad (12.4)$$

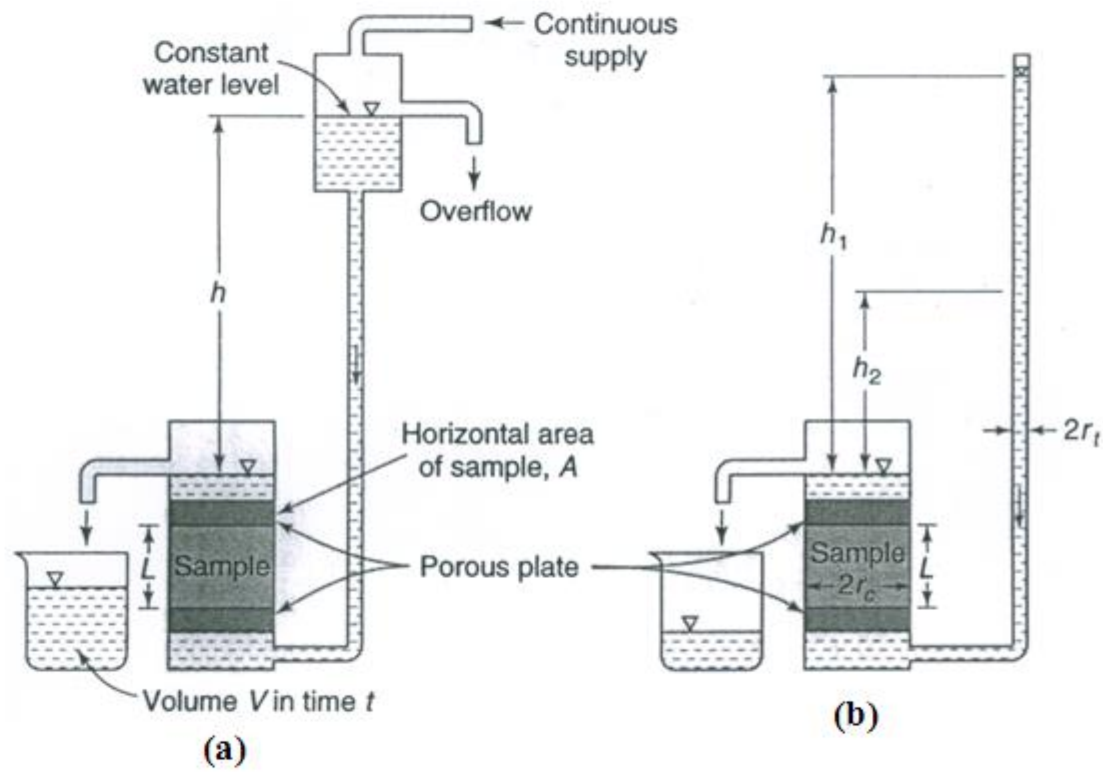

Fig. 12.2. Permeameters for measuring saturated hydraulic conductivity of geologic or soil samples: (a) Constant-head permeameter; (b) Falling-head permeameter. (Source: Mays, 2012)

After equating Eqns. (12.3 and 12.4) and integrating, we have:

$$K = \frac{\pi r_t^2 \times L}{\pi r_c^2 \times t} \times \ln \frac{h_1}{h_2} \qquad (12.5)$$

Where L, r_t, and r_c are shown in Fig. 12.2b, and t is the time interval for the water level in the tube to fall from h_1 to h_2.

Permeameter results may bear little relation to actual field hydraulic conductivities. Undisturbed samples of the unconsolidated subsurface formation (aquifer or non-aquifer material) are difficult to obtain, while disturbed samples are not representative of actual field conditions because they experience changes in porosity, packing, and grain orientation, which modify hydraulic conductivities. Note that one or even several samples from an aquifer may not represent the overall hydraulic conductivity of an aquifer. Variations of several orders of magnitude frequently occur for different depths and locations in an aquifer (Todd, 1980). Also, directional properties of hydraulic conductivity cannot be recognized by the laboratory methods.

12.3 Field Methods

12.3.1 Tracer Test

Field determination of hydraulic conductivity can be made by measuring the time interval for a water tracer to travel between two observation wells or test holes. For the tracer, a dye such as sodium fluorescein, or a salt such as calcium chloride is convenient, inexpensive, easy to detect and safe. Fig. 12.3 shows the cross section of a portion of an unconfined aquifer with groundwater flowing from Hole A toward Hole B. The tracer is injected as a slug in Hole A, after which water samples are taken from Hole B to determine the time taken by the tracer to reach Hole B. As the tracer flows through the aquifer with an average interstitial velocity or seepage velocity (V_s), V_s needs to be computed and it is given as follows:

$$V_s = \frac{K}{n_e} \times \frac{h}{L} \qquad (12.6)$$

Where, K = hydraulic conductivity of the aquifer, n_e = effective porosity of the aquifer, h = head difference between the two holes/observation wells (Fig. 12.3), and L = distance between the two holes/observation wells (Fig. 12.3).

However, V_s can also be calculated as:

$$V_s = \frac{L}{t} \qquad (12.7)$$

Where, t is the time taken by the tracer to travel from Hole A to Hole B.

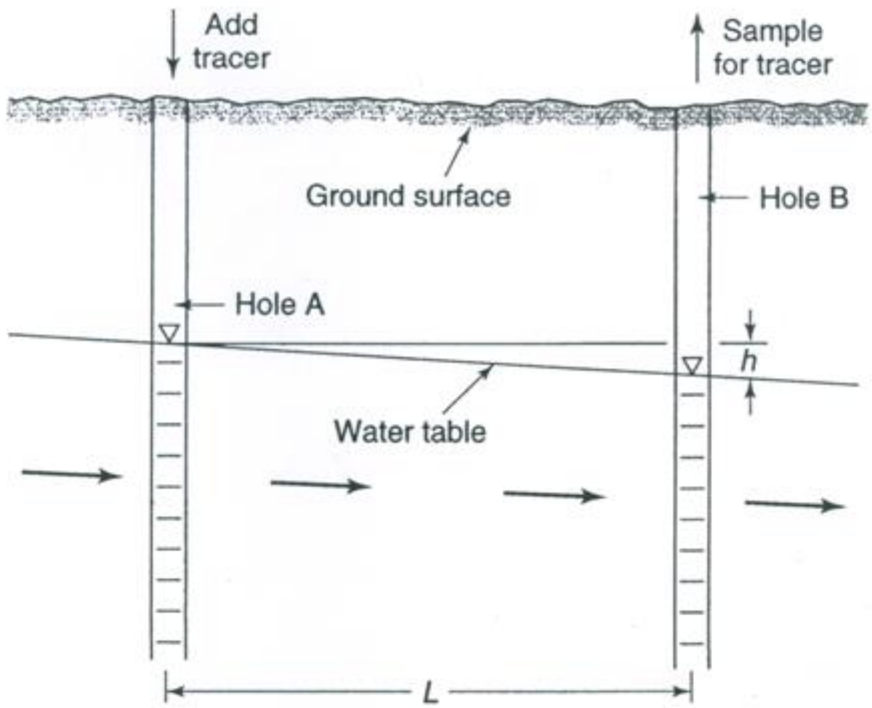

Fig. 12.3. Illustration of a tracer test in an unconfined aquifer for determining hydraulic conductivity. (Source: Mays, 2012)

Equating Eqns. (12.6) and (12.7) and solving for K yields:

$$K = \frac{n_e \times L^2}{ht} \qquad (12.8)$$

Although the tracer test is simple in principle, its results are only approximations because of serious constraints in the field. Therefore, this test should be conducted considering the following limitations (Todd, 1980):

(1) The holes/observation wells need to be close together; otherwise, the travel time interval can be excessively long.

(2) Unless the flow direction is accurately known, the tracer may miss the downstream hole entirely. In this case, multiple sampling holes can help, but it will increase the cost and complexity of conducting the tracer test.

(3) If the aquifer is stratified with layers with differing hydraulic conductivities, the first arrival of the tracer will result in the hydraulic conductivity considerably larger than the average hydraulic conductivity of the aquifer.

An alternative tracer technique, which has been successfully applied under field conditions, is the point dilution method (Todd, 1980). In the point dilution method, a tracer is introduced into an observation well and thoroughly mixed with the groundwater present in the observation well. Thereafter, as water flows into and from the well, repeated measurements of tracer concentration are made. Using these data, a dilution curve is plotted. The groundwater velocity can be obtained from the analysis of the dilution curve. Using the groundwater velocity, measured water-table gradient and Darcy's law, we can obtain a localized estimate of the aquifer hydraulic conductivity as well as the direction of groundwater flow.

Example Problem:

A tracer test was conducted in an unconfined aquifer to determine its hydraulic conductivity. For this, two observation wells were installed 30 m apart and the hydraulic heads at these two locations were measured as 20.5 m and 18.4 m, respectively. During the test, it was found that the tracer injected in the first observation well arrived at the second observation well in 180 hours. If the effective porosity of the aquifer is 18%, calculate the hydraulic conductivity of the unconfined aquifer.

Solution:

Given: Hydraulic head difference between the two observation wells (h) = 20.5 m – 18.4 m = 2.1 m, distance between the two observation wells (L) = 30 m, effective porosity (n_e) of the aquifer = 18% = 0.18, and the time taken by the tracer to travel a distance of L (t) = 180 h = $180 \div 24$ = 7.5 days.

Using Eqn. (12.8) for computing the hydraulic conductivity of the aquifer (K) and substituting the above values, we have:

$$K = \frac{\eta_e \times L^2}{ht} = \frac{0.18 \times 30^2}{2.1 \times 7.5} = 10.29 \text{ m/day, Ans.}$$

12.3.2 Auger-Hole Method

The auger-hole method involves the measurement of the change in water level after the rapid removal of a volume of water from an unlined cylindrical hole. If the soil is loose, a screen may be necessary to maintain the test-hole geometry. The method is relatively simple and is most adapted to shallow water-table conditions. The value of hydraulic conductivity (K) obtained is essentially horizontal hydraulic conductivity (K_h) in the immediate vicinity of the test hole.

Figure 12.4 illustrates an auger hole and the dimensions required for the computation of hydraulic conductivity. The hydraulic conductivity is given as (Todd, 1980):

$$K = \frac{C}{864} \times \frac{dy}{dt} \qquad (12.9)$$

Where $\frac{dy}{dt}$ is the measured rate of rise in cm/s and the factor 864 yields K values in m/day. The factor C is a dimensionless constant governed by the variables shown in Fig. 12.4 and its value can be obtained from the standard table given in Todd (1980) or Mays (2012).

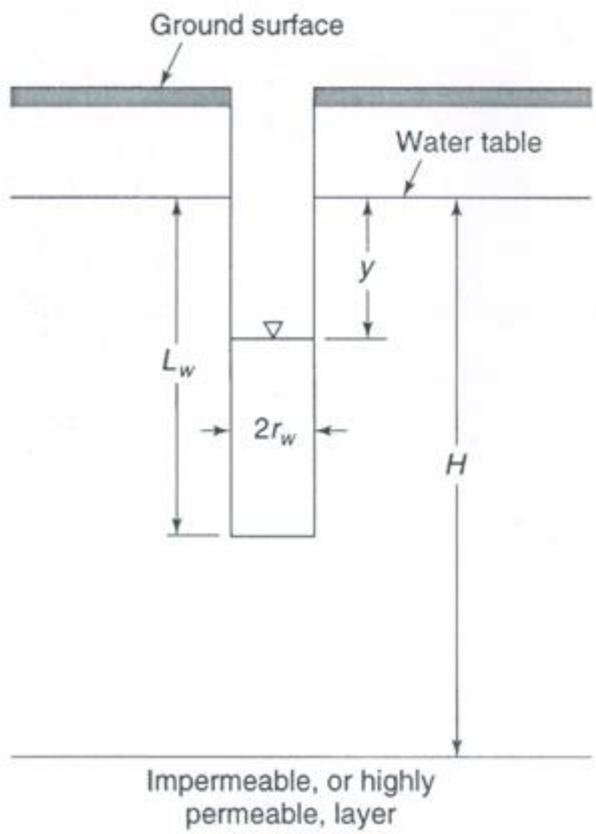

Fig. 12.4. Schematic of an auger hole and its dimensions for determining aquifer hydraulic conductivity. (Source: Mays, 2012)

Several other techniques similar to the auger-hole method have been developed in which water level changes are measured after an essentially instantaneous removal or addition of a volume of water. With a small-diameter pipe driven into the ground, K can be found by the piezometer method, or tube method (van Schilfgaarde, 1974).

12.3.3 Slug Test

Pumping tests are typically expensive to conduct because of the installation costs of wells. Where a pumping test cannot be conducted, the slug test serves as an alternative approach for determining aquifer parameters. However, the aquifer parameters obtained by slug tests are representative of a smaller area (the area in the vicinity of the well in which slug tests are conducted). Nevertheless, slug test has been used for several years as a cost-effective and quick method of estimating the hydraulic properties of confined and unconfined aquifers. More recently (since the 1980s) it has gained even more popularity in: (i) obtaining estimates of hydraulic properties of contaminated aquifers where treating the pumped water is not desirable or feasible, and (ii) field investigations of low-permeability materials, particularly for studies of potential waste storage or disposal sites (Mays, 2012). The materials at these sites may have a hydraulic conductivity which is too low to be determined by pumping tests.

Slug test consists of measuring the recovery of head in a well after near instantaneous change in head at that well. A solid object (slug) is rapidly introduced into or removed from the well, causing a sudden change (increase or decrease) in the water level in the well. Tests can also be performed by introducing an equivalent volume of water into the well; or, an equivalent volume of water can be removed from the well, causing a sudden decrease in the water level. Following the sudden change in head, the water level returns to the static water level. While the water level is returning to the static level, the head is measured as a function of time (referred to as the response data). These response data are used to determine the hydraulic properties of the aquifer using one of several methods of analyses. Various methods have been developed for the analysis of slug-test data obtained from different slug-test designs in confined and unconfined aquifers. A comprehensive description about the methodology of slug tests and their data analysis can be found in Butler (1998), while a summary of slug tests and their applications is presented in Mays (2012) and Fetter (2000).

12.3.4 Pumping Test

To date, pumping test is the most reliable method for determining aquifer hydraulic conductivity. In the pumping test designed for aquifer parameter determination, a pumping well is pumped and the resulting drawdown is measured in one or more observation wells located at varying distances from the pumping well (within its radius of influence). The time-drawdown data thus obtained at a given location are analyzed to determine hydraulic parameters of confined, unconfined and leaky aquifers. A properly designed pumping test can also yield the hydraulic parameters of leaky confining layers (aquitards). Thus, an integrated K value over a sizable aquifer section can be obtained by pumping tests. Unlike the laboratory methods, the aquifer is not disturbed by pumping test, and hence the reliability of pumping test is superior to the laboratory methods. The details of pumping test and the determination of aquifer parameters from pumping-test data analysis are given in Lessons 13 and 14, respectively.

Lesson 13 Overview of Field Pumping Test

13.1 Introduction

As mentioned in Lesson 12, pumping test is the most reliable and standard method for determining hydraulic parameters of different aquifer systems. Proper knowledge of the lithology and types of aquifers present in an area or basin are pre-requisite for designing and conducting efficient pumping tests, which in turn can ensure good-quality pumping-test data. Good-quality pumping-test data are vital for the determination of accurate or dependable aquifer parameters, which are the key to the accurate hydraulic characterization of an aquifer system. In this lesson, the fundamentals of pumping test are discussed. For the detailed information about pumping tests, interested readers are referred to Kruseman and de Ridder (1994), Roscoe Moss Company (1990), Batu (1998), Kasenow (2001), and Freeze and Cherry (1979). The methods of analyzing different types of pumping-test data for determining hydraulic parameters of aquifer systems will be discussed in Lesson 14.

Pumping test (sometimes also called field pumping test) can be defined as a field investigation in which a well is pumped in a specific fashion and the resulting drawdowns are measured in the pumping well itself and/or observation wells installed at different locations over the groundwater basin under investigation. Many modern books on groundwater/hydrogeology mostly use the terminology 'Aquifer test' instead of the widely-used terminology 'Pumping test'. Although the use of both the terms is recommended, the 'aquifer test' is a broader term which encompasses non-conventional tests also. Note that the phrases 'pumping test' and 'pump test' convey completely different meanings, and hence they should be used cautiously!

13.2 Purpose of Pumping Test

The pumping test or aquifer test is the only standard method available to date for determining the hydraulic characteristics of various aquifer systems [e.g., T, K, S or S_y, leakage factor (B) and hydraulic resistance (C)] and those of production wells (e.g., well parameters, safe well yield, etc.). Long-term time-drawdown pumping tests can also provide information about the presence of subsurface hydraulic barriers, if any, as well as the existence of a boundary and its type (recharge, impermeable or leaky boundary). Properly placed observation wells at different locations and in different directions can provide information about the degree of heterogeneity and anisotropy of aquifer systems. Further, step-drawdown tests can provide important information about the hydraulic characteristics of production wells (i.e., aquifer loss coefficient, well loss coefficient, well efficiency, well specific capacity, and safe well yield) and the condition of production wells.

13.3 Types of Pumping Tests

There are primarily four types of pumping tests/aquifer tests. They are as follows:

(1) Time-Drawdown Test:

(a) Interference Test

(b) Distance-Drawdown Test

(c) Single Well Test

(2) Recovery Test

(3) Step-Drawdown Test or Variable Rate Test

(4) Injection Tests:

(a) Time-Groundwater Level Rise Test

(b) Step-Injection Test

Aquifer tests (1) and (2) are mainly used for determining various hydraulic parameters of different aquifer systems (viz., confined, leaky confined and unconfined aquifers). However, the step-drawdown test is used for determining the hydraulic characteristics of production wells and for evaluating the condition of existing production wells.

Instead of a production well, if an injection well is used to conduct a pumping test, it is called an injection test. The injection test can be designed to observe 'time-groundwater level rise' in an injection well itself or in nearby observation wells. In addition, the injection test can be designed to conduct a 'step-groundwater level rise' or 'step-injection' test. Since in injection tests, water is injected into a production well instead of pumping, it is not technically sound to call it a pumping test; aquifer test is a better terminology to assimilate this type of test. Thus, the term 'aquifer test' includes both pumping tests and injection tests.

When aquifer tests 1(a, b, c), (2), and 4(a) are conducted until an equilibrium (steady-state) or quasi-equilibrium (quasi steady-state) condition is reached, they are called steady-state tests. On the other hand, when these tests are conducted for a relatively short period and are completed before the steady-state or quasi steady-state condition is reached, they are called unsteady-state tests or transient tests.

The type of pumping test/aquifer test performed depends on the purpose, available resources (labor, money, instruments/equipment, etc.) and time, and site-specific limitations. Site-specific limitations, for instance, could be limited water disposal facilities, water quality constraints, or noise restrictions.

13.4 Tests for Determining Hydraulic Parameters of Aquifers

13.4.1 Steady and Unsteady Time-Drawdown Tests

Time-drawdown pumping tests are suitable for the determination of aquifer parameters. Based on the type of data yielded by time-drawdown pumping tests, these tests fall into two categories: (a) unsteady-state tests or transient tests, and (b) steady-state tests or equilibrium tests. In both the categories, the pumping rate is kept constant throughout the duration of the test. In unsteady-state tests, groundwater level changes in response to a constant pumping rate are measured over a period of time. The data obtained from transient tests are useful for determining almost all the hydraulic parameters of aquifer systems.

In steady-state tests, pumping is continued until near-equilibrium conditions are reached (i.e., there are negligible changes in groundwater levels with time). In practice, steady-state tests approach only quasi-steady-state conditions in most aquifer systems. This is because true equilibrium may never be obtained under field conditions because of ever-present aquifer recharge or discharge in a groundwater system. The data obtained from steady-state tests can be used to calculate only aquifer transmissivity or hydraulic conductivity and, in some cases, leakance (leakage coefficient); storage coefficient of an aquifer system cannot be determined from these data.

While conducting time-drawdown tests, drawdown can be measured either in one or more observations wells (called ‘interference test’) or in the pumping well itself (called ‘single well test’). They are described below.

13.4.1.1 Interference Test

The measurement of groundwater level changes (drawdowns) in response to pumping in one or more observation wells is known as an interference measurement as shown in Fig. 13.1. Depending upon the nature of the interference test, it can be steady-state test and/or unsteady-state test. Unlike pumping well measurements, interference measurements contain no turbulent flow components, and hence the drawdown measured in observation wells is the representative of that in aquifers. This is the reason that interference-test data are preferred for determining aquifer parameters. For interference tests, it is essential that the observation wells should be completed in the same aquifer as the pumping well.

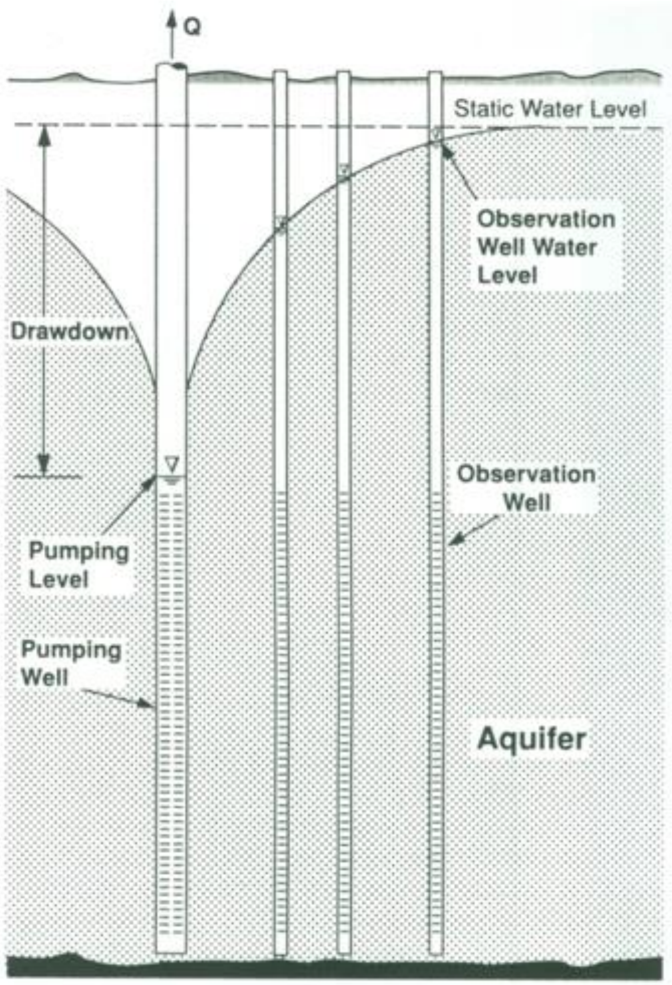

Fig. 13.1. Illustration of an interference pumping test.

(Source: Roscoe Moss Company, 1990)

Determination of aquifer transmissivity (T) or hydraulic conductivity (K) from the steady-state test data usually requires the steady drawdown data from two observation wells or the steady drawdown data from at least one observation well in addition to the pumping well; the drawdown measured in a pumping well needs to be corrected for well losses. However, the steady drawdown data from one observation well or pumping well can also be used for computing K or T, if the radius of influence is known. On the other hand, the determination of aquifer parameters including the storativity (storage coefficient) requires transient drawdown data from one observation well only.

As a rule, the more observation wells available for measurement during a time-drawdown test, the more reliable the information obtained on aquifer characteristics. Since real-world aquifer systems exhibit significant heterogeneity, it is essential that the time-drawdown measurements during a pumping test should be done at as many locations as possible and that time-drawdown tests should be conducted at various sites over a basin or sub-basin so as to account for the aquifer heterogeneity. If the observation wells are oriented in different directions away from the pumping well, aquifer anisotropy can also be determined.

13.4.1.2 Distance-Drawdown Test

In distance-drawdown tests, drawdown is measured in three or more observation wells located at different radial distances from a pumping well, and thereby a set of distance-drawdown data is obtained. This test as such is not conducted separately in the field; rather one can create such dataset from the time-drawdown measurements at different locations during a time-drawdown test. Thus, depending on the nature of time-drawdown tests, the distance-drawdown data can be of two types: distance-steady drawdown data or distance-unsteady drawdown data; the former dataset can yield only aquifer transmissivity (T) or hydraulic conductivity (K), whereas the latter dataset can yield both T and storage coefficient (S). The distance-drawdown data are useful for cross-checking the aquifer parameters obtained from time-drawdown data. In addition, the distance-drawdown data can be used to calculate ‘radius of influence’ (R_0) and ‘well loss’ (and hence ‘well efficiency’).

13.4.1.3 Single Well Test

If the time-drawdown aquifer test is conducted in such a way that the drawdown is measured only in a pumping/production well itself, it is known as a single well test. Single well test is normally conducted when there are no observation wells in a basin or sub-basin and it is not possible to install observation wells due to money and/or time constraints. Hence, single well tests are less expensive than the interference tests.

It should be noted that the drawdown in a pumping well consists of two types of head losses: (a) aquifer loss or formation loss (drawdown due to laminar flow in the aquifer which is known as 'theoretical drawdown'), and (b) well loss (drawdown due to turbulent flow in the immediate vicinity of the pumping well through the screen and/or gravel pack as well as due to flow inside the well to the pump intake) as shown in Fig. 13.2. Since the well loss is associated with turbulent flow, it is proportional to an n^{th} power of the pumping rate (well discharge) where n is a constant greater than one. Thus, the total drawdown in a pumping well or well drawdown (s_w) is given as (Fig. 13.2):

$$s_w = \text{Aquifer Loss} + \text{Well Loss} \qquad (13.1)$$

Or,

$$s_w = BQ + CQ^n \qquad (13.2)$$

Where, B = aquifer/formation loss coefficient, Q = pumping rate (well discharge), and C = well loss coefficient which is a function of the radius, construction and condition of the pumping well.

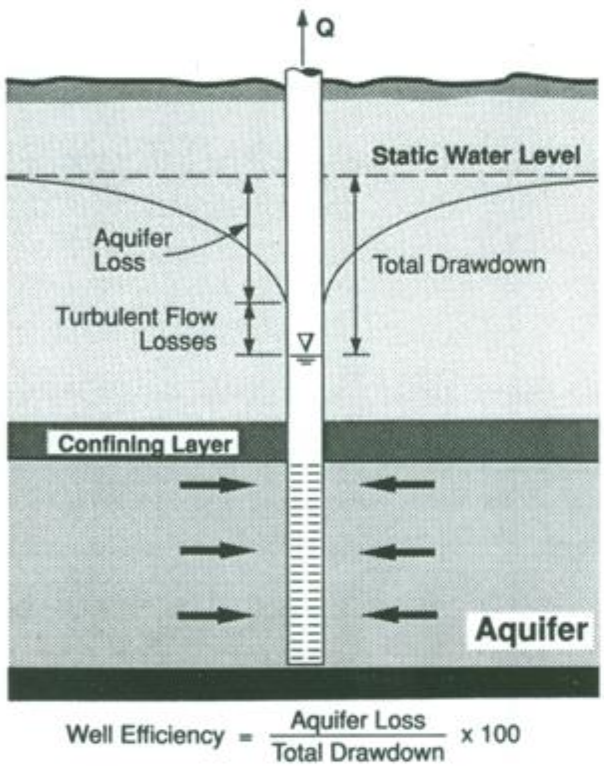

Fig. 13.2. Components of drawdown in a pumping well.

(Source: Roscoe Moss Company, 1990)

It is obvious from Eqn. (13.2) that the aquifer loss increases linearly with increasing pumping rate, while the well loss increases non-linearly with increasing pumping rate. Therefore, the well loss can be a sizeable component of the well drawdown (i.e., total drawdown) when pumping rates are large. The first term (aquifer loss) on the right hand side of Eqn. (13.2) can be calculated by using Thiem equation (if the well drawdown is steady) or Theis equation (if the well drawdown is unsteady). However, the second term (well loss) can be determined by step-drawdown test (described later in this lesson) or can be estimated from distance-drawdown data. Well losses can be minimized by proper design and development of pumping wells. However, clogging or deterioration of well screens can increase well losses in old pumping wells.

Thus, the drawdown measured in pumping wells is not the representative of that in the aquifer. Consequently, single well test is generally avoided unless the value of well loss for a particular pumping well is known so that the measured well drawdowns could be corrected to be representative of aquifer drawdowns. Note that the time-drawdown data obtained from single well tests should be used for the determination of T or K only, because these data do not yield reliable estimates of S.

13.4.2 Recovery Test

Recovery test is conducted at the end of a time-drawdown aquifer test. It is an unnsteady-state aquifer test in which groundwater rise is measured with time in a pumping/production well or in an observation well after pumping has been stopped (Fig. 13.3). It is apparent from Fig. 13.3 that if the pumping of a well is stopped, the groundwater level in the aquifer gradually starts increasing and it should theoretically return to its pre-pumping groundwater level; this time period is known as recovery period or recovery phase. From the water-level data measured during recovery period, ‘residual drawdown’ (difference between the pre-pumping groundwater level and the depth to groundwater after the pump is stopped) or ‘recovery’ (difference between the extrapolated time-drawdown curve and the residual drawdown) is computed at different times during recovery period (Fig. 13.3). The ‘time-residual drawdown’ data or the ‘time-recovery’ data thus obtained are used for determining aquifer parameters.

In general, the data obtained during the recovery period are more reliable than during the pumping period due to the lack of water-level fluctuations caused by discharge variations and the absence of turbulence. Like the time-drawdown data from single well tests, the time-recovery data measured in pumping wells can yield only T or K. However, if the time-recovery data are measured in observation wells, both T and S can be determined.

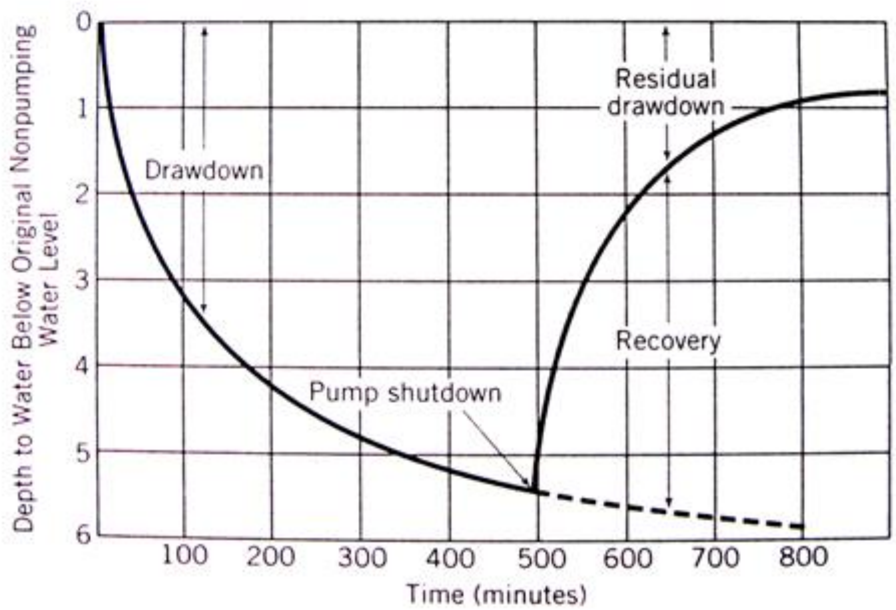

Fig. 13.3. Illustration of time-drawdown and recovery tests.

(Source: Schwartz and Zhang, 2003)

Since additional expenses are not required for conducting a recovery test, it is always recommended to carry out this test during an aquifer-test program for a basin or sub-basin. The recovery test provides an independent and additional estimate of aquifer parameters, which can be compared with the aquifer parameters obtained from time-drawdown data to gain confidence in the analysis.

13.4.3 Injection Tests

In the area where artificial recharge projects using the well injection technique are running, injection tests can be conducted in that area. The test procedures and the equations used for analyzing injection well test data are similar to those of drawdown tests, except that the injection pressure head (i.e., difference between the static groundwater level and the groundwater level during injection) is substituted for drawdown, and the injection rate for pumping rate (Roscoe Moss Company, 1990). Thus, the data of time-groundwater level rise test can be used for determining hydraulic parameters of aquifer systems. Similarly, the data of step injection test can be used for determining the hydraulic characteristics of production wells.

13.5 Test for Determining Well Hydraulic Characteristics

13.5.1 Step-Drawdown Test

In a step-drawdown test (or variable rate test), changes in the drawdown of a pumping/production well are measured corresponding to the changes in pumping rate (i.e., well discharge) as illustrated in Fig. 13.4. This test is normally conducted by pumping a production well at successively greater pumping rates (i.e., $Q_1 < Q_2 < Q_3 < Q_4 < Q_5$) for 5 to 8 steps (pumping rates). The entire test is done in one day. Generally, the drawdown for a particular step (pumping rate) is measured until a steady-state or quasi-steady-state condition is reached. Alternatively, this test can also be conducted by pumping a production well for about 1-2 hours at each successively greater pumping rate and observing unsteady drawdown at the end of each step (i.e., after 1-2 hours of pumping). However, the analysis of 'discharge-unsteady drawdown' data becomes somewhat complicated. Thus, a set of 'discharge-drawdown' data for a particular pumping well is obtained from a step-drawdown test.

The 'discharge-drawdown' data obtained from a step-drawdown test are analyzed to determine the hydraulic characteristics of a production well such as

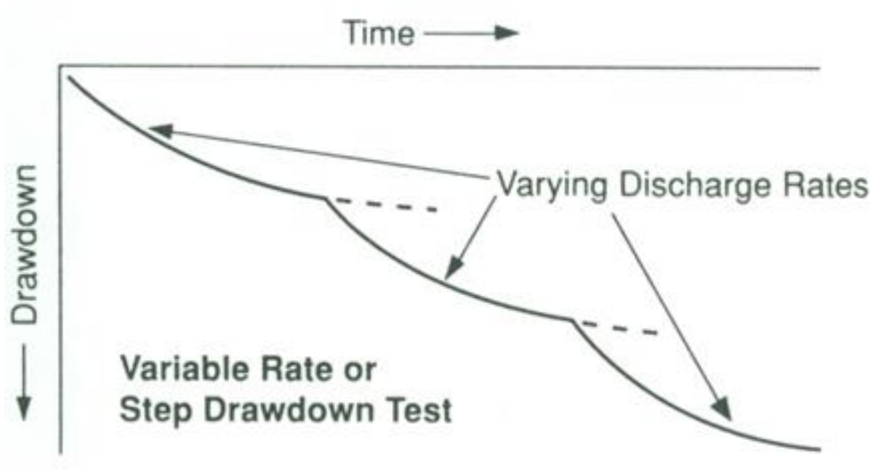

Fig. 13.4. Illustration of a step-drawdown test.

(Source: Roscoe Moss Company, 1990)

safe well yield, aquifer loss and well loss coefficients, specific capacity of the well (ratio of the well discharge to the corresponding drawdown), well efficiency (ratio of the theoretical drawdown to the observed well drawdown as shown in Fig. 13.2), and the relationships between drawdown and discharge, between well efficiency or specific capacity and discharge as well as the evaluation of the condition of existing production wells. Thus, the step-drawdown tests are of great significance for the design and monitoring of pumping plants as well as for estimating site-specific safe aquifer yields over a basin or sub-basin.

13.5.2 Critical Pumpage and Safe Well Yield

Based on the step-drawdown test data, a safe pumping rate or safe well yield for a production well can be determined. The safe well yield is generally defined as the highest pumping rate at which equilibrium conditions can be maintained in the aquifer system (Roscoe Moss Company, 1990). When the equilibrium rate is exceeded, the pumping level continues to decline at a rate proportional to the well and aquifer parameters. When this decline is quite rapid (e.g., several centimeters per minute), the well may dewater in a relatively short time period, with the pump 'breaking suction'; the latter condition is very serious and could result in the collapse of well casing.

The pumping rate (well discharge) at which a rapid decline in groundwater level occurs is known as critical pumpage or critical discharge (Roscoe Moss Company, 1990). This condition varies widely from one area to another as well as from one well to another in an area. For example, in the areas having high-yielding aquifers (i.e., high values of T and S), the full capacity of a pump may not be able to create a critical pumpage situation. However, in the areas having low-yielding aquifers (i.e., low values of T and S), a rapid dewatering of the well may be achieved quite easily. Besides these factors, water quality (e.g., saltwater intrusion or contaminated water migration) also limit the safe pumping rate. This situation necessitates reduced pumping rate below the potential of production wells. Note that the safe pumping rate or safe well yield should always be less than the critical pumpage in order to ensure sustainable groundwater utilization in a basin.

13.6 Design of Pumping Test

Proper planning and design of a pumping test is essential for ensuring good-quality pumping-test data. Besides the adequate knowledge of the local lithology and types of aquifers, a proper selection of production wells to be pumped and the observations wells for drawdown measurement is also necessary. The latter requires the field information such as depth, design and condition of pumping wells; availability of water disposal facility; and the number of observation wells and their depth, design and location. On top of it, some additional decisions are to be made prior to the execution of pumping tests. These points are described in this section.

13.6.1 Placement of Observation Wells and Number of Tests

Ideally, observation wells should be placed in four quadrants surrounding the production well at radial distances ranging from 3 m to 300 m (Roscoe Moss Company, 1990). Spacing between the observation wells should be closer near to the pumping well where drawdown changes are the greatest. Observation wells should be placed considering both the duration of a pumping test and aquifer parameters, because the shape of the cone of depression depends upon these two major factors. For instance, the cones of depression in the aquifers having low transmissivity are steep and limited compared to the broad and flat cones of depression in the aquifers having high transmissivity (Fig. 13.5). Similarly, if the duration of a pumping test is longer, the cone of depression is expected to extend up to a larger distance from the pumping well in comparison with the short-duration pumping test.

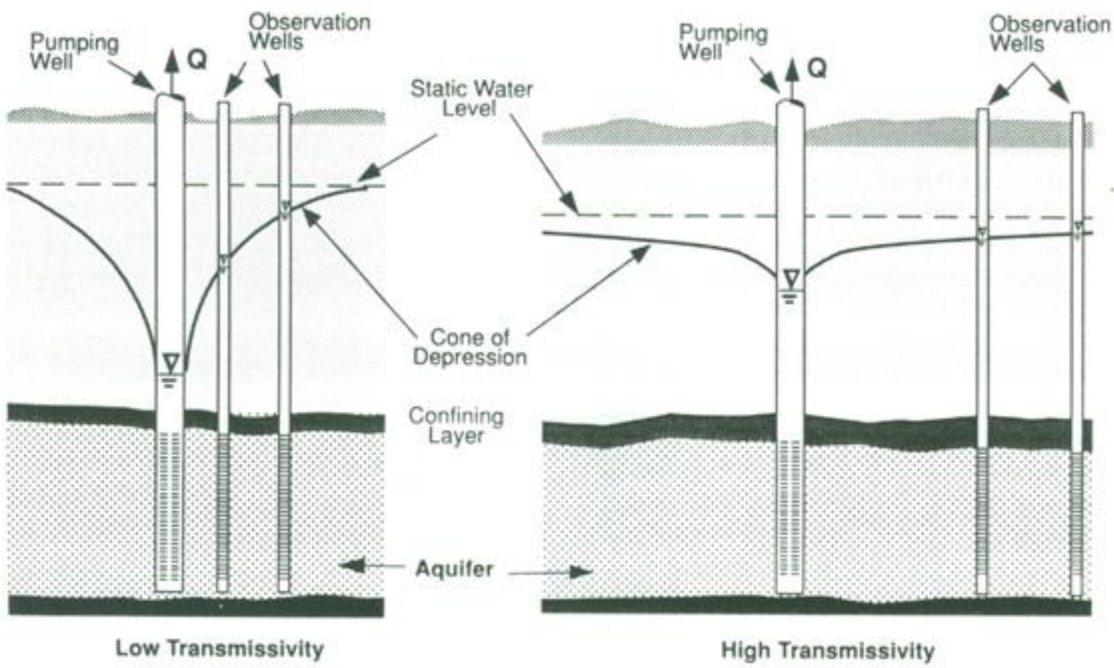

Fig. 13.5. Placement of observation wells for a pumping test.

(Source: Roscoe Moss Company, 1990)

Moreover, proper knowledge of the aquifer type is also important for the placement of observation wells. For example, groundwater level changes in unconfined aquifers may be measurable within only relatively short distances from the pumping well (e.g., 30 to 150 m), whereas confined aquifers may show significant changes in groundwater level hundreds (100 to 250 or more) to thousands of meters away from the pumping well (Roscoe Moss Company, 1990). Furthermore, if hydrogeologic boundaries are known or anticipated, it is desirable to place observation wells between the boundary and the pumping well. In most cases, placing observation wells at about 10 to 100 m from the center of the pumping well provides good results. The observation wells should be placed in the same aquifer in which the pumping well is installed. Also, both the pumping well and the observation well should fully penetrate the aquifer under investigation so as to avoid the complexity in the analysis of pumping-test data.

As far as the number of pumping tests (or aquifer tests) and the number of observation wells are concerned, they depend on the amount of information desired, degree of precision expected, heterogeneity/anisotropy of the aquifer, and the availability of fund and time for investigation. In general, the number should be as many as techno-economically feasible for a better understanding of the spatial variation in aquifer parameters over a basin.

13.6.2 Pumping Rate and Test Duration

The rate of pumping during aquifer tests should be such that an appreciable drawdown occurs in most of the observation wells placed at different radial distances. It should also be ensured that the drawdowns measured during an aquifer test are not affected by any other factors. If any external factor affects the drawdown measured in a given observation well, the data become erroneous (noisy) and such data are usually not considered for analysis.

As to the duration of aquifer tests, it depends on both the test purpose and the hydraulic properties of the aquifer. Since the hydraulic properties of an aquifer are to be determined by aquifer tests, they are not known prior to conducting aquifer tests. Therefore, the duration of aquifer tests mainly depends on whether one wishes to conduct unsteady-state tests or steady-state tests. The duration of unsteady-state tests is shorter (a few hours only) than that of steady-state tests which can vary from several hours to days. Essentially, the duration of an aquifer test should ensure that sufficient number of drawdown data with significant differences from one another is obtained so that the data are well spread in the graph to be plotted during analysis. Under average conditions, a leaky aquifer should be pumped for 15 to 20 hours, a confined aquifer should be pumped for 1 day, and an unconfined aquifer for 3 days or more in order to achieve steady-state (equilibrium) flow condition (Roscoe Moss Company, 1990). As such, there exist no hard and fast universal guidelines for deciding the duration of aquifer tests. However, the guidelines suggested in Table 13.1 can be followed to ensure that the aquifer-test duration exceeds the minimum pumping time mentioned in this table.

Table 13.1. Guidelines for deciding the duration of aquifer tests

(Source: Todd, 1980)

Sl. No.	Predominant Aquifer Material	Minimum Pumping Time
1	Silt and Clay	170 hours
2	Fine Sand	30 hours
3	Medium Sand and Coarser Materials	4 hours

13.6.3 Measurement of Pumping Rate and Groundwater Level

The essential measurements taken during any aquifer test are time, pumping rate (well discharge), and depth to groundwater level (i.e., drawdown or groundwater rise). Measurements of the time of start, stop and pumping interval must be made with a reasonable accuracy (±0.1 min). The pumping rate or well discharge should preferably be kept constant throughout the aquifer test. During an aquifer test, well discharge is measured at regular intervals and an adjustment is made to keep it constant. If the well discharge is small, it can be accurately measured by the volumetric method. However, if the well discharge is large, a water meter or an orifice meter installed in the delivery pipe of the pump, or a 90º V-notch (fitted with a conveyance channel wherein pumped water is diverted for disposal) can be used. The depth to groundwater level can be measured either manually using a water-level indicator and/or automatically by a water-level recorder ('clock and chart type water-level recorder' or 'water-level sensor').

13.6.4 Schedule of Data Collection

Once an aquifer test is initiated, the groundwater levels in a pumping well or an observation well decline (after start of pumping) or recover (after stop of pumping) very rapidly during first one or two hours. Therefore, initially groundwater level is measured at short time intervals and thereafter the time interval is gradually increased as the pumping proceeds. The data-collection schedule presented in Table 13.2 should be followed for measuring groundwater levels in pumping wells and/or nearby observation wells during an aquifer test. Frequency of measurement is less important for the observation wells located at larger distances from the pumping well.

Table 13.2. Frequency of groundwater-level measurement during aquifer tests (Source: Roscoe Moss Company, 1990)

Time Since Pumping Started or Stopped	Time Interval
0-5 min	1 min
5-60 min	5 min
60-120 min (i.e., 1-2 hours)	15 min
2-6 hours	1 hour
6 hours - End of Test	2 hours

13.7 Response of Bounded Aquifers to Pumping

The general theory of groundwater hydraulics assumes that the aquifer is infinite in areal extent. In reality, however, aquifers are not infinite because they can be cut by tight faults (e.g., fault barriers) or they can end abruptly due to changes in geology (e.g., limited aquifer conditions or impermeable bedrock) These impermeable boundaries of an aquifer system effectively halt the spread of the cone of depression and significantly affect the pattern of drawdown in the vicinity of pumping wells (Todd, 1980; Fetter, 1994; Schwartz and Zhang, 2003). Similarly, if an aquifer system is bounded by a surface-water body (e.g., a fully penetrating stream, lake, or reservoir) or an adjacent segment of aquifer having considerably high transmissivity or storativity, these recharge boundaries can halt the spread of a cone of depression by providing a source of recharge to the aquifer. They also influence the pattern of drawdown in the vicinity of pumping wells.

In general, the theory of well hydraulics cannot cope with the presence of one of above-mentioned aquifer boundaries. The method of images, which plays an important role in the mathematical theory of electricity, is employed in conjunction with the principle of superposition for assessing the influence of aquifer boundaries on the well flow (Ferris, 1959). This theory permits treatment of the aquifer that is limited in one or more directions. However, the additional assumption of straight-line boundaries is added, which renders aquifers of rather simple geometric form.

13.7.1 Image Well Theory

Boundaries are considered to be either a recharge boundary or an impermeable boundary (or a barrier boundary). A recharge boundary is a region in which the aquifer is replenished. An impermeable boundary or a barrier boundary is an edge of the aquifer where it terminates either by thinning or abutting a low-permeability formation or has been eroded away (Fetter, 1994).

When a pumping well is located near a recharge or barrier (impermeable) boundary, there are considerable deviations in the radial-flow pattern towards a pumping well. As a result, the solution to the flow towards wells under such conditions becomes complicated and the radial-flow equations (Theim or Theis equations) cannot be applied. For boundaries, the wells that create the same effect as a boundary are called image wells. The theory of image well was formulated (Ferris, 1959) to tackle flow problems in bounded aquifers using analytical techniques. In this theory, the effect of a recharge or leaky boundary is simulated by considering an imaginary recharging well, which is known as a 'recharging image well' (Fig. 13.6). Similarly, the effect of an impermeable boundary is simulated by considering an imaginary pumping well, which is known as a 'discharging image well' (Fig. 13.7). Thus, an image well (imaginary recharging well or imaginary pumping well) creates a hydraulic system equivalent to the effects caused by a physical boundary on the flow system. In essence, image wells enable us to transform the aquifer of finite extent to the aquifer to infinite extent. Consequently, the radial flow equations can be applied to analyze the well flow near aquifer boundaries. This is described below for the two types of aquifer boundaries.

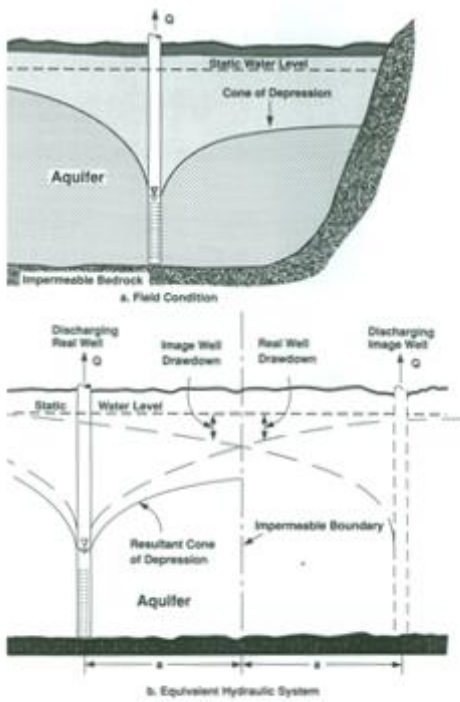

Fig. 13.6. Pumping well near a recharge boundary.

(Source: Roscoe Moss Company, 1990)

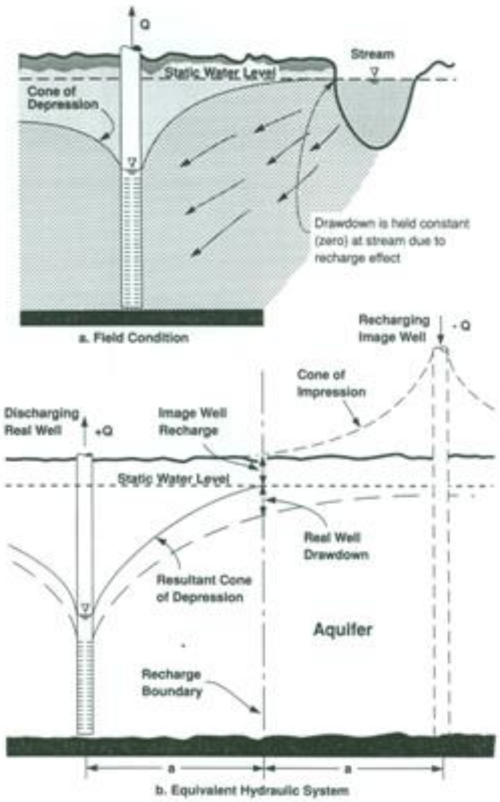

Fig. 13.7. Pumping well near an impermeable boundary.

(Source: Roscoe Moss Company, 1990)

13.7.2 Pumping Near Recharge Boundary

Figure 13.6 shows a pumping well in an aquifer bounded by a recharge boundary. As the recharge boundary provides a source of recharge to the aquifer, once the cone of depression extends to the recharge boundary, it initially stops the spread of a cone of depression and then starts reducing the rate of drawdown. When the rate of recharge from the boundary becomes equal to the rate of pumping (well discharge), steady-state (equilibrium) flow condition is established in the aquifer and the drawdown becomes stationary. As mentioned above, the recharge boundary can be simulated by a recharging image well located an equivalent distance away from the recharge boundary but on the opposite side (Fig. 13.6).

Using the Theis equation, the unsteady drawdown in an observation well under the influence of a recharge boundary can be given as follows:

$$s(r,t) = \frac{Q}{4\pi T}\left[W\left(\frac{4Tt}{r^2 S}\right) - W\left(\frac{4Tt}{r_i^2 S}\right)\right] \qquad (13.3)$$

Using the Cooper-Jacob equation (Cooper-Jacob's approximation for the Theis well function evaluation), we have:

$$s(r,t) = \frac{Q}{4\pi T}\left[\ln\frac{2.25Tt}{r^2 S} - \ln\frac{2.25Tt}{r_i^2 S}\right] \qquad (13.4)$$

Where, s(r,t) = drawdown in an observation well under the influence of a recharge boundary, r = distance of the observation well from the real well (i.e., pumping well), r_i = distance of the observation well from the image well (i.e., recharging image well), and the remaining symbols have the same meaning as defined earlier.

If steady-state flow condition is established in the aquifer, the drawdown will stabilize and will not change with time any longer. In this case, the steady drawdown in an observation well under the influence of a recharge boundary can be given as:

$$s(r) = \frac{Q}{2\pi T}\ln\frac{r_i}{r} \qquad (13.5)$$

If the distance between the pumping well (real well) and the recharge boundary is a and f is the angle between the line joining the real and image wells and the line joining the real well and observation well (Fig. 13.8), then Eqn. (13.5) can be expressed as:

$$s(r) = \frac{Q}{2\pi T}\ln\left(\frac{\sqrt{4a^2 + r^2 - 4ar\cos\phi}}{r}\right) \qquad (13.6)$$

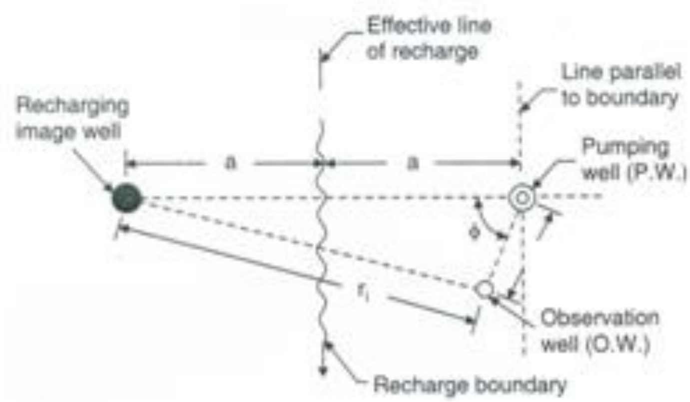

Fig. 13.8. Typical configuration of pumping well, observation well and image well in an aquifer bounded by a recharge boundary. (Source: Raghunath, 2007)

Moreover, the expression for steady drawdown in a pumping well (i.e., steady drawdown at the well face) under the influence of a recharge boundary can be obtained as follows by substituting $r = r_w$ and $r_i = 2a - r_w$ in Eqn. (13.5):

$$s_w = \frac{Q}{2\pi T} \ln\left(\frac{2a - r_w}{r_w}\right) \qquad (13.7)$$

13.7.3 Pumping Near Impermeable Boundary

Figure 13.7 shows a pumping well in an aquifer bounded by an impermeable boundary. Impermeable boundaries have the most dramatic impact on the drawdown of a pumping well for the aquifer with no source of vertical recharge. As the well withdraws water only from the aquifer storage, the drawdown continues as a function of the logarithm of time (Fetter, 1994). The impermeable boundary is simulated by a discharging image well located an equivalent distance away from the boundary but on the opposite side (Fig. 13.7). The unsteady drawdown in an observation well under the influence of an impermeable boundary can be expressed as:

$$s(r,t) = \frac{Q}{4\pi T}\left[W\left(\frac{4Tt}{r^2 S}\right) + W\left(\frac{4Tt}{r_i^2 S}\right)\right] \qquad (13.8)$$

Using the Cooper-Jacob equation (Cooper-Jacob's approximation for the Theis well function evaluation), we have:

$$s(r,t) = \frac{Q}{4\pi T}\left[\ln\frac{2.25Tt}{r^2 S} + \ln\frac{2.25Tt}{r_i^2 S}\right] \qquad (13.9a)$$

Or,

$$s(r,t) = \frac{Q}{4\pi T}\left[2\ln\frac{2.25Tt}{r^2 S} + \ln\frac{r^2}{r_i^2}\right] \qquad (13.9b)$$

Where, r_i = distance of the observation well from the image well (i.e., discharging image well), and the remaining symbols have the same meaning as defined earlier.

Furthermore, the unsteady drawdown in a pumping well (i.e., unsteady drawdown at the well face) under the influence of an impermeable boundary can be expressed as:

$$s_w(t) \text{ or } s(r_w,t) = \frac{Q}{4\pi T}\left[2\ln\frac{2.25Tt}{r_w^2 S} + \ln\frac{r_w^2}{(2a - r_w)^2}\right] \qquad (13.10)$$

13.7.4 Impact of Aquifer Boundaries on Time-Drawdown Curve

Figure 13.9 shows a theoretical straight-line plot of drawdown as a function of time on the semi-logarithmic paper. It is evident from this figure that the impact of a recharge boundary is to slow down (retard) the rate of drawdown. The change in drawdown can become zero if the well is supplied entirely with recharged water. In contrast, the impact of a barrier boundary or an impermeable boundary on flow in some region of the aquifer is to accelerate the drawdown rate (Fig. 13.9). In this case, the groundwater level declines faster than the theoretical drawdown curve (straight line shown in Fig. 13.9).

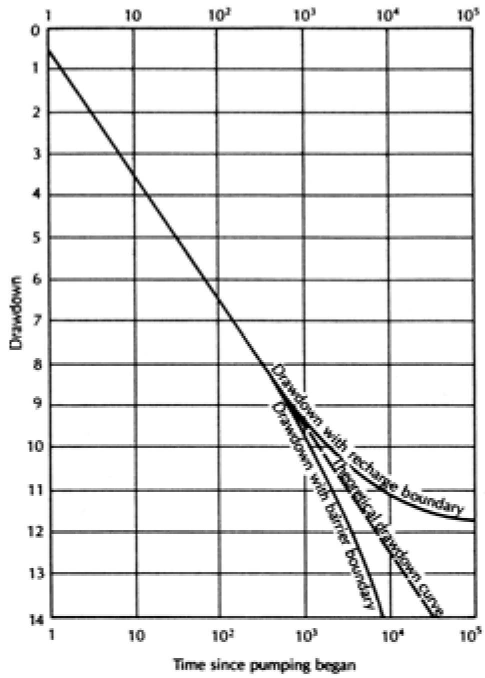

Fig. 13.9. Impact of recharge and impermeable boundaries on the time-drawdown curve. (Source: Redrawn from Fetter, 1994)

13.8 Advantages and Disadvantages of Pumping Tests

The advantages of the pumping test method are self-evident. A pumping test provides in situ parameter values and these values are, in effect, averaged over a large and representative volume (Freeze and Cherry, 1979). From a single pumping test, one can obtain both T and S values. Also, in aquifer-aquitard systems (i.e., leaky aquifer systems), it is possible to get very important leakage properties of the system (e.g., leakage factor, hydraulic conductivity and storage coefficient of aquitards, and leakance). Moreover, the step-drawdown test provides hydraulic characteristics of production wells, including safe well yield.

However, there are two disadvantages of the pumping test (Freeze and Cherry, 1979): one scientific and one practical. The scientific limitation is that the pumping test interpretation is not unique. For example, similarity in timedrawdown response could be found for leaky, unconfined and bounded systems. Thus, unless clear geologic evidence is available, a unique prediction of the effects of any pumping test would be difficult. The fact that a theoretical curve can be matched by the pumping test data in no way proves that the aquifer satisfies the assumptions on which the curve is based. The practical disadvantage of the method lies in its expense, including money, time and labor. Therefore, when only K or T values are required such as in the case of geotechnical and contamination studies or flownet analysis, simple and inexpensive methods like slug test should be used. These simple and cheaper methods are also useful when local conditions do not permit pumping tests or when preliminary and quick results are desired.

Lesson 14 Analysis of Pumping-Test Data

14.1 Introduction

Considering the pivotal role of groundwater in the world's water supply and its gradual depletion coupled with growing contamination, there is an urgent need to investigate the reaction of aquifers to various human activities in terms of both quantity and quality of groundwater so as to avoid severe and often irreversible damages to the mankind and ecosystem. To achieve this broad goal, a prior knowledge of the hydraulic properties of different aquifer systems is a basic necessity for almost all groundwater-related studies. Further, groundwater processes being hidden and highly complex in nature, modeling plays an important role in the planning, design and management of groundwater systems. Adequate knowledge of aquifer parameters is also indispensable for successful and reliable modeling results, and thereby ensuring proper management of vital groundwater resources.

As discussed in Lesson 12, there are several methods for the determination of hydraulic parameters of aquifer systems. However, the pumping test (or 'aquifer test') is the standard and most widely used method for determining the hydraulic parameters of aquifers, viz., transmissivity (T), hydraulic conductivity (K), storage coefficient (S), specific yield (S_y) and leakage factor (B). As mentioned in Lesson 13, the pumping test yields aquifer parameters averaged over a large and representative volume of the aquifer, and hence it is more reliable than the methods providing essentially point estimates (e.g., slug/bail tests and laboratory methods). Different types of pumping tests are available which provide varying types of pumping-test data for confined, unconfined and leaky aquifer systems. Depending on the type of pumping-test data and the type of aquifer in which the test is conducted, a wide range of methods are available for analyzing pumping-test data in order to determine aquifer parameters. Table 14.1 summarizes commonly used methods for analyzing pumping-test data obtained from confined, unconfined and leaky confined aquifers. The detailed discussion on each of the methods is beyond the scope of this course, and hence only selected methods are discussed in this lesson. Interested readers may refer to Kruseman and de Ridder (1994), Fetter (1994), Batu (1998), Kasenow (2001), Schwartz and Zhang (2003), and Michael et al. (2008) for further details about the methods described in this lesson, together with the discussion on other methods of pumping-test data analysis.

In the past, the analyses of pumping-test (aquifer-test) data for determining aquifer parameters or for determining hydraulic characteristics of production wells were done manually only, which is cumbersome and somewhat subjective. However, with a rapid advancement in the computer technology and numerical techniques, it is possible to perform such analyses using a PC (laptop or desktop). Commercial software packages such as AquiferTest developed by the Waterloo Hydrogeologic, Inc., Canada (http://www.swstechnology.com), AQTESOLV (http://www.aqtesolv.com/) and Aquifer^{win32} (http://www.aquifer-test.com/), among some others, are available which enable us to analyze different types of pumping-test data easily and efficiently in considerably less time. These software packages are based on either graphical approaches or numerical approaches to aquifer-test data analysis. In addition, the developer of this course, Prof. Madan Kumar Jha, has copyrighted a user-friendly software package named GA-AquiAnalyzer which facilitates the analysis of aquifer-test data by the genetic algorithm (GA) technique (Samuel and Jha, 2003)

Table 14.1. Commonly used methods for pumping-test data analysis

Sl. No.	Type of Aquifer	Type of Pumping-Test Data	Name of Methods
1	Confined Aquifer	(i) Time-Drawdown data	• Theis Type Curve Method • Cooper-Jacob Straight-Line Method
		(ii) Unsteady Distance-Drawdown data	• Cooper-Jacob Straight-Line Method
		(iii) Quasi-Steady/Steady Distance-Drawdown data	• Thiem Method • Graphical Method
		(iv) Recovery data: - Time-Residual Drawdown data - Time-Recovery data	Residual Drawdown-Time Ratio Method Cooper-Jacob Straight-Line Method

2	Unconfined Aquifer without Delayed Yield	(i) Time-Drawdown data	• Theis Type Curve Method • Cooper-Jacob Straight-Line Method
		(ii) Unsteady Distance-Drawdown data	• Cooper-Jacob Straight-Line Method
		(iii) Quasi-Steady/Steady Distance-Drawdown data	• Thiem Method • Graphical Method
		(iv) Recovery data: - Time-Residual Drawdown data - Time-Recovery data	Residual Drawdown-Time Ratio Method Cooper-Jacob Straight-Line Method
3	Unconfined Aquifer with Delayed Yield	(i) Time-Drawdown data	• Type-Curve Method • Neuman Straight-Line Method
		(ii) Quasi-Steady/Steady Distance-Drawdown data	• Thiem Method • Graphical Method
4	Leaky Confined Aquifer without Storage in Aquitards	(i) Time-Drawdown data	• Walton Type-Curve Method • Hantush Inflection-Point Method
		(ii) Quasi-Steady/Steady Distance-Drawdown data	• Type-Curve Method
5	Leaky Confined Aquifer with Storage in Aquitards	(i) Time-Drawdown data	• Hantush Type-Curve Method
		(ii) Quasi-Steady/Steady Distance-Drawdown data	• Type-Curve Method

14.2 Determination of Confined Aquifer Parameters

As shown in Table 14.1, the hydraulic parameters of confined aquifer systems can be determined using time-drawdown data, distance-drawdown data and recovery data. The methods used for analyzing these pumping-test data are discussed in this section.

14.2.1 Theis Type-Curve Method

It is a graphical method for analyzing time-drawdown data. In this method, the field-data curve is matched with the standard curve known as Theis type curve (Fig. 14.1) and then the hydraulic parameters of confined aquifers viz., T and S can be determined using the Theis equation [Eqn. (11.8)] as follows:

Theis equation can be rearranged as: $T = \frac{Q}{4\pi s} W(u)$ (14.1)

Also, $u = \frac{r^2 S}{4Tt}$ can be rearranged as: $S = \frac{4Ttu}{r^2}$ (14.2)

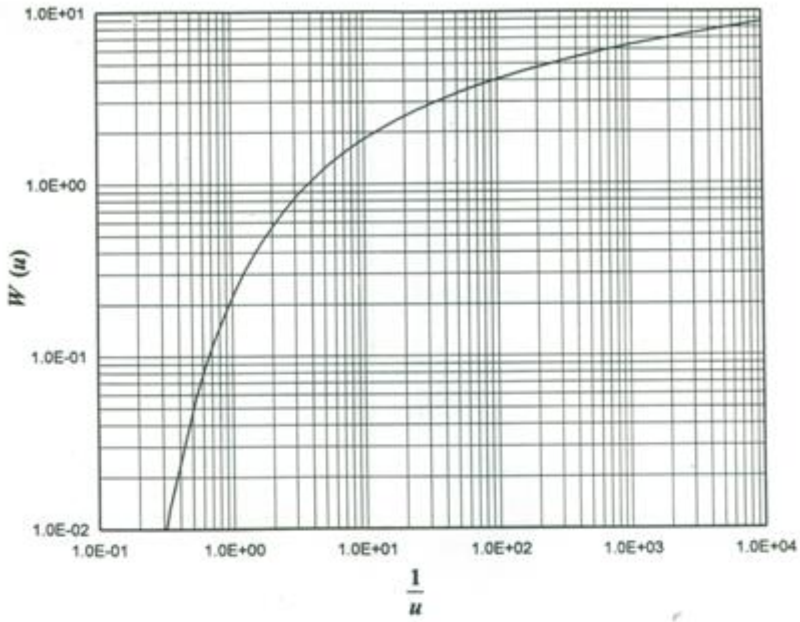

Fig. 14.1. Theis Type Curve for confined aquifers. (Source: Roscoe Moss Company, 1990)

The step-by-step procedure for determining confined aquifer parameters from time-drawdown data by the Theis Type-Curve method is as follows:

Step 1: Construct Theis Type Curve by plotting W(u) and u on the log-log graph paper as shown in Fig. 14.1. Alternatively, obtain a copy of this curve from the literature.

Step 2: Plot field-data curve using observed values of drawdown (s) versus r^2/t on the log-log graph paper having the same scale as the Type Curve.

Step 3: Superimpose the transparent field-data curve on the Type-Curve sheet, keeping coordinate axes of the two graphs parallel to each other. Adjust the field-data curve until a best fit of field data points to the Type Curve.

Step 4: Select an arbitrary 'match point' on the Type Curve and note down the corresponding coordinates (s and r^2/t) from the field-data curve, and W(u) and u from the Type Curve. Note that the selection of (1,1) match point on the Type Curve simplifies the calculation.

Step 5: Finally, substitute the values of these coordinates and the value of Q in Eqn. (14.1) to calculate T. Thereafter, substitute the values of the known variables in Eqn. (14.2) to obtain S.

14.2.2 Illustrative Example 1

Problem: A time-drawdown pumping test was conducted in a groundwater basin. A pumping well tapping a non-leaky confined aquifer was pumped at a constant rate of 200 L/s and drawdowns were measured in an observation well located 45 m away from the pumping well. The measured drawdowns are summarized in Table 14.2.

Table 14.2. Time-Drawdown data of an observation well

Time since pumping started (min)	Drawdown (m)	Time since pumping started (min)	Drawdown (m)
2	0.37	24	2.37
3	0.58	30	2.60
4	0.75	40	2.78
5	0.89	50	2.90
6	1.03	60	3.06
7	1.12	80	3.10
8	1.26	120	3.14
10	1.41	180	3.20
14	1.69	240	3.26
18	2.15	360	3.33

Calculate transmissivity (T) and storage coefficient (S) of the confined aquifer at 45 m location by the Theis Type-Curve Method.

Solution: The above time-drawdown data are analyzed by the Theis Type-Curve Method with the help of AquiferTest software. The matching of the field-data curve with the Theis Type-Curve using AquiferTest software is shown in Fig. 14.2. From this analysis, the value of T is obtained as 1373.76 m^2/day and that of S is obtained as 0.0027, which are automatically yielded by the software once reasonable matching is achieved.

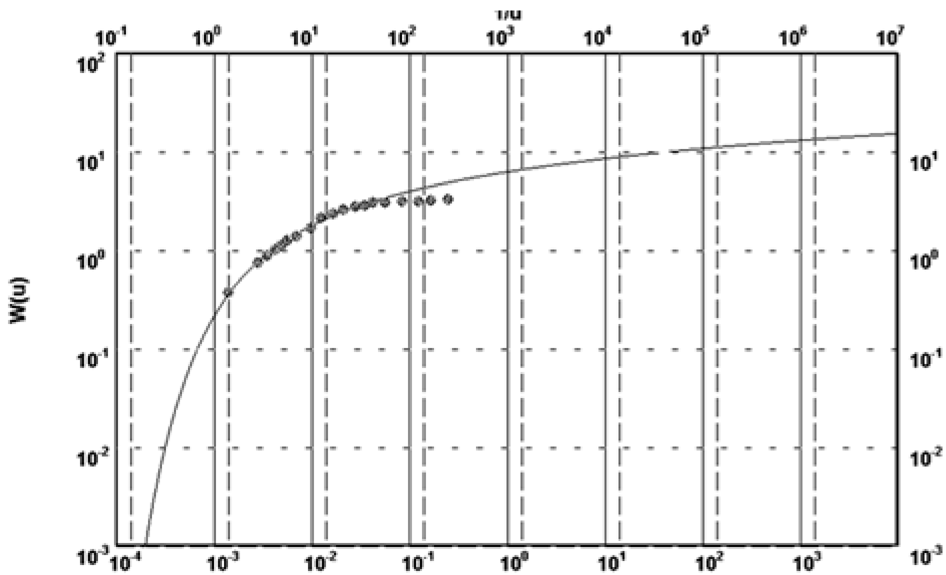

Fig. 14.2. Matching the field data with the Theis Type Curve.

14.2.3 Cooper-Jacob Straight-Line Method for Time-Drawdown Data

The Cooper-Jacob equation [Eqn. (11.11)] expresses drawdown (s) as a linear function of ln(t) or log(t) if the limiting condition ($u \leq 0.01$) is satisfied. This can be true for the large values of t and/or small values of r. Thus, the straight-line plot of drawdown (s) versus time (t) on the semi-logarithmic paper can occur after sufficient time has elapsed since the start of pumping. In case of multiple observation wells, the closer observation wells will meet the conditions earlier than the more distant ones.

The step-by-step procedure for determining confined aquifer parameters from time-drawdown data by the Cooper-Jacob straight-line method is as follows:

Step 1: Plot a field-data curve (s versus t) on the semi-logarithmic graph paper with the time on the X-axis (logarithmic scale) and the drawdown on the Y-axis (arithmetic scale).

Step 2: Fit a straight line to the field-data points.

Step 3: Extend the fitted straight line backward to intercept the zero-drawdown line and designate this time t_o.

Step 4: Compute the change in the value of the drawdown per log cycle (i.e., Δs) from the slope of the straight line.

Step 5: Finally, compute the values of T and S by using the following equations:

$$T = \frac{2.3Q}{4\pi\Delta s} \tag{14.3}$$

and

$$S = \frac{2.25Tt_o}{r^2} \tag{14.4}$$

14.2.4 Illustrative Example 2

Problem: To demonstrate the application of the Cooper-Jacob straight-line method, let's consider the same problem as mentioned in Illustrative Example 1. In the present case, we have to calculate transmissivity (T) and storage coefficient (S) of the confined aquifer at 45 m location by the Cooper-Jacob straight-line method.

Solution: Following the step-by-step procedure of the Cooper-Jacob straight-line method, a graph of drawdown versus log(t) is prepared and a straight line is fitted through the data points (after eliminating the data that considerably deviate from the straight line) as shown in Fig. 14.3.

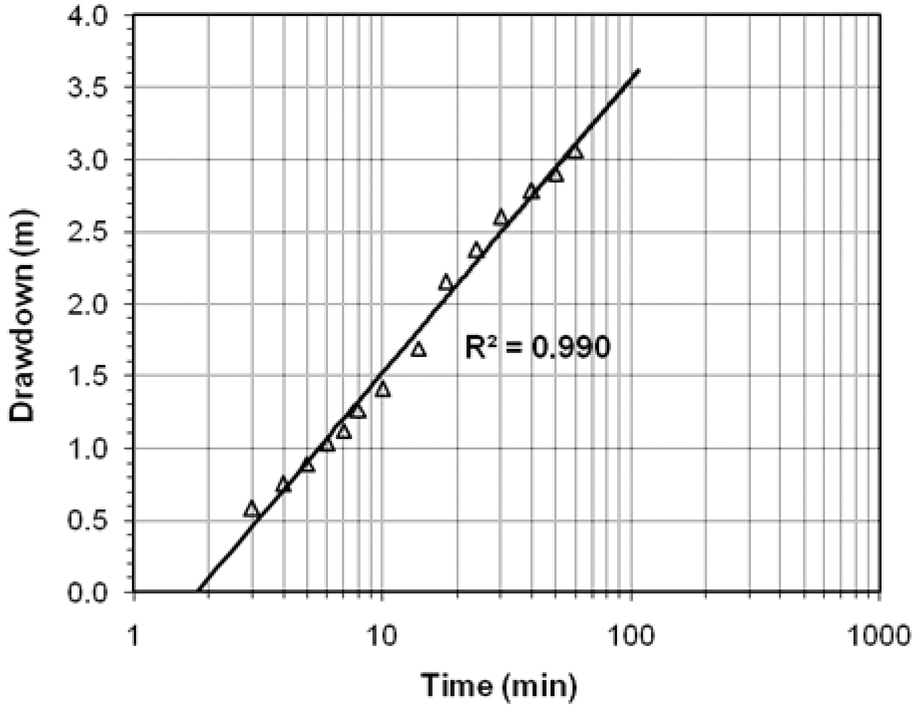

Fig. 14.3. Illustration of the Cooper-Jacob straight-line method.

From the graph, we have t_0 (time corresponding to the zero drawdown) = 1.8 min, and Δs (drawdown per log cycle) = 3.55−1.5 = 2.05 m. Therefore, transmissivity (T) of the confined aquifer is:

$$T = \frac{2.3Q}{4\pi\Delta s} = \frac{2.3\times(200\times 60\times 60\times 24/1000)}{4\pi\times 2.05} = \frac{39744}{25.761} = 1542.80\ \mathrm{m^2/day},\ \mathrm{Ans.}$$

Now, storage coefficient (S) of the confined aquifer is:

$$S = \frac{2.25Tt_0}{r^2} = \frac{2.25\times\dfrac{1542.80}{24\times 60}\times 1.8}{45^2} = 0.00214,\ \mathrm{Ans.}$$

Check: $$u = \frac{r^2 S}{4Tt} = \frac{45^2 \times 0.00214}{4 \times \frac{1542.80}{60 \times 24} \times 10} = \frac{4.3335}{42.8556} = 0.101$$

Since in the maximum value of u is larger than 0.01 (validity criterion of the Cooper-Jacob straight-line method), the Cooper-Jacob straight-line method is not strictly applicable to the present problem. Nevertheless, in this example, the value of S obtained by the Cooper-Jacob straight-line method is quite comparable with that yielded by the Theis Type Curve method, but the value of T is overestimated by the Cooper-Jacob straight-line method.

14.2.5 Cooper-Jacob Straight-Line Method for Distance-Drawdown Data

If the drawdown is measured at the same time in several observation wells, it is found to vary with the distance from the pumping well in accordance with the Theis equation. If simultaneous measurements of the drawdown are made at a given time in three or more observation wells, the Cooper-Jacob straight-line method for time-drawdown data can be used after a minor modification. For example, let’s assume that two observation wells are located at distances r_1 and r_2 from the pumping well where drawdowns are measured at some time t as s_1 and s_2, respectively. Using the Cooper-Jacob equation [Eqn. (11.11b)], we have:

$$s_1 = \frac{2.3Q}{4\pi T} \log \frac{2.25Tt}{r_1^2 S} \qquad (14.5)$$

and $$s_2 = \frac{2.3Q}{4\pi T} \log \frac{2.25Tt}{r_2^2 S} \qquad (14.6)$$

From Eqns. (14.5) and (14.6), we have:

$$s_1 - s_2 = \frac{2.3Q}{2\pi T} \log \frac{r_2}{r_1} \qquad (14.7)$$

Now, the Cooper-Jacob straight-line method described in Section 14.2.2 can be used as follows to determine confined aquifer parameters from the distance-drawdown data:

Step 1: Plot a drawdown (s) versus distance (r) field-data curve on the semi-logarithmic graph paper. Distance is plotted as a logarithmic scale on the X-axis, and drawdown is plotted on a linear scale on the Y-axis.

Step 2: Fit a straight line to the field-data points.

Step 3: Extend the fitted straight line to intercept the zero-drawdown line and designate this distance r_0.

Step 4: Calculate the change in the value of the drawdown per log cycle (i.e., Δs) from the slope of the straight line.

Step 5: Finally, calculate the values of T and S by using the following equations:

$$T = \frac{2.3Q}{2\pi \Delta s} \qquad (14.8)$$

and

$$S = \frac{2.25Tt}{r_0^2} \qquad (14.9)$$

It should be noted that t in Eqn. (14.9) denotes a specific time since the start of pumping when the drawdowns are measured in multiple observation wells located in one direction at varying distances from the pumping well. These unsteady distance-drawdown data can yield both T and S. However, if instead of unsteady drawdowns steady or quasi-steady drawdowns are measured in multiple observation wells, the Thiem method can be directly used to calculate only T from such distance-drawdown data. The Thiem equation is applied for each pair of the steady (or quasi-steady) distance-drawdown data to obtain T values and then the mean of the T values is calculated which is taken as a representative aquifer parameter.

14.2.6 Illustrative Example 3

Problem: During a pumping test conducted in a confined aquifer, the aquifer was pumped at a constant rate of 280 m^3/h. After 180 minutes of pumping, drawdowns were simultaneously measured in nine observation wells located at different radial distances from the pumping well as shown in Table 14.3. Using the observed distance-drawdown data, determine transmissivity (T) and storage coefficient (S) of the confined aquifer.

Table 14.3. Unsteady Distance-Drawdown data

Distance (m)	3	15	30	45	60	75	90	120	150
Drawdown (m)	10.73	7.42	6.00	5.17	4.58	4.13	3.76	3.18	2.73

Solution: Following the procedure of the Cooper-Jacob straight-line method for distance-drawdown data mentioned above, a graph of drawdown versus log(r) is prepared and a straight line is fitted through the data points as shown in Fig. 14.4.

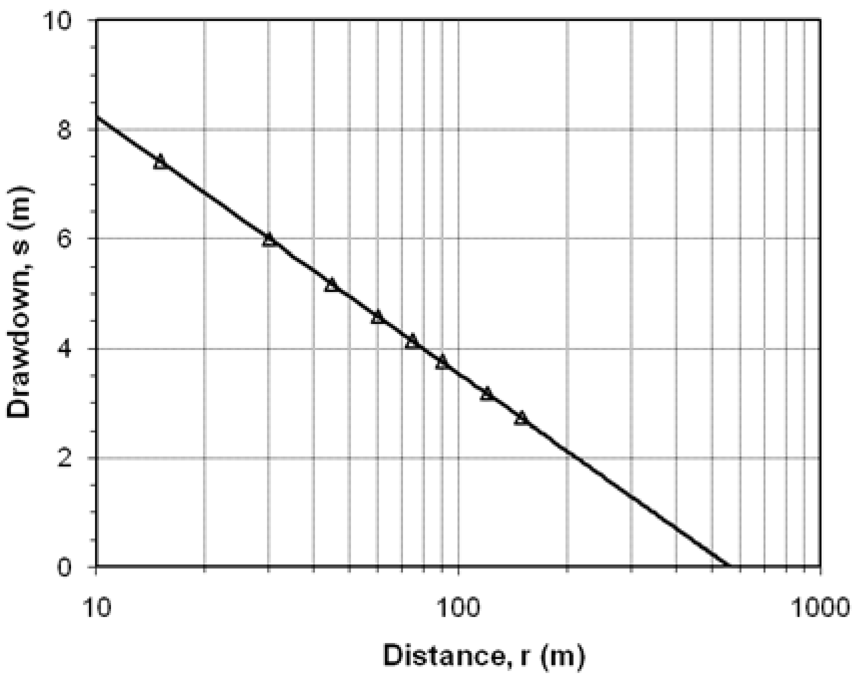

Fig. 14.4. Straight-line fitting to the distance-drawdown data.

From the graph, we have r_0 (distance corresponding to the zero drawdown) = 578 m, and Δs (drawdown per log cycle) = 8.25−3.5 = 4.75 m. Therefore, transmissivity (T) of the confined aquifer is:

$$T = \frac{2.3Q}{2\pi\Delta s} = \frac{2.3 \times 280}{2\pi \times 4.75} = 21.578\ \mathrm{m^2/h} = 21.578 \times 24 = 517.87\ \mathrm{m^2/day},\ \mathrm{Ans.}$$

Now, storage coefficient (S) of the confined aquifer is:

$$S = \frac{2.25Tt}{r_0^2} = \frac{2.25 \times 517.87 \times \dfrac{180}{60 \times 24}}{578^2} = 4.36 \times 10^{-4},\ \mathrm{Ans.}$$

14.2.7 Residual Drawdown-Time Ratio Method for Recovery Data

To calculate the behavior of an aquifer after pumping has been stopped, i.e., during recovery phase, an imaginary recharging well with the same constant flow rate is superimposed on the pumping well, which is supposed to continue production (i.e., pumping) at the same constant rate. Thus, the two flow rates cancel each other, and in essence represent an idle well. As mentioned in Lesson 13, recovery test can be conducted either in the pumping well itself or in nearby observation wells. The analysis of recovery data obtained from pumping wells can yield only T or K not the storage coefficient (S) because of significant well losses in pumping wells. However, the analysis of recovery data obtained from observation wells can yield both T (or K) and S.

If t is the time since pumping starts and t¢ is the time since pumping stops, the residual drawdown (s¢) (Fig. 14.5) in a confined aquifer at any time (t') after the end of pumping (i.e., during recovery period) can be obtained from the principle of superposition as follows:

$$s' = \frac{Q}{4\pi T}\left[\int_{u}^{\infty}\frac{e^{-u}}{u}du - \int_{u'}^{\infty}\frac{e^{-u'}}{u'}du'\right] \tag{14.10}$$

Where,

$$u = \frac{r^2 S}{4Tt} \tag{14.11}$$

and

$$u' = \frac{r^2 S}{4Tt'} \tag{14.12}$$

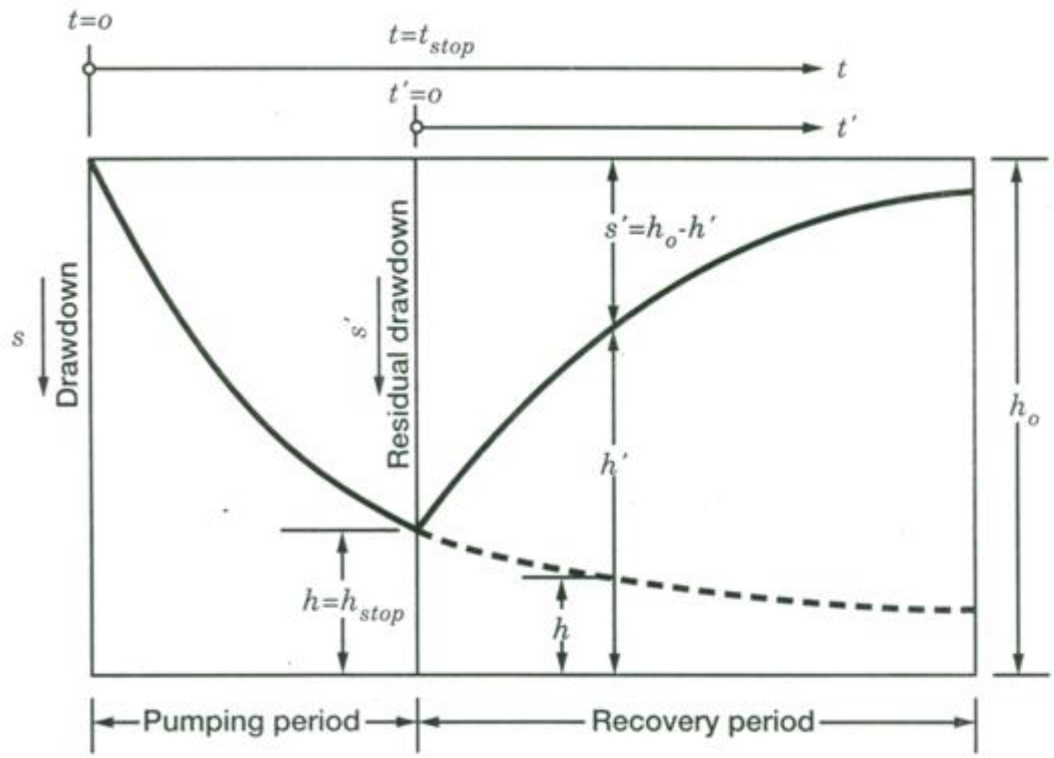

Fig. 14.5. Schematic diagram of the aquifer recovery after the pump is turned off. (Source: Batu, 1998)

Since recovery measurements are made in a pumping well or in a nearby observation well, the value of r (distance of the measurement point from the pumping well) is normally small. A small value of r generally leads to a small value of u¢, which enables us to take advantage of the Cooper-Jacob approximation of the Theis well function. Applying the Cooper-Jacob assumption, Eqn. (14.10) reduces to:

$$s' = \frac{2.3Q}{4\pi T}\log\left(\frac{t}{t'}\right) \qquad (14.13)$$

From Eqn. (14.13), transmissivity (T) can be calculated as:

$$T = \frac{2.3Q}{4\pi s'} \log\left(\frac{t}{t'}\right) \qquad (14.14)$$

The step-by-step procedure for determining transmissivity and storage coefficient using recovery data is as follows:

Step 1: Plot a residual drawdown (s¢) versus time ratio (t/t¢) curve on the semi-logarithmic graph paper with the time ratio on the X-axis (logarithmic scale) and the residual drawdown on the Y-axis (arithmetic scale).

Step 2: Fit a straight line to the field-data points.

Step 3: Calculate the value of the residual drawdown per log cycle ($\Delta s'$) from the slope of the straight line.

Step 4: Calculate the transmissivity (T) as:

$$T = \frac{2.3Q}{4\pi \Delta s'} \qquad (14.15)$$

As mentioned above, Storage coefficient (S) can be calculated only when the recovery data are measured in an observation well. The drawdown (s_p) when pumping is stopped at time (t_p) can be expressed as:

$$s_p = \frac{2.3Q}{4\pi T} \log \frac{2.25 T t_p}{r^2 S} \qquad (14.16)$$

Once T is known from Eqn. (14.15), the storage coefficient (S) is calculated as:

$$S = \frac{2.25 T t_p}{r^2} 10^{-\left(\frac{4\pi T s_p}{2.3Q}\right)} \qquad (14.17)$$

14.2.8 Illustrative Example 4

Problem (Modified from Schwartz and Zhang, 2003): A pumping test was conducted in a confined aquifer and drawdowns were measured during both pumping period and recovery period in the pumping well (Table 14.4) as well as in an observation well (Table 14.5). In Tables 14.4 and 14.5, the first column contains the time since the pumping started (t), the second column contains the drawdown (s) during the pumping period, the third column contains the time since pumping stopped (t’), the fourth column contains the time ratio (t/t’), and the last column contains the residual drawdown (s’) during the recovery period.

Table 14.4. Time-Drawdown and Time-Residual Drawdown data from a pumping well (Source: Modified from Schwartz and Zhang, 2003)

t (min)	s (m)	t¢(min)	t/t¢	s¢(m)
20	3.44	20	41.00	0.46
80	3.54	80	11.00	0.30
140	3.60	140	6.71	0.24
195	3.60	195	5.10	0.21
255	3.60	255	4.14	0.18
315	3.66	315	3.54	0.16
375	3.72	375	3.13	0.15
435	3.72	435	2.84	0.14
495	3.72	495	2.62	0.12
560	3.72	560	2.43	0.10

616	3.75	616	2.30	0.10
668	3.78	668	2.12	0.10
737	3.81	737	2.10	0.07
800	3.81	800	2.00	0.07

Table 14.5. Time-Drawdown and Time-Residual Drawdown data from an observation well (Source: Modified from Schwartz and Zhang, 2003)

t (min)	s (m)	t¢(min)	t/t¢	s¢(m)
5	0.02	5	161.00	0.54
10	0.07	10	81.00	0.50
15	0.10	15	54.33	0.47
20	0.12	20	41.00	0.44
25	0.15	25	33.00	0.42
30	0.17	30	27.67	0.40
40	0.20	40	21.00	0.37
50	0.22	50	17.00	0.35
60	0.24	60	14.33	0.33
70	0.26	70	12.43	0.31
80	0.28	80	11.00	0.30
90	0.29	90	9.89	0.29
100	0.30	100	9.00	0.27
110	0.32	110	8.27	0.27
120	0.33	120	7.67	0.26
180	0.38	180	5.44	0.21
240	0.41	240	4.33	0.19
300	0.44	300	3.67	0.16
360	0.46	360	3.22	0.15
420	0.48	420	2.90	0.14
480	0.50	480	2.67	0.12
540	0.52	540	2.48	0.11
600	0.53	600	2.33	0.11
660	0.54	660	2.21	0.10
720	0.55	720	2.11	0.09
800	0.57	800	2.00	0.09

A constant pumping rate of 270 m^3/h was maintained during the pumping period. The observation well was situated 22 m away from the pumping well. Determine hydraulic parameters of the confined aquifer using the above two sets of the recovery data.

Solution: Residual drawdown (s¢) and time ratio (t/t¢) data obtained from the pumping well and the observation well were plotted on the semi-logarithmic graph paper as illustrated in Figs. 14.6 and 14.7, respectively. Thereafter, a straight line was fitted to the residual drawdown versus time ratio curve in each figure. After selecting two suitable points on each graph as indicated in Figs. 14.6 and 14.7, we have Ds¢= 0.43 − 0.13 = 0.30 m in the pumping well from Fig. 14.6 and Ds¢= 0.53 − 0.28 = 0.25 m in the observation well from Fig. 14.7. From the question, pumping rate (Q) = 270 m^3/h. Given these data, the transmissivity (T) of the confined aquifer can be determined from the recovery data of the pumping well as:

$$T = \frac{2.3Q}{4\pi\,\Delta s'} = \frac{2.3\times 270\times 24}{4\pi\times 0.30}$$ = 3953.41 m^2/day, Ans.

As we know that the storage coefficient (S) cannot be determined from the recovery data of the pumping well.

Similarly, the transmissivity (T) of the confined aquifer can be determined from the recovery data of the observation well as:

$$T = \frac{2.3Q}{4\pi\,\Delta s'} = \frac{2.3\times 270\times 24}{4\pi\times 0.25}$$ = 4744.09 m^2/day, Ans.

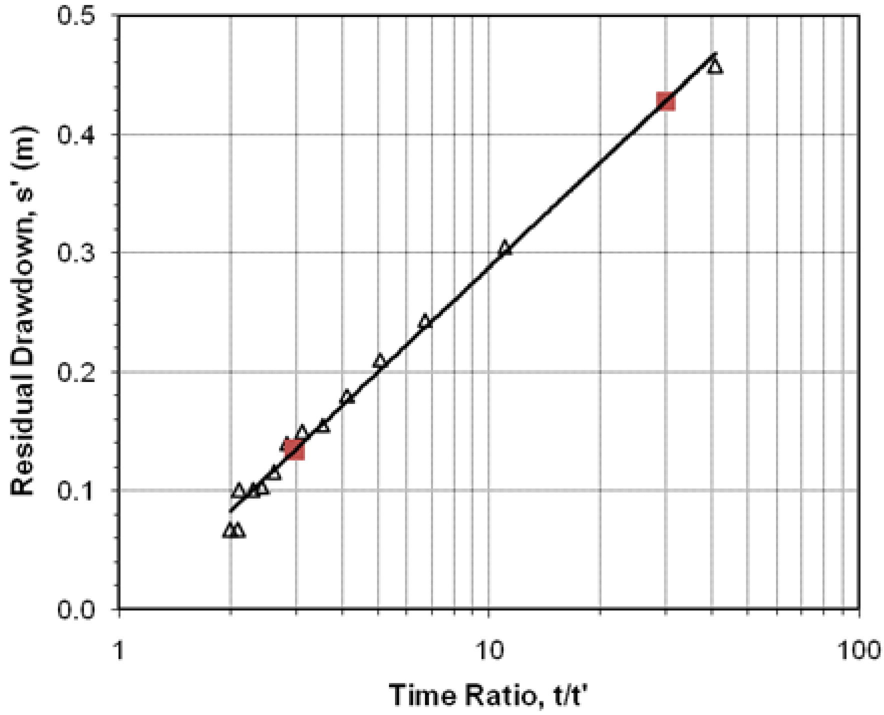

Fig. 14.6. Straight-line fitting to the residual drawdown-time ratio data obtained from the pumping well.

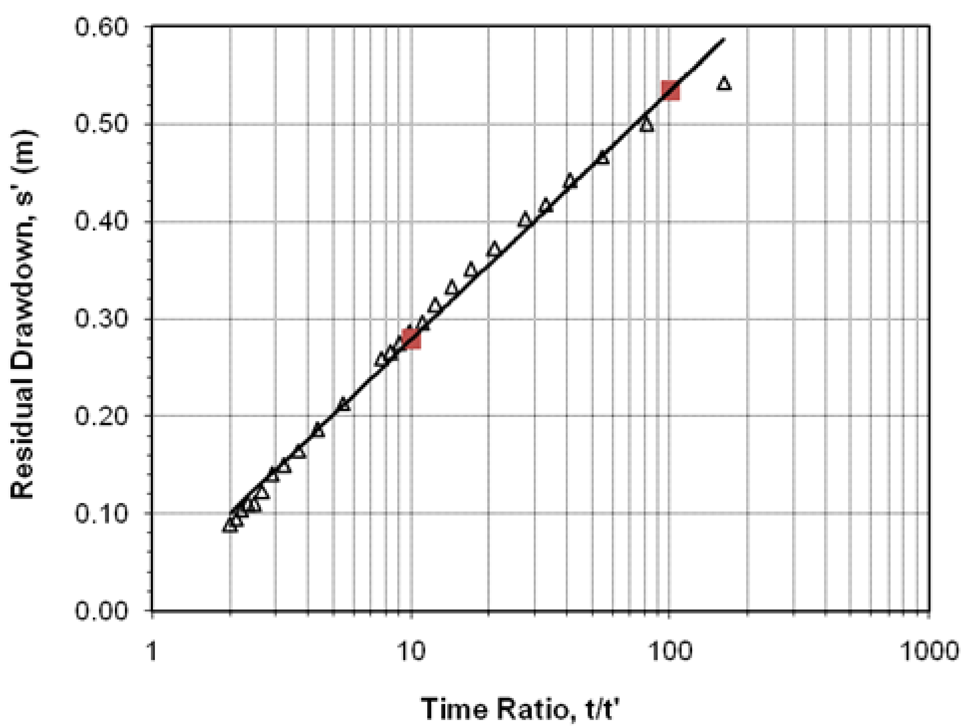

Fig. 14.7. Straight-line fitting to the residual drawdown-time ratio data obtained from the observation well.

Now, from the question we have: t_p = 800 min, s_p = 0.57 m (Table 14.5), Q = 270 m^3/h = 270/60 = 4.5 m^3/min, r = 22 m, and T = 4744.09 m^2/day = 4744.09/(24´60) = 3.29 m^2/min as computed above. Using these data, the storage coefficient (S) of the confined aquifer can be determined as:

$$S = \frac{2.25Tt_p}{r^2}10^{-\left(\frac{4\pi T s_p}{2.3Q}\right)}$$

$$= \frac{2.25\times 3.29\times 800}{22^2}\times 10^{-\left(\frac{4\pi\times 3.29\times 0.57}{2.3\times 4.5}\right)}$$

$$= 12.235\times 10^{-2.277} = 0.0647,\ \textbf{Ans.}$$

14.2.9 Cooper-Jacob Straight-Line Method for Recovery Data

The Cooper-Jacob straight-line method can be used to analyze the time-recovery data obtained from a pumping well or an observation well. The step-by-step procedure for analyzing time-recovery data by the Cooper-Jacob straight-line method is the same as that for analyzing the time-drawdown data described in Section 14.2.3, except that the time is measured since pumping stopped (Fig. 14.5), and ‘recovery’ is used instead of drawdown. Finally, Eqn. (14.3) is used to determine T and Eqn. (14.4) is used to determine S from the time-recovery data. In this case also, storage coefficient (S) cannot be determined using the time-recovery data obtained from a pumping well.

Note that ‘recovery’ is calculated as the difference between the extrapolated time-drawdown curve and the ‘residual drawdown’ (Fig. 14.5). Residual drawdown at time t after the pump is turned off is the difference between the static groundwater level (i.e., pre-pumping groundwater level) and the depth of groundwater at time t.

14.3 Determination of Unconfined Aquifer Parameters

14.3.1 Unconfined Aquifer without Delayed Yield

The methods mentioned above for confined aquifers are also applicable to the unconfined aquifers exhibiting no delayed yield. However, in this case, it is necessary to check whether correction in the observed drawdowns is required or not. This checking is done by using the following criteria:

Case A: If $\frac{s_{max}}{h_0}$ 0.02, then no correction in the observed drawdown is needed. Here, s_{max} = maximum drawdown at a given location during pumping test and h_0 = initial saturated thickness of the unconfined aquifer. Therefore, the observed time-drawdown data of unconfined aquifers as such can be used for determining unconfined aquifer parameters (K and S_y) by the Theis Type-Curve method, the Cooper-Jacob Straight-Line method, or the methods for analyzing recovery data.

After determining T (transmissivity) and S (storage coefficient) from any of the above methods, the hydraulic conductivity (K) of the unconfined aquifer can be obtained by dividing T with initial saturated thickness of the unconfined aquifer (h_0). The value of S will be equal to the specific yield (S_y) of the unconfined aquifer. Thus, K and S_y of unconfined aquifers can be determined from time-drawdown pumping-test data. Also, the Thiem method can be used for determining T or K from steady or quasi-steady distance-drawdown data.

Case B: If > 0.02, then the observed drawdown must be corrected using Eqn. (14.18) in order to obtain drawdowns in the equivalent confined aquifer so that the methods used for confined aquifers can be applicable to unconfined aquifers as well.

$$s' = s - \frac{s^2}{2h_0} \qquad (14.18)$$

Where, = corrected drawdown (i.e., drawdown in the equivalent confined aquifer), s = drawdown measured in the unconfined aquifer, and h_0 = initial saturated thickness of the unconfined aquifer.

After correcting the observed/measured drawdowns of the unconfined aquifer, the corrected time-drawdown data can be used to determine unconfined aquifer parameters (K and S_y) by the Theis Type-Curve method, the Cooper-Jacob Straight-Line method, or the methods for analyzing recovery data as mentioned in Case A above. Similarly, the Thiem method can be used for determining T or K of unconfined aquifers from steady or quasi-steady distance-drawdown data measured in unconfined aquifers.

14.3.2 Unconfined Aquifer with Delayed Yield

If the time-drawdown data obtained from unconfined aquifers exhibit significant delayed yield, the above-mentioned methods cannot be used for determining aquifer parameters. In this case, different methods have been suggested for reliable results which are: (i) Type-Curve Method, and (ii) Neuman Straight-Line Method. Excellent descriptions of these methods can be found in Fetter (1994), Schwartz and Zhang (2003) and Batu (1998).

14.4 Identification of Aquifer Boundaries

A common field problem is to identify and locate aquifer boundaries. Pumping-test data can provide quantitative answers to this problem by the application of image well theory and unsteady (transient) flow equations. As discussed in Lesson 13, if a pumping well and an observation well are located near an unknown impermeable aquifer boundary, the real pumping well and the image well are considered to be operating simultaneously and the drawdowns caused by the real and image pumping wells are additive, resulting in an effective discharge boundary of no flow. When determining the distance to a barrier boundary, the image well is assumed to pump water out of the aquifer at the same rate as the water is withdrawn by the real pumping well. Similarly, when estimating the distance to a recharge boundary, the image well is assumed to inject water into the aquifer at the same rate as water is withdrawn by the real pumping well. Thus, the resulting drawdown caused by the real pumping well and simulated ‘build up’ of water caused by the recharging image well equals an effective recharge boundary with zero drawdown in the aquifer.

According to the law of times, the time of occurrence of zero drawdown or equal drawdown is directly proportional to the square of the distances of the observation wells from the pumping well and is independent of the rate of pumping. That is,

$$\frac{r^2}{t_p} = \frac{r_i^2}{t_i} \qquad (14.19)$$

Where, r = distance of the observation well from the real well (pumping well), r_i = distance of the observation well from the image well, t_p = time since pumping started to any selected drawdown before the boundary influences the well drawdown (Fig. 14.8), and t_i = time since pumping started where the divergence of the time-drawdown curve from the Type curve equals the selected drawdown (Fig. 14.8).

The value of r_i can be obtained from Eqn. (14.19) and half the value is taken approximately as the distance of the aquifer boundary from the pumping well.

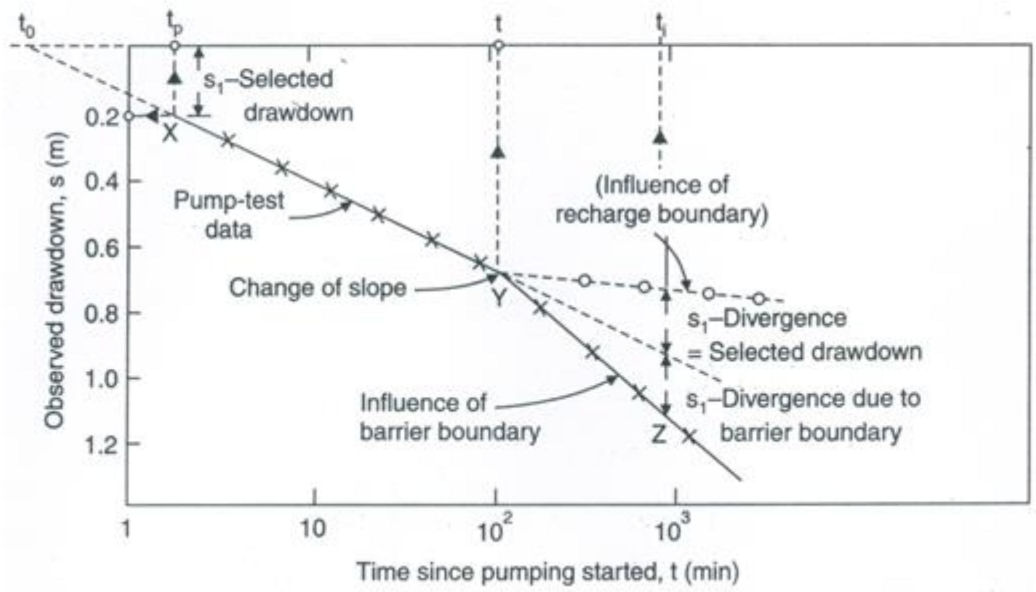

Fig. 14.8. Identification of aquifer boundaries using semi-logarithmic plot of time-drawdown data. (Source: Raghunath, 2007)

Alternatively, if the time t_0 corresponding to the zero drawdown is selected from the Cooper-Jacob's time-drawdown curve and t is the time at which a change of slope (i.e., divergence) is indicated (Fig. 14.8), the distance of the aquifer boundary from the pumping well can be obtained from the following equation:

$$\frac{r^2}{t_0} = \frac{r_i^2}{t} \qquad (14.20)$$

It should be noted that the distance r_i obtained from Eqn. (14.19) or Eqn. (14.20) gives an arc on which the image well lies. Data from two or more observation wells are required to locate the image well from the intersection of the three arcs (Fig. 14.9). Then, the location of the aquifer boundary is found to be midway and perpendicular to the line joining the pumping well and the image well as shown in Fig. 14.9.

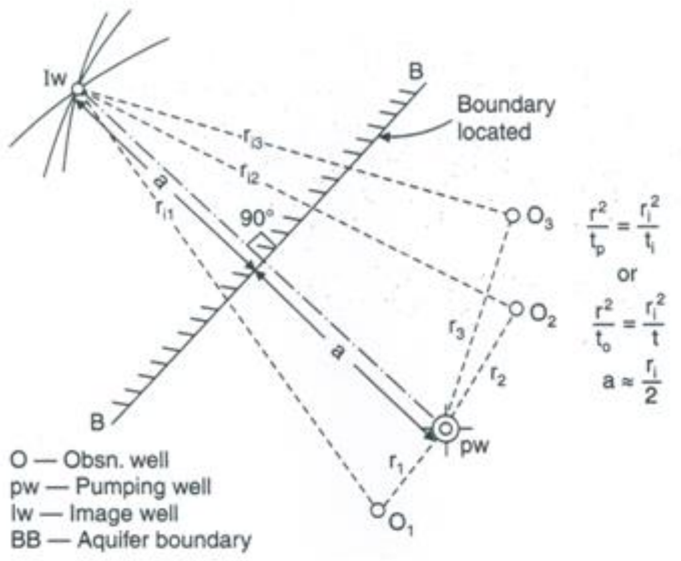

Fig. 14.9. Finding the location of an aquifer boundary.

(Source: Raghunath, 2007)

Lesson 15 Well Interference and Multiple Well Systems

15.1 What is Well Interference?

The cases of well hydraulics considered so far have involved only one well pumping from an aquifer system. However, there are often several wells tapping the same aquifer and located within the radii of influence of the wells, which result in intersecting cones of depression. When the cones of depression of two or more nearby pumping wells overlap, the well is said to interfere with another well. Well interference increases drawdown, and hence pumping lift is increased. At any given point in a confined aquifer, the total drawdown due to simultaneous pumping of multiple wells is calculated as a sum of the drawdowns caused by individual wells. Since the Laplace equation (i.e., steady-state groundwater flow in homogeneous and isotropic aquifer systems) is linear, the superposition of drawdown effects is found by simple addition. In Fig. 15.1, the well interference for a three-well system is presented graphically in which the individual drawdown curves are shown as dotted lines, while the composite drawdown due to the simultaneous pumping of three wells is shown as solid lines. For a group of wells forming a well field, the drawdown can be determined at any point in the area of influence if well discharges are known, or vice versa (Todd, 1980).

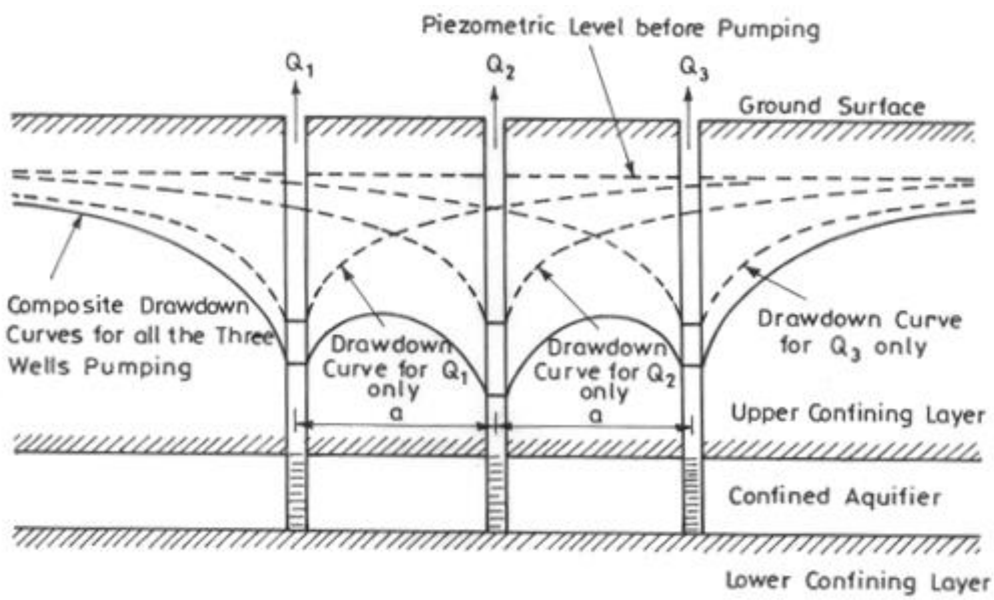

Fig. 15.1. Well interference in a well field having three pumping wells. (Modified from Todd, 1980)

15.1.1 Well Interference in Confined Aquifer Systems

From the principle of superposition, the drawdown at any point in the area of influence caused by the pumping of several wells is equal to the sum of the individual drawdowns caused by each pumping well, which is mathematically expressed as follows (Todd, 1980):

$$s = s_a + s_b + s_c \ldots\ldots\ldots\ldots\ldots\ldots + s_n \tag{15.1}$$

Where, s = total drawdown at a given point due to the pumping of multiple wells, and s_a, s_b, s_c, …., s_n are individual drawdowns at the point caused by the pumping of wells a, b, c, …., n, respectively.

15.1.2 Unconfined Aquifer and Well Interference

The linear superposition principle [Eqn. (15.1)] is valid only for confined aquifer systems, in which the value of aquifer transmissivity does not change with drawdown. In unconfined (water-table) aquifer systems, if the drawdown is significant compared to its initial saturated thickness, the use of linear superposition will result in a predicted composite drawdown that is less than the actual composite drawdown. As a decrease in the saturated thickness of an unconfined aquifer reduces the aquifer transmissivity, the multiple-well system in this aquifer will result in a composite hydraulic gradient greater than that of an equivalent confined system in order to compensate for a reduced value of aquifer transmissivity. Thus, when two or more wells are discharging groundwater from an unconfined aquifer with intersecting cones of depressions, the composite drawdown predicted by Eqn. (15.1) is always an estimate-in-error of the actual drawdown. Therefore, the following steps are followed to calculate the composite drawdown due to well interference in unconfined aquifers (Kasenow, 2001):

Step 1: Determine the theoretical confined drawdown (steady or unsteady) using known T (i.e., Kh_0) and S_y values for each production well as if they were pumping groundwater in isolation.

Step 2: Determine a resulting sum for these confined drawdowns.

Step 3: Correct this resulting sum to determine total unconfined drawdown at the observation point, which includes well interference drawdowns:

$$s = h_0 - \left(h_0^2 - 2s'h_0\right)^{0.5} \qquad (15.2)$$

Where, s = drawdown in the unconfined aquifer [L]; s'= drawdown in the equivalent confined aquifer [L], and h_0 = initial saturated thickness of the aquifer [L].

Step 4: Finally, to determine the dewatering component of the drawdown, subtract the result obtained in Step 2 from the result obtained in Step 3.

The above procedure is necessary to be followed while computing composite drawdown due to well interference in unconfined aquifer systems; otherwise a large error may occur (Kasenow, 2001).

15.2 Salient Applications of Well Interference

- In designing well-field layouts, it is necessary to take into account well interference. The water level in a well during pumping determines the length of suction pipe necessary to carry groundwater to the ground surface. The characteristics of the pump and the horsepower requirements of the motor also depend on the depth to the pumping level; considerably high energy is required for withdrawing groundwater from deeper depths.

- Generally, the well field designed for water supply purposes should be spaced as far apart as possible to minimize well interference, which in turn will minimize drawdowns. If wells are spaced too closely together, the amount of well interference could be very high.
- For drainage (or dewatering) wells, however, the well field is designed to increase well interference so as to enhance the drainage or dewatering effect.
- Aligning wells parallel to a line source of recharge (e.g., river, lake) would result in less well interference compared to a perpendicular configuration of wells.

15.3 Analysis of Multiple Well Systems

As mentioned earlier, if there are several wells in a given well field, the drawdown at any point is the sum of the drawdowns due to individual pumping wells. The drawdown depends on the pumping pattern, i.e., number of pumping wells, their pumping rates and their arrangement. Solutions can be obtained using steady-state or unsteady (transient) flow equations depending on the field situation. Multiple well systems are used for lowering the groundwater level in a given area to facilitate subsurface drainage or excavation for foundation work, mining, etc. Steady-state solutions for multiple well systems are presented in this section for three major cases: (i) drawdown for the well systems parallel to a line source, (ii) well discharges for different well configurations, and (iii) required drawdown for the well systems used for dewatering.

15.3.1 Well Systems Parallel to Line Source

Wells may be closely spaced (resulting in well interference) and all the wells may be connected to a common supply pipe to meet the large demand of water supply. For an array of a number of equally-spaced fully penetrating wells, all discharging at the same rate, parallel to a line source (Fig. 15.2), steady drawdown in the confined aquifer at any point (x, y) is given as (Forchheimer, 1908 as referred in Raghunath, 2007):

$$s = \frac{Q}{2\pi Kb} \ln \frac{\cosh \frac{2\pi}{a}(x+d) - \cos \frac{2\pi y}{a}}{\cosh \frac{2\pi}{a}(x-d) - \cos \frac{2\pi y}{a}} \qquad (15.3)$$

Where, s = steady drawdown at the observation point (x,y), [L]; a = spacing between the wells, [L]; Q = discharge of each well, [L^3T^{-1}]; and d = distance of the observation point from the line source, [L].

For unconfined aquifers, Eqn. (15.3) is expressed as:

$$h_0^2 - h_w^2 = \frac{Q}{4\pi K} \ln \frac{\cosh \frac{2\pi}{a}(x+d) - \cos \frac{2\pi y}{a}}{\cosh \frac{2\pi}{a}(x-d) - \cos \frac{2\pi y}{a}} \qquad (15.4)$$

Where, h_w= water level in the well during pumping from the well bottom [L].

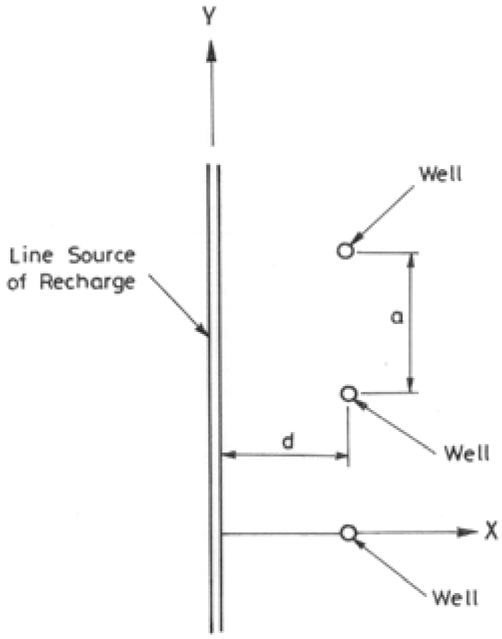

Fig. 15.2. Well parallel to a line source of recharge.

15.3.2 Well Discharge for Different Well Configurations

Muskat in 1973, as referred in Raghunath (2007), developed analytical solutions (Eqns. 15.5 to 15.7) for well discharges considering various well patterns localized near the centre of a well field of radius R_0 (i.e., radius of influence for each well) such that for each well the head at the external boundary can be taken to be (Fig. 15.3). It was assumed that all the wells fully penetrate a confined aquifer, have the same diameter and drawdown, and discharge for the same period of time. Three configurations (linear, triangular and square) of closely spaced multiple wells as shown in Fig. 15.3 are discussed below as three cases.

(1) Case 1: Discharge of the two wells spaced at a distance a (a R_0)

For Confined Aquifers:

$$Q_1 = Q_2 = \frac{2\pi Kb(H - h_w)}{\ln \frac{R_0^{\ 2}}{r_w a}} \qquad (15.5a)$$

Where, K = hydraulic conductivity of the aquifer [LT^{-1}]; H = head at external boundary (i.e., water level in the well before pumping from the bottom of the well) [L]; and $H\text{-}h_w = s_w$ = drawdown of single well at a given discharge Q [L].

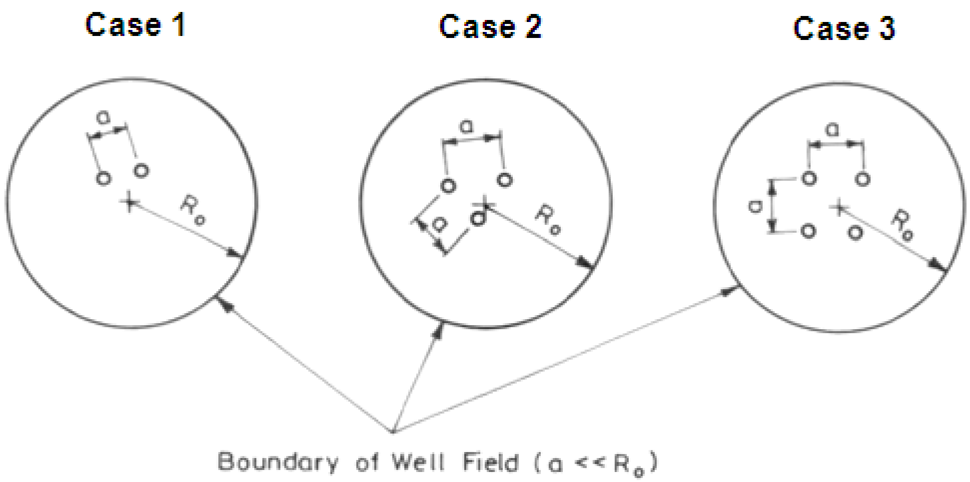

Fig. 15.3. Three configurations of wells closely spaced in a well field.

For Unconfined Aquifers:

Eqn. (15.5a) can also be applied to unconfined aquifers by replacing H with $H^2/2b$ and h_w with /2b (Raghunath, 2007), which results in:

$$Q_1 = Q_2 = \frac{\pi K(H^2 - h_w^2)}{\ln \frac{R_0^{\ 2}}{r_w a}} \qquad (15.5b)$$

(2) Case 2: Discharge of the three wells spaced at a distance a (a R_0)

For Confined Aquifers:

$$Q_1 = Q_2 = Q_3 = \frac{2\pi Kb(H - h_w)}{\ln \frac{R_0{}^3}{r_w a^2}} \qquad (15.6a)$$

For Unconfined Aquifers:

$$Q_1 = Q_2 = Q_3 \frac{\pi K\left(H^2 - h_w^2\right)}{\ln \frac{R_0{}^3}{r_w a^2}} \qquad (15.6b)$$

(3) Case 3: Discharge of the four wells spaced at a distance a (a R_0)

For Confined Aquifers:

$$Q_1 = Q_2 = Q_3 = Q_4 = \frac{2\pi Kb(H - h_w)}{\ln \frac{R_0{}^4}{\sqrt{2} r_w a^3}} \qquad (15.7a)$$

For Unconfined Aquifers:

$$Q_1 = Q_2 = Q_3 = Q_4 = \frac{\pi K\left(H^2 - h_w^2\right)}{\ln \frac{R_0{}^4}{\sqrt{2} r_w a^3}} \qquad (15.7b)$$

Note that as the number of wells in the group increases, the mutual interference between wells becomes more, which results in the reduction of production capacity of individual wells.

15.3.3 Multiple Well Systems for Dewatering

Design of dewatering systems has great importance in drainage, mining and foundation engineering. It involves a number of pumping wells for accomplishing the dewatering objective. The principle of superposition is used to calculate drawdown and required well discharge. For a confined aquifer, the principle of superposition yields (Charbeneau, 2000):

$$s = \frac{1}{2\pi T}\sum_{i=1}^{N} Q_i \ln\frac{R_0}{r_i} \qquad (15.8)$$

Where, s = drawdown of the system of wells [L], Q_i = discharge from the i^{th} well [LT^{-3}], R_0 = radius of influence [L], and r_i = radius of the i^{th} well [L].

If the discharge from each well is the same, Eqn. (15.8) can be written as:

$$s = \frac{Q}{2\pi T}\sum_{i=1}^{N} \ln\frac{R_0}{r_i} \qquad (15.9)$$

Where, Q is the discharge from each well. Eqn. (15.9) is important because it indicates that with a multiple number of wells, all pumping at the same rate, the drawdown at any point depends only on the geometry of the system.

(1) Circular Well System

Jacob (1950) analyzed the dewatering problem with a number of pumping wells arranged in a circle as shown in Fig. 15.4. It was assumed that each well is pumping at the same rate. We are often interested in the drawdown at the centre of the system of wells, which might correspond to the centre of an excavation for example, and the drawdown at each of the wells and at midpoint between wells on the circle of the system.

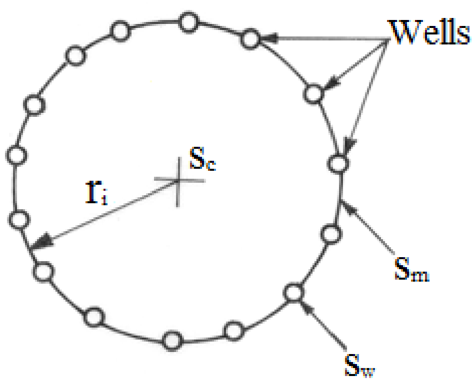

Fig. 15.4. Geometry for a circular dewatering system.

- For the drawdown at the centre of the system of wells, the radii from each of the wells to the centre (r_i) are the same (Fig. 15.4). Thus, Eqn. (15.9) can be written as (Charbeneau, 2000):

$$s_c = \frac{Q}{2\pi T}\sum_{i=1}^{N}\ln\left(\frac{R_0}{r_i}\right) = \frac{Q}{2\pi T} N \ln\left(\frac{R_0}{r_i}\right) \tag{15.10}$$

Where, s_c = drawdown at the center of the system of wells [L].

- For the drawdown at each well, Eqn. (15.9) becomes:

$$s_w = \frac{Q}{2\pi T}\ln\left(\frac{R_0}{r_w}\right) + \frac{Q}{2\pi T}\sum_{i=1}^{N-1}\ln\left(\frac{R_0}{r_i}\right) \tag{15.11}$$

Where, s_w = drawdown at each well [L], and r_w = radius of each well [L].

Jacob (1950) used several trigonometric identities to demonstrate that Eqn. (15.11) reduces to:

$$s_w = s_c + \frac{Q}{2\pi T}\ln\left(\frac{r_i}{Nr_w}\right) \qquad (15.12)$$

- For the drawdown at midpoint between the wells, Eqn. (15.9) becomes:

$$s_m = \frac{Q}{2\pi T}\sum_{i=1}^{N}\ln\left(\frac{R_0}{r_i}\right) \qquad (15.13)$$

Where, s_m = drawdown at midpoint between the wells [L].

Again, using trigonometric identities, it can be shown that Eqn. (15.13) reduces to:

$$s_m = s_c - \frac{Q}{2\pi T}\ln(2) \qquad (15.14)$$

It should be noted that $s_w > s_c > s_m$.

(2) Linear Well System

For a linear well system, let's consider a line of wells with a constant well spacing of a and the number of wells in the line N (Fig. 15.5). The number of wells (N), well spacing (a), and the length of line (L) are related as N = L/a so that the length of the line is considered to extend a distance a/2 beyond the last well at each end. We are usually interested to find out the drawdown at an arbitrary point away from the line of wells (Charbeneau, 2000).

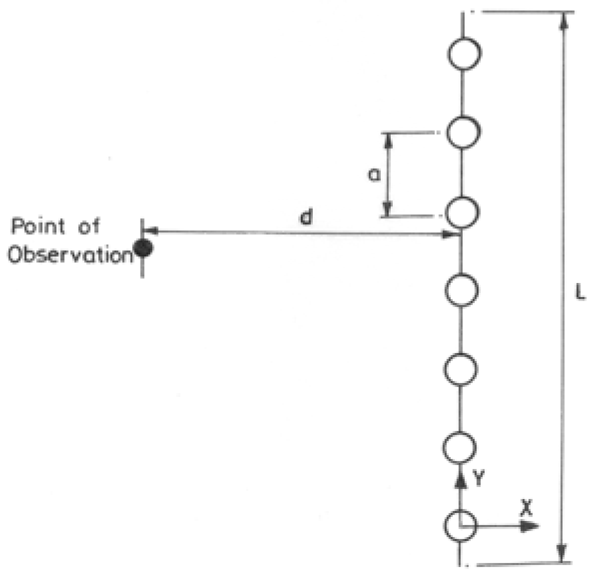

Fig. 15.5. Geometry for a linear dewatering system.

The drawdown at any arbitrary distance d from one end of the row (chosen as the origin) satisfies Eqn. (15.9), and we have the following expression for confined aquifer systems (Charbeneau, 2000):

$$\frac{2\pi Ts}{Q} = \sum_{i=1}^{N} \ln\left(\frac{R_0}{r_i}\right) = \frac{1}{2}\sum_{i=1}^{N} \ln\left(\frac{R_0^{\ 2}}{r_i^2}\right) = \frac{1}{2}\sum_{i=1}^{N} \ln\left(\frac{R_0^{\ 2}}{d^2 + y_i^2}\right) \qquad (15.15)$$

The summation can be approximated by the integral because both are equal to N as $\sum_{i=1}^{N} \cong \int_0^L \frac{dy}{a}$. Thus, we have:

$$\frac{2\pi Ts}{Q} \cong \frac{1}{2}\int_0^L \ln\left(\frac{R_0{}^2}{d^2+y^2}\right)\frac{dy}{a} = \frac{L}{2a}\left[\ln\left(\frac{R_0^2}{L^2+d^2}\right)+2\left\{1-\frac{d}{l}\tan^{-1}\frac{L}{d}\right\}\right] \tag{15.16}$$

As L/a = N and NQ = $Q_{T,}$ Eqn. (15.16) can be written as:

$$s = \frac{Q_T}{4\pi T}\left[\ln\left(\frac{R_0^2}{L^2+d^2}\right)+2\left\{1-\frac{d}{l}\tan^{-1}\left(\frac{L}{d}\right)\right\}\right] \tag{15.17}$$

Where, Q_T = total discharge [LT^{-3}].

Similarly, for unconfined aquifer systems, we have:

$$h_0^2 - h_w^2 = \frac{Q_T}{2\pi K}\left[\ln\left(\frac{R_0^2}{L^2+d^2}\right)+2\left\{1-\frac{d}{l}\tan^{-1}\left(\frac{L}{d}\right)\right\}\right] \tag{15.18}$$