16

Ground Water Hydrology and Construction of Wells and Tubewells

16.1. Definition and General Introduction

Ground water is the underground water that occurs in the saturated zone of variable thickness and depth, below the Earth's surface. Cracks and pores in the existing rocks and unconsolidated crystal layers, make up a large underground reservoir, where part of precipitation is stored.

Ground-water is largely tapped for irrigating crops in India. So much so that about 46% of our total irrigated area, gets its irrigation water from this source. Most of our **minor irrigation schemes***, make use of this source of supply. Besides its use for irrigation, ground water is also used as a source of water supply for municipal purposes.

The ground water is utilised through wells and tubewells. Various lifting devices, such as those using animal, manual, diesel, or electric power, may be used, so as to bring the underground supplies to the surface.

The use of open wells is a traditional method of tapping ground water in areas where ground watertable is high. Manual, animal, wind, diesel or electric power can be used for lifting water from open wells. The use of tubewells, however, is a subsequent development in the techniques of tapping ground water, and certainly requires diesel or electric power.

16.2. Occurrence of Ground Water

The rainfall that percolates below the ground surface, passes through the voids of the rocks, and joins the watertable. These voids are generally inter-connected, permitting the movement of the ground-water. But in some rocks, they may be isolated, and thus, preventing the movement of water between the interstices. The mode of occurrence of ground-water therefore, depends largely upon the type of formation, and hence depends upon the geology of the area.

The possibility of occurrence of ground-water mainly depends upon two geological factors; *i.e.* (*i*) the *porosity* of the rocks; and (*ii*) *the permeability* of the rocks, as explained below:

16.2.1. Porosity. The porosity of a rock, which is the major geological criteria for occurrence of ground-water, is a quantitative measurement of the interstices or voids present in the rock. It is generally defined as *the percentage of the voids present in a given volume of aggregate*. Mathematically, it can be expressed as :

* Schemes involving CCA up to 2000 hectares are **minor irrigation schemes**; those involving CCA between 2000 to 10,000 hectares are **medium irrigation schemes**; and those involving CCA greater than 10,000 hectares are **major irrigation schemes.**

$$\text{Porosity} = \frac{\text{Total volume of voids in the aggregate, } i.e. \text{ the volume of water required to saturate the dry sample } (V_v)}{\text{Total volume of the aggregate } (V)}$$

It is generally denoted by the letter n.

$$\therefore \quad n = \frac{V_v}{V} \times 100 \text{ (per cent)} \quad \quad ...(16.1)$$

Porosity, in fact depends upon the shape, packing, and degree of sorting of the component grains in a given material. Uniform and well sorted grains (Fig. 16.1 A) give rise to higher porosity; whereas, heterogeneous grains with irregular arrangement (Fig. 16.1B) decrease the porosity.

The porosity of rocks and unconsolidated materials may vary considerably. It may be less than a per cent or more than 50%. But generally, it does not exceed 40% except in very poorly compacted materials. In general, a porosity greater than 20% is considered to be large, and below 5% as small, and between 5 to 20% as medium. The porosity values for a few common types of rock formations are given in table 16.1.

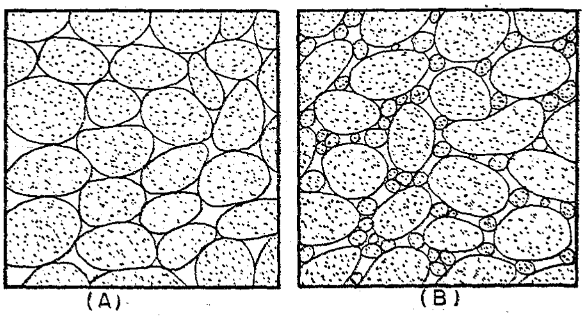


Fig. 16.1. Diagrammatic sketch of well sorted (A) and poorly sorted (B) deposits. Porosity of A is greater than that of B, because the grains of A are all about equal in size. In B, smaller grains fill the spaces between the larger grains, thus reducing the volume of void space.

Table 16.1. Porosity values of a few rock formations

S. No.	*Type of rock formation*	*Porosity*
1.	Granite, Quartzite	1.5%
2.	Slate, Shale	4%
3.	Limestone	5 to 10%
4.	Sandstone	10 to 15%
5.	Sand and Gravel	20 to 30%
6.	Only Gravel	25%
7.	Only Sand	35%
8.	Clay and Soil	45%

16.2.2. Permeability and Transmissibility. As stated above, the ground water can get stored in the underground rocks, only if, they are sufficiently porous. In other words, water is get stored in the pores (voids) of the rocks. The *porosity* of the rock, thus, defining the maximum amount of water that can be stored in the rock. The porosity, however, in itself, does not ensure the storage of underground water. Infact the water can enter into a rock (with any amount of porosity) only if the rock permits the flow of water through it, *i.e.* it depends on whether the rock is *permeable* or not. It may be clarified here that a rock which is porous, may or may not be permeable. For example, Shale is a porous rock, but its pore spaces are so minute that the rock remains impermeable. The *size of the pores*, is thus, quite an important factor, and it should be sufficiently large to make the rock permeable.

The *permeability* is, therefore, defined as the ability of a rock or unconsolidated sediment, to transmit or pass water through itself. Transmissibility is another term which represents the same physical meaning, but only differing mathematically, as explained below :

The capability of the entire soil of full width (b) and depth (d), (*i.e.* area bd) is represented by *permeability*; while that of the soil of unit width and full depth (*i.e.* $b = 1$ and $d = d$, *i.e.* $A = d$) is known as *transmissibility*.

The permeability is measured in terms of **coefficient of permeability** which will be defined a little later, in article 16.6. Various methods including constant head permeameter and variable head permeameter, are used to measure permeability.

16.3. Zones of Under-ground Water

As we move down below the surface of the Earth towards its centre, water is found to exist in different forms in different regions.

With regard to the existence of water at different depths, the Earth's crust can be divided into various zones, namely,

(*i*) Zone of rock fracture; and

(*ii*) Zone of rock flowage.

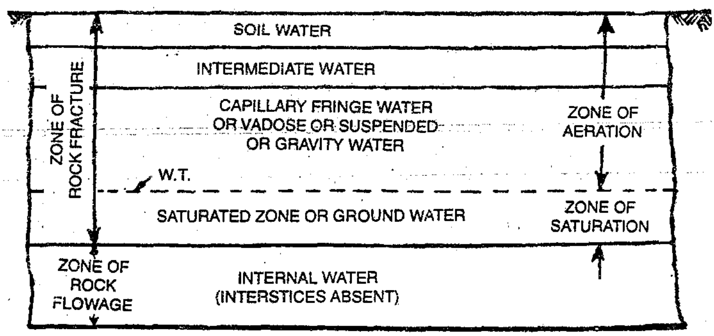


Fig. 16.2. Zones of under-ground water.

The depth of the *zone of rock flowage* (*i.e.* the zone in which the rocks undergo permanent deformation) is not accurately known but is generally estimated as many miles. Interstices are probably absent in this zone, because the stresses are beyond the elastic limits and the rocks remain in a state of plastic flow. Water present in this zone is known as *internal water*, and a hydraulic engineer has nothing to do with this water.

Above the zone of rock flowage, there lies the *zone of rock fracture*. In this zone, the stresses are within the elastic limits, and the interstices do exist. Water is stored in the voids, the amount of which depends upon porosity. The maximum depth of this zone below the ground surface, varies in the range of about 100 metres or less to 1,000 metres or more.

In crystalline rocks, most of the water is met within 100m of the surface; while in sedimentary rocks, it is found upto depths of 1,800 m or so, although very less quantity is found below 1,000 m or so.

The zone of rock fracture can be further sub divided into two zones; one is the *zone of saturation*, *i.e.* the zone below the watertable, and the other is the *zone of aeration*, *i.e.* the zone above the watertable.

In the *zone of saturation*, water exists within the interstices, and is known as ground water. This is the most important zone for a ground water hydraulic engineer, because he has to tap out this water. Water in this zone is under hydrostatic pressure.

The space above the watertable and below the surface is known as the *zone of aeration*. Water exists in this zone by molecular attraction. The gravity water moves through this zone, and the water in this zone is not at hydrostatic pressure. The thickness of this zone varies from almost none in marshy and low lying areas to about 300 metres or so in arid regions.

This zone is also divided into three classes depending upon the number of interstices present. The *capillary fringe* is the belt overlying the zone of saturation and it does contain some interstitial water, and is thus a continuation to the zone of saturation; while the depth from the surface which is penetrated by the roots of vegetation is known as the *soil zone*. The remainder intermediate part is the *intermediate zone*.

These zones have already been explained in article 2.14 and the students may refer back.

16.4. Movement of Ground Water and its Velocity

16.4.1. The Watertable. The static level of water in wells penetrating the zone of saturation, is called the watertable. The watertable is often described as the subdued replica of the surface topography. It is generally higher under the hills and lower under the valleys, and a contour map of the watertable in any area may look like the surface topography.

The watertable is thus the surface of a water body which is constantly adjusting itself towards an equilibrium condition, with the water moving from the higher points to the lower points. If there were no recharge to or outflow from the ground water in a basin, the watertable would eventually become horizontal. But, few basins have uniform recharge conditions at the surface, as some areas receive more rain than others; and some portions of the basin have more permeable soil. Thus, when intermittent recharge does occur, mounds and ridges do form in the watertable under the areas of greatest recharge. Subsequent recharge creates additional mounds, perhaps at other points in the basin, and the flow patten is further changed. Various other factors, such as : variations in permeability of aquifers; impermeable strata ; influence of lakes, streams, and wells, etc; do make the watertable less and less horizontal. *All this gives us a picture of watertable constantly adjusting towards equilibrium* (*i.e.* horizontal). Because of the low flow rates in most of the aquifers, this equilibrium is rarely attained before additional disturbances occur.

16.4.2. Movement of Ground Water. As discussed above, the watertable is generally not horizontal, and has high and low points in it, *i.e.* it is not in equilibrium. In order that the equilibrium is approached, water moves inside the ground from the high points on the watertable to the points lower down, as shown in Fig. 16.3. The rate at which such movement occurs is dependent upon two factors, *i.e.* (*i*) on the ability of the porous medium to pass water through it, *i.e.* on the *permeability*; and (*ii*) on the

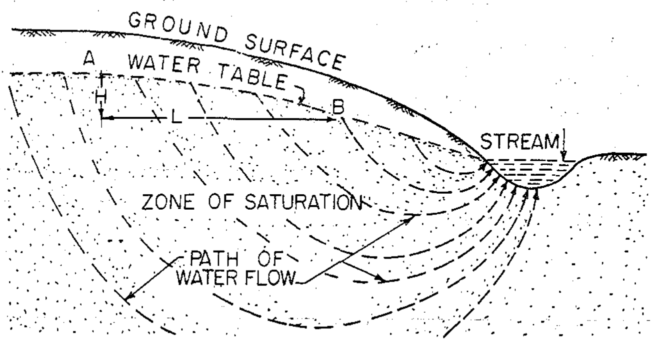


Fig. 16.3. Cross-sectional diagram showing the flow of ground water in a uniformly permeable material. The difference in elevation between point A and B of the water-table is H and the distance between them in L. The hydraulic gradient is H/L.

driving force or *hydraulic gradient* (I) usually expressed as the ratio between the difference in elevation (H_L) of the two points on the watertable (in the direction of flow), and the distance between then (L). This was stated as such by Darcy, as the important **Darcy's law**, which is discussed below :

16.4.3. Darcy's Law for determining Ground Water Velocity. On the basis of experimental evidence, Mr. H. Darcy, a French Scientist enunciated in 1865, a law governing the rate of flow (*i.e.* the discharge) through soils. According to him, this discharge was directly proportional to the head loss (H_L), and the area of cross- section (A) of the soil, and inversely proportional to the length of the soil sample (L). In other words,

$$Q \propto \frac{H_L}{L} \cdot A$$

But $\frac{H_L}{L}$ represents the rate of loss of head, *i.e.* the hydraulic gradient (I)

$\therefore$ $$Q \propto I \cdot A$$

or $$Q = K \cdot I \cdot A \qquad ...(16.2)$$

where, K is the proportionality constant and was found to be changing with the type of soil, and hence represented a property of the soil, called *permeability* or *coefficient of permeability*.

The above equation becomes dimensionally compatible, if K has the units of L/T, *i.e.* say cm/sec, *i.e.* the units of velocity.

The Darcy's law has been demonstrated to be valid only for *laminar** flow conditions, which as far as soils are concerned, is not at all a serious problem, because the flow in sands, silts and clays is invariably laminar.

Dividing both sides of equation (16.2) by A, we get

$$\frac{Q}{A} = K \cdot I$$

or $$v = K \cdot I \qquad ...(16.2\ a)$$

* For laminar flow and turbulent flow differences, please refer to any book on Fluid Mechanics.

where v is the discharge velocity, and is not the actual flow velocity through the soil medium, since the flow occurs through the voids of cross-sectional area A_v and not in A itself. The permeability of the soil can then be viewed as this superficial velocity under a unit hydraulic gradient.

If A_v is the area of the voids, then $A_v \cdot v_a = A \cdot v$, where v_a is the actual velocity of flow of water through the soil. Then

$$v = v_a \cdot \frac{A_v}{A} \quad \text{...(16.3)}$$

when A is large in comparison, we can safely assume that the ratio of the area of the void (A_v) to the total area (A) is the same as the ratio of the volume of the voids (V_v) to the total volume (V), *i.e.* equal to porosity (n).

Hence, $\frac{A_v}{A} = n$ (porosity)

Substituting this value is eqn. (16.3), we get

or $$\boxed{v = n \cdot v_a} \quad \text{...(16.5)}$$

Knowing the value of v from equation (16.2a) and dividing it by porosity n, the actual velocity of flow (v_a) of water through the soil, can be worked out.

16.4.4. Empirical Formulas for Ground Water Velocity Determination. Before Darcy came into picture, certain empirical formulae based upon the experimental results were the only way to find out the velocity of ground water flow. The formulae which were commonly used, are :

(1) **Slichter's formula.** According to which

$$v_a = K' I \frac{D^2_{10}}{\mu} \quad \text{...(16.5)}$$

where v_a = velocity of ground water flow in m/day.
K' = a constant.
I = slope of the hydraulic gradient line.
D_{10} = effective size of the particles in the aquifer in mm. (*i.e.* the hypothetical size which is larger than 10% of the particles in the sample, *i.e.* only 10% of the particles will pass through this size).
μ = viscosity of water depending on temperature.

(2) **Hazen's formula.** Formula in M.K.S. or S.I. system is

$$v_a = \frac{K'' I D^2_{10}}{60} \times (1.8T \div 42) \quad \text{...(16.6)}$$

where v_a = velocity of ground water flow in m/day.
T = temperature in °C.

values of K' and K'' in M.K.S. or S.I. system are approximately 400 and 1000, respectively.

Example 16.1. *Find out the velocities of the ground water flow with the following data, using Slichter and Hazen's constants as 400 and 800, respectively.*

Viscosity coefficient of water at ground water temperature of 10° C	= *1*
Effective size of the particles in the aquifer	= *0.1 mm*
Hydraulic gradient	= *1 in 80*

Solution. (*a*) Using Slichter's formula *i.e.* eqn. (16.5), we have

$$v_a = \frac{K' I \cdot D^2_{10}}{\mu}$$

$$= \frac{400 \times \frac{1}{80} \times (0.1)^2}{1} = \frac{400 \times 0.01}{80} = \mathbf{0.05\ m/day} \quad \textbf{Ans.}$$

(b) Using Hazen's formula, i.e. Eq. (16.6), we have

$$v_a = \frac{K'' I \cdot D^2_{10}}{60} (1.8T + 42), \text{ where } T \text{ is in °C}$$

$$= \frac{800 \times \frac{1}{80} \times 0.01}{60} (1.8 \times 10 + 42) \qquad [\because T = 10°\text{C}]$$

$$= \frac{800 \times \frac{1}{80} \times 0.01 \times 60}{60} = \mathbf{0.1\ m/day} \quad \textbf{Ans.}$$

16.4.5. Permeability Values and Relation between K and T. From Darcy's law, the coefficient of permeability may be defined as the rate of flow of water through a unit cross sectional area of the water- bearing material under a unit hydraulic gradient, and at a temperature* of 20°C. Various approximate average values of permeability coefficient for different types of rock (soil) formations are given in table 16.2. There may be vide variations in these values, depending upon the type of field soil.

Table 16.2. Permeability Coefficient (K) Values

S. No.	*Type of rock formation*	*App. Av. value of K in cm/sec*
1.	Granite, Quartzite	0.6×10^{-5}
2.	Slate, Shale	4×10^{-5}
3.	Limestone	4×10^{-5}
4.	Sandstone	0.004
5.	Sand and Gravel	0.4
6.	Only Gravel	4.0
7.	Only Sand	0.04
8.	Clay and Soil	0.04×10^{-5}

* This accounts for the viscosity of water, which decreases with the increase in temperature, thereby causing permeability to increase with the increase in temperature.

The term *transmissibility* introduced by Theiss is measured by the **coefficient of transmissibility (T),** which is defined as the rate of flow of water through a vertical strip of the water bearing material (*i.e.* aquifer) of unit width and full depth (d), under a unit hydraulic gradient and at a temperature of 20°C.

The relation between K and T is simple, and is given by

$$T = Kd \quad \text{...(16.7)}$$

16.5. Drainage of Ground Water

By the term 'drainage of ground water', we generally mean extracting the water from below the watertable through wells, infiltration galleries, springs, etc. The water, is thus, drained from the ground-water reservoir, either under a natural phenomenon (like spring), or it can be drained artificially by constructing wells, and lifting water through them. The water so drained may then be used to fulfil irrigation or municipal needs.

16.6. Ground Water Yield

The interstices present in the given formation get filled up with water during the process of ground-water replenishment. If all these voids are completely filled with water, then it is known as a saturated formation. The water contained in these voids is drained by digging wells under the action of gravity drainage. When these saturated formations are drained under the action of gravity drainage, it is found that the volume of water so drained is less than the volume of the void space, as indicated by its porosity. This is because of the fact, that the entire water contained in these voids cannot be drained out by the mere force of gravity. Some of the water is always retained by these interstices due to their molecular attraction. The water so retained is known as **pellicular water.**

16.6.1. Specific Yield. The volume of ground water extracted by gravity drainage from a saturated water bearing material is known as the **yield**, and when it is expressed as ratio of the volume of the total material drained, then it known as the *specific yield.*

$$\therefore \text{ Specific yield} = \frac{\text{Volume of the water obtained by gravity drainage}}{\text{total volume of the material drained or dewatered}} \times 100 \quad \text{...(16.8)}$$

16.6.2. Specific Retention or Field Capacity. On the other hand, the quantity of water retained by the material against the pull of gravity is termed as the *specific retention* or the *field capacity;* and this is also expressed as the percentage of the total volume of the material drained.

Specific retention or field capacity

$$= \frac{\text{Volume of the water held against gravity drainage}}{\text{total volume of the material drained}} \times 100 \quad \text{...(16.9)}$$

It is evident that the sum of the specific yield and the specific retention is equal to its porosity.

16.6.3. Specific Retention of Different Kinds of Formations. As has been said earlier, the specific retention is the amount of the water held between the grains due to molecular attraction. This film of water is thus held by molecular adhesion on the walls of the interstices. Therefore, the amount of this water will depend upon the total interstitial surface in the rock. If the total interstitial surface is more, the specific retention will be more, and *vice versa.*

Now, if the effective size of the soil grains decreases, the surface area between the interstices will increase, thus, causing more specific retention and less specific yield.

It, therefore, follows that, in fine soils like clay, the specific retention would be more, and hence, it would result in very small specific yield.

The reverse is also true when the grain size increases, the interstitial surface area reduces, and therefore, the specific retention reduces, and hence specific yield increases. It, therefore, follows that in a large particle soil like *coarse gravel*, the specific retention would be small, and it would result in large specific yield.

This conclusion is very important from practical stand point because it follows from this *that a water bearing formation of coarse gravel would supply large quantities of water to wells, whereas the clay formations although saturated and of high porosity would be of little value in this respect.* Hence, the location of wells depends considerably upon the type of neighbouring formations.

16.6.4. Determination of Specific yield of a Formation. It is difficult to determine accurately the specific yield in the laboratory, because it is difficult to obtain an undisturbed sample, and also the short-sample columns used in the laboratory cannot duplicate the very long capillary tubes actually existing in the field. And hence, the field observations are made through the medium of pumping tests at site, in order to determine accurate and reliable values of specific yields, as will be explained later.

16.7. Aquifers and their Types

A permeable stratum or a geological formation of permeable material, which is capable of yielding appreciable quantities of ground water under gravity, is known, as an *aquifer*. The term 'appreciable quantity' is relative, depending upon the availability of ground-water. In the regions, where ground-water is available with great difficulty, even fine-grained materials containing very less quantities of water may be classified as the principal aquifers.

When an aquifer is overlain by a confined bed of impervious material, then this confined bed of overburden is called an **aquiclude**, as shown in Fig. 16.4.

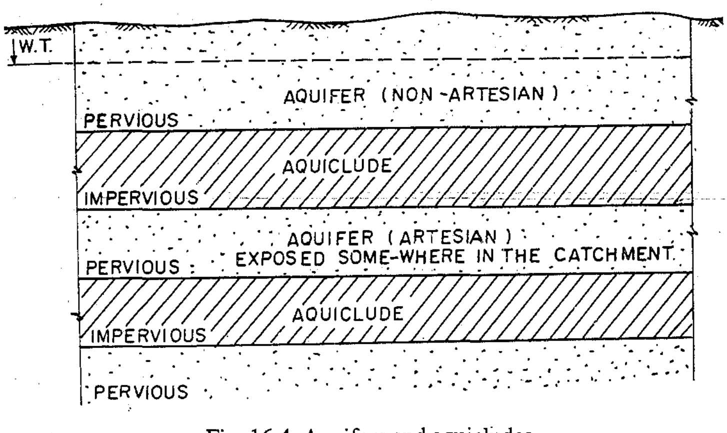


Fig. 16.4. Aquifers and aquicludes.

The amount of water yielded by a well, excavated through an aquifer, depends on many factors; some of which, such as the well diameter are inherent in the well itself. But all other things being equal, the permeability and the thickness of the aquifer are the most important.

Aquifers vary in depth, lateral extent, and thickness; but in general, all aquifers fall into one of the two categories, *i.e.*

1. Unconfined or Non-artesian aquifers, and
2. Confined or Artesian aquifers.

16.7.1. Unconfined Aquifers or Non-artesian Aquifers. The top most water bearing stratum, having no confined impermeable over burden (*i.e.* aquiclude) lying over it, is known as an *unconfined* aquifer or *non-artesian* aquifer. (Refer Fig. 16.5).

The ordinary gravity wells of 2 to 5 m diameter, which are constructed to tap water from the top most water bearing strata, *i.e.* from the unconfined aquifers,. are known as *unconfined* or *non-artesian* wells. The water level in these wells will be equal to the level of the watertable. Such wells are, therefore, also known as *wells* or *gravity wells.*

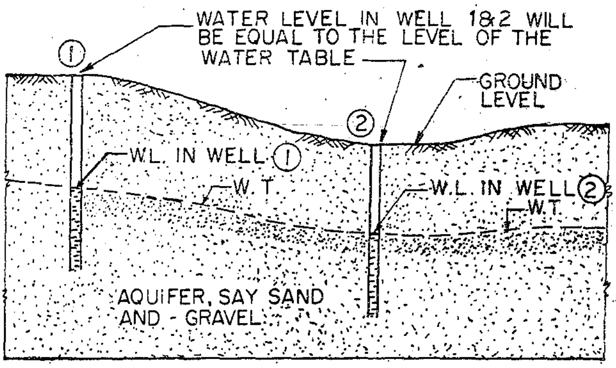


Fig. 16.5. Non-artesian or Unconfined aquifers and wells.

16.7.2. Confined Aquifers or Artesian Aquifers. When an aquifer is confined on its upper and under surface by impervious rock formations (*i.e.* aquicludes), and is also broadly inclined so as to expose the aquifer somewhere to the catchment area at a higher level for the creation of sufficient hydraulic head, it is called a *confined aquifer* or an *artesian aquifer*. A well excavated through such an aquifer, yields water that often flows out automatically, under the hydrostatic pressure, and may thus, even rise or gush out of surface for a reasonable height. However, where the ground profile is high, the water may remain well below the ground level. The former type of artesian well, where water gushes out automatically, is called a **flowing well.**

The level to which the water will rise in an artesian well is determined by the highest point on the aquifer, from where it is fed from the rain falling in the catchment. However, the water will not rise to this full height, because the friction of the water moving through the aquifer uses up some of the energy.

The question whether it will be a *flowing artesian well* or a *non-flowing artesian well* depends upon the topography of the area, and is not the inherent property of the artesian aquifer. In fact, if the *pressure surface** lies above the ground surface, the well will be a flowing artesian well; whereas, if the pressure surface is below the ground

* It is the imaginary surface which represents the magnitude of the hydrostatic pressure available along the artesian aquifer. It is represented by the line joining the various piezometric heads in various tightly cased wells tapping the aquifer.

surface, the well will be artesian but non- flowing, and will require a pump to bring the water to the surface, as shown in Fig. 16.6. Such non-flowing artesian wells are sometimes called as sub-artesian wells.

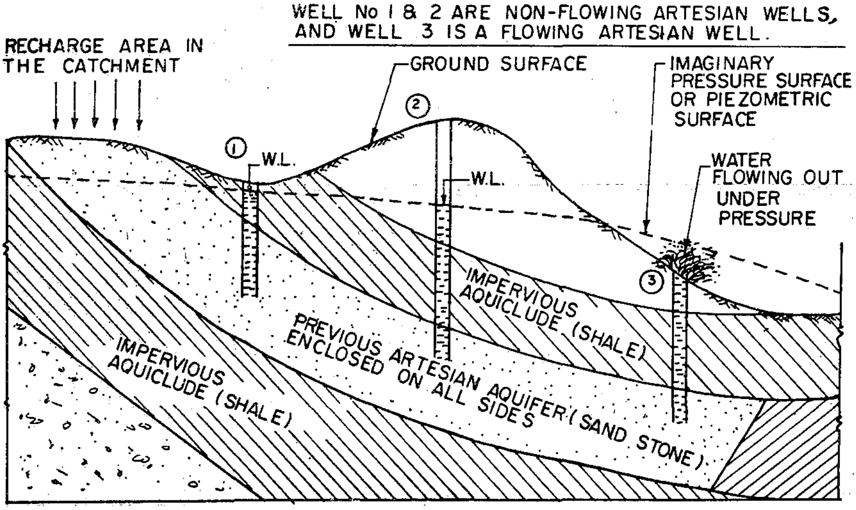


Fig. 16.6. Confined or Artesian aquifers and wells.

16.7.3. Perched Aquifers. Perched aquifer is a special case which is sometimes found to occur within an unconfined aquifer.

If within the zone of saturation, an impervious deposit below a pervious deposit is found to support a body of the saturated material, then this body of the saturated material, which is a kind of aquifer, is known as a *perched aquifer*. The top surface of the water held in the perched aquifer is known as the *perched watertable*. This is shown in Fig. 16.7.

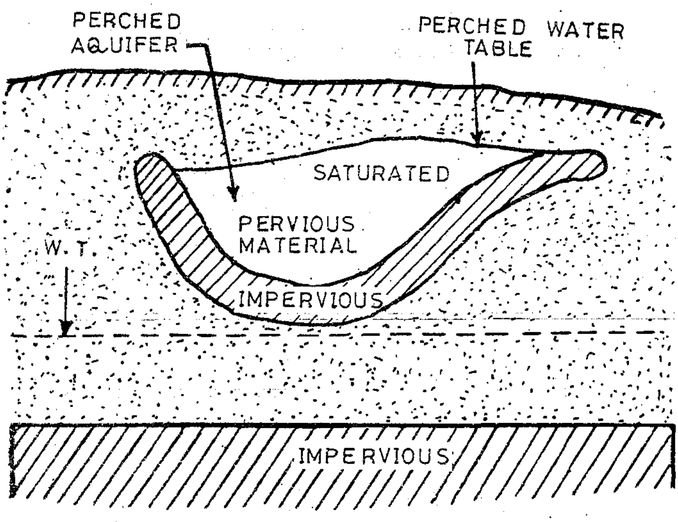


Fig. 16.7. Perched aquifer.

16.8. Certain Other Important Terms Connected with Ground Water

16.8.1. Specific Capacity. Specific capacity of a well is the rate of flow from a well per unit of drawdown. It should be determined for the fall of the first metre, as it is not the same for all the drawdowns.

16.8.2. Coefficient of Storage. Now, we introduce a very important term known as storage coefficient, which may be denoted by A.

Water discharged from an aquifer or recharged into an aquifer represents the change in its storage volume. For unconfined aquifers, this change can be easily determined. Knowing the fall or the rise of the watertable in a given time (t) and multiplying it with the average specific capacity during this time, the change of the storage volume in a time t can be found out.

But in artesian aquifers, (assuming that the aquifer itself remains saturated throughout the drainage), changes in pressure produce only small changes in storage volume. Thus, the hydrostatic pressure within an aquifer partially supports the overburden, while the remainder is supported by the solid structure of the aquifer, When the hydrostatic pressure is reduced, such as by pumping water from a well penetrating the aquifer, the aquifer load increases. The aquifer gets compressed and forces some water from it. In addition, lowering of the pressure causes a small expansion and subsequent release of water. This water yielding capacity of an artesian aquifer can be expressed by its storage coefficient.

The storage coefficient (A) for an artesian aquifer is equal to the volume of the water released from the aquifer of unit cross- sectional area and of the full height of the aquifer when the piezometric surface declines by unity. In general, storage coefficient is defined as the volume of water that an aquifer releases or stores per unit surface area of the aquifer per unit change in the component of head normal to that surface.

16.9. Measurement of Yield of Underground Sources (Aquifers)

The yield of an aquifer can be estimated in the following different ways ;

(*i*) *On the basis of flow velocity of the ground water;* and

(*ii*) *By performing pumping tests in the field.*

These methods are discussed below.

16.9.1. Estimation of the Yield by Estimation of the Velocity of Ground Water. If a well is penetrated through an aquifer, the water will rush into it with a velocity say v; and if A is the area of the aquifer opening into the well, then Q will be given as :

$$Q = v \, . \, A \qquad ...(16.10)$$

where v is the discharge velocity into the well, and is given by the eqn. 16.4) as :

$$\boxed{v = n \cdot v_a}$$

where n = porosity of the soil medium

v_a = actual flow velocity of ground water.

Eventually, the discharge of the well excavated through the given aquifer is given as:

$$Q = n \cdot v_a \cdot A \qquad \text{...(16.11)}$$

In the above equation, the velocity of the ground water flow (v_a) can be estimated by using Slichters or Hazen's empirical equations (eqn. 16.5 and 16.6, respectively); or it can better be measured in the actual field by using chemical tracers, such as a dye ; or by using electrical resistivity methods.

The time (t) taken by a **chemical tracer** to travel a given known distance (S) between two observation wells, will directly indicate the ground flow velocity as, $v_a = \frac{S}{t}$. Such a test can also help to determine K, since $v = K \cdot I$; or $K = \frac{v}{I} = \frac{n \cdot v_a}{I}$ where $I = \frac{H_L}{S}$ where H_L is the difference of water surface elevations of the two wells.

Example 16.2. *In a field test, a time of 6 hours was required for a tracer to travel through an aquifer from one well to another. The observation wells were 42m apart, and the difference in their water levels was found to be 0.42 m. Compute (i) the discharge velocity; (ii) the coefficient of permeability (K). Given the porosity of the soil medium as 20%. (iii) Also compute the value of coefficient of intrinsic permeability for the aquifer in Darcy's, if viscosity of water* $\nu = 0.01\ cm^2/s$.

Solution. Velocity of the ground water flow $= v_a = \frac{S}{t} = \frac{42\text{ m}}{6\text{ h}}$

$$= \frac{4200}{6 \times 60 \times 60}\text{ cm/sec} = 0.194\text{ cm/sec}$$

The discharge velocity $= v = n \cdot v_a = 0.2 \times 0.194$ cm/sec $=$ **0.0388 cm/sec** **Ans.**

Hydraulic gradient between the wells $= I = \frac{H_L}{S} = \frac{0.42\text{ m}}{42\text{ m}} = \frac{1}{100}$

using : $v = K \cdot I$, we have $0.0388\text{ cm/sec} = K \times \frac{1}{100}$

or $\qquad K = 100 \times 0.0388\text{ cm/sec} = 3.88\text{ cm/sec}.$

$\therefore$ coefficient of permeability = **3.88 cm/sec** **Ans.**

Intrinsic permeability coefficient is given by the eqn.

$$K_0 = \frac{K \cdot \nu}{g} \qquad \text{...(16.12)}$$

$$= \frac{3.88\text{ cm/sec} \times 0.01\text{ cm}^2/\text{sec}}{981\text{ cm/sec}^2}$$

$$= \frac{3.88 \times 0.01}{981}\text{ cm}^2 = 3.96 \times 10^{-5}\text{ cm}^2$$

Since $\mathbf{9.87 \times 10^{-9}\ cm^2 = 1}$ **darcy**, we have

K_0 in darcy $= \frac{3.96 \times 10^{-5}}{9.87 \times 10^{-9}}$ darcys $=$ **4007 darcys** **Ans.**

16.9.2. Estimation of Yield by Pumping Tests. The yield of an aquifer can be estimated by conducting two direct practical tests in the field, which can directly indicate the well yield from the given aquifer. These tests are:

(*a*) Pumping test ; (*b*) Recuperating test

Both these tests are discussed below:

(*a*) **Pumping test.** A well is, first of all, constructed through the aquifer, of which the yield is to be estimated. Huge amount of water is drawn from the well, so as to cause heavy drawdown in its water level. The rate of pumping is changed and so adjusted that the water level in the well becomes constant. In this condition of equilibrium, the rate of pumping will be equal to the rate of yield, and hence, the rate of pumping will directly give us the yield of the well at a particular drawdown.

But it is very difficult and almost impractical to adjust the rate, so as to keep the well water level constant.

(*b*) **Recuperating test.** In this method, water is first of all drained from the well at a fast rate, so as to cause sufficient drawdown. The pumping is then stopped. The water level in the well will start rising. The rise is noted at regular intervals of time, till the initial level is reached. Knowing the area of the well and the rise of the water level, the volume of the water yielded in that given time interval, can be worked out at different drawdowns.

It is found that the yield is higher at higher drawdowns. These tests are generally conducted during the driest periods of years, so as to know the yield under the worst conditions.

In addition to the above two direct practical tests for determining the yield of a well or an aquifer, another pumping test with one, two, or more observation wells can be performed to determine the characteristics of an aquifer. Although, dominant hydraulic properties of the aquifer can be computed even by the use of one observation well with a main pumping well, yet more precise results could be obtained by installing two or more observation wells at varying distances from the main central pumped well. To obtain precise & reliable results, the pumping in the main well may be continued for a long period of 15 to 20 hours or more, to obtain study conditions of water levels in the wells. However, it is not necessary to wait till steady conditions are reached, because even when the duration of the pumping is small, aquifer parameters can be computed by using the appropriate methods/formulas. The test results consist of recording of the drawdowns in the observation wells, which can be used to compute the aquifer characteristics by using the appropriate theoretical formulas, developed by Dupuit, Thiem, Thies, etc., as shall be discussed in the following articles.

THEORETICAL APPROACH TO COMPUTATION OF COEFFICIENT OF PERMEABILITY (K) AND DISCHARGE CAPACITY OF AN AQUIFER(Q)

Dupuit, Thiem, and Theis have developed the theoretical analysis of ground water flow, which is discussed in the following articles. The theory & formulas developed by them can be used to compute K value for a given aquifer by performing a field test of observing water levels in one or two observation well(s) at the known pumped discharge, in the main well. This K value of the given aquifer can be used to compute Q value for a well or a tubewell to be driven through that aquifer, at any given drawdown, such as 1 m, when it will indicate the specific capacity of the well. We will first of all derive the equations of Thiem, and then switch to the equations of Dupuit, although Dupuit's equations were put forth prior to the Thiem's equations.

16.10. Thiem's Equilibrium Formulas for Unconfined as well as Confined Aquifers.

16.10.1. Thiem's Formula for Unconfined Aquifer case *(for Watertable or Gravity wells).*

Let a non-artesian well be driven, and water pumped heavily so as to cause sufficient drawdown. When the water level in the well decreases, the water level in the neighbourhood will also fall down; forming what is called as an **inverted cone of depression** all around the well, as shown in Fig. 16.8. The base of this cone is a circle of radius R, known as the *circle of influence*, and the inclined side is known as the **drawdown curve.**

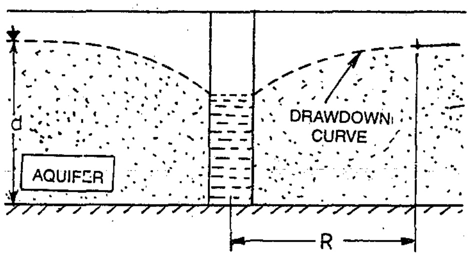


Fig. 16.8. Inverted cone of depression.

In the method suggested by Thiem, two observation wells lying within the circle of influence of the main pumped well are to be driven. Let these wells be numbered as 1 and 2 (Fig. 16.9) and let them be at distances of r_1 and r_2 from the main well (centre to centre distance). Let d be the depth of the well or the aquifer, below the static watertable.

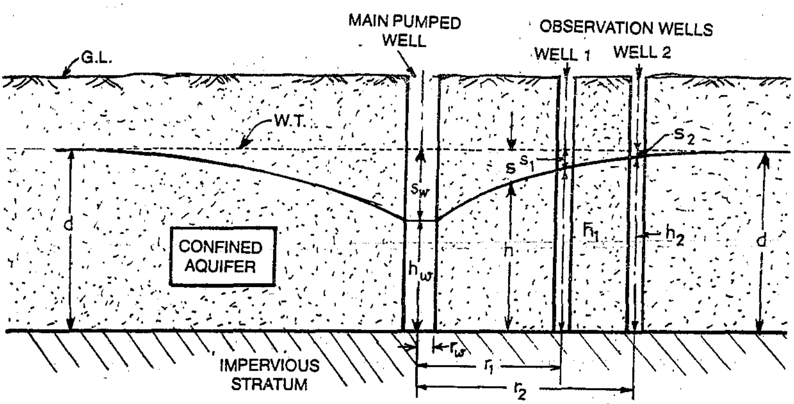


Fig. 16.9. Unconfined aquifer case of Thiem's formula.

Now, let the main well be pumped at a sufficient rate, so as to cause heavy drawdown. Then, let the pumping be so adjusted that the equilibrium conditions are reached. In other words, the rate of pumping becomes equal to the rate of yield, and thus causing the water level to attain a constant value. Since the formula involves the use of equilibrium conditions, it is known as the *equilibrium formula*. Let s_1 and s_2 be the drawdowns in the two corresponding observation wells, at this equilibrium stage.

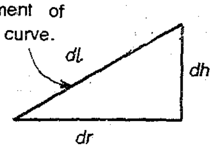
From Darcy's law, the flow through any concentric cylindrical section of the water bearing material is given by Eq. (16.2) as :

$$Q = KIA \qquad \text{where } I = \text{Hydraulic gradient}$$

Using cylindrical co-ordinates, we take r as the radius of any cylinder, and h as the height of the cone of depression at a distance r from the main well.

*Assuming that the inclination of the water surface is small**, so that the tangent can be used in place of sine for the hydraulic gradient in Darcy's law, we have

$$I = \frac{dh}{dl} \approx \frac{dh}{dr}.$$



$$\therefore \qquad I = \frac{dh}{dr}$$

Also *assuming that the water flows through the full height of the aquifer (below the W.T. of course) and also that the flow is radial and horizontal (by horizontal flow we mean that the velocity distribution is assumed to be uniform as shown in Fig. 16.10).**, the area of flow (A) is equal to $2\pi rh$.

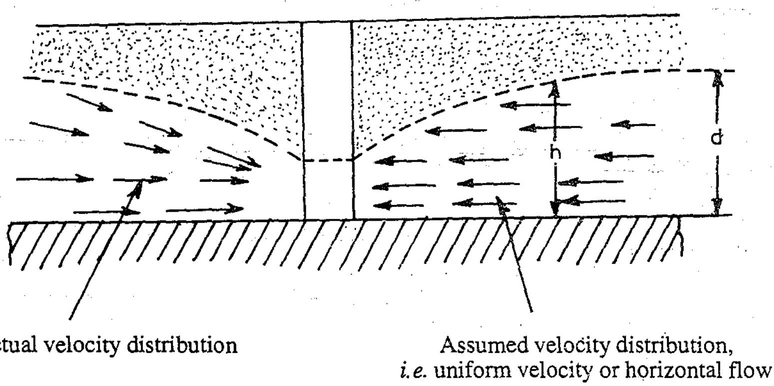


Fig. 16.10.

Substituting these values of I and A in Darcy's law, we get

$$Q = KIA = K \cdot \frac{dh}{dr} \cdot 2\pi r \cdot h$$

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

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

Integrating between the limits r_1 and r_2 and h_1 and h_2, we get

$$\int_{r_1}^{r_2} \frac{dr}{r} = \int_{h_1}^{h_2} \frac{2\pi K}{Q} \cdot h dh$$

where Q = Constant when steady conditions have reached.

K = Permeability of soil, which is assumed to be constant *at all places and at all times* *i.e.* (homogeneous soil), and in all directions, *i.e.* (isotropic soil), by assuming the soil to be homogeneous and isotropic

* Evidently, the above assumptions shall be quite true to the actual conditions in most of the flow region except in the immediate neighbourhood of the well.

Therefore $$\int_{r_1}^{r_2} \frac{dr}{r} = \frac{2\pi K}{Q} \int_{h_1}^{h_2} h\,dh$$

or $$\left|\log_e r\right|_{r_1}^{r_2} = \frac{2\pi K}{Q} \left|\frac{h^2}{2}\right|_{h_1}^{h_2}$$

or $$\log_e \frac{r_2}{r_1} = \frac{2\pi K}{Q}\left[\frac{(h_2^2 - h_1^2)}{2}\right] = \frac{\pi K}{Q}\left[h_2^2 - h_1^2\right]$$

or $$K = \frac{Q \cdot \log_e \frac{r_2}{r_1}}{\pi\,(h_2^2 - h_1^2)}$$

or $$\boxed{Q = \frac{\pi K\,(h_2^2 - h_1^2)}{2.3 \log_{10} \frac{r_2}{r_1}}} \quad ...(16.13)$$

This important Thiem's formula can be further simplified, if required, as follows :

$$(h_2^2 - h_1^2) = (h_2 + h_1) \cdot (h_2 - h_1)$$

But $$h_2 - h_1 = s_1 - s_2$$

and if the amount of drawdown is small compared to the saturated thickness of the water bearing material, then h_2 and h_1 are nearly equal and each is approximately equal to this saturated thickness, say d.

Therefore, $h_1 + h_2 \approx d + d = 2d$

or $$(h_2^2 - h_1^2) \approx (s_1 - s_2)\,2d = 2d\,(s_1 - s_2)$$

Putting this value in equation (16.13), we get

$$Q \approx \frac{2\pi \cdot Kd\,(s_1 - s_2)}{2.3 \log_{10} \frac{r_2}{r_1}} \quad ...(16.14)$$

Introducing the coefficient of transmissibility (T) instead of K, *i.e.*

$T = Kd$, we have

$$Q \approx \frac{2\pi\,T\,(s_1 - s_2)}{2.3 \log_{10} \frac{r_2}{r_1}} \quad ...(16.14a)$$

This is another form of Thiem's formula for the unconfined aquifers. The various assumptions that we have made in its derivation are again summarised below :

1. The aquifer is homogeneous, isotropic and of infinite and areal extent, so that its coefficient of transmissibility or permeability is constant everywhere.

2. The well has been sunk through the full depth of the aquifer and it receives water from the entire thickness of the aquifer.

3. Pumping has continued for a sufficient time at a uniform rate, so that the equilibrium stage or steady flow conditions have reached.

4. Flowlines are radial and horizontal, and flow is laminar.

5. The inclination of the water surface is small so that its tangent can be used in place of sine for the hydraulic gradient in Darcy's equation.

16.10.2. Thiem's Formula for Confined Aquifer Case. (*For Pressure or Artesian wells*) The above formula has to be slightly modified in the case of an artesian aquifer. The conditions of a confined aquifer case are shown in Fig. 16.11.

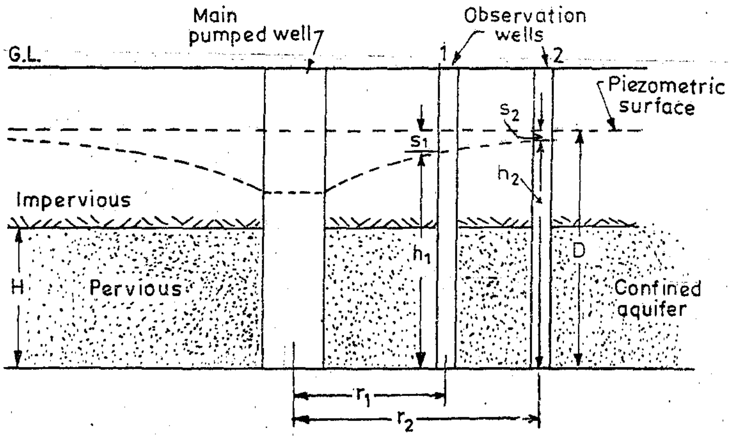


Fig. 16.11. Confined aquifer case for Theim's formula.

In a confined aquifer, the flow is actually radial and horizontal (Refer Fig. 16.12) and, therefore, it has not to be assumed as such, as it was in the unconfined case. Rest of the assumptions remain the same and hold good in this case also.

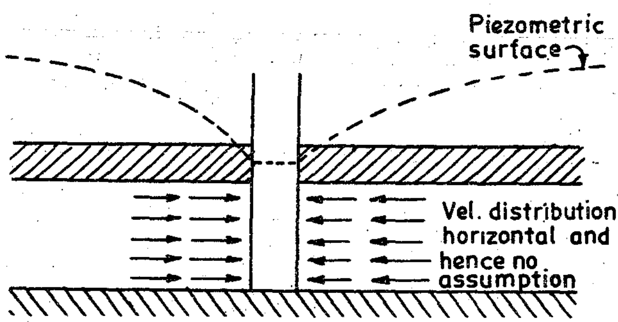


Fig. 16.12.

$$Q = KIA = K \cdot \frac{dh}{dr} 2\pi rH, \text{ because}$$

water is drawn from the cylinder of radius r and Height $= H$.

where H = height of the confined aquifer.

Now $$Q = 2\pi K.H.r. \frac{dh}{dr}$$

or $$\frac{dr}{r} = \frac{2\pi KH}{Q} \cdot dh$$

Integrating between the limits of r_1 and r_2, we get

$$\int_{r_1}^{r_2} \frac{dr}{r} = \frac{2\pi KH}{Q} \int_{h_1}^{h_2} dh$$

or $$\left| \log_e r \right|_{r=r_1}^{r=r_2} = \frac{2\pi KH}{Q} \left| h \right|_{h=h_1}^{h=h_2}$$

or $$\log_e \frac{r_2}{r_1} = \frac{2\pi KH}{Q} \left[h_2 - h_1 \right]$$

or $$\boxed{Q = \frac{2\pi KH\,(h_2 - h_1)}{2.3 \log_{10} \frac{r_2}{r_1}}} \qquad ...(16.15)$$

But $h_2 - h_1 = s_1 - s_2$

Therefore, $$Q = \frac{2\pi KH\,(s_1 - s_2)}{2.3 \log_{10} \frac{r_2}{r_1}} \qquad ...(16.16)$$

Also $$Q = \frac{2\pi T\,(s_1 - s_2)}{2.3 \log_{10} \frac{r_2}{r_1}} \qquad ...(16.16\ a)$$

$[\because \; KH = T]$

where $s_1 - s_2$ is the difference of water levels between the two observation wells after the steady conditions have reached.

Limitations. Various assumptions have been made in the derivation of the above Thiem's formulae. In actual practice, however, none of these conditions may get fulfilled; say for example, an aquifer is not fully homogeneous, or the well might have been dug half way through the aquifer, or permeability may not be uniform, or the ground watertable may be inclined and thus, the base of the cone may not be a circle, or the equilibrium conditions might have not fully reached.

However, it is very difficult to assess the effects of these factors, and despite the various limiting assumptions, Thiem's formula is widely used in ground water problems and many of its limitations are removed by appropriate adjustments.

16.11. Dupuit's Original Equilibrium Formulas

The formulae put forward by Thiem and which have been derived earlier for a gravity well as well as for a pressure well are the refined forms of the original Dupuit's formulae.

We will now discuss the original Dupuit's formulas. In Dupuit' formulas, no observation wells (as constructed in Thiem's formula) are constructed. The main well is pumped out so as to get sufficient drawdown, and then the rate of pumping is so adjusted as to establish equilibrium conditions (*i.e.* the rate of inflow becomes equal to the rate of outflow, and the water level in the well becomes constant).

All the assumptions which have been made in the Theim's formulas hold good for the Dupuit's formulas also. The only difference is that the integration which was done between the limits of r_1 and r_2 (radii of two observation wells) in Theim's formulas is changed, and the integration is done between the limits r_w and R, where r_w is the radius of the main pumped well and R is the radius of influence.

The **radius of influence** is the distance from the centre of the pumped well to the point, where the drawdown is zero or is inappreciable. The complete derivations of the Dupuit's formulae for a gravity well as well as for a pressure well are given below :

16.11.1. Dupuit's Formula for Gravity Well or Unconfined Acquifer Case (Refer Fig. 16.13).

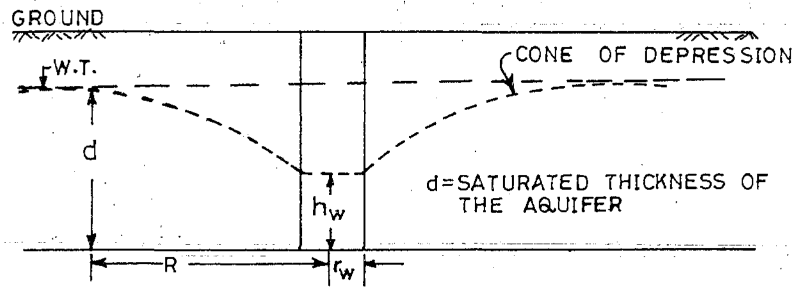


Fig. 16.13. Unconfined aquifer case for Dupuit's formula.

$$Q = K.I.A.$$

or $$Q = K \cdot \frac{dh}{dr} \cdot 2\pi rh$$

where K = Coefficient of permeability

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

Integrating between the limits r_w and R, we get

$$\int_{r_w}^{R} \frac{dr}{r} = \frac{2\pi K}{Q} \int_{h_w}^{d} h.dh$$

or $$\left| \log_e r \right|_{r_w}^{R} = \frac{2\pi K}{Q} \left| \frac{h^2}{2} \right|_{h_w}^{d}$$

or $$\log_e \frac{R}{r_w} = \frac{\pi K}{Q} \cdot \left[d^2 - h_w^{\ 2} \right]$$

or $$2.3 \log_{10} \frac{R}{r_w} = \frac{\pi K}{Q} \left[d^2 - h_w^{\ 2} \right]$$

or $$K = \frac{2.3\, Q \log_{10} \frac{R}{r_w}}{\pi (d^2 - h_w^2)}$$

or $$\boxed{Q = \frac{\pi K (d^2 - h_w^{\ 2})}{2.3 \log_{10} \frac{R}{r_w}}} \qquad ...(16.17)$$

Since the value of R is not easily assessible, various arbitrary values have been assigned to R by various investigators. Slitcher gives it as 500 ft. (150m) and Tolman calls it as 1000 ft. (300 m). But a more realistic picture is obtained ; when $R = CQ$, or $R \propto Q$

where, C = is a constant and
Q = is the discharge.

Putting $R = CQ$ in equation (16.17), we get

$$Q = \frac{\pi K (d^2 - h_w^2)}{2.3 \log_{10}\left(\frac{CQ}{r_w}\right)} \qquad ...(16.18)$$

Q can be determined by Hit and Trial method.

16.11.2. Dupuit's Formula for Pressure Well or Confined Aquifer Case. (Refer Fig. 16.14).

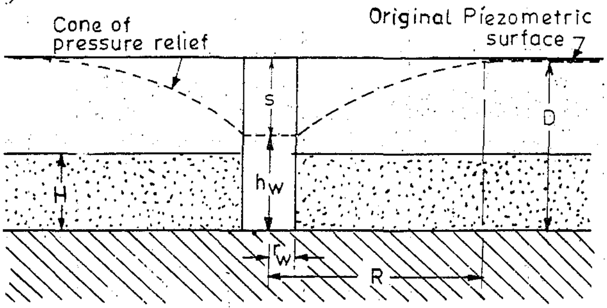


Fig. 16.14. Confined aquifer case for Dupuit's formula.

$$Q = KIA$$

or $$Q = K \cdot \frac{dh}{dr} 2\pi \cdot rH$$

or $$\frac{dr}{r} = \frac{2\pi KH}{Q} \cdot dh$$

Integrating between r_w and R, we get

$$\int_{r_w}^{R} \frac{dr}{r} = \frac{2\pi K}{Q} \cdot H \int_{h_w}^{D} dh;$$

where D = depth of the well or height of the aquifer below the original peizometric surface

or $$\left|\log_e r\right|_{r=r_w}^{r=R} = \frac{2\pi K}{Q} \cdot H \left|h\right|_{h=h_w}^{h=D}$$

or $$2.3 \log_{10} \frac{R}{r_w} = \frac{2\pi KH}{Q}\left[(D - h_w)\right]$$

or $$K = \frac{2.3\, Q \log_{10} \frac{R}{r_w}}{2\pi.H.\,(D - h_w)}$$

or $$\boxed{Q = \frac{2\pi KH\,(D - h_w)}{2.3 \log_{10} \frac{R}{r_w}}} \qquad ...(16.19)$$

or $$Q = \frac{2\,\pi KHs}{2.3\log_{10}\left(\frac{R}{r_w}\right)} \quad ...(16.20)$$

where H = total height of the confined aquifer.
h_w = artesian pressure in the well.
r_w = radius of the well.
D = initial artesian pressure at the bottom of the aquifer or the initial height of the piezometric surface from the bottom of the well.
s = drawdown = $(D - h_w)$

16.12. Partial Penetration of an Aquifer by a Well

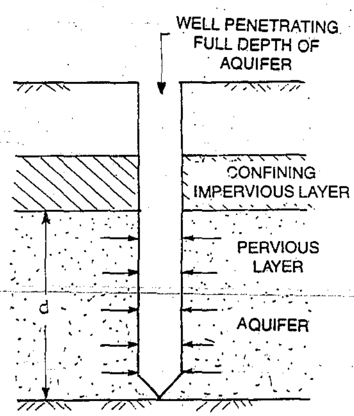
From eqn. (16.17), we have

$$Q = \frac{\pi K\,(d^2 - h_w^{\,2})}{2.3\log_{10}\left(\frac{R}{r_w}\right)} \quad \text{(for unconfined wells)}$$

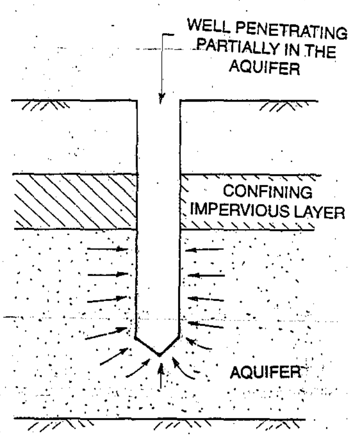
and from eqn. (16.19), we have

$$Q = \frac{2\pi KH\,(D - h_w)}{2.3\log_{10}\left(\frac{R}{r_w}\right)} \quad \text{(for confined wells)}$$

The above equations have been derived, respectively for a gravity well and for an artesian well, which penetrate throughout the aquifer. But, if a well does not penetrate up to the bottom of the aquifer, these formulas will not be applicable, as the nature of the flow will become three dimensional. It will not only be radial but will also have an upward component, as shown in Fig. 16.15. The yield of such a well is found to be more than that of a fully penetrating well of the same depth.



(*a*) Showing flow lines in a well fully penetrating the aquifer.



(*b*) Showing flow lines in a well partly penetrating the aquifer.

Fig. 16.15.

Kozeny has given a correction factor, and according to him the discharge Q_P through such a well is given as follows :

(*a*) Partially penetrating gravity well. (*b*) Partially penetrating artesian well.

Fig. 16.16.

Discharge (Q_P) for a partially penetrating gravity well (Fig. 16.16 (*a*)].

$$= \left[\frac{\pi \cdot K \cdot (d_1^2 - h_w^2)}{2.3 \log_{10}\left(\dfrac{R}{r_w}\right)}\right]\left[1 + 7 \,.\sqrt{\frac{r_w}{2d_1}} \cdot \cos\frac{\pi d_1}{2d}\right] \quad \ldots(16.21)$$

where d_1 = Actual penetration depth below the watertable

d = Actual depth of the aquifer below the watertable.

Similarly, the discharge (Q_P) for a partially penetrating artesian well [Fig. 16.16 (*b*)].

$$= \left[\frac{2\pi \cdot KH_1\,(D - h_w)}{2.3 \log_{10}\left(\dfrac{R}{r_w}\right)}\right]\left[1 + 7 \,.\sqrt{\frac{r_w}{2H_1}} \cdot \cos\frac{\pi H_1}{2H}\right] \quad \ldots(16.22)$$

where H = Full depth of the confined aquifer.

H_1 = Depth upto which the well penetrates.

D, h_w, r and R have the same meaning as given earlier.

16.13. Spherical Flow in a Well

Fig. 16.17 shows a special case of partially penetrating well, where the well just penetrates up to the top surface of the semi-infinite porous medium. In this case, $H_1 = 0$, and the equation (16.19) is not applicable because the flow towards the well is purely spherical. The discharge Q_s from such a well can be calculated from the equation,

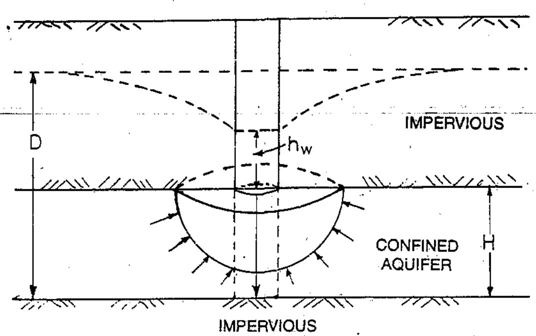


Fig. 16.17. Speherical flow in a well.

$$Q_s = 2\pi K.r_w.\,(D - h_w) \quad \ldots(16.23)$$

Whereas in the case of a simple radial flow in a fully penetrating well, the discharge is given by the equation (16.19) as :

$$Q = \frac{2\pi K.H.\,(D - h_w)}{2.3 \log_{10}\left(\frac{R}{r_w}\right)}$$

Now $$\frac{Q_s}{Q} = \frac{2\pi K r_w \cdot (D - h_w)}{\left(\frac{2\pi KH\,(D - h_w)}{2.3 \log_{10}\left(\frac{R}{r_w}\right)}\right)} = \frac{2.3\, r_w \log_{10}\left(\frac{R}{r_w}\right)}{H}$$

or $$\frac{Q_s}{Q} = 2.3\left(\frac{r_w}{H}\right)\log_{10}\left(\frac{R}{r_w}\right) \qquad ...(16.24)$$

For example, if

$r_w = 10\text{ cm} = 0.1\text{ m}$; and

$R = 100\text{ m}$

Then $\frac{R}{r_w} = 1{,}000$; and let

H = thickness of the confined aquifer = 20 m (say)

Then $$\frac{Q_s}{Q} = 2.3\left(\frac{0.1}{20}\right)\log_{10}(1000)$$

$$= \frac{2.3}{200} \times 3.0 = \frac{6.9}{200} \approx \frac{1}{30}$$

This shows that the yield in a spherical flow is much less than that in a radial flow. Hence, *the spherical flow is much less efficient than the radial flow.*

16.14. Interference Among Wells

If two or more wells are constructed in such a way that they are near to each other and their cones of depressions interact, they are said to interfere. Such interference of wells decreases the discharges of such interfering wells.

Muskat has proposed the following formulas for computation of discharges from such interfering wells. These formulas have been found to yield reliable results.

(*A*) For Confined Aquifers (*i.e.* Artesian Wells)

(1) *For two artesian identical* wells at a distance B apart,*

$$Q_1 = Q_2 = \frac{2\pi KH \cdot (D - h_w)}{2.3 \log_{10}\left(\frac{R^2}{r_w \cdot B}\right)} \qquad ...(16.25)$$

(2) *For three artesian identical wells at distances B apart, in a pattern of equilateral triangle,*

$$Q_1 = Q_2 = Q_3 = \frac{2\pi KH\,(D - h_w)}{2.3 \log_{10}\left(\frac{R^3}{r_w \cdot B^2}\right)} \qquad ...(16.26)$$

* Wells of the same diameter, drawdown, and discharge over the same period of time.

(3) *For three artesian identical wells, at distances B apart in a straight line*

$$Q_1 = Q_3 = \left[\frac{\frac{2\pi KH}{2.3}(D-h_w)\log_{10}\left(\frac{B}{R_w}\right)}{\log_{10}\left(\frac{R}{B}\right)\cdot\log_{10}\left(\frac{B}{r_w}\right)+\log_{10}\left(\frac{B}{2r_w}\right)\cdot\log_{10}\left(\frac{R}{r_w}\right)}\right] \quad ...(16.27)$$

and

$$Q_2 = \left[\frac{\frac{2\pi KH}{2.3}(D-h_w)\cdot\log_{10}\left(\frac{B}{2r_w}\right)}{2\cdot\log_{10}\left(\frac{R}{B}\right)\cdot\log_{10}\left(\frac{B}{r_w}\right)+\log_{10}\left(\frac{B}{2r_w}\right)\cdot\log_{10}\left(\frac{R}{r_w}\right)}\right] \quad ...(16.28)$$

where, Q_1 and Q_3 are the discharges of the outer-wells, and Q_2 is the discharge of the middle well.

Total discharge = $Q_1 + Q_2 + Q_3$.

(B) **For Unconfined Aquifers** (*i.e.* **Gravity Wells**)

All the above formulas can be applied to the unconfined aquifers by replacing HD by $\frac{d^2}{2}$, and $H.h_w$ by $\frac{h_w^2}{2}$.

(1) *For two identical gravity wells at distance B apart, formula would therefore, become*

$$Q_1 = Q_2 = \frac{2\pi K\left(\frac{d^2}{2}-\frac{h_w^2}{2}\right)}{2.3\log_{10}\left(\frac{R^2}{r_w\,.\,B}\right)}$$

or

$$Q_1 = Q_2 = \left[\frac{\pi K(d^2-h_w^2)}{2.3\log_{10}\left(\frac{R^2}{r_w\cdot B}\right)}\right] \quad ...(16.29)$$

(2) *For three identical gravity wells at distances B apart, in a pattern of equilateral triangle, formula would be*

$$Q_1 = Q_2 = Q_3 = \frac{2\pi K\left(\frac{d^2}{2}-\frac{h_w^2}{2}\right)}{2.3\log_{10}\left(\frac{R^3}{r_w\cdot B^2}\right)}$$

$$= \frac{\pi K(d^2-h_w^2)}{2.3\log_{10}\left(\frac{R^3}{r_w\cdot B^2}\right)} \quad ...(16.30)$$

16.15. Surface of Seepage and Free Surface Curve

The surface throughout which the pressure is atmospheric is known as the *free surface.*

Let us consider an unconfined aquifer and let AD be the position of the original free water surface (*i.e.*, the watertable) which is approximately horizontal. Let a gravity well of radius r_w be constructed throughout the depth of this aquifer. Let d be the height from the bottom of the well to the ground watertable.

Let the water be pumped from this well (Fig. 16.18). After the pumping, the water will stand in the rest of the bore holes along the line $AB'C'D$, marked as "Free Surface Curve" provided the bore holes were only just deep enough to reach the free surface. Thus, in the aquifer above the surface $AB'C'D$, there is no ground water except capillary moisture.

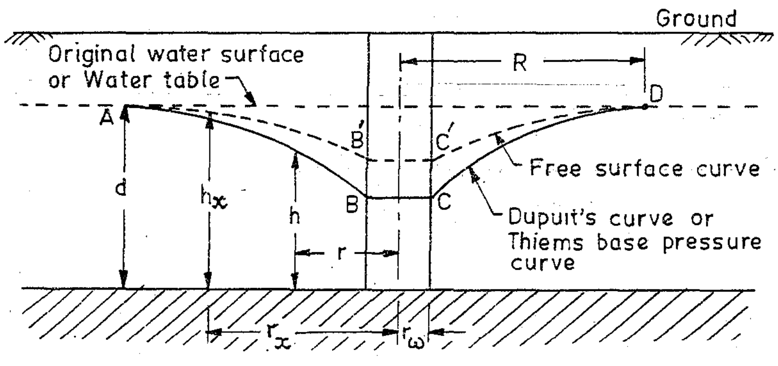


Fig. 16.18. Free surface curve Vs Dupuit's base pressure curve.

But according to the Dupuit's formula, the water level in the pumped well is not found at $B'C'$ level, but at a slightly lower level, *i.e.* at BC, where $ABCD$ is the cone of depression or Dupuit's base pressure curve. Hence, the drawdown in the pumped well is slightly more than the drawdown of the ground watertable immediately adjacent to the well [See Fig 16.19]. This vertical surface of the ground, forming outside of the well hole, which is exposed between the water surface in the well and the free surface is known as the *surface of seepage.*

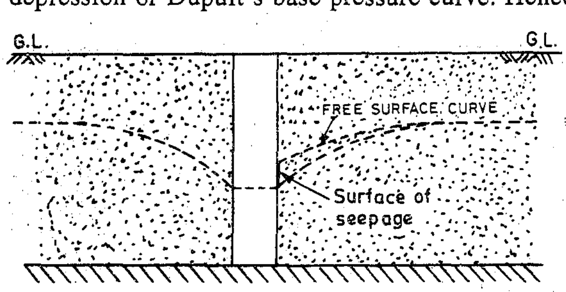


Fig. 16.19. Surface of seepage.

The difference between the actual free surface and the Dupuit's base pressure curve, in a gravity well, arises due to the Dupuit's assumption of 'horizontal and radial flow'. In other words, in an unconfined aquifer, the velocity distribution will not be horizontal near the well but Dupuit assumed it to be so, and this deviation from the actual field conditions gives rise to the difference between the actual watertable and the Dupuit's computed watertable.

The actual watertable deviates more and more from the Dupuit's computed watertable in the direction of flow, as shown in Fig. 16.20. The fact that the actual watertable

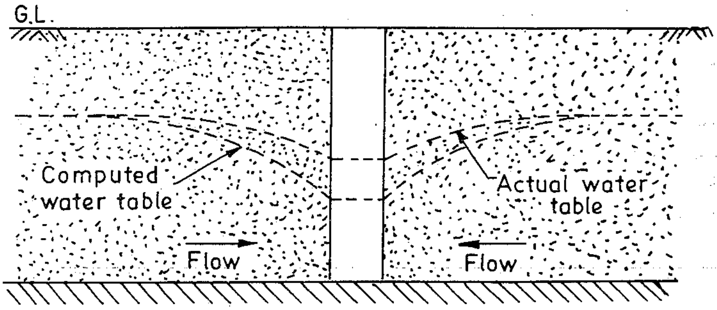


Fig. 16.20. Actual watertable and computed watertable.

lies above the Dupuit's computed watertable can be explained by the fact the Dupuit's flows are all assumed horizontal, whereas the actual prevailing velocities of the same magnitude have downward vertical component, so that a greater saturated thickness is required for the same discharge.

This discrepancy indicates that the watertable does not follow the parabolic path (as given by Dupuit). Nevertheless, for flatter slopes, where the sine and tangents nearly equal, it closely predicts the watertable position except near the outflow.

Equation of free surface curve. The free surface of water in the cone of depression surrounding a pumped well is the surface of water under the atmospheric pressure. And this free surface curve does not coincide with the Dupuit's base pressure curve but lies slightly above the same. The free surface curve is represented by the following equation:

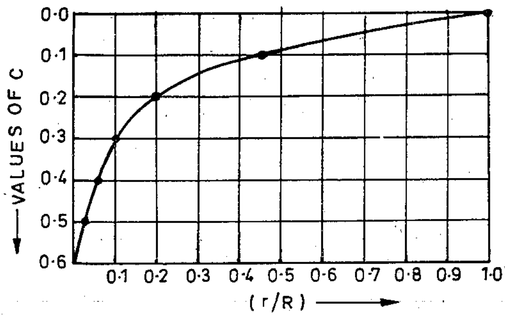


Fig. 16.21. Value of C in Eq. (16.31).

$$Q = \left[\frac{\pi K}{2C}\right]\left[\frac{(d-h)\,d}{\log_{10}\left(\frac{R}{0.1d}\right)}\right] \quad ...(16.31)$$

where (r, h) is any point on the curve.

R = Radius of influence.

K = Permeability coefficient.

C = a constant, the value of which depends upon the value of $\frac{r}{R}$.

The values of C in the above equation can be obtained by the curve shown in Fig. 16.21.

16.16. Well Loss and Specific Capacity

Well loss. When water is being pumped out of a well, the draw- down caused includes not only the drawdown which is given by the logarithmic drawdown curve (Equation 16.20), but also includes a certain drawdown caused by the flow of water through the well screen and axial movement within the well. The drawdown caused by the flow through the screen and the axial movement within the well brings the water level in the well from AB to CD (Refer Fig. 16.22). This vertical drawdown AC or BD is known as the well loss. The magnitude of this well loss may be taken equal to C_2Q^2, as the flow in the vicinity of the well face is turbulent.

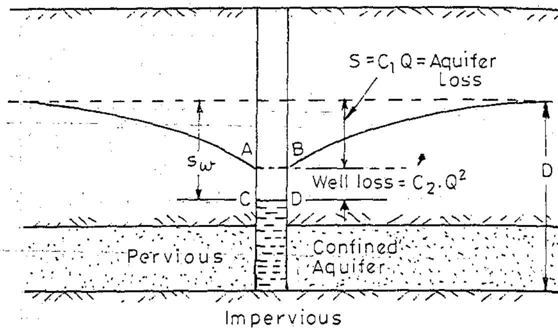


Fig. 16.22.

The total drawdown can then be obtained by adding the two drawdowns.

The first drawdown is obtained from Equation (16.20); according to which

$$Q = \frac{2\pi KHs}{2.3 \log_{10}\left(\frac{R}{r_w}\right)}$$

or
$$s = \frac{2.3\, Q \log_{10} \frac{R}{r_w}}{2\pi KH} = C_1 \cdot Q$$

where C_1 is a constant $= \dfrac{2.3 \log_{10} \frac{R}{r_w}}{2\pi KH}$

The total drawdown is then given as

$$s_w = C_1Q + C_2 \cdot Q^2 \qquad ...(16.32)$$

where C_2Q^2 = **well loss**; and

C_1Q is known as **aquifer loss** or *formation loss*

In case of an unclogged or unencrusted well screen, where the screen size is compatible to the surrounding porous media, the well loss is mainly caused by the axial movement inside the well up to the pump intake; because the well loss caused by the water entering the well through the screen is very small. However, when clogging of screen increases, the well loss increases, which adversely affects the pump efficiency.

Specific Capacity. The specific capacity of a well is defined as the well yield per unit of drawdown. Hence, the

$$\text{Sp. Capacity} = \frac{\text{discharge of the well}}{\text{Drawdown}} = \frac{Q}{C_1Q + C_2Q^2}$$

$$\therefore \quad \text{Sp. Capacity} = \left[\frac{1}{C_1 + C_2Q}\right] \qquad ...(16.33)$$

The Equation (16.33) clearly shows that the sp. capacity of the well is not constant but decreases as the discharge increases.

16.17. Efficiency of a Well

It has been discussed in the previous article that discharge from a well is approximately proportional to the drawdown s, (where $s = C_1Q$ neglecting well loss). The discharge per unit of drawdown was called Sp. capacity of the well; this specific capacity will be different for different well designs. For determining the best drawdown discharge conditions for a well, the well may be operated under varying drawdown conditions, and then a graph may be plotted between discharge and drawdown (called yield drawdown curve) as shown in Fig. 16.23.

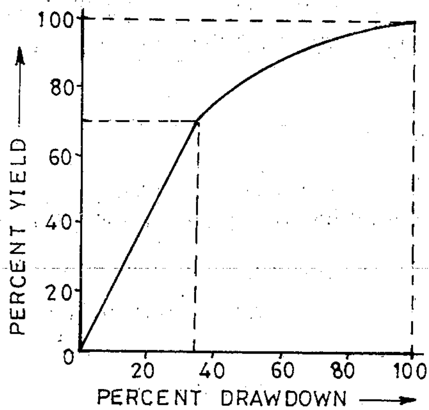


Fig. 16.23. Yield drawdown curve.

The curve obtained is a straight line up to a certain stage of drawdown, beyond which the drawdown increases disproportionately to the yield. This places an optimum and efficient limit to the drawdown, which may be allowed to be created in a well. This is generally found to be 70% of the maximum drawdown which can be created in a well.

Example 16.3. *A pumping test was made in a medium sand and gravel to a depth of 15 m where a bed of clay was encountered. The normal ground water level was at the surface. Observation holes were located at distances of 3 m and 7.5 m from the pumped well. At a discharge of 3.6 litres/sec from the pumping well, a steady state was attained in about 25 hrs. The drawdown at 3 m was 1.65 m and at 7.5 m was 0.36 m. Compute the coefficient of permeability of the soil.*

Solution. $Q = 3.6$ litres/sec

$$r_1 = 3\text{ m};\quad s_1 = 1.65\text{ m};\ h_1 = 15 - 1.65 = 13.35\text{ m}$$

$$r_2 = 7.5\text{ m},\ s_2 = 0.36\text{ m};\ h_2 = 15 = 0.36 = 14.64\text{ m}$$

Using Thiem's formula for unconfined aquifers, *i.e.* equation (16.13), we get

$$Q = \left[\frac{\pi K}{2.3}\right]\left[\frac{(h_2^2 - h_1^2)}{\log_{10}\left(\frac{r_2}{r_1}\right)}\right]$$

or $$\frac{3.6}{1000}\text{ m}^3/\text{sec} = \frac{\pi K}{2.3}\left[\frac{(14.64)^2 - (13.35)^2}{\log_{10}\left(\frac{7.5}{3}\right)}\right] = \frac{\pi K}{2.3}\left[\frac{27.99 \times 1.29}{0.398}\right]$$

or $$K = \left[\frac{2.3 \times 0.398}{\pi \times 27.99 \times 1.29}\right] \times \frac{3.6}{1000} \times 100\text{ cm/sec} = 0.00289\text{ cm/sec.}$$ **Ans.**

Example 16.4. *A well penetrates into an unconfined aquifer having a saturated depth of 100 metres. The discharge is 250 litres per minute at 12 metres drawdown. Assuming equilibrium flow conditions and a homogeneous aquifer, estimate the discharge at 18 metres drawdown. The distance from the well where the drawdown influences are not appreciable may be taken to be equal for both the cases.*

(U.P.S.C., Engg. Services, 1969)

Solution. $d = 100$ m

$s_1 = 12$ m $\qquad s_2 = 18$ m

$Q_1 = 250$ litres/minute, $\qquad Q_2 = ?$

Using Dupuit's formula for unconfined aquifers, *i.e.* eqn. (16.17), we have

$$Q = \frac{\pi K\,(d^2 - h_w^{\,2})}{2.3 \log_{10}\left(\dfrac{R}{r_w}\right)}$$

In the first case ; drawdown = 12 m

$$\therefore \qquad h_w = (100\text{ m} - 12\text{ m}) = 88\text{ m}$$

$$\therefore\ 250 \text{ litres/minute} = \frac{\pi K\,[(100)^2 - (88)^2]}{2.3 \log_{10}\left(\dfrac{R}{r_w}\right)}$$

(Here R and r_w are the same for both the cases)

or
$$\frac{\pi K}{2.3 \log_{10}\left(\dfrac{R}{r_w}\right)} = \frac{250}{(100)^2 - (88)^2} = \frac{250}{188 \times 12} \qquad \ldots(A)$$

In the 2nd case : Drawdown = 18 m

$$h_w = 100 - 18 = 82\text{ m}$$

$$\therefore \qquad Q_2 = \left[\frac{\pi K \times [(100)^2 - (82)^2]}{2.3 \log_{10}\left(\dfrac{R}{r_w}\right)}\right]$$

Putting the value from (A), we get

$$Q_2 = \left[\frac{250}{188 \times 12}\right][(100)^2 - (82)^2]$$

$$= \frac{250 \times 182 \times 18}{188 \times 12} = 363 \text{ litres/minute} \qquad \textbf{Ans.}$$

Example 16.5. *A 30cm diameter well penetrates 25 m below the static watertable. After 24 hours of pumping @ 5400 litres/minute, the water level in a test well at 90 m is lowered by 0.53 m, and in a well 30 m away the drawdown is 1.11m. (a) What is the transmissibility of the aquifer?* (*Engg. Services, 1968*)

(b) Also determine the drawdown in the main well.

Solution. Since the well penetrates 25 m below the static watertable, it evidently is the case of an unconfined aquifer. The Thiem's discharge eqn. for such a well is given by eqn. (16.13) as:

$$Q = \frac{\pi K.\,(h_2^{\,2} - h_1^{\,2})}{2.3 \log_{10} \dfrac{r_2}{r_1}}$$

where
$$h_2 = d - s_2 = 25 - 0.53 = 24.47\text{ m}$$
$$h_1 = d - s_1 = 25 - 1.11 = 23.89\text{ m}$$
$$r_2 = 90\text{ m}$$
$$r_1 = 30\text{ m}$$
$$Q = 5400\text{ l/min} = 5.4\text{ m}^3/\text{min} = 0.09\text{ m}^3/\text{s}$$

Substituting the given values in the above eqn., we get

$$0.09 = \frac{3.14 \times K. [(24.47)^2 - (23.89)^2]}{2.3 \log_{10} \frac{90}{30}}$$

or $$K = \frac{0.09 \times 2.3 \times \log_{10} 3}{3.14 [(24.47)^2 - (23.89)^2]} = 1.121 \times 10^{-3} \text{ m/s}$$

Now $$T = Kd = 1.121 \times 10^{-3} \times 25 \text{ m}^2/\text{s}$$
$$= 0.028 \text{ m}^2/\text{s} = \mathbf{1.68 \text{ m}^2/\text{min}} \quad \textbf{Ans.}$$

To determine the drawdown in the main well, use the above eqn. as

$$Q = \frac{\pi K (h_1^2 - h_w^2)}{2.3 \log_{10} \frac{r_1}{r_w}} \qquad \begin{bmatrix}\text{using } h_1 \text{ in place of } h_2 \\ \text{and } h_w \text{ in place of } h_1\end{bmatrix}$$

or $$0.09 = \frac{1.121 \times 10^{-3} [(23.89)^2 - h_w^2]}{2.3 \log_{10} \frac{30}{0.15}}$$

or $$h_w = 12.08 \text{ m}$$

$\therefore$ $$s_w = d - h_w = 25 - 12.08 = \mathbf{12.92 \text{ m}}$$

Hence, the drawdown in the main well = **12.92 m** **Ans.**

Example 16.6. *60 cm diameter well is being pumped at a rate of 1360 litres/minute. Measurements in a nearby test well were made at the same time as follows. At a distance of 6 m from the well being pumped, the drawdown was 6 m, and at 15 m the drawdown was 1.5 m. The bottom of the well is 90 m below the ground watertable. (a) Find out the coefficient of permeability. (b) If all the observed points were on the Dupuit curve, what was the drawdown in the well during pumping? (c) What is the sp. capacity of the well? (d) What is the rate at which water can be drawn from this well?*

Solution. From Eqn. (16.13), we have Thiem's formula for unconfined aquifers:

$$Q = \frac{\pi K [h_2^2 - h_1^2]}{2.3 \log_{10} \frac{r_2}{r_1}}$$

Here $r_1 = 6$ m $\qquad r_2 = 15$ m

$s_1 = 6$ m $\qquad s_2 = 1.5$ m

$d = 90$ m

$Q = 1{,}360$ litres/minutes $= 1.36 \text{ m}^3/\text{min}$

$\therefore$ $h_1 = 90 - 6 \text{ m} = 84$ m

$h_2 = 90 - 1.5 \text{ m} = 88.5$ m

(*a*) $$1.36 = \frac{[\pi K (88.5)^2 - (84)^2]}{2.3 \log_{10} \frac{15}{6}}$$

or $$\pi K = \frac{1.36 \times 2.3 \times 0.398}{172.5 \times 4.5} = 1.603 \times 10^{-3}$$

and $$K = 0.51 \times 10^{-3} \text{ m/min} \quad \textbf{Ans.}$$

(*b*) Now $r_w = 0.3$ m

$r_2 = 15$ m

or $h_2 = 88.5$ m

$h_w = ?$

Using
$$Q = \frac{\pi K\,(h_2^2 - h_w^2)}{2.3 \log_{10} \dfrac{r_2}{r_w}}$$

we get
$$Q = \frac{\pi\, K\,[(88.5)^2 - h_w^2]}{2.3 \log_{10} \dfrac{15}{0.3}}$$

But $Q = 1.36$

and $\pi K = 1.603 \times 10^{-3}$

$$\therefore \quad 1.36 = \frac{1.603 \times 10^{-3}\,[(88.5)^2 - h_w^2]}{2.3 \log_{10} (50)}$$

or
$$\frac{1.36 \times 2.3 \times 1.69}{1.603 \times 10^{-3}} = (88.5)^2 - h_w^2$$

or $3{,}290 = 7{,}820 - h_w^2$

or $h_w^2 = 7{,}820 - 3{,}290 = 4{,}530$

or $h_w = 67.4$ m

$\therefore$ Drawdown in the pumped well

$= 90 - 67.4 = \mathbf{22.6\ m}$ **Ans.**

(*c*) *Specific capacity of the well.* It is the discharge for a unit (*i.e.* 1 m) drawdown in the pumped well.

Let us first find out the value of R

Use Dupuit's equation for unconfined aquifers, *i.e.* eq. (16.17) as :

$$Q = \frac{\pi K\,(d^2 - h_w^2)}{2.3 \log_{10} \left(\dfrac{R}{r_w}\right)}$$

$$\therefore \quad 1.36 = \frac{\pi K\,[(90)^2 - (67.4)^2]}{2.3 \log_{10} \left(\dfrac{R}{0.3}\right)}$$

or
$$1.36 = \frac{1.603 \times 10^{-3} \times 157.4 \times 22.6}{2.3 \log_{10} \left(\dfrac{R}{0.3}\right)}$$

or
$$\log_{10} \left(\frac{R}{0.3}\right) = \frac{1.603 \times 157.4 \times 22.6}{2.3 \times 1{,}360} = 1.824$$

Taking antilog, we get

$$\frac{R}{0.3} = 66.7$$

or $R = 20.01$

Say $\mathbf{R = 20\ m.}$

Now, specific capacity

$$= Q_{unitdrawdown} = \frac{\pi K\,[(90)^2 - (89)^2]}{2.3\,\log_{10}\left(\frac{20}{0.3}\right)}$$

$$= \frac{1.603 \times 10^{-3} \times 179 \times 1}{2.3 \times 1.824} = 68.3 \times 10^{-3}\ \text{m}^3/\text{min}.$$

Hence, the specific capacity

$$= \mathbf{68.3\ litre/minute} \quad \textbf{Ans.}$$

(*d*) *Maximum discharge* will occur when

$$h_w = 0$$

$$\therefore \quad Q_{max} = \frac{\pi K\,[(90)^2 - (0)^2]}{2.3\,\log_{10}\left(\frac{20}{0.3}\right)}$$

$$= \frac{1.603 \times 10^{-3} \times 8100}{2.3 \times 1.824} = 3.09\ \text{m}^3/\text{min}$$

Hence, the maximum rate of discharge

$$= \mathbf{3{,}090\ litres/minute} \quad \textbf{Ans.}$$

16.18. Non-Equilibrium Formula for Aquifers (Unsteady Radial Flows)

The main drawback of the equilibrium formulas given by Thiem and Dupuit, was the problem to attain equilibrium conditions, since it is not an easy job to do so. The pumping has to be continued at a uniform rate for a very long time so as to achieve steady flow conditions. Further research was, therefore, carried out to simplify the process and to calculate the yield in some other way.

A major advancement in this field was made by Thies when he developed his non-equilibrium formula by introducing the time factor t. This formula was derived by Thies in 1935, by comparing the flow of water with the flow of heat by conduction. The formula evolved by him involved the solution of a complicated integral, the solution of which requires the use of various tables and graphs.

Later, Jacob derived the same formula by directly using the hydraulic concept. He also slightly modified the Thies formula by making a slight approximation, so as to simplify it. The final formula which was arrived at the end, by Jacob, is given as :

$$s = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt}{r^2 \, . \, A} - 0.5772\right] \qquad ...(16.34)$$

where s = Drawdown in the observation well after a time t, from the start of pumping in the main well

T = Coefficient of transmissibility of the aquifer

Q = Constant discharge pumped out from the main pumped well.

A = Coefficient of storage of the measured drawdown.

r = Radial distance of the observation well from the main pumped well.

The derivation of this formula is given below :

Derivation. Let us consider a generalised free-body diagram of the flow system in the vicinity of a discharging well, as shown in Fig. 16.24.

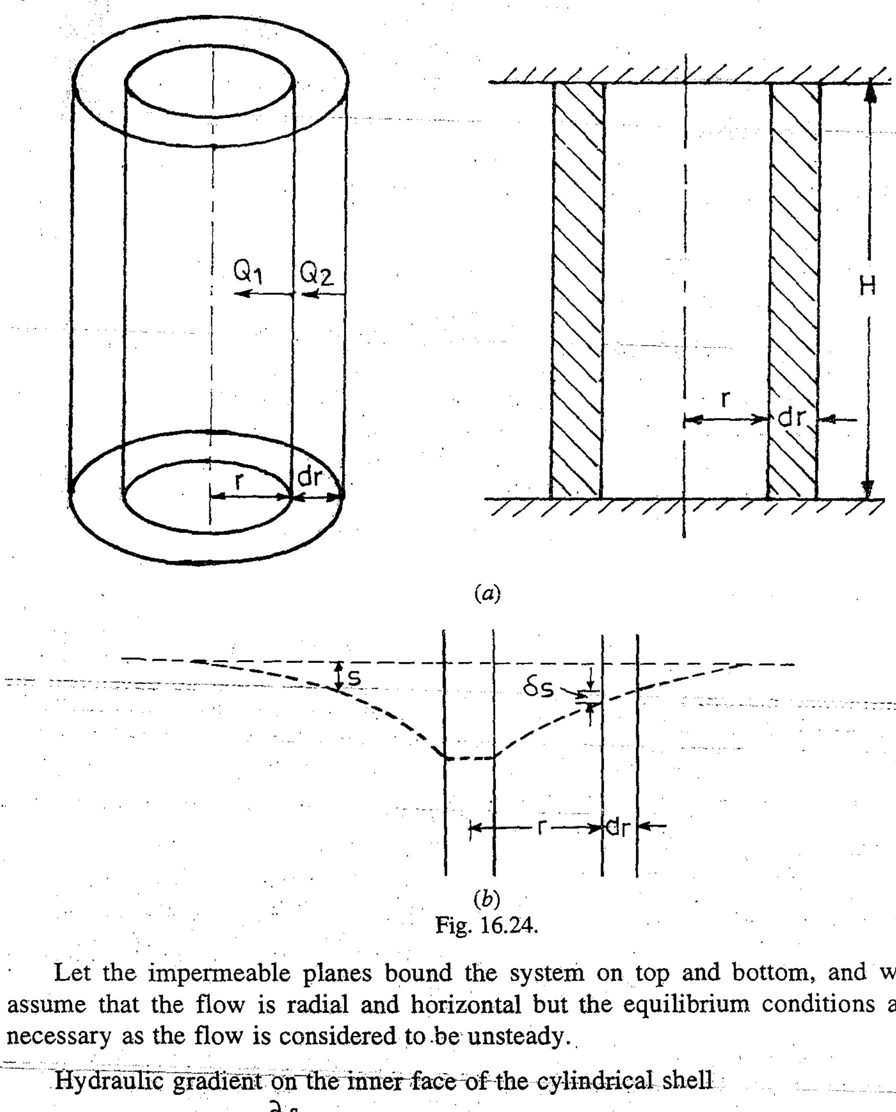


Fig. 16.24.

Let the impermeable planes bound the system on top and bottom, and we also assume that the flow is radial and horizontal but the equilibrium conditions are not necessary as the flow is considered to be unsteady.

Hydraulic gradient on the inner face of the cylindrical shell

$$= I_1 = \frac{\partial s}{\partial r}$$

It should be negative because as s is increasing, r is decreasing

$$\therefore \qquad I_1 = -\frac{\partial s}{\partial r}$$

Hydraulic gradient at the outer face $= I_2$

$$= \left(I_1 + \frac{\partial I_1}{\partial r} \cdot dr\right) \qquad \text{(with –ve sign)}$$

or $$I_2 = -\left(\frac{\partial s}{\partial r} + \frac{\partial_s^2}{\partial_r^2} \cdot dr\right)$$

Now Q_1 = Discharge from the inner face

$$= K I_1 A$$

$$= K\left(-\frac{\partial s}{\partial r}\right) \cdot 2\pi r \cdot H \qquad \left[\begin{array}{l} \because \ 2\pi r = \text{Surface area} \\ H = \text{Full depth of the aquifer} \end{array}\right]$$

$\therefore$ $$Q_1 = -2\pi KH.r.\frac{\partial s}{\partial r} = -2\pi T.r.\frac{\partial s}{\partial r} \qquad [\because \ Kd = T]$$

or $$Q_1 = -2\pi r T\frac{\partial s}{\partial r} \qquad ...(i)$$

Similarly, $Q_2 = K\, I_2\, A = K\, I_2\, 2\pi\,(r + dr)\, H = T\,.\, I_2\, 2\pi\,(r + dr)$

$$= -T\left(\frac{\partial s}{\partial r} + \frac{\partial_s^2}{\partial_r^2}\, dr\right) 2\pi\,(r + dr) \qquad ...(ii)$$

subtracting (*ii*) from (*i*), we get

$$Q_1 - Q_2 = -2\pi r T \cdot \frac{\partial s}{\partial r} + T\,.\left[\frac{\partial s}{\partial r} + \frac{\partial_s^2}{\partial_r^2} \cdot dr\right] 2\pi\,(r + dr)$$

$$= -2\pi r T.\frac{\partial s}{\partial r} + 2\pi r T\frac{\partial s}{\partial r} + 2\pi r.T\frac{\partial_s^2}{\partial_r^2}\, dr. + 2\pi.T.dr\frac{\partial s}{\partial r} + 2\pi T.d_r^2\,\frac{\partial_s^2}{\partial_r^2}$$

$$= 2\pi r T.\frac{\partial_s^2}{\partial_r^2}\, dr + 2\pi T dr \cdot \frac{\partial s}{\partial r} + 2\pi T.d_r^2 \cdot \frac{\partial_s^2}{\partial_r^2}$$

Neglecting $2\pi.T.\frac{\partial_s^2}{\partial_r^2}\, d_r^2$, as the differentials of the higher order are very small, we get

$$Q_1 - Q_2 = 2\pi r T.\frac{\partial_s^2}{\partial_r^2} \cdot dr + 2\pi T.dr.\frac{\partial s}{\partial_r} = 2\pi T.\left[r\,.\frac{\partial_s^2}{\partial_r^2}\, dr + \frac{\partial s}{\partial r}.dr\right]$$

$$= 2\pi r T.dr.\left[\frac{\partial_s^2}{\partial_r^2} + \frac{1}{r}\frac{\partial s}{\partial r}\right] \qquad ...(iii)$$

$Q_1 - Q_2$ (*i.e.* the difference in the rate of flow through the inner and outer faces of the cylindrical shell) must be drawn from the storage within the shell (Principle of Conservation of Mass).

Therefore, $Q_1 - Q_2$ = Rate of change of volume of this storage of the thin cylindrical shell of thickness *dr*.

$$= \frac{(2\pi r.dr)\ A.\partial s}{\partial t}$$

[Because A = Coeff. of storage, *i.e.* the volume of water drained through a unit-cross-section and full depth, when pressure falls by unity.

In this case,

pressure fall is equal to ∂s; and the total volume drained = (surface area drained) $\times A \times$ fall of pressure

or Change of volume in a time ∂t

$$= (2\pi r.dr.)\, A.\partial s$$

or Rate of change of volume

$$= \frac{2\pi r.dr.A.\partial s}{\partial t}\Bigg]$$

Equating these two values of $Q_1 - Q_2$ given by eqn. (*iii*) and (*iv*), we get

$$2\pi rT.dr.\left[\frac{\partial_s^2}{\partial_r^2} + \frac{1}{r}\cdot\frac{\partial s}{\partial r}\right] = \left[2\pi r.dr.A.\frac{\partial s}{\partial r}\right]$$

or
$$\frac{\partial_s^{\,2}}{\partial_r^{\,2}} + \frac{1}{r}\cdot\frac{\partial s}{\partial r} = \frac{A}{T}\cdot\frac{\partial s}{\partial t}$$

This is the differential equation governing this unsteady flow and can be solved for the boundary conditions of our given problem *i.e.* $s = 0$ before pumping begins and s approaches zero as r approaches infinity after pumping begins, *i.e.* $s = 0$ at $t = 0$ and $s \to 0$ as $r \to \infty$.

The solution of this differential equation for a constant pumping rate Q is given by the expression.

$$s = \frac{Q}{4\pi T}\int_{x=\frac{r^2A}{4Tt}}^{x=\infty} \frac{e^{-x}}{x}\cdot dx$$

The above solution is obtained after complicated mathematical calculations, which are beyond the scope of this book. The integral involved above is called **well-function.**

Now we can again write

$$s = \frac{Q}{4\pi T}\int_{x=\frac{r^2A}{4Tt}}^{x=\infty} \frac{e^{-x}}{x}\cdot dx = \frac{Q}{4\pi T}\left[\int\frac{e^{-x}}{x}\cdot dx\right]_{\text{at } x=\frac{r^2A}{4Tt}} \quad \because \left|\int\frac{e^{-x}}{x}\,dx\right|_{\text{at } x=\infty} = 0$$

But $\int\frac{e^{-x}}{x}\,dx$ is given by the following series:

$$\int\frac{e^{-x}}{x}\,dx = \left[-0.5772 - \log_e x + x - \frac{x^2}{2\lfloor 2} + \frac{x^3}{3\lfloor 3}\cdots\right]$$

Hence, the drawdown,s, after a given time t and at a distance r from the centre of the pumped well is given by the equation

$$s = \frac{Q}{4\pi T}\left[-0.5772 - \log_e x + x - \frac{x^2}{2\lfloor 2} + \frac{x^3}{3\lfloor 3} + \dots\right] \quad \dots(16.34a)$$

$$\text{where} \quad x = \frac{r^2 \cdot A}{4T \cdot t}$$

This equation, as derived by Thies, involved complicated mathematical calculations and was later simplified by Jacob as follows :

Modified Non-Equilibrium Formula. It is evident that x decreases as t increases, and Jacob found that the terms beyond $\log_e x$ in equation (16.34a) are not appreciable and can be neglected for larger values of t. Using this approximation, equation (16.34a) can be reduced to

$$s = \frac{Q}{4\pi T}\left[-0.5772 - \log_e x\right] \text{ where } x = \frac{r^2 A}{4Tt}$$

$$= \frac{Q}{4\pi T}\left[\log_e \frac{1}{x} - 0.5772\right] \text{ where } x = \frac{r^2 A}{4Tt}$$

or
$$s = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt}{r^2 A} - 0.5772\right] \quad \dots(16.34)$$

This is the important final equation and must be remembered. If, in an observation well at a distance r from the main pumped well, the drawdowns are respectively s_1 and s_2 at times t_1 and t_2 after the pumping was started in the main well, then

$$s_1 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_1}{r^2 . A} - 0.5772\right] \quad \dots(v)$$

and
$$s_2 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r^2 A} - 0.5772\right] \quad \dots(vi)$$

Subtracting (v) from (vi), we get

$$s_2 - s_1 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r^2 A} - \log_e \frac{4Tt_1}{r^2 A}\right] = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r^2 A} \cdot \frac{r^2 A}{4Tt_1}\right]$$

$$= \frac{Q}{4\pi T}\log_e \frac{t_2}{t_1} = \frac{2.3Q}{4\pi T}\log_{10}\frac{t_2}{t_1}$$

or
$$s_2 - s_1 = \frac{2.3Q}{4\pi T}\log_{10}\frac{t_2}{t_1} \quad \dots(16.35)$$

The above formula holds good for larger values of t. It is evident from the above equation that if the drawdowns are noted for various values of t on the given observation well at a distance r from the main pumped well, and a graph is plotted between log t and s, it will be a straight line, with the limitation that the initial values (*i.e.* when t is small) may not exactly lie on a straight line. From this straight line, two values, of t_1 and t_2 and corresponding values of s_1 and s_2 can be read out, and knowing the value of T, Q can be worked out easily.

The straight line can be produced so as to cut the X-axis at point P as shown in Fig. 16.25. Now let the value of t at P be represented by t_0.

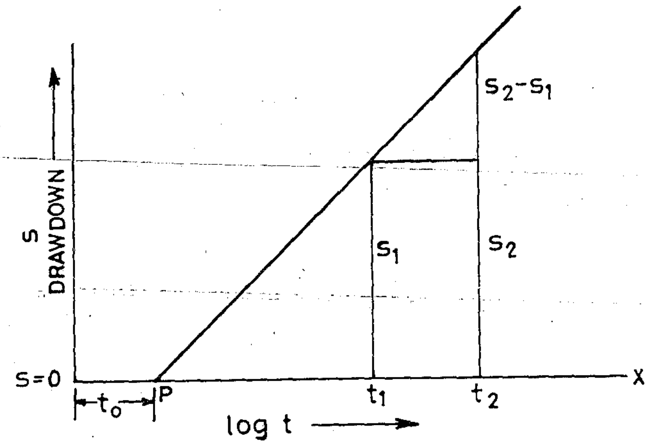


Fig. 16.25.

Now, we know that

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

at $\quad s = 0, t = t_0$

$$\therefore \quad 0 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_0}{r^2 A} - 0.5772\right]$$

Since $\quad \frac{Q}{4\pi T} \neq 0$

$$\log_e \frac{4Tt_0}{r^2 A} - 0.5772 = 0$$

or $\quad \log_e \frac{4Tt_0}{r^2 A} = 0.5772$

or $\quad \frac{4Tt_0}{r^2 A} = e^{0.5772}$

or $\quad A = \frac{4Tt_0}{r^2\, e^{0.5772}} = \frac{2.25Tt_0}{r^2}$

Hence, $\quad \boxed{A = \frac{2.25Tt_0}{r^2}} \quad$ ...(16.36)

The coefficient of storage A, can be worked out by this equation.

Example 16.7. *In an artesian aquifer, the drawdown is 1.2 metres at a radial distance of 10 metres from a well after two hours of pumping. On the basis of Thies' non-equilibrium equation, determine the pumping time for the same drawdown (i.e. 1.2 m) at a radial distance of 30 metres from the well.*

Solution. The Thies' non-equilibrium equation (16.34) is

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

In the given question, the drawdown is the same in both the observation wells, therefore,

well (1)	well (2)
$r_1 = 10$ m	$r_2 = 30$ m
$t_1 = 2$ hr.	$t_2 = ?$

Now $$s_1 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_1}{r_1^2 . A} - 0.5772\right]$$

$$s_2 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r_2^2 A} - 0.5772\right]$$

But $s_1 = s_2$

$$\therefore \quad \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_1}{r_1^2 A} - 0.5772\right] = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r_2^2 A} - 0.5772\right]$$

or $$\log_e \frac{4Tt_1}{r_1^2 A} = \log_e \frac{4Tt_2}{r_2^2 A}$$

or $$\frac{4Tt_1}{r_1^2 A} = \frac{4Tt_2}{r_2^2 A}$$

or $$\frac{t_1}{r_1^2} = \frac{t_2}{r_2^2}$$

Putting the respective values, we get

or $$\frac{2\text{ hr}}{(10)^2} = \frac{t_2}{(30)^2}$$

or $$t_2 = \frac{(30)^2}{(10)^2} \times 2\text{ hr} = 9 \times 2\text{ hr} = 18\text{ hr}$$

$\mathbf{t_2 = 18\ hr}$ **Ans.**

Example 16.8. *A well is located in a 30 m thick confined aquifer of permeability 35 m/day and storage coefficient of 0.004. If the well is pumped at the rate of 1500 litres per minute, calculate the drawdown at a distance of 40 m from the well after 20 hours of pumping.* *(U.P.S.C. Civil Services, 1991)*

Solution. Using Jacob's eqn. (16.34), we have

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

where s = drawdown = ?

H = depth of aquifer = 30 m

K = 35 m/day

A = storage coeff = 0.004

$Q = 1500\ l/\text{min} = \frac{1.5}{60}\ \text{m}^3/\text{sec} = \mathbf{0.025\ m^3/s}$

$r = 40$ m

$t = 20\text{ hr} = 20 \times 3600\text{ secs} = 72000\text{ secs}$

$$T = K \cdot H = 35 \times 30 \text{ m}^2/\text{day}$$
$$= \frac{1050}{60 \times 60 \times 24} \text{ m}^2/\text{sec} = 0.012153 \text{ m}^2/\text{s}$$

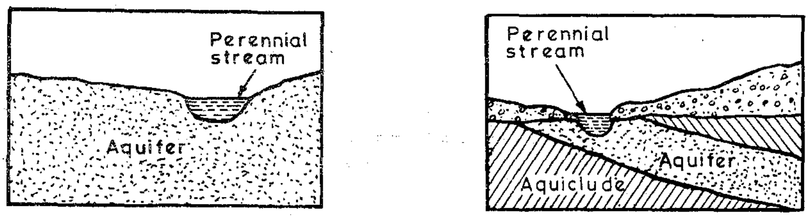
Substituting values, we get

$$s = \frac{0.025}{4 \times 3.14 \times 0.012153}\left[\log_e \frac{4 \times 0.012153 \times 72000}{(40)^2 \times 0.004} - 0.5772\right]$$
$$= 0.163\,[6.3042 - 0.5772] = 0.163 \times 5.724 = \mathbf{0.94\ m.} \quad \textbf{Ans.}$$

16.19. The Method of Images - Its Use in Ground Water Analysis for Areally Limited Aquifers.

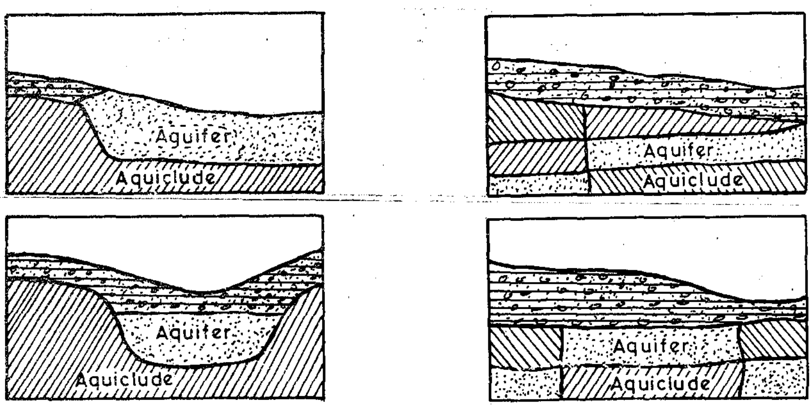
The equilibrium as well as the non-equilibrium formulas, derived in the previous articles, for analysing ground water pumping, have been derived on the assumption that the aquifer being pumped is of *infinite areal* extent. This assumption is generally not fulfilled in the actual field except in the cases of a few sedimentary rock aquifers. *In actual practice, the continuity of aquifers is usually broken by the existence of formation boundaries, folds & faults, or the dissection by surface streams.* The existence of such geological structures limit the areal extent of aquifers to a few kilometers or more in consolidated strata. In unconsolidated materials and particularly in the glaciated areas, the infinite areal extent is seldom met with. Consequently, it becomes necessary to consider the effects caused by the existence of such geological structures on the movement of ground water, before the foregoing formulas (derived in the previous articles) can be applied to the problems of flow in *really limited aquifers.*

Thus, when an aquifer is intersected by a perennial stream or some other water body with sufficient flow, it will prevent development of the cone of depression beyond that surface



(*i*) Unconsolidated strata (*ii*) Consolidated strata

(*a*) Positive boundary caused by the intersection of an aquifer by a perennial stream.



(*i*) Unconsolidated strata (*ii*) Consolidated strata

(*b*) Negative boundary caused by the intersection of an aquifer by impervious strata.

Fig. 16.26. Positive and Negative geological boundaries in aerially llmited aquifers.

source. The intersection of the aquifer with the stream will then become a geological boundary, and is called a **positive boundary**, since it will help in preventing drawdowns over large areas. On the other hand, when some impervious aquiclude formation does intersect an aquifer, it will stop the flow from the aquifer area bounded by it, and hence such a contact is known as a **negative boundary**. Examples of positive & negative contacts are reflected in Fig. 16.26 (*a*) and (*b*), respectively.

Strictly speaking, most geologic boundaries except for some faulted structures, do not occur as abrupt straight line demarcations, yet they can be assumed to be so without involving much error, since the area covered by a well being pumped, is relatively small.

By considering the geologic boundary as a straight line demarcation, it has further been possible to solve the flow pattern by substitution of a hypothetical system that satisfies the limits of the real system. The **method of images**, devised byLlord Kelvin in his work on electrostatic theory, is a convenient tool for the solution of boundary problems.

Let us for example, consider an idealized section of an aquifer which is inter-sected by a surface stream in Fig. 16.27. To act as an effective positive boundary, the flow in the surface stream must be equal or exceed the withdrawal of the well, because any flow

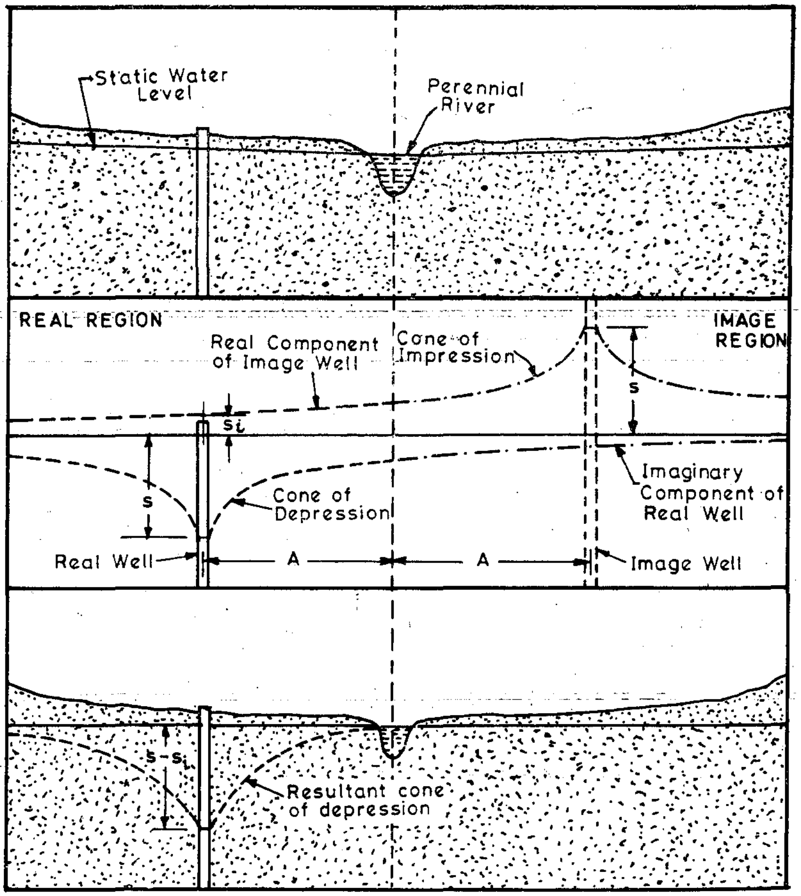


Fig. 16.27. Idealized section of an aquifer which is intersected by a perennial stream alongwith a hypothetical well system for the solution of this type of flow problems.

below the well yield would result in drying up of the stream, eliminating the said boundary. The stream is now assumed to be of infinitesimal width, *i.e.* ~~equivalent to~~ a line source. At this line source, there will be zero drawdown, since the river is perennial. Any system that can satisfy this boundary limit will offer the solution to the real problem.

As shown by the central Fig. in Fig block 16.27, the real and bounded aquifer has been replaced by an *imaginary aquifer of infinite areal extent and an imaginary recharging well, called* **image well**, placed on the opposite side of and equidistant from the boundary. As illustrated, the imaginary recharge well returns water to the aquifer at the same rate as it is withdrawn by the real discharge well. The image well, consequently produces a build up of water level at the boundary that is exactly equal to, and cancels the drawdown of the real well. This system results in obtaining zero drawdown at the boundary, which satisfies the limit of the real problem.

The real components of the cone of depression of the real well and the cone of impression of the image well are shown as solid lines in the region of real values. to obtain the resultant cone of depression; *i.e.* to evaluate the drawdown at any point in the real region, we shall have to add algebraically the real components of the two depression cones. *The resultant cone of depression is thus found to be steepened on the riverside of the well and flattened on the landward side.*

Note. The actual observed drawdown in a well bounded by such a river boundary on pumping, will hence be found to be less than what is computed by the Dupuit's or Thiem's formulas i.e. s. The reduction being equal to the drawdown caused by the image well at the location of the real ~~well at~~ distance 2A apart ; say s_i. Hence, actual drawdown in the real well will be equal to $s - s_i$ inplace of s. It use has been exhibited in solving numerical example 16.9.

Similarly, the utilisation of an image well in solving the problem of an aquifer bounded by an impervious strata is reflected in Fig. 16.28.

Example 16.9. *A 0.5m diameter well fully penetrates a 30m thick confined aquifer of hydraulic conductivity 20m/day. Due to continuous pumping, if a drawdown of 1.0 m is registered in the well, what will be the rate of pumping when the well is located at a distance of 50 m from a perennial stream.* (*Civil Services, 1999*)

Solution. In this case, the aquifer is not of infinite areal extent, but is inter-sected by a perennial stream at a distance of 50 m from the well, which will reduce the drawdown in the well than what is computed by Dupuit's or Thiem's formulas.

If s represents the drawdown computed by Dupuit's or Thiem's formulas, and s_i represents the drawdown caused at the location of the real well by the image well (at a distance of $2A = 2 \times 50$ m $= 100$ m), then we can write;

The actual drawdown in the well of aerially limited aquifer

$$= s - s_i = 1.0 \text{ m (given)} \quad ...(i)$$

Thiem's formula is now applied to the image well being pumped at a discharge of Q, to find out its impact (i.e. drawdown, s_i) at a distance $2A$, by using Eq. 16.14, as :

$$Q \approx \frac{2\pi.K.d.\,(s_1 - s_2)}{2.3 \log (r_2/r_1)} \quad (i.e. \text{ Eq. } 16.14)$$

where, $s_1 = s$ (image well)

$s_2 = s_i$ (impact of image well at distance 2A apart)

$r_1 = r = 0.25$ m (given), since the real well as well as its image well are of 0.5 m dia.

$r_2 = 100$ m (*i.e.* $2A$).

$d = 30$ m (given) ; and $K = 20$ m/day (given)

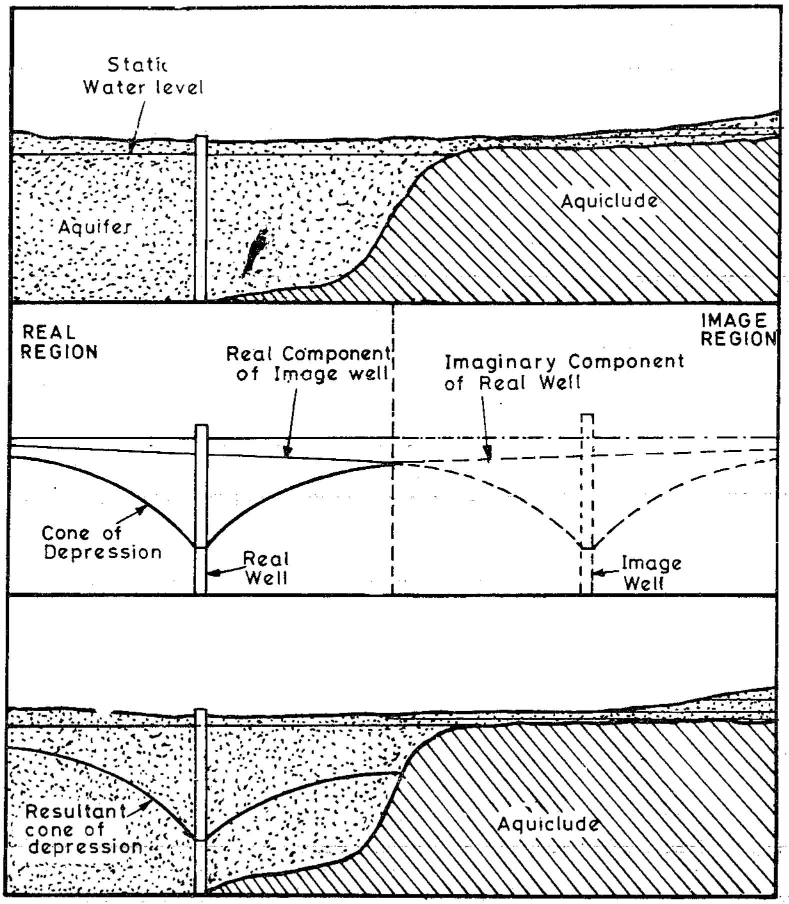


Fig. 16.28. Idealized section of an aquifer bounded by an 'impervious formation', shown along with a hypothetical well system for the solution of this type of flow problems.

Substituting the values, we get

$$Q = \frac{2\pi \times 20 \times 30\,(s - s_i)}{2.3 \log_{10} \frac{100}{0.25}} \text{ m}^3/\text{day}$$

or $$Q = 629.92\,(s - s_i) \qquad \text{...}(ii)$$

But from (i)

$$s - s_i = 1 \text{ m}$$

$\therefore$ $\mathbf{Q = 629.92\,m^3/day = 6.30\,lps.}$ **Ans.**

16.20. Recharging of Underground Storage

Just as the artificial surface reservoirs are constructed by building dams in order to store the surplus surface waters; in the same manner, artificial underground reservoirs are now-a-days developed by *artificial recharge*, for storing water underground. The development of such a reservoir may be advantageous as compared to the development of a dam reservoir, because of the following reasons;

(i) Much purer water can be obtained from an underground reservoir source.

(*ii*) No space is required for building such a reservoir.

(*iii*) The cost of building such a reservoir by recharging the aquifers may be considerably less than the cost of the surface reservoirs. Moreover, in an underground reservoir, the aquifer in which the water is stored shall itself act as a distribution system for carrying the water from one place to another, and as such, the necessity of constructing pipe lines or canals (as is required in a surface reservoir) is completely eliminated. On the other hand, extra cost is involved for pumping out the ground water. However, the reduction in the first cost due to elimination of a huge distribution network may sometimes be so high as to offset the subsequent cost of pumping, which is required in such underground reservoirs.

(*iv*) The water lost in evaporation from an underground reservoir is much less than the water lost from a surface reservoir.

(*v*) The raising of the watertable by artificial recharge may help in building pressure barriers to prevent sea water intrusion in the coastal areas.

Artificial recharge of ground water is, therefore, preferred and encouraged in modern days, so as to augment the natural available underground yield, for management of water supply systems. Artificial recharging technique is under intensive research, and is being increasingly used in France, Israel, Federal Republic of Germany, U.K. etc.

16.20.1. Methods of Recharging. The three methods which are generally adopted for ground water recharging are discussed below :

(1) **Spreading method.** This method consists in *spreading* the water over the surfaces of permeable open land and pits, from where it directly infiltrates to rather shallow aquifers. In this method, the water is temporarily stored in shallow ditches, or is spread over an open area by constructing low earth dykes (called *percolation bunds*). The stored water, slowly and steadily, percolates downward, so as to join the nearby aquifers. The recharging rate depends upon the permeability of the spread area and on the depth of water stored, and is generally less, say of the order of 1.5 m/day, though rates as high as 22.5 m/day have been possible. Certain chemicals, when added to the soil, may help in increasing the recharging rate and are under research.

(2) **Recharge-well method.** This method consists in injecting the water into bore holes, called recharge wells. In this method of recharge, the water is therefore, fed into recharge wells by gravity or may be pumped under pressure to increase the recharge rate, if surface conditions permit. The recharge wells used for this purpose, are just like ordinary production wells. Infact the ordinary production wells are many a times directly used for recharge during the off season, when the water is not required for use. *Recharge-well method* is certainly preferred when the *spreading method* cannot yield appreciable recharge, because of low permeable areas. High recharge rates can be obtained with this method. Moreover, this method may help in injecting water into the aquifers, and also where it is most needed. This method is widely practised in Israel.

The water to be used in the recharge well should, however, be purer than that is required in the first method.. This water should be free of suspended matter, so as to avoid clogging of the well screens. Since the recharge well injects the water directly into the aquifer, the water used for recharging must also be free from bacteria. Hence, if the treated sewage is used for recharge, it should generally be bacteriologically pure.

(3) **Induced Infiltration method.** The third method which is sometimes used for recharge is that of the induced infiltration, which is accomplished by increasing the watertable gradient from a source of recharge. In this method, Renney type* wells are

* Ranney well is explained in the next article.

constructed near the river banks. The percolating water is collected in the well through radial collectors and is then discharged as recharge into a lower level aquifer 'B' for storage, as shown in Fig. 16.29.

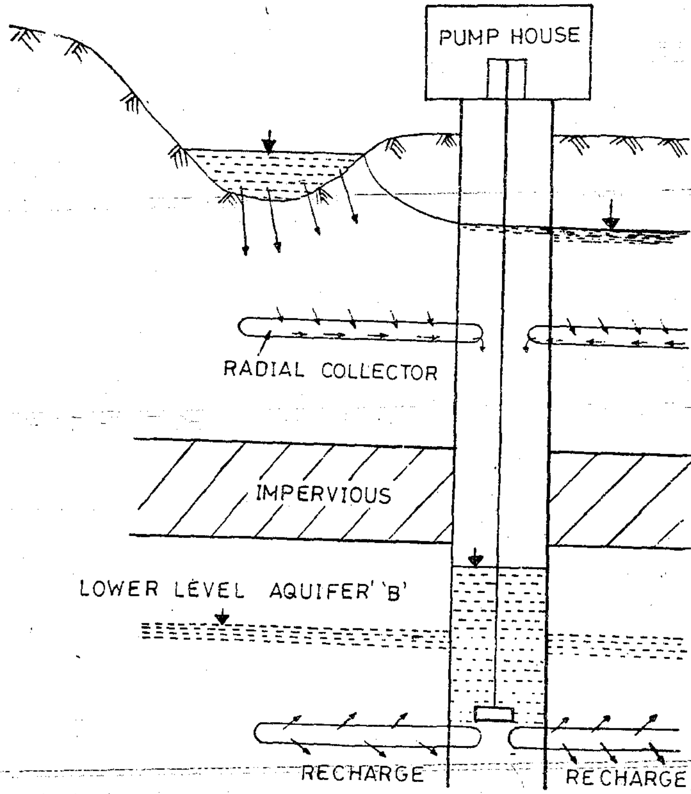


Fig. 16.29. Induced infiltration method of recharge.

VARIOUS FORMS OF UNDERGROUND SOURCES AND THEIR EXPLOITATION

The underground water is generally available in the following forms:

(1) Infiltration galleries; (2) Infiltration wells;
(3) Springs; and (4) Wells including tubewells.

These forms are discussed below :

16.21. Infiltration Galleries

Infiltration galleries are horizontal or nearly horizontal tunnels constructed at shallow depths (3 to 5 metres) along the bank of the river through the water bearing strata, as shown in Fig. 16.30(*a*). They are sometimes called *horizontal wells.*

These galleries are generally constructed of masonry walls with roof slabs, and derive their water from the aquifer by various porous drain pipes. These pipes are generally covered with gravel, so as to prevent the entry of the fine sand particles into the pipe. These tunnels or galleries are generally laid at a slope, and the water collected in them is taken to a sump well, from where it is pumped, treated and distributed to the consumers. These infiltration galleries are quite useful when water is available in sufficient quantity just below the ground level or so.

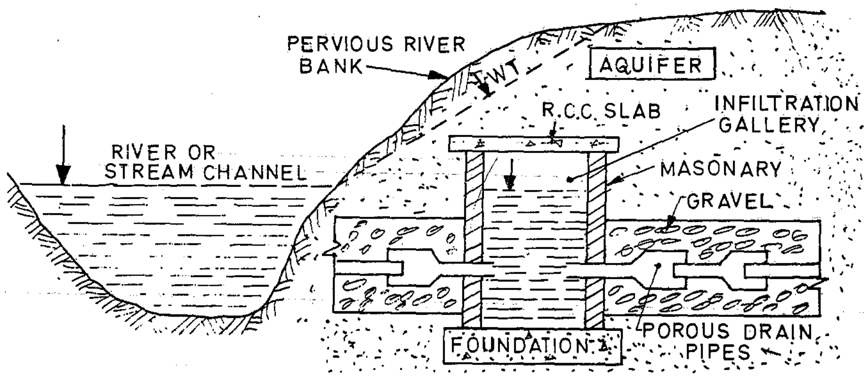


Fig. 16.30 (*a*). Section of an infiltration gallery.

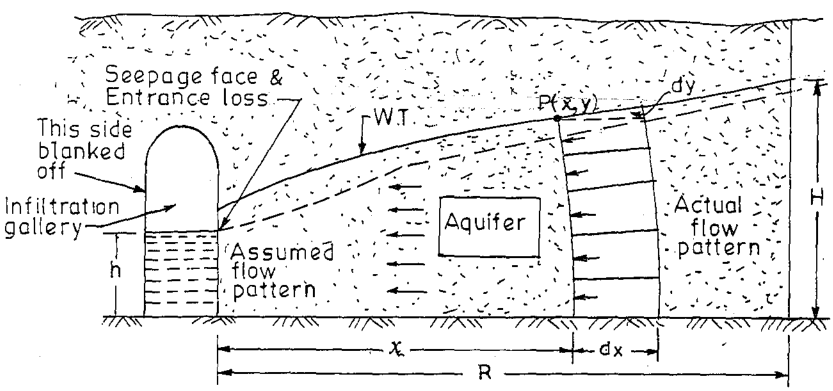


Fig. 16.30 (*b*). Flow pattern in an infiltrating gallery.

The flow pattern into an infiltration gallery under pumping equilibrium conditions is shown in Fig. 16.30 (*b*), as the drawdown curve follows the Dupuit's curve. The discharge, from the gallery can be computed by using Darcy's law, as

$$Q = K\,i\,A$$

Now, consider a point P on the drawdown curve having coordinates (x, y). On considering a small element of width dx, the head gradient will be dy/dx, and the area of soil through which flow will occur will be $y \,.\, L$, where L is the length of the gallery ($\perp$ to paper).

$\therefore$ Discharge $Q = \int K \,.\left(\dfrac{dy}{dx}\right).\,(y.L).$

or $$Q \times \int_{x=0}^{x=R} dx = K.L. \int_{y=h}^{y=H} y\,dy$$

where x varies between O to R, where R is the radius of influence.

y varies between h and H ; where

h = depth of water in the gallery on pumping at equilibrium

H = Static head *i.e.* height of initial water level above the bottom of gallery.

$$\therefore \qquad Q \left| x \right|_{x=0}^{x=R} = K.L \left| \frac{y^2}{2} \right|_{y=h}^{y=H}$$

$$\therefore \qquad Q.R = K.L \left(\frac{H^2 - h^2}{2} \right)$$

or

$$Q = K.L. \left(\frac{H^2 - h^2}{2R} \right) \qquad \text{...(16.37)}$$

where Q = Discharge

K = Permeability coefficient of the aquifer

L = length of the gallery ($\perp$ to Fig.)

H = Initial depth of water level

h = Final depth of water level

R = Radius of influence

Sometimes, horizontal perforated pipes are laid in place of rectangular tunnels, and their perforations are covered with gravel so as to prevent sand entry. These pipes may be called infiltration pipes, and are useful when the available ground water is small in quantity.

16.22. Infiltration Wells

Infiltration wells are the shallow wells constructed in series along the banks of the rivers, in order to collect the river water seeping through their bottoms, as shown in Fig. 16.31.

These wells are generally constructed of brick masonry with open joints. They are generally covered at the top and kept open at the bottom, as shown in Fig 16.32. For inspection purposes, manholes are provided in the top cover.

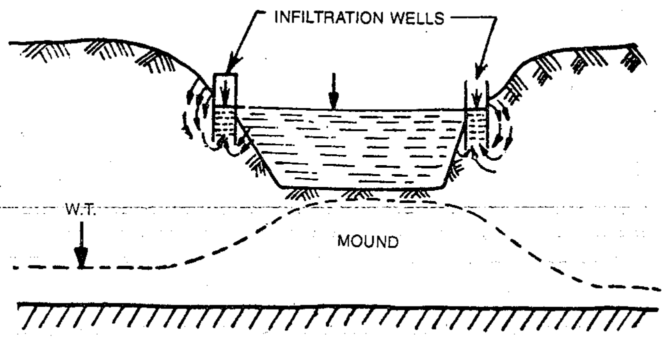


Fig. 16.31. Location of infiltration wells.

The various infiltration wells are connected by porous pipes to a *sump well,* called **jack well**, as shown in Fig. 16.33. The water reaching the jack well from different infiltration wells is lifted, treated and distributed to the consumers.

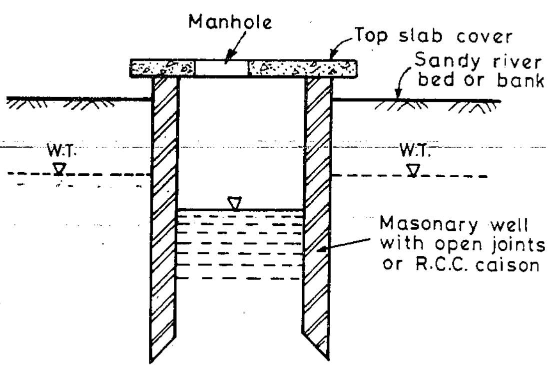


Fig. 16.32. Section of an infiltration well.

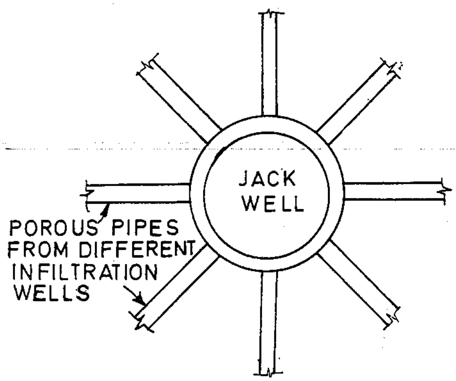


Fig. 16.33. Plan of a jack well drawing water from several infiltration wells.

Ranney Wells. A new technique that has recently started is to construct a vertical well of 3 to 6 m in diameter with horizontal radial collectors, and is known as a **radial well.** Such a well is sunk up to the required level and *plugged at the bottom.* Horizontal perforated steel pipes are then driven just at the level of the aquifer in the well by powerful hydraulic jacks. The length of these pipes or *radial collectors* may be of the order of 60 to 80 m or so. About 10 collectors can be installed at one level. Some other set or sets of such radial collectors can be installed at other levels, if possible, so as to increase the yield. The inner end of each collector pipe is fitted with a sluice valve which can be operated from the pump house above. The inflow of water into the well is thus controlled. The water from the well obtained by this method is generally clean, fresh and free from bacterial contamination. These wells are, therefore, very useful for drawing water from polluted streams. This type of well construction is very common in *France*, and is sometimes referred to as *French system of tapping underground water.*

A patented type of a radial well is known as a **ranney well**, or a ranney collector. It consists of an R.C.C. caisson of 4.3 m (13′) in diameter, 0.45 m (18") thick, which is sunk into the ground up to the required level, and from which radial collectors are projected, as explained above.

16.23. Springs

The natural outflow of ground water at the Earth's surface is said to form a *spring.* A pervious layer sandwiched between two impervious layers, give rise to a natural spring. A spring indicates the outcropping of the watertable.

The springs are generally capable of supplying very small amounts of water, and are, therefore, generally not regarded as sources of water supplies. However, good developed springs may sometimes be used as water supply sources for small towns, especially in hilly areas. Certain springs, sometimes discharge hot water due to the presence of sulphur in them. These **hot springs** (such as the one in Sohana in Haryana; a group on the bed and bank of Sutlej river at Tattapani near Simla ; and also another at Manikaran near Manali on Parvati river in H.P. state) usually emit sulphur mixed water (warm to boiling), and hence cannot be used for water supplies, though sometimes useful for taking dips for the cure of certain skin ailments.

16.23.1. Formation and Types of Springs. Springs are usually formed under three general conditions of geological formations, as explained below :

(*a*) **Gravity springs.** When the ground watertable rises high and the water overflows through the sides of a natural valley or a depression (as shown in Fig. 16.34), the spring formed is known as a gravity spring. The flow from such a spring is variable with the rise or fall of watertable.

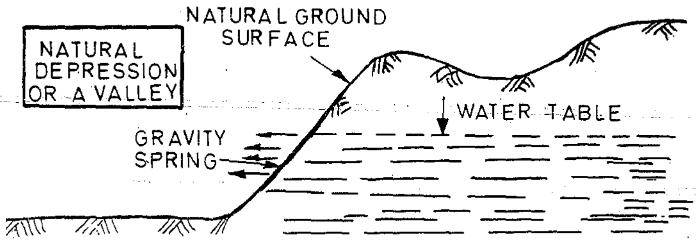


Fig. 16.34. Gravity spring.

(*b*) **Surface springs**. Sometimes, an impervious obstruction or stratum, supporting the underground storage, becomes inclined (such as shown in Fig. 16.35), causing the watertable

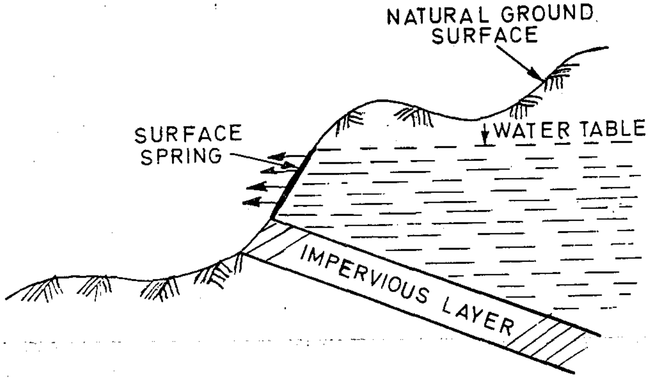


Fig. 16.35. Surface spring.

to go up and get exposed to the ground surface. This type of a spring is known as a *surface spring*. The quantity of water available from such springs is quite uncertain.

(*c*) **Artesian springs**. When the above storage is under pressure (*i.e.* the water is flowing through some confined aquifer), such as shown in Fig. 16.36, the spring formed is known as *artesian spring*. This type of springs are able to provide almost uniform quantity of water. Since the water oozes out under pressure, they are able to provide higher yields and may be thought of as the possible sources of water supply.

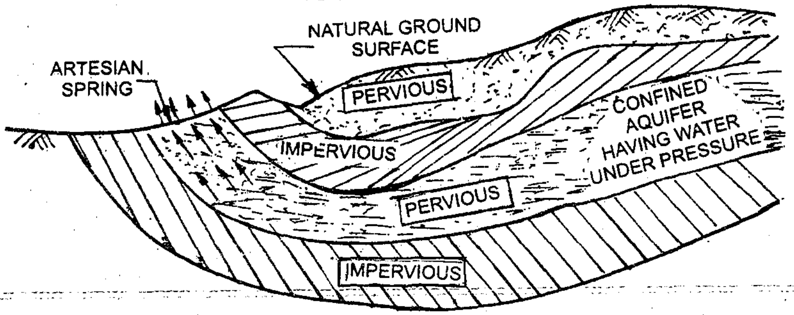


Fig. 16.36. Artesian spring.

When a spring issues out of the ground, a lake or a pond gets formed. Pucca masonry walls are, therefore, constructed on four sides of this tank or pond which is covered at the top. Proper cleaning of the area and arrangements for excluding the surface waters from entering the spring should be properly made, so as to avoid contamination of spring water. The water collected in the tank or the pond can be carried through pipes for meeting the water demand.

WELLS

A water well is a hole usually vertical, excavated in the Earth for bringing ground water to the surface. The wells may be classified into two types :

(1) Open wells ; and (2) Tube wells.

16.24. Open Wells or Dug Wells

Smaller amount of ground water has been utilised from the ancient times by open wells. Open wells are generally open masonry wells, having comparatively bigger diameters, and are suitable for low discharges of the order of 1—5 litres per second. The diameter of open wells generally vary from 2 to 9 m, and they are generally less than 20 m in depth. The walls of an open well may be built of precast concrete rings or in brick or stone masonry. Their thickness generally varies from 0.45 to 0.75 m, according to the depth of the well (See Fig. 16.37)

The yield of an open well is limited because such wells can be excavated only to a limited depth where the ground water storage is also limited.

Moreover, in such a well, the water can be withdrawn only at the critical velocity for the soil. Higher velocities cannot be permitted as that may lead to disturbance of soil grains and consequent subsidence of the well lining in the hollow so formed. The limit placed on the velocity, therefore, also limits the maximum possible safe discharge of an open well.

One of the recent methods used to improve the yield of an open well is to put in a 8 to 10 cm diameter bore hole in the centre of the well, so as to tap additional water from an aquifer or from the fissures in the rock. If a clay or a kankar layer is available at a smaller depth to support the open masonry well, a bore hole can be made in its centre so as to reach the sand strata. Such an arrangement will not only give a structural support to the open well but will also considerably increase its yield. Depending upon the availability of such a provision, the open wells may be classified into the following two types :

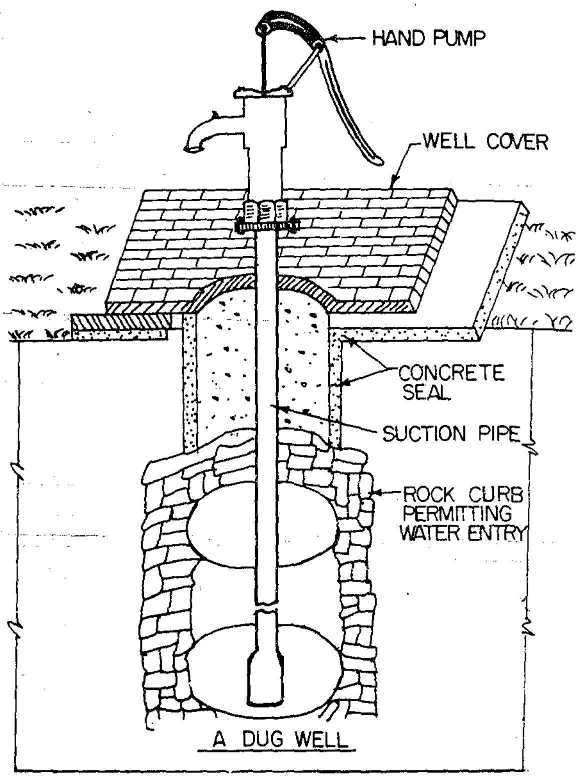


Fig. 16.37. Open well fitted with hand pump.

(*a*) *Shallow open wells* ; and (*b*) *Deep open wells.*

Shallow well is the one which rests in a pervious stratum and draws its supply from the surrounding material. On the other hand, a deep well is the one which rests on an impervious '*mota*' layer and draws its supply from the pervious formation lying below the mota layer, through a bore hole made into the '*mota*' layer, as shown in Fig. 16.38. The term "mota layer," also sometimes known as "Matbarwa" or "Magasan", refers to a layer of clay, cemented sand, kankar or other hard materials, which are often found lying a few metres below the watertable in the sub-soil. These names are not applied to the layers of hard material lying

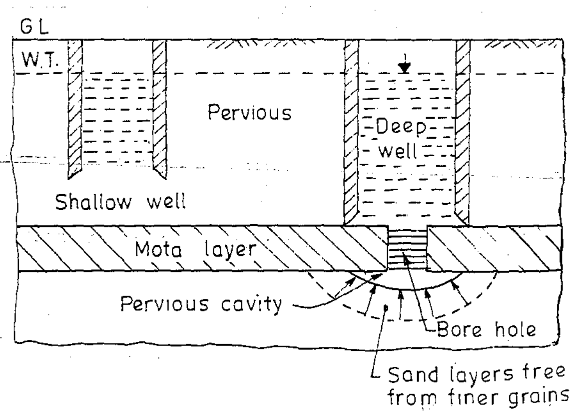


Fig. 16.38. Shallow and deep dug wells.

above the watertable. The main advantage of such a mota layer lies in giving structural support to the open well resting on its surface. It is useful for unlined and partly lined wells, and is indispensable for a heavy masonry well, which would not remain stable under steady use without such a support. The mota layer may either be continuous or may be localised, and are generally found in different thicknesses and depths at different places.

The nomenclature of shallow and deep dug wells is purely technical and has nothing to do with the actual depth of the well. A "shallow dug well" might be having more depth than a "deep dug well".

Since a shallow well draws water from the topmost water bearing stratum, its water is liable to be contaminated by the rain water percolating in the vicinity, which may take with it minerals or organic matter from decomposing animals and plants, etc. The water in a deep well, on the other hand, is not liable to get such impurities and infections. Secondly, the pervious formations below the mota layer generally contain greater quantities of ground water, yielding high specific yield. Hence, greater discharge and greater supplies can be obtained from a deep well as compared to those from a shallow well.

Water is generally drawn from dug or open wells by means of a bucket and a rope. However, due to the possible surface contamination of water in an uncovered well and also the individual buckets adding contamination to the water, such open wells have been covered in many parts of India and fitted with *hand pumps* (Fig. 16.37). A hand pump uses a reciprocating type of a pumping arrangement to lift water and is described in details in chapter 7 of "Water Supply engineering" by the same author.

16.24.1. Cavity Formation in Open Wells. Consider a well from which no water is being withdrawn. The water level in such a well will obviously be the same as is the static watertable outside the well. Now, if a discharge is withdrawn from this well at a constant rate, the level in the well will go down and stabilise at a lower level than that of the outside watertable. The head difference between these two levels is called the **depression head** (Fig. 16.39). Under the influence of this head difference, water enters the well from outside, so as to fill the gap created by the withdrawn water. As the water from the surrounding soil travels towards the well, there is a gradual loss of head, and water surface drops towards the well. Since the same discharge is passing through the reducing soil areas as it approaches the well, there is a gradual increase in the flow velocity towards the well. Now according to Darcy's law, this velocity can gradually increase only if the hydraulic gradient gets gradually increased. Hence, the water surface will fall gently in the beginning and will fall more and more rapidly as it approaches the well. The surface of watertable surrounding the well, therefore, takes up a curved shape and is called the **cone of depression**. At a certain distance from the well, there is no appreciable depression of watertable. This distance from the central line of the well is called the *radius of influence* of the well.

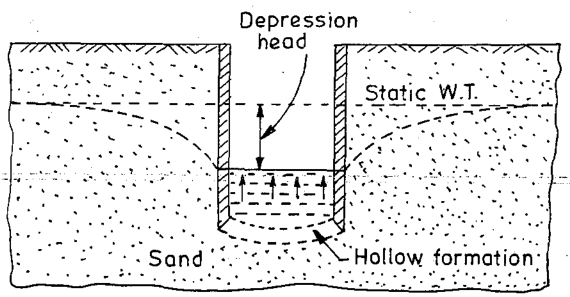


Fig. 16.39

The velocity of percolating water into the well depends upon the depression head. If more amount of water is withdrawn from the well and thereby increasing the depression head, higher flow velocities will prevail in the vicinity of the well. Thus, at a certain rate of withdrawal, it is very much possible that the flow velocity may exceed the critical velocity for the soil, thereby causing the soil particles to lift up. As more and more sand particles are lifted, a hollow is created in the bottom of the well, resulting in the increased effective area, so that ultimately, the velocity falls below the critical value and then no further sand goes out of the well.

As pointed out earlier, the formation of such hollows beneath the wells are dangerous in shallow wells, because there is always a danger of subsidence of the well lining. *The maximum rate of withdrawal from such wells is therefore, limited.*

In case of a deep well resting on mota layer, the cavity or hollow formation below the bore hole (Fig. 16.38) is not dangerous, because the well lining remains supported on the mota layer. Hence, a hollow, much larger in area than the cross-sectional area of the well, may safely form in deep wells, and thereby giving higher yields. In a shallow well of an equivalent yield, the well area will have to be increased equal to the area of the cavity under the deep well, which would make it costlier.

16.24.2. Construction of Open Wells. From the construction point of view, the open wells may be classified into the following three types :

Type I. Wells with an impervious lining, such as masonry lining, and generally resting on a mota layer.

Type II. Wells with a pervious lining, such as the dry brick or stone lining, and fed through the pores in the lining.

Type III. No lining at all *i.e.*, a Kachha well.

All these three types of wells are discused below :

Type I. *Wells with impervious lining.* They provide the most stable and useful type of wells for obtaining water supplies. For constructing such a well, a pit is first of all excavated, generally by hand tools, up to the soft moist soil. Masonry lining is then built up on the *kerb* upto a few metres above the ground level. A "Kerb" is a circular ring of R.C.C., timber or steel having a cutting edge at the bottom and a flat top, wide enough to support the thickness of the well lining called "steining". The kerb is then descended into the pit by loading the masonry by sand bags, etc. As excavation proceeds below the kerb, the masonry sinks down. As the masonry sinks down, it may be corrected by adjusting the loads or by removing the soil from below the kerb which may be causing the tilt. The well lining (steining) is generally reinforced with vertical steel bars.

After the well has gone up to the watertable, further excavation and sinking may be done either by continuously removing the water through pumps, etc., or the excavation may be carried out from top by the *Jhams*. A *Jham* is a self-closing bucket which is tied to a rope and worked up and down over a pulley. When the *Jham* is thrown into the well, its jaws strike the bottom of the well, dislodging some of the soil mass. As the *Jham* is pulled up, the soil cuttings get retained but the water oozes out. The sinking is continued till the mota layer is reached. A smaller diameter bore hole is then made through the mota layer in the centre of the well, which is generally protected by a timber lining.

Sometimes, when mota layer is not available, shallow wells may be sunk as described above upto a required depth, and partly filled with gravel or broken ballast. This will function as a filter through which water will percolate and enter the well but the sand particles will be prevented from rising up.

In a pucca well, lined with an impervious lining on its sides, the flow is not radial. The water enters only from the bottom and the flow becomes spherical when once the cavity has been formed at the bottom.

Type II. *Wells with pervious lining.* In this type of wells, dry brick or stone lining is used on the sides of the well. No mortar or binding material is used. The water, thus enters from the sides through the pores in the lining. The flow is, therefore, radial. Such wells are generally plugged at the bottom by means of concrete. If the bottom is not plugged, the flow pattern will be a combination of radial flow and a spherical flow. *Such wells are generally suitable in strata as of gravel or coarse sand.* Such a well is constructed in finer soils, so as to prevent the entry of sand into the well along with the seeping water.

Type III. *Kachha wells.* These are temporary wells of very-shallow depths, and are generally constructed by cultivators for irrigation supplies in their fields. Such wells can be constructed in hard soils, where the well walls can stand vertically without any support. They can, therefore, be constructed only where the watertable is very near to the ground. Though they are very cheap and useful, yet they collapse after sometime, and may sometimes prove to be dangerous.

16.24.3. Yield of an Open Well. As discussed in article 16.9, the yield of an open well can be determined:

(*i*) by *estimating the velocity of the ground water* :

(*ii*) by performing pumping tests, such as *equilibrium pumping test* or *recuperating test.* Both these tests were briefly described in article 16.9, and are elaborated in more details in this article.

(*a*) **Equilibrium pumping test.** A pump is first of all installed, so as to draw sufficient supplies of water from the open well, and to cause heavy drawdown in its water level. The rate of pumping is then changed and so adjusted that the water level in the well becomes constant. In this condition of equilibrium, the rate of pumping will be equal to the rate of yield of the well at a particular drawdown. Knowing this yield say Q_1 at a certain known drawdown, say s_1, the yield (Q) at any given drawdown (s) can be evaluated as follows :

By Darcy's law $Q = K.I.A = K \cdot \frac{s}{L} \cdot A = \frac{K}{L}.A.s.$

or $$Q = C.A.s \qquad ...(16.38)$$

where s is the depression head or the drawdown in the well.

If Q_1 is the known discharge at a certain known drawdown s_1, we have

$$Q_1 = C.A.s_1 \qquad ...(i)$$

Q_1 and s_1 are known and A is the area of cross-section of the well. In case a cavity is formed, the area A is taken as $\frac{4}{3}$ times the actual cross-sectional area of the bottom of the well. Knowing Q_1, A and s_1 in Eq. (*i*) above, the value of C can be calculated. Hence, the discharge Q at any other value of depression head (s) can be easily worked out.

From the above equation, it is also evident that the velocity increases with the depression head (s). The value of s, for which the velocity becomes equal to the critical value, is called the *critical depression head.* Generally, the depression head is kept equal to $\frac{1}{3}$ times the critical depression head; and such a head is known as the *working head.*

Hence, the maximum yield or critical yield corresponds to the critical depression head, and the maximum safe yield corresponds to the working head.

(*b*) **Recuperating test.** Although the pumping test gives accurate value of the safe yield, it sometimes becomes very difficult to adjust the rate of pumping, so as to keep the well water level constant. In such circumstances, recuperating test is adopted.

In this method, water is first of all drained from the well at a fast rate so as to cause sufficient drawdown. The pumping is then stopped. The water level in the well will start rising. The time taken by the water to come back to its normal level or some other measured level is then noted. The discharge can then be worked out as below :

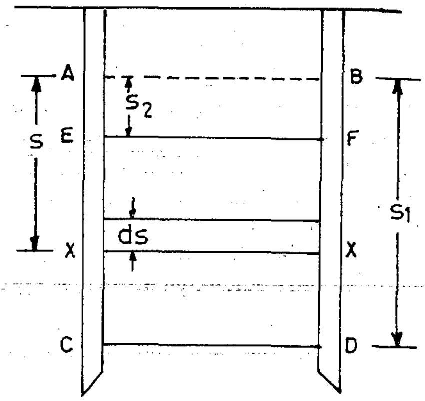


Fig. 16.40.

With reference to Fig. 16.40.

Let $AB =$ Static water level in the well before the pumping was started

$CD =$ Water level in the well when the pumping was stopped

$s_1 =$ Depression head in the well at the time the pumping was stopped

$EF =$ Water level in the well at the noted time (say after a time T from when the pumping is stopped)

$s_2 =$ Depression head in the well at time T after the pumping is stopped.

Let X-X be the position of the water level at a time t after the pumping was stopped and let the corresponding depression head be s.

Let ds be the decrease in the depression head in a time dt after the time t.

Hence, in a time t after the pumping is stopped, the water level recuperates by $(s_1 - s)$. It again recuperates by ds in a time dt after this.

$\therefore$ Volume of water entering the well in the small interval of time (dt)

$$= dV = A.ds \quad ...(1)$$

where A is the cross-sectional area of the well at the bottom.

Also, if Q is the rate of recharge into the well at the time t under a depression head s, then the volume of water entering the well in this small time interval is

$$= dV = Q.dt$$

But $Q \propto s$

$$\therefore \quad Q = C' . s \quad ...(2)$$

when C' is a constant depending on the soil through which water enters the well.

$$\therefore \quad dV = C' . s . dt \quad ...(3)$$

Equating (1) and (3), we get

$$-A.ds = C' .s.dt$$

(The –ve sign indicates that s decreases as t increases)

or
$$\frac{C' .dt}{A} = -\left(\frac{ds}{s}\right)$$

Integrating between the limits

$$t=0, \quad s=s_1, \quad t=T, \quad s=s_2$$

we get, $$\frac{C'}{A}\int_0^T dt = -\int_{s_1}^{s_2}\frac{ds}{s} \qquad \text{or} \qquad \frac{C'}{A}\cdot\left|t\right|_0^T = -\left|\log_e s\right|_{s_1}^{s_2}$$

or $$\frac{C'}{A}(T) = -\log_e\frac{s_2}{s_1} = -2.3\log_{10}\frac{s_2}{s_1} = 2.3\log_{10}\frac{s_1}{s_2}$$

$\therefore$ $$\boldsymbol{\frac{C'}{A} = \frac{2.3}{T}\log_{10}\frac{s_1}{s_2}} \qquad \text{...(16.39)}$$

Knowing the values of s_1, s_2 and T from the above test, the value of $\frac{C'}{A}$ can be calculated. $\frac{C'}{A}$ is called the specific yield or the specific capacity of the open well in cumecs per sq-m of area under a unit depression head. Knowing the value of $\frac{C'}{A}$, the discharge Q for a well under a constant depression head s can be calculated as follows:

$$Q = C'.s$$

or $$Q = \left(\frac{C'}{A}\right)A.s \qquad \text{...(16.40)}$$

or $$Q = \left(\frac{2.3}{T}\log_{10}\frac{s_1}{s_2}\right)A.s \qquad \text{...(16.41)}$$

A and s are known, discharge for any amount of drawdown (s) can be easily worked out.

In the absence of recuperation test, the following rough values of $\frac{C'}{A}$, as given by Marriot, can be used.

Table 16.3

Type of soil	$\frac{C'}{A}$ *(i.e. specific capacity or yield) in cubic metres per hour per sq. metre of area under unit drawdown*
Clay	0.25
Fine sand	0.50
Coarse sand	1.00

(*iii*) The third method which is useful for determining the yields of open wells as well as those of tubewells involves the use of equations developed by Dupuit, Thiem, etc., as discussed in the previous articles.

Example 16.10. *Design an open well in coarse sand for a yield of 0.004 cumec when operated under a depression head of 3 metres.*

Solution.

The discharge required from the well

$$= 0.004 \text{ cumec} = 0.004 \text{ cubic metre per second}$$

$$= 0.004\times 60\times 60 \text{ cubic metres per hour} = 14.4\ \text{m}^3/\text{hr}.$$

From Table 16.3, the value of $\frac{C'}{A}$ for coarse sand may be taken as

$$1.0\ \text{m}^3/\text{hr}/\text{m}^2/\text{m of depression head.}$$

Also, Depression head $= s = 3$ m.

Now from Eq. (16.40) $Q = \left(\frac{C'}{A}\right) A.s$

$$\therefore \quad 14.4 = 1 \times A \times 3$$

or $$A = \frac{14.4}{3} = 4.8 \text{ m}^2.$$

If d_w is the diameter of the well, then $\frac{\pi d_w^2}{4} = 4.8$

or $$d_w = \sqrt{\frac{4.8 \times 4}{\pi}} = 2.48 \text{ m} ; \textbf{ say 2.5 m} \quad \textbf{Ans.}$$

Example 16.11. *During a recuperation test, the water level in an open well was depressed by pumping by 2.5 metres and is recuperated by an amount of 1.6 metres in 70 minutes.*

(a) Determine the yield from a well of 3 m diameter under a depression head of 3.5 metres.

(b) Also determine the diameter of the well to yield 10 litres/second under a depression head of 2.5 metres.

Solution.

From Equation (16.39) $\frac{C'}{A} = \frac{2.3}{T} \log_{10} \frac{s_1}{s_2}$

where s_1 = Initial drawdown = 2.5 m

s_2 = Final drawdown = 2.5 − 1.6 = 0.9 m

T = Time = 70 minutes = 70 × 60 = 4,200 sec.

$$\therefore \quad \frac{C'}{A} = \frac{2.3}{4{,}200} \log_{10} \frac{2.5}{0.9} = \frac{2.3}{4{,}200} \log_{10} 2.778 = 0.244 \times 10^{-3}$$

(*a*) Yield from a well of 3 m diameter, under a depression head of 3.5 m, is obtained from equation (16.40) as

$$Q = \left(\frac{C'}{A}\right) A.s = 0.244 \times 10^{-3} \times \left(\frac{\pi}{4} \times 3^2\right) \times 3.5 = 6.02 \times 10^{-3} \text{ m}^3/\text{s}$$

$$= 6.03 \text{ litres/sec.} \quad \textbf{Ans.}$$

(*b*) If Q = 10 litres/sec

s = 2.5 m

$$Q = \left(\frac{C'}{A}\right) \cdot A \cdot s$$

$$10 \times 10^{-3} = (0.244 \times 10^{-3})\, A \times 2.5$$

or $$A = \frac{10}{0.244 \times 2.5} = 16.4 \quad \text{or} \quad \frac{\pi}{4} d^2 = 16.4$$

or $$d = 4.57 \text{ m} ; \text{ say } 4.6 \text{ m}$$

Hence, the diameter of the well required = 4.6 m **Ans.**

16.25. Tubewells

The discharge from an open well is generally limited to 3 to 6 litres/sec. Mechanical pumping of small discharges available in open wells is not economical. To obtain large discharge mechanically, *tubewell, which is a long pipe or a tube, is bored or drilled deep into the ground, intercepting one or more water bearing stratum.* The discharge of an open well is smaller, because : (*i*) open wells can tap only the topmost or at the most the next lower water bearing stratum. (*ii*) water from open wells can be withdrawn only at velocity equal to or smaller than the critical velocity for the soil, so as to avoid the danger of well subsidence. But in the tubewells, larger discharges can be obtained by getting a larger velocity as well as a larger cross-sectional area of the water bearing stratum. Since, we have an enormous storage of ground water in India, the tubewells provide excellent method of providing water supplies, although they are generally used for irrigation.

16.25.1. Tube-wells in Alluvial Soils. Most of our land, especially the entire area from the Himalayas to the Vindhya mountains (such as the Indo-Gangetic plain), coastal areas, Narmada valley, etc., consist of deep alluvial soils. The subsoil water slowly penetrates and is stored in the porous sand and gravel beds which are extensively found in India, except that in the desert areas. Tubewells can be easily installed in such soils and are very useful for irrigation. It is in this context that the tubewells are assuming greater and greater importance for tapping our ground water resources, especially in alluviums.

Deep tubewells are as deep as 70 to 300 m, and tap more than one aquifer. They are usually constructed in our country by the State Governments and are called State tubewells. Such wells may yield as high as 200 to 220 litres/sec. The general average yield from the standard tubewells is however of the order of 40 to 45 litres/sec. A 300 m deep tubewell has been constructed at Allahabad (U.P.) at the edge of the river Ganga and is yielding at about 140 litres/sec. The diameter of the hole is 0.6 m upto 60 m depth and then 0.56 m below 60 m. The diameter of the strainer is 0.25 m and drawdown is 10 m. Such deep tubewells are drilled by *heavy duty rotary drilling rigs* (direct rotary as well as reverse rotary). *Percussion drilling rigs* may also be used in hard boulder areas. There exist about 50,000 deep tubewells in our country, and every year about 1,000 such wells are being added. Most of our deep tubewells have been constructed by using mild steel slotted pipe screens with gravel packing.

Besides the deep tubewells, **shallow tubewells**, having 20 to 70m depth and tapping only one aquifer, are also constructed, usually by private individual cultivators. Such wells may yield as high as 15–20 litres/sec, if located at proper places. Each well irrigates about 8 hectares. Such shallow tubewells are usually drilled by *light rigs, cable tool drills, water jet methods*, and *hand boring devices*. There exist about 50 lakh shallow tubewells in India, and every year about 2.5 lakh such wells are being added.

16.25.2. Tubewells in Hard Rocky Soils. It is very difficult to construct a tubewell irrigation system in rocky areas. Therefore, in rocky areas, tubewells are resorted to only when, there are no other alternate sources of water. Hence, in rocky areas, only isolated holes of 10 to 15 cm diameter are drilled using *down the hole hammer rigs* (DTH rigs). They are usually in depth range of 100m, although tubewells up to 300 m depths have been successfully bored. Such tubewells are called **bored wells** because the bore hole is able to stand on its own in the bottom portion, and a tube is pushed only in the upper weathered zone. These wells usually depend on joints & fissures in the rocks for their water supply. Even with a heavy drawdown of 20 to 30 m, such wells are generally not able to yield more than 5–10 litres/sec, except when they tap some embedded aquifer. Such tubewells have mostly been constructed in southern States of our country.

16.25.3. Types of Tubewells. Depending upon the entry of the water through a cavity or a screen, the tubewells can be broadly classified into the following two categories : .

(1) *Cavity type tubewells;* and (2) *Screen type tubewells.*

Both these types of tubewells are discussed below:

(1) **Cavity Type Tubewells.** A cavity type tubewell draws water from the bottom of the well, and not from the sides, as is done by a screen well. The flow in a cavity well, therefore, is essentially spherical, and not radial like that of a screen well.

Moreover, since in a gravity well, the water is drawn from the bottom of the well, such a well can tap only one water bearing stratum. Such a tubewell however, is very economical, as it requires only plain well pipe, which is lowered into the bore made through the ground strata up to the required depth, so as to tap the requisite aquifer.

The principle behind the working of a cavity-type tubewell is essentially similar to that of a deep open well, with the only difference that whereas an open deep well taps the first aquifer, just below the mota layer, a cavity tubewell need not do so, and may even tap the lower or still lower stratum, as shown in Fig. 16.41

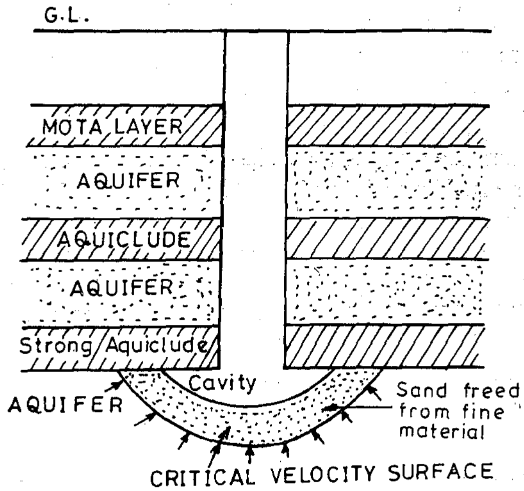


Fig. 16.41. Cavity type tubewell.

A cavity type tubewell essentially consists of a pipe bored through the soil and resting on the bottom of a strong clay layer. A cavity is formed at the bottom, and the water from the aquifer enters the well pipe through this cavity, as shown in Fig. 16.41. In the initial stage of pumping, fine sand comes out with water and consequently a hollow or a cavity is formed. As the spherical area of the cavity increases outwards, the radial critical velocity decreases for the same discharge, thus reducing the flow velocity and consequently stopping the entry of sand. Hence, the flow in the beginning is sandy but becomes clean with the passage of time.

The cavity of such a tubewell, however needs to be **developed** carefully and slowly by using a *centrifugal pump* rather than a *compressor* or a *turbine pump*.

To begin with, the water is pumped from the well at a low discharge rate. When the discharging water becomes clear, the drawdown may be increased slightly, which may result in further sand being drawn out. The process is repeated till the normal drawdown and clear discharge is obtained. The pumping is then stopped for an hour or so, and then resumed again. The discharge after restarting may again contain sand. The pumping is continued till the water is clear again. The procedure may be repeated till the tubewell is fully developed, which is shown by the sand free discharge coming out even on the resumption of pumping after a closing interval.

Cavity type tubewells can however be used only for small supplies, particularly for domestic supplies.

(2) **Screen Type Tubewells.** Screen type tubewells are most widely adopted and have been extensively constructed in our country particularly for irrigation purposes. So much so that whenever we refer to a tubewell, we generally mean a "screen well". All the State tubewells constructed in U.P. from where the technique of tubewell construction started in 1931, are exclusively of this type. Such a well can easily tap a number of aquifers, and hence does not depend only on one aquifer, like a cavity well. Screen type tubewells can be further divided into the following two types :

(*i*) *strainer tubewells;* and (*ii*) *slotted pipe gravel-pack tubewells.*

(*i*) A **strainer tubewell** uses strainer lengths lowered into the bore hole and located opposite the water bearing formations, whereas, plain pipe lengths are located opposite the non water bearing formations, as shown in Fig 16.42. A bail plug is provided at the bottom. Water enters into the well through these strainers from the sides, and the flow is *radial*.

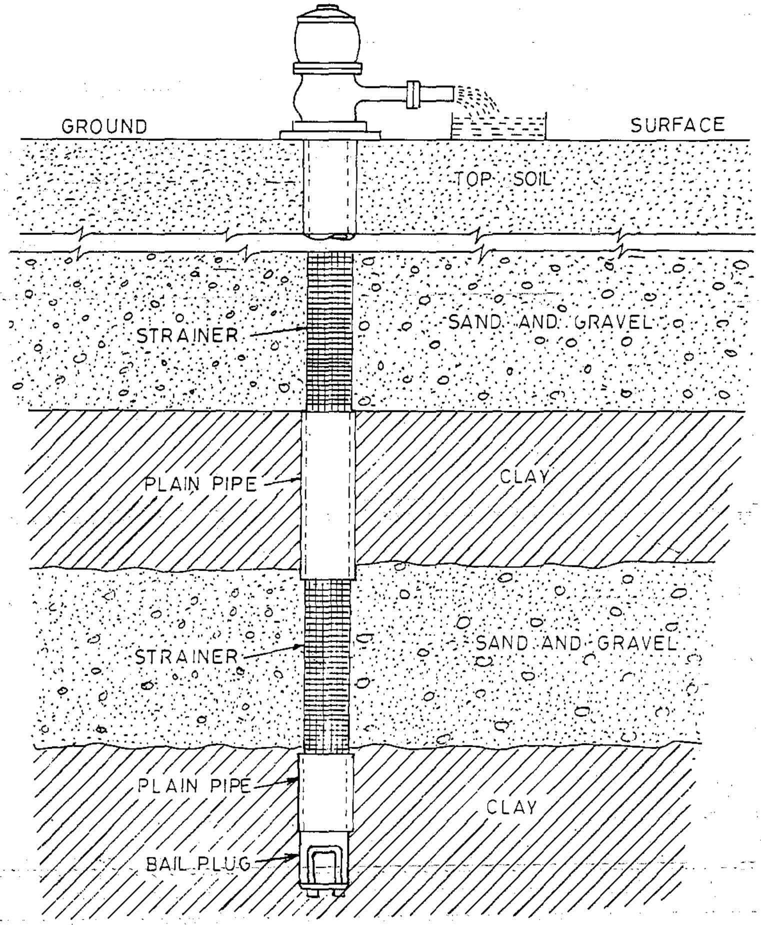


Fig. 16.42. A strainer tubewell.

A **strainer** (Fig 16.43) essentially *consists of a perforated or a slotted pipe with a wire mesh wrapped round the pipe with a small annular space between the two.* The wire screen prevents the sand particles from entering the well. The water, therefore, enters the well pipe through the fine mesh and the particles of size larger than the size of the mesh are prevented from entering the well. This reduces the danger of sand removal, and hence larger flow velocities can be permitted.

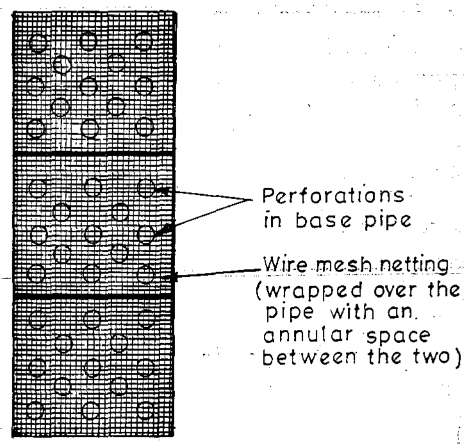


Fig. 16.43. A typical strainer.

The perforated pipe is made to have the cross-sectional area of its openings equal to that in the wire mesh, so that no change of velocity occurs between the two. An annular space between the pipe and the wire mesh is certainly required, otherwise the wires of the mesh will cover a large part of the area of the pipe openings.

A strainer type tubewell is generally unsuitable for fine sandy strata, because in that case, the size of the mesh openings will have to be considerably reduced, which may result in choking of the strainer; and if the mesh openings are kept bigger, the well will start discharging sand.

(*ii*) A **slotted pipe gravel pack tubewell**, on the other hand, uses a *slotted pipe without being covered by any wire mesh.* Such slotted pipe lengths are located opposite the water bearing formations, as is done with the strainers in a strainer tubewell. These slotted pipes should strictly be referred to as "screen pipes", although, however, they are often called as "*slotted pipe strainers*" or "*pipe strainers*", thereby having little scope to differentiate between such slotted pipes (*i.e. screen pipes*) and the perforated pipes with mesh coverings (*i.e. strainers*).

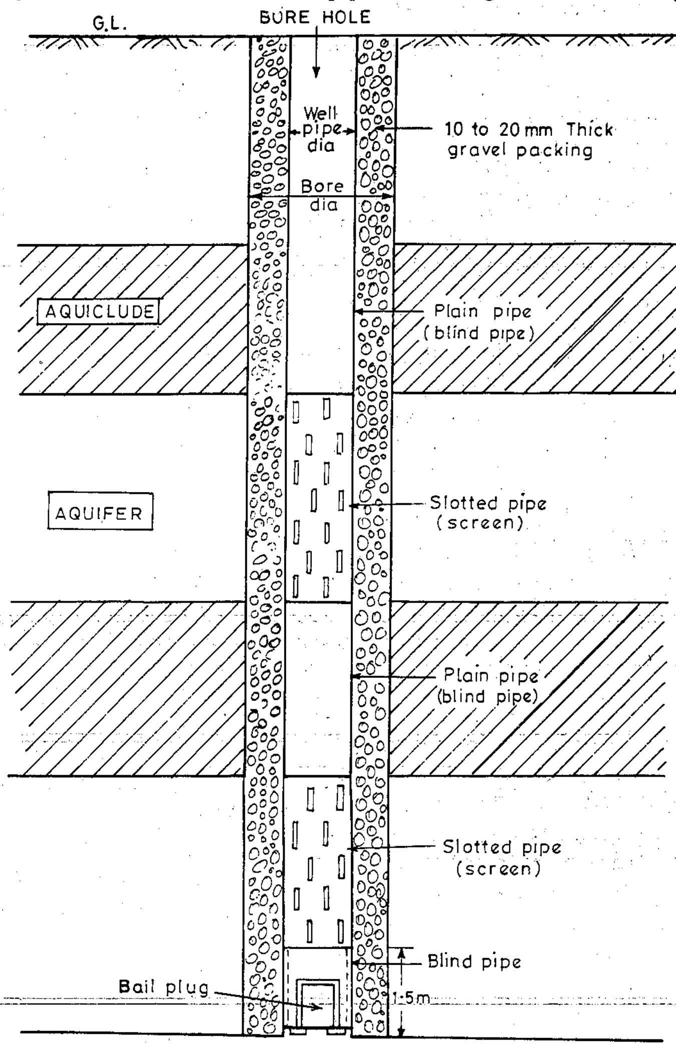


Fig. 16.44. Slotted pipe gravel pack tubewell.

After placing the assembly of the plain and slotted pipes in the bore hole, a mixture of gravel and bajri (called gravel shrouding) is poured into the bore hole between the well pipe assembly and the casing pipe, so as to surround the well pipes by a designed optimum thickness of gravel pack. The gravel packing is specifically required around the screen pipes, but since the material is poured from the ground level into the bore hole, the pack shall be installed in the entire depth of the well below the top level of the shallowest screen, as shown in Fig. 1644. Sometimes, the gravel pack is provided even above the top level of the shallowest screen, and up to the ground level, so as to obtain a stable and an efficient tubewell.

Gravel pack wells are generally provided in fine aquifers, where the effective grain size (D_{10}) may be less than 0.25 mm, and uniformity coefficient (C_u) may be 2.0 or less. *Gravel packing also enables the use of single size slots in the screen pipes, irrespec-*

tive of the fact that fine as well as coarser aquifers are tapped simultaneously and in alternating layers. Gravel pack is therefore highly preferred for deep tubewells, which tap more than one aquifer. Due to this reason, most of the State deep tubewells have been constructed in our country in fine alluvial soils, by providing cast in situ gravel packs, throughout the full height of the wells.

Precast gravel packs have also come into the market, where the slotted pipes are mounted with *prepack gravel filters*, popularly called *Ashim filters*. In these filters, graded hard gravel grains are coated and bonded together with water proof chemicals over the outer surface of the slotted pipe, providing a highly permeable screen. Such prepack gravel filters are found suitable for shallow and medium deep tubewells in aquifers composed of fine sand, coarse sand and gravel. Prepacking is claimed to be providing economy in the quantity of gravel used and protects against dislodging of the gravel envelope. However, such filters are so far not being used in larger projects in our country.

16.25.4. Design of a Well Screen. A well screen constitute the most important part of a tubewell, since it serves as the intake structure for the entry of water into the well. The design of the screen is largely influenced by the characteristics of the water bearing formations. Dry sieve analysis of the aquifer sample obtained during drilling of the bore hole, is therefore carried out to plot the grain size distribution curve for the aquifer. The results will help us to determine the design specifications of the well screen as well as to design the gravel pack.

From the grain size distribution curve of the aquifer, the following characteristics of the aquifer are determined :

(*i*) Effective size (D_{10})

(*ii*) Uniformity Coefficient (C_u)

(*iii*) D_{50} size (mean particle size) of the aquifer.

The *effective size* (D_{10}) is that sieve size through which 10% of the particles shall pass, and 90% are retained. 90% of the particles in the aquifer are thus coarser than this size, and hence retained on the sieve of this size.

The *uniformity coefficient* (C_u) is defined as the ratio of the sieve size passing 60% of the aquifer material, to the sieve size passing 10% of the aquifer material. In other words,

$$C_u = \frac{D_{60}\ (40\%\ \text{retained})}{D_{10}\ (90\%\ \text{retained})} \qquad \text{...(16.42)}$$

This ratio was proposed by Hazen (1893) to be a quantitative expression of the degree of the assortment of the water bearing sand, as an indicator of porosity. The value of C_u for complete assortment (one grain size) is 1; while for fairly even grained sand, it ranges between 2 to 3. For heterogeneous sand, the value will be higher. *Generally a material is classified as uniform, if its uniformity coefficient C_u is equal to or less than 2.*

Designing the Size of the Slot Openings of a Screen. Fixing an optimum size for the slot openings of a screen is an important component of the screen design, and it depends upon the size of the aquifer material. Oversized slots will pump finer material indefinitely, and it will be difficult to obtain clear water. Undersized slots will provide more resistance to the flow of ground water into the well, resulting in more head loss and corrosion. Fine slots are also blocked by small sand and silt particles. The problem of clogging is reduced as the size of the openings is increased. *Therefore, theoretically, the slot openings should be as wide as possible.* The optimum value is determined by matching the size of the opening with the grain size distribution of the material surrounding the screen. *In practice, the slot size varies from values as low as 0.2 mm to as large as 5 mm.* In India, two sizes *i.e.* 1.6 mm and 3.2 mm are generally available in the market. The criteria for selecting the size of the slot openings for non-gravel pack and gravel pack well is given below :

(*a*) **Slot openings for non-gravel pack wells.** The optimum slot opening size is chosen as the one which retains 40% of the sand (*i.e.* equal to D_{60} size of the aquifer) if the ground water is non-corrosive; and equal to the D_{50} size of the aquifer if the ground water is corrosive.

The above optimum selected slot size may vary for different aquifers, when more than one aquifer is to be tapped, thereby indicating the use of different widths for slots to be made in the pipe lengths to be rested against the different aquifers, Practically this is a difficult proposition, and hence uniform width of slots designed on the basis of the finest aquifer material are often provided.

The above adoption of D_{60} size for the slots indicates that 60% of the formation material shall be pumped out during the development of the well, which will result in the removal of all the fine particles from around the well screen.

(*b*) **Slot openings for gravel pack wells.** The slot size for well screens for gravel pack wells is determined on the basis of the grain size distribution curve of the gravel material used for the gravel pack. On this curve, a point is located indicating the 90% size of the gravel to be retained (*i.e.* D_{10} size for the gravel material), which indicates the optimum slot size for the well screen. The actual size of the slots may be fixed within ± 8% of the D_{10} size of the gravel pack.

Gravel pack design. The gravel pack should however also be designed before designing the size of the slots to be made in the well screens. The gravel pack is usually designed on the basis of **Pack Aquifer ratio** (PA ratio), which is usually defined as the ratio of D_{50} size of the gravel pack material to the D_{50} size of the aquifer material.

$$\therefore \quad \text{PA ratio} = \frac{D_{50} \text{ of the gravel}}{D_{50} \text{ of the aquifer}} \qquad \text{...(16.43)}$$

Several recommendations have been made by different investigators for optimum values of PA ratio. The Central Board of Irrigation and Power (1967) has for Indian conditions, suggested the following criteria for PA ratio.

(*i*) PA ratio should be between 9 and 12.5 for uniform aquifers having $C_u \leq 2.0$ ...(16.44)

(*ii*) PA ratio should be between 12 and 15.5 for graded aquifers having $C_u > 2.0$ ...(16.45)

The uniformity coefficient of the gravel material is also preferably kept to be equal to or less than 2.5, because a higher value shall cause seggregation of the gravel shrouding during pouring, which will result in poor efficiency of operation of the well. Such gravel materials, having $C_u \leq 2.5$ are difficult to obtain, and hence gravel pouring requires a lot of field control. A numerical example on the design of the gravel pack has been solved in the next article to make its design very clear.

The *thickness of the gravel pack* should normally be fixed from practical considerations at about 7.5 cm, and in no case should exceed about 20 cm. A thicker envelope does not materially increase the well yield, nor will it reduce the possibility of sand pumping, because the controlling factor is the ratio of the grain size of the pack material and that of the aquifer. Too thick a gravel pack, instead of giving any advantages, may make the final development of the well more difficult.

Example 16.12. *The results of sieve analysis test carried out on a 500 gm sample of underground aquifer, proposed to be tapped for installation of a tubewell, are given in the table below :*

Size of the sieve in mm	*Mass of material retained in gm*
> 2.54	0.0
1.80	6.0
0.30	15.0
0.25	320.0
0.21	5.0
0.16	50.0
0.12	34.0
< 0.12	70.0
	500.0

Design the size of the gravel pack, and the slot size for the slotted screen pipes.

Solution. The given sieve analysis data is analysed, as shown in table 16.4. A distribution curve is then plotted between grain size in mm (on log x-axis) and % age finer (on y-axis), as shown in Fig. 16.45.

Table 16.4

Size of the sieve in mm	*Mass of material retained in gm*	*% age of material retained* $\frac{(2)}{500} \times 100$	*Cummulative % age retained*	*% age finer (p) 100 – col (4)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*
> 2.54	0	0.0	0	100
1.80	6	1.2	1.2	98.8
0.30	15	3.0	4.2	95.8
0.25	320	64.0	68.2	31.8
0.21	5	1.0	69.2	30.8
0.16	50	10.0	79.2	20.8
0.12	34	6.8	86.0	14.0
< 0.12	70	14.0	100.0	0.0
Σ	500 gm	100%		

From the drawn curve, the following characteristics of the aquifer material are read out as :

$$D_{60} = 0.27$$
$$D_{50} = 0.265$$
$$D_{10} = 0.102$$

$$\therefore \quad C_u = \frac{D_{60}}{D_{10}} = \frac{0.27}{0.102} = \mathbf{2.64}$$

Since $C_u \geq 2.0$, we should use P.A. ratio for designing the gravel pack lying between 12 and 15.5

$$\text{Hence, P.A. ratio} = \frac{D_{50} \text{ of gravel pack}}{D_{50} \text{ of aquifer}} = 12 \text{ and } 15.5$$

$$\therefore \quad 12 = \frac{D_{50} \text{ of gravel pack}}{0.265}$$

$\therefore$ D_{50} of gravel pack $= 12 \times 0.265 = \mathbf{3.18\ mm}$

$$\text{Also} \quad 15.5 = \frac{D_{50} \text{ of gravel pack}}{0.265}$$

$\therefore$ D_{50} of gravel pack $= 15.5 \times 0.265 = \mathbf{4.11\ mm}$

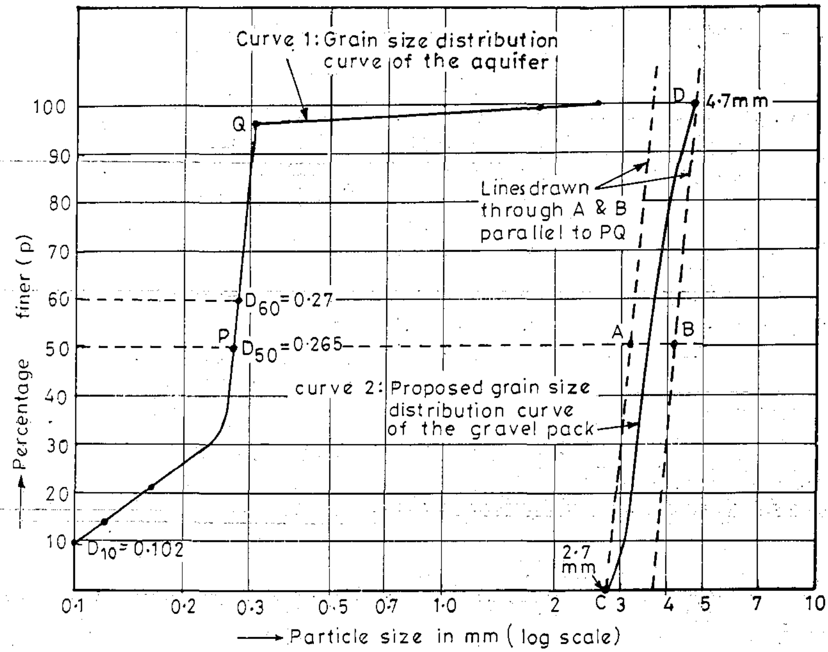


Fig. 16.45. Grain size distribution curves for the aquifer and the gravel pack.

In other words, D_{50} of the gravel pack should lie between these two limiting values; *i.e.* 3.18 mm and 4.11 mm. These values are marked on 50% horizontal line, as *A* and *B*. Lines (dotted) are drawn through these two points, parallel to the central portion of the grain size curve of the aquifer material. These two enveloping curves are the limiting curves for the grain size distribution curve of the gravel pack. The minimum size of the gravel between these two curves is represented by point *C* as 2.7 mm; and the maximum size is represented by point *D* as 4.7 mm. *Hence, the gravel pack size should vary between 2.7 mm and 4.7 mm.* The gravel should be screened such that the gravel size ranges between 2.7 mm and 4.7 mm. The proposed grain size curve for the gravel shall be somewhat as shown by the firm line *CD*. D_{10} size of this curve is approximately equal to 3 mm; and hence the screen having standard 3.2 mm wide slots may be used.

C_u for the gravel is also computed as $= \dfrac{D_{60}}{D_{10}} = \dfrac{3.6 \text{ mm}}{3 \text{ mm}} = 1.2$ (O.K.). This value should be ≤ 2.5 for good design and the drawn curve may be adjusted between the two envelope curves to achieve satisfactory value of C_u.

Designing the length and size of the screen. The total length of the screen to be provided for a tubewell shall be primarily controlled by the available thickness of the aquifers, since this length cannot exceed the aquifers thickness. It shall further be governed by the *area of the screen openings,* because to pass a given discharge, the screen length will be less if the area of its openings per m length is more, and *vice-versa.*

The area of the screen openings per m length of the screen usually veries between 15 to 20% of the screen area, which equals $\pi.d$, where d is the dia of the screen pipe. If the area of the openings is kept more than about 15 to 20 %, the structural strength of the well screen reduces, which consequently reduces the life of the screen.

The diameter of the screen is selected to satisfy the essential basic requirement that sufficient open area be provided in the screen, so as to limit the entrance velocity to a safe permissible limit. The optimum safe entrance velocity for a given aquifer, is related to the coefficient of permeability (K) of the aquifer, as shown in Table 16.5.

Table 16.5 Optimum Screen Entrance Velocity (v_e)

Coefficient of Permeability (K) of the aquifer in cm/s	*Optimum entrance velocity (v_e) in cm/s*
0.02	1.5
0.05	2.0
0.09	3.0
0.14	4.0
0.18	4.5
0.24	5.0
0.28	5.5

For most of the available sandy aquifers, K value varies between 0.05 to 0.10 cm/s, and hence v_e generally varies between 2 cm/s to 3 cm/s.

When entrance velocity exceeds this safe optimum value, the frictional loss through the screen openings shall become higher, and there will be more incrustation and corrosion.

The entrance velocity is therefore calculated by dividing the design discharge per m length of the screen by the total area of the openings in the screen per m length of the screen. If the value works out to be more than the safe optimum value, the dia of the screen is increased, so as to increase the open area to achieve the optimum value of the entrance velocity. The guiding values of the screen diameter for different discharges, as suggested by USBR (Ahrens, 1970) are given in Table 16.6.

Table 16.6 Recommended Values of Screen Diameter

Discharge in l/min	*Recommended screen diameter in cm*
0 to 475	10
475 to 1125	15
1125 to 3000	25
3000 to 5250	30
5250 to 9500	35
9500 to 13,300	40

The minimum length of the screen can finally be designed by using the equation

$$\text{Min. length of the screen} = h = \frac{Q}{A_0 \cdot v_e} \qquad \text{...(16.46)}$$

where Q = Design discharge of the tubewell

A_0 = Area of the screen openings per m length of the screen

$= \pi . d \times$ percent of openings (p)

v_e = optimum entrance velocity for the given K value of the aquifer for non-gravel wells. For gravel pack wells, the mean K value for the aquifer and for the gravel pack is considered, to assume the value of v_e.

The above calculated minimum strainer length should be adjusted within the available aquifer depths by screening the available depth(s) by about 75 to 90% for confined aquifers. The percentage should be increased with the increase in thickness. Say for example, 75% screening is satisfactory for 8 m thick aquifers, and 90% for

20 m thick aquifers. Atleast 0.3 m aquifer depth at top as well as at bottom of the aquifer should be left unscreened to safeguard against the error in the placement of the screen during installation. The pumping water level should never fall below the top of the aquifer. The screen is usually located at the centre of the aquifer.

In unconfined aquifers, the bottom $\frac{1}{3}$ to $\frac{1}{2}$ depth of the aquifer is usually screened.

Example 16.13. *A fully penetrating well in a confined sandy aquifer has a maximum discharge capacity of 1200 l/min. The aquifer is overlain and underlain by impervious formations. The thickness of the aquifer is 20m. Design the length of the well screen assuming the percentage of the open area of the available strainer to be 15%, and bore hole dia as 15 cm.*

Solution. $Q = 1200\,\text{l/min} = \frac{1.2}{60}\,\text{m}^3/\text{s} = 0.02\,\text{m}^3/\text{s}$

A_0 = Area of the openings per m length of the screen

$= (\pi \cdot d)\, p = \pi\,(0.15\,\text{m}) \times 15\%$

$= \pi \times 0.15 \times 0.15\,\text{m}^2/\text{m} = \mathbf{0.071}\,\text{m}^2/\text{m}$ length of the screen

v_e = safe entrance velocity

= may be assumed to be 2 cm/s since K for sand is usually 0.04 cm/s

$\therefore \quad v_e = 0.02$ m/s

Using eqn. (16.46), we have

$$h = \text{Min. length of the screen} = \frac{Q}{A_0 \cdot v_e}$$

$$= \frac{0.02}{0.071 \times 0.02} = 14.2\,;\ \mathbf{15\ m}\ \text{(say)}$$

Since the aquifer thickness is 20m, and the minimum required screen length is 15m, it would be prudent to use 18 m length of the screen, which shall be about 90%* of the aquifer depth. The screen may be provided in the central 18 m depth of the aquifer, leaving one metre depth of the aquifer unscreened at both the ends.

16.25.5. Types of Well Screens. (1) **Slotted Pipe Screens.** As stated earlier, all the gravel pack wells normally utilise *slotted pipe screens*. The slot size is also decided somewhere near the D_{10} size of the designed gravel material. The slotted pipes to be used are generally made of mild steel. IS:8110-1976 provides details in respect of such screens. The standard slot size commonly adopted in India is 1.6 mm or 3.2 mm wide, and 10 cm long. The minimum spacing between slots is 3 mm. The slots are so arranged as to obtain an even distribution of flow over the entire periphery of the screen. They are distributed in groups of 3 or 4, and arranged so that the slots of one group are not in line with, those of the adjacent row, so as to maintain adequate strength in the well pipe. A typical view of the arrangement of slots is shown in Fig 16.46

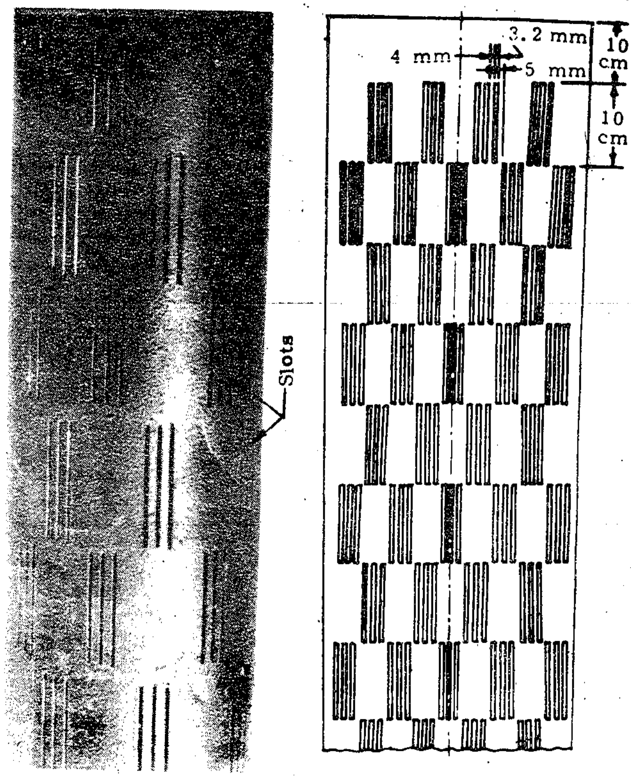
The slotted pipe is threaded at both ends. The bottom end of the slotted pipe is fitted with a blind pipe of 1.25m length, with a cap called *bail plug* at the bottom. The bail plug has an eye, fixed inside, which facilities the extraction of the tubewell assembly incase of failure.

Slotted pipe screens with prepack filters called **Ashim filters**, have also come into the market, and may be used for comparatively smaller and medium depth wells. A

* Confined aquifers are screened by about 75–90% of the thickness, whereas unconfined aquifers are screened in their bottom 1/3 to 1/2 of the thickness. Percentage of the confined aquifer to be screened increases with the increase in the thickness.

photographic view of such filters is shown in Fig 16.47. In such screens, mild steel slotted pipes are coated with 15–20 mm thick precast gravel layers, as to provide highly permeable screens. The mild steel pipe is usually slotted to have an open area of 25% with slots of 3.2 mm width. Different grades of filters are available to suit different sized aquifer particles. Such screens are, however, very costly, and hence adopted only for smaller individual wells.

Brass screens and **stainless steel screens**, utilising brass or stainless steel pipes, with slots as in mild steel pipes, are also available in the market. These screens are quite costly but are more durable and less liable to corrosion, incrustation and consequent contamination of water due



(*a*) General view (*b*) Layout of slots as per IS:8110-1976

Fig. 16.46. Mild steel slotted pipe screen.

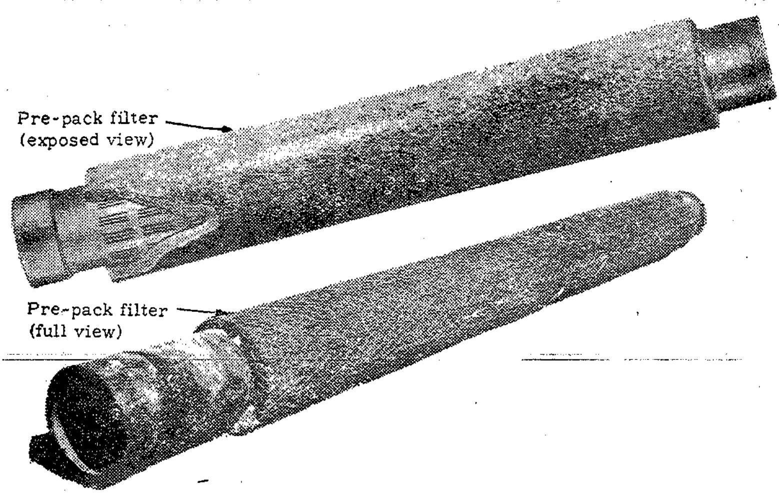


Fig. 16.47. Ashim's slotted pipe prepack filter screens.

to rusting. Hence, such screens may be preferred for tubewells to be used for water supplies. Such screens are manufactured from 4 mm thick copper or stainless steel sheets, provided with designed openings (slots), which is rolled and welded to make the circular tube. The ends of the tube length (unslotted) are provided with outer threads for joining the tubes through a socket. Screens with different slot sizes and diameters are available in the market to suit the requirement of the design based on grain size distribution curve of the aquifer or the gravel pack.

(2) **Strainer Type of Well Screens.** Aquifers have traditionally been screened by utilising the strainer type of well screens. Such screens contain very small & narrow openings, to exclude the removal of those aquifer particles which are larger than the screen openings. Such screens usually have a double system of openings, consisting of slotted or perforated cylindrical pipes or shells, which are covered by fine wire mesh, wound round the pipe. The water has to initially enter the openings of the *outer wire jacket*, and then pass through the openings of the pipe shell. *Such strainer wells are usually provided without gravel packs, although it is not an absolute necessity, and even stainer wells can be gravel packed, like those using slotted pipes.*

Several types of strainers, which are generally used in tubewell construction, are:

(*i*) *Continuous-slot type of strainers.*

(*ii*) *Louver or shutter type of strainers;*

(*iii*) *pipe strainers with a cover of wire jacket, like that used in an agricultural strainer;* and

(*iv*) *Coir rope strainers.*

These types are briefly discussed below;

(*i*) **Continuous slot type of strainers** consist of cold drawn wire wound round a suitable cylindrical frame made of rods of iron, steel, brass etc; At each point where the wire crosses the rods, the two are welded and jointed to produce a one piece rigid unit. In order to avoid clogging, V-shaped openings are usually provided by using triangular shaped wire in the outer jacket. Such screens can be made practically in any width from 1.5 mm onward.

The well loss in such strainers is much-lower than those in the pipe strainers, but these are much costlier. A photographic view of such a strainer is shown in Fig. 16.48.

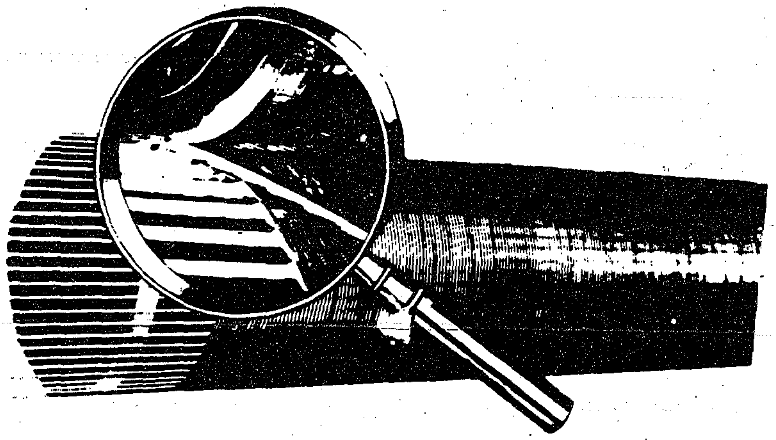


Fig. 16.48. Photoview of a continuous slot type of well strainer.

(*ii*) **Louver type of stainers** have openings, which are rows of louvers. The openings may be oriented either at right angles or parallel to the axis of the screen. The openings are produced in the wall of the welded tube by stamping process, using a die. A photo view of such a screen is shown in Fig 16.49. Such a screen provides lesser choice of the opening sizes, because sizes of dies are generally limited. Such a screen also provides lesser percentage of the open area than the one provided by the continuous slot type. The openings of such a screen also tend to clog during the development of the well, where the aquifer contains appreciable quantity of sand. There screens are, however, best suited when provided with gravel pack, as is done with the slotted pipe screens.

The continuous slot type of strainers, as well as the louver type of strainers, though are hydraulically more efficient, but have not become popular in developing countries like India, owning to their high costs.

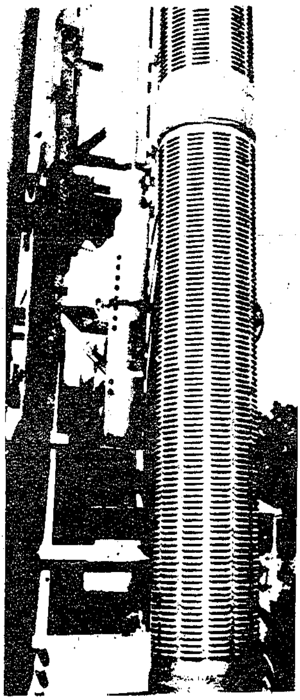


Fig. 16.49. Photoview of a louver type of well screen.

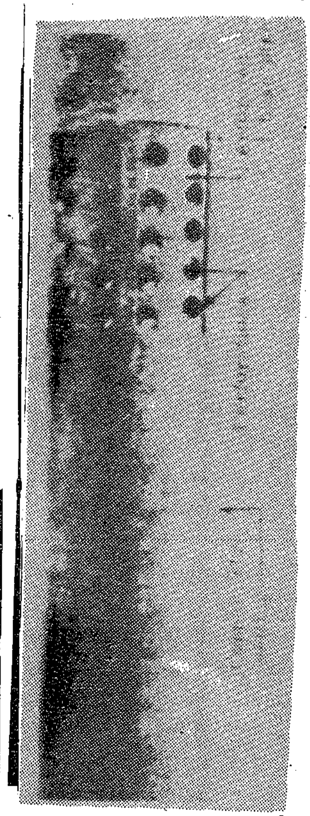


Fig. 16.50. Partially exposed photoview of an agricultural strainer.

(*iii*) **Pipe strainers with outer fine mesh jackets,** utilise perforated metal or pvc pipes surrounded by fine wire mesh, with some annular space between the two. Several designs are available in the market, and the most commonly adopted type is known as the **agricultural strainer.** A photoview of such a strainer is shown in Fig 16.50. It consists of a perforated galvanised iron pipe, on which steel or iron rods or strips of 1 cm width and 3mm thickness are welded at fixed intervals around the circumference of the pipe. The perforations are circular and made with a drilling machine. The perforations are arranged in such a manner, as not to be covered by the rods. Trapezoidal shaped bronze, brass or copper wire is wound round the series of rods, and securely shouldered, to provide a jacket of non-corrosive alloys.

The type of wire mesh to be used depends upon the aquifer particle size. Usually, three types of wire nettings suitable for fine, medium and coarse aquifers are available in the market.

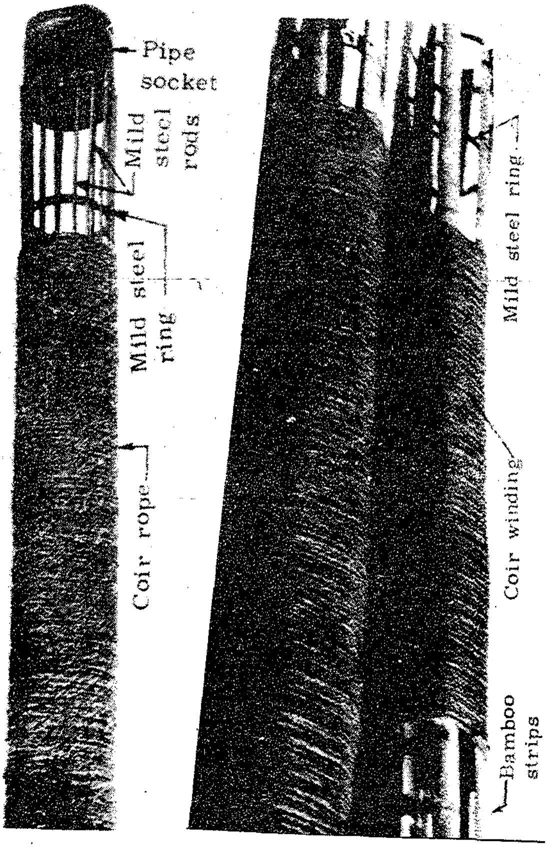
In the above design, since the wire netting is not in direct contact with the perforated pipe over which it is wrapped, the area of perforations is not decreased by the netting in front of the perforations. The arrangement therefore, provides good hydraulic efficiency. Such a strainer could last for 10 years or so.

The biggest disadvantage of such a strainer however, is that it involves bimetallic construction, since the pipe base is of steel and the outer mesh jacket is of brass or bronze, etc. Such bimetallic construction causes electrolytic action and corrosion of the steel pipe. To overcome this difficulty, several designs by using the same type of metal in the base pipe as well as in the net jacket have been commercially produced, but are generally very costly, since single metal construction in brass or copper considerably increases the cost of the strainer. Such costly single metal strainers are however, widely used in developed countries and are sold in several brand names.

(*iv*) **Coir rope strainers** are low cost strainers, generally used for shallow irrigation tubewells, particularly in deltaic regions of developing countries. Such a strainer essentially consists of a cylindrical frame made of iron rods or bamboo strips, wound round by coir rope of 3 to 5 mm diameter, as shown in Fig. 16.51.

The disadvantage of such a strainer is its short life of 3 to 5 years. The cause of its failure is the rusting of iron bars of the supporting frame, and the loosening of the coir rope which expands on wetting. Use of *nylone rope*, and coating the iron rods with bitumen have been found to increase its life to a good extent. Since the coir rope strainers cannot withstand the high pressure created during the development of the tubewell by an air compressor, wells using such screens shall have to be developed only by pumps.

Closely mounted *polythene rings* of 6 to 10 mm width have also been used in place of coir rope to make such a stainer to be more resistant to corrosion and deterioration.



(*a*) Coir rope strainer with mild steel frame (*b*) Coir rope stainer with bamboo frame.

Fig. 16.51. Coir rope strainers

16.25.6. Construction and Boring of Tubewells. With respect to the method of boring adopted, the tubewells can be classified into the following three categories;

(1) *driven tubewells;*

(2) *Jetted tubewells;* and

(3) *drilled tubewells.*

These three types of borings are discussed below :

16.25.6.1. Driven Tubewells. A driven tubewell consists of a pipe and a well point fixed to the lower end of the pipe, as shown in Fig 16.52. The assembly is forced down the ground to penetrate into the water bearing formation, by driving it with a wooden maul, drop hammer, or other suitable means.

If the water level is 7 m or more below the ground surface, the well pipe should be more than 5 cm in diameter, so that a jet or a cylinder may be inserted and submerged to permit the pump to function.

A special device, called a *cap* or a *drive head* is provided at the top of the pipe assembly during the driving operation, so as to protect the pipe during hammering. After each length of the pipe is hammered into the ground, the cap is removed and additional sections are attached, and driven as required.

The pipe is kept full of water at all times. In its descent through the aquicludes, the water in the pipe shall not flow out through the screen, since the screen is sealed by the impervious formation; but as the well point reaches the water bearing formation, some water will flow out of the pipe, and water remaining in the pipe drops to a static level. This is a signal to the driller that water bearing formation has been reached.

To develope the well, the driller attaches a small pitcher pump to the pipe. A considerable amount of the sandy water may be pumped for a short time, but if a good aquifer has been tapped, continued pumping should result in clear sand free water.

Driven tubewells yield very small discharges, and are suitable only for individual domestic supplies. Their construction is limited to shallow depths in soft unconsolidated formations free from boulders, and other obstructions.

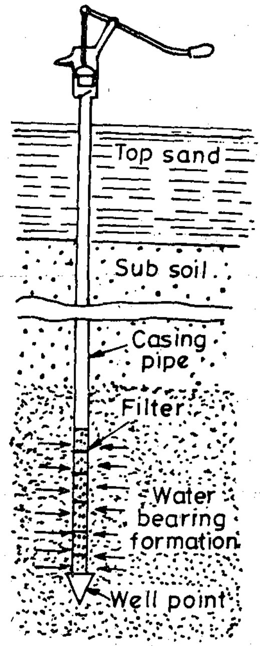


Fig. 16.52. Driven tubewell

16.25.6.2. Jetted tubewells. A jetted tubewell may be constructed either with a hand operated equipment or power driven machines, depending upon the soil formation and size and depth of the well bore.

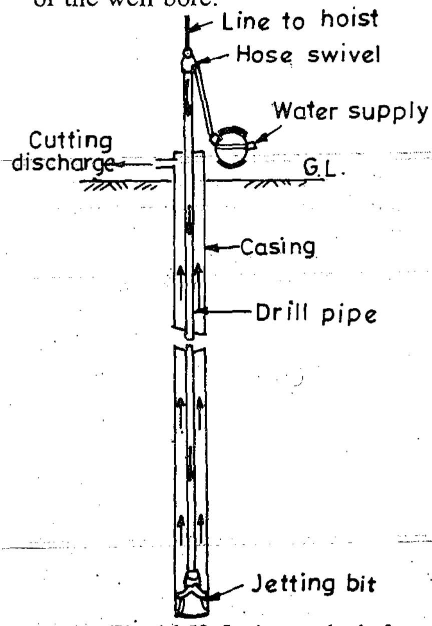


Fig. 16.53. Jetting method of drilling shallow tubewells.

A hole in the ground is made by the cutting action of a stream of water, which is pumped into the well through a small dia pipe, and forced against the bottom of the hole through the nozzle of a jetting bit (Fig 16.53). A casing pipe is provided during boring to prevent the caving of the bore/hole.

After the hole has been jetted down to the designed depth, the well assembly consisting of blank pipes and screen, is lowered, and *the outer casing is pulled out.*

Jetted tubewells have small yields, and their construction is feasible only in unconsolidated formations.

16.25.6.3. Drilled tubewells. Deep and high capacity wells are constructed by drilling. Various techniques are employed in drilling the well hole. Different techniques have comparative advantages and disadvantages over each other, depending upon the type of formation to be drilled. Therefore, each well should be treated as an individual project, and one particular method adopted depending upon its suitability. Some of the **drilling methods** commonly used, are described below :

(1) **Standard method or Cable tool method of drilling.** This method of drilling the well hole is known as *percussion drilling*; because in this method, the

well hole is made by percussion (*i.e.* by hammering and cutting). This method is very useful for cutting consolidated rocks from soft clay to hardest rocks, and is generally unsuitable in loose formations, such as unconsolidated sand and gravel or quick sand. This method becomes ineffective in loose material, because the loose material slumps and caves around the *drilling bit*. The drill bit has a chiesel sharp edge which breaks the rock by impact when alternately lifted and dropped. This drilling bit is connected at the lowest end of the entire 'falling and rising arrangement' known as the *string of tools*. [Refer Fig. 16.54 (*a*) and (*b*)]. From top to bottom, the string of tools consists of a rope socket, a set of jars, a drill stem, and the drilling bit.

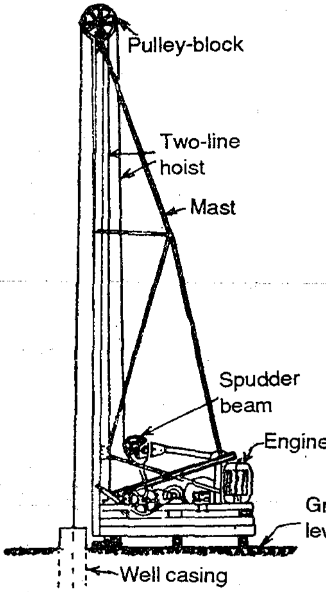


Fig. 16.54 (*a*) Basic components of Percussion Drilling Rig.

Tools are made of steel and are joined with tapered box and pin screw joints. The entire assembly weighs several tonnes. The most important tool of the entire assembly is the drilling bit (or drill) as it does the actual rock cutting. The drill stem is the long steel bar which adds weight and length to the drill, so that it can cut rapidly and vertically.

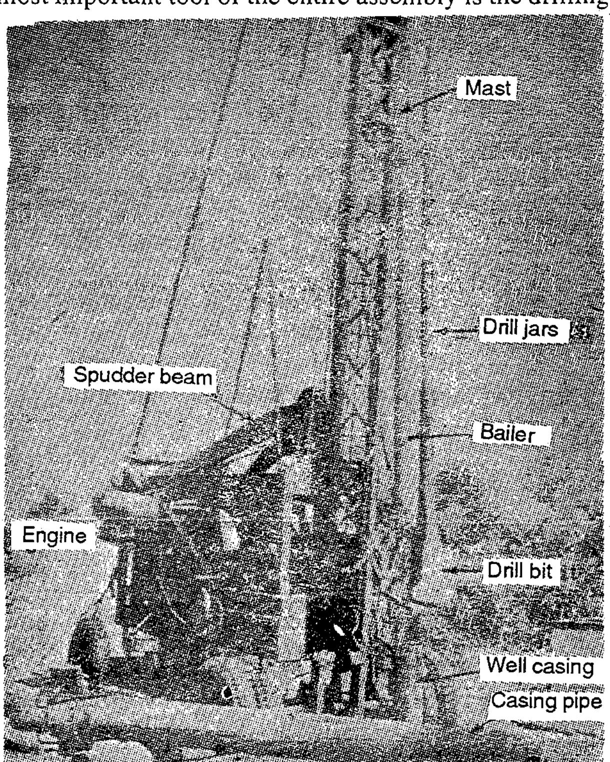


Fig. 16.54 (*b*). Photographic view of a Percussion Drilling Rig. The string of tools is seen to the right of the mast.

The set of jars have no direct effect on the drilling. They only loosen the tools when they stick in the hole. A rope or a cable is fastened at the upper end to the rope socket and to a dead man (or a heavy weight) at the lower end.

The entire assembly of tools is suspended from an assembly of a mast and walking beam, etc. This assembly, known as *drilling rig*, in turn, is generally mounted on a truck, as to make it easily portable. The mast should be sufficiently high so as to allow the longest of tools to be hoisted.

As the drilling proceeds, the tools makes 40 to 60 strokes per minute, from a height of 0.4 to 1 m. Water is sometimes added in the hole so as to form a paste with the cuttings, thus reducing friction on the falling bit. After the bit has cut 1 to 2 m through a formation, the string of tools is lifted out and the hole is cleaned and cleared of the cuttings by means of a *bailer*. The process is known as bailing out the hole.

A bailer essentially consists of a pipe with a valve at the bottom and a ring at the top. When lowered into the well, the valve permits the cuttings to enter the bailer but prevents them from escaping the bailer. After it is filled with cuttings, it is lifted up to the surface and emptied.

In unconsolidated formations, casing should be driven down and maintained near the bottom of the hole to avoid caving. Casing is driven down by means of drive clamps fastened to the drill stem. The up and down motion of the tools, striking the top of the casing, protected by a drive head, sinks the casing. On the bottom of the casing, a drive shoe is fastened to protect the casing, as it is being driven.

(2) **Hydraulic rotary or Direct rotary method of drilling.** This is the fastest method of drilling and is especially useful in unconsolidated formations. The method involves a continuously rotating hollow bit, through which, a mixture of clay and water or mud is forced. The bit cuttings are carried up in the hole by the rising mud. No casing is required during drilling because the mud itself makes a lining on the walls of the hole which prevents caving.

The drill bit is connected to a hollow steel rod (or drill stem), which, in turn, is connected at the top to a square rod, known as the Kelley [Refer Fig. 16.55 (*a*)]. The drill is rotated by a rotating table which fits closely around the Kelley and allows the drill rod to slide down, as the hole progresses.

The drilling rig, such as shown in Fig. 16.55 (*b*), consists of a mast, rotating table, a pump for forcing the mud, a hoist and the engine. The mud, after it emerges out of the hole, is carried to a tank where the cuttings settle out and the mud can be repumped into the hole.

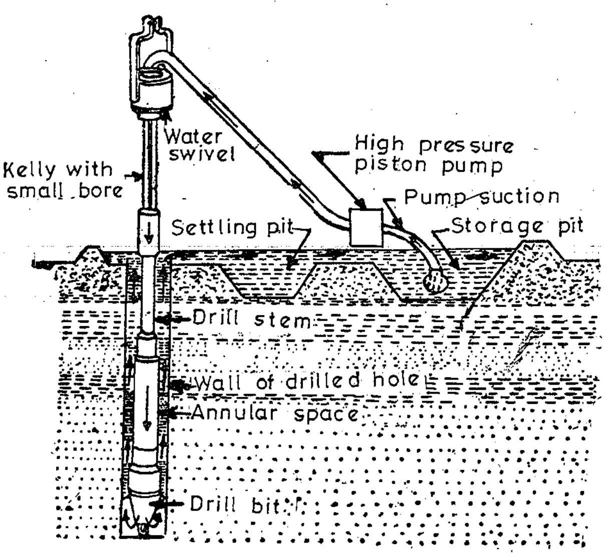


Fig. 16.55 (a). Schematic sketch illustrating the basic principles of Direct Rotary Drilling.

After the drilling is completed, the casing is lowered into the hole. The clay deposited in the well-walls during mud pumping, is removed by washing it with water. Water containing some chemicals like sodium hexametaphophate is forced through the drill rod and the washings come out through the preforations of the casing. When the washing at one level is completed, the bit is raised, and the process repeated.

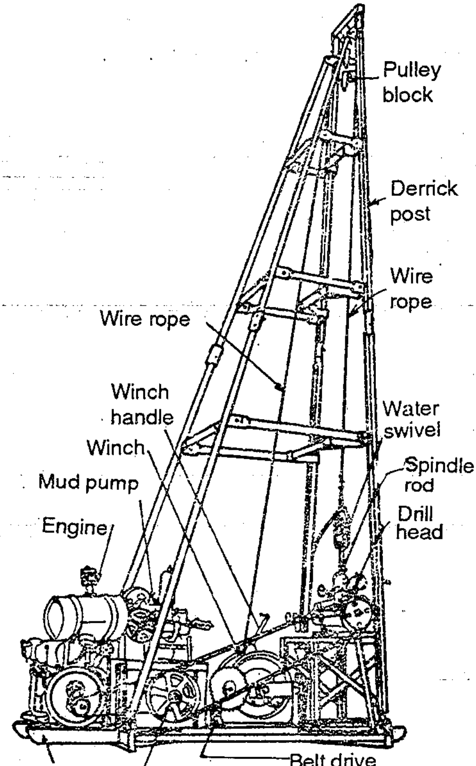


Fig. 16.55 (*b*). Skid mounted light duty Rotary drilling rig.

(3) **Reverse rotary method of drilling.** A modification of the hydraulic rotary method is known as the reverse rotary method. This is gaining popularity day by day. *It is quite useful for making large wells (diameter up to 1.2 m. app.) in unconsolidated formations.*

The tools consists of a hollow drill, a drill pipe and water swivel. In this method, the cuttings are removed by water through a suction pipe called the *drill pipe*. The equipment consists of a mast or a derrick, a centrifugal pump, necessary water and power arrangement, and the requisite casing pipe.

The hole is driven by pumping water under pressure through the drill bit, while it is churned up and down. The walls of the hole are supported by hydrostatic pressure acting against a film of fine grained material deposited on the walls by the drilling water. Cuttings are removed by the

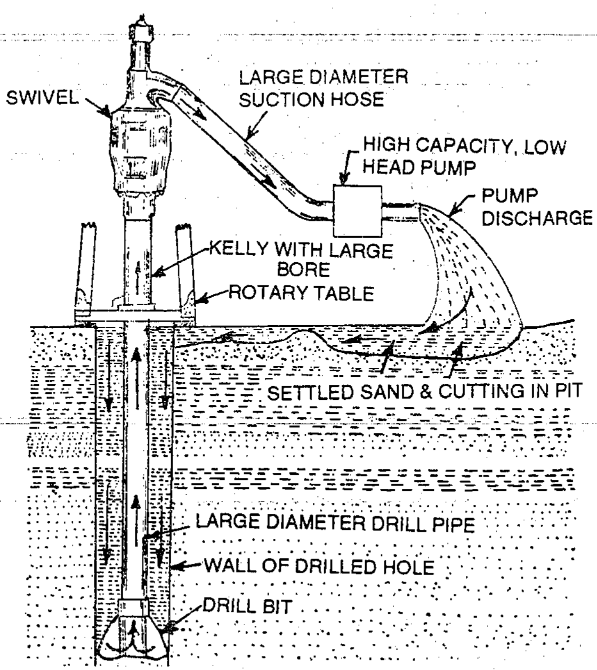


Fig. 16.56. Schematic sketch illustrating the basic principle of Reverse Rotary Driling.

water, and after the mixture (water + cuttings) comes out to the surface, it is passed through a settling tank (Refer Fig 16.56)

The sand settles out here but the fine grained particles are recirculated, so as to help in stablising the walls. Casing and cleaning of the walls, etc. is the same as in the hydraulic rotary method.

Comparison of cable tool and hydraulic rotary methods of drilling. *Advantages of Cable Tool Method are given below:*

1. A more accurate sample of the formation can be obtained.
2. Lesser amount of water is required during operations.
3. Cable tool rig is lighter and easy to transport.
4. Very useful for consolidated rocks and less useful for loose formations.
5. For shallow tubewells in unconsolidated materials too, it comes out to be cheaper.

Advantages of Hydraulic Rotary Method are given below :

1. Can be used for larger holes up to 1.5 m diameter.
2. Can be best used for drilling test holes, because the hole can be abandoned with minimum cost.
3. Rotary drilled hole can be gravel packed, which increases its specific capacity, and keeps the fine particles away, thus causing less sand trouble.
4. Casing is to be driven only after the hole completion, and hence, can be set at any desired depth.
5. It is the fastest method of drilling and especially useful in unconsolidated formations.
6. It can handle alternate hard and soft formations with ease and the danger of accidents is lesser. In quick sands, clays, etc., cable tool method is likely to give troubles, as there is a danger of freezing.

Verticality of the bore hole during drilling must by ensured, irrespective of the method adopted for drilling the bore hole. A common specification allows a deviation of 15 cm from the vertical in a length of 30m.

16.25.6.4. Well Log.

During drilling a well hole, the description of the material encountered in sequence through the drilling, has to be recorded, and this record is called a well log. A well log will, therefore, record the different types of formations with their correct depths of occurrences. Samples of drill cuttings at different depths, at regular intervals of 1.5 to 2 m, are also collected, to determine the exact nature of the rocks being drilled.

The prepared well log will help in designing and commissioning the tubewell :

16.25.7. Installing Well Screens. Well casings and well screens are installed on the basis of the well log. If the strata conditions, as revealed by the well log, warrant a cavity type tubewell, no screen will be necessary.

In such a cavity well therefore, the casing pipe used in the drilling, may either be left as it is, or if it is costly, then a smaller dia blind well pipe may be inserted, and the casing pipe withdrawn.

In screen tubewells, strainers or slotted pipes will be required, and the procedures for installing such well strainers shall vary with the design of the well and the method employed for drilling. The screens or slotted pipes shall have to be located opposite the water bearing strata, in order to draw water from the strata. This is done by assembling together the whole length of the screens and the blank pipes in exactly the same order and the lengths, in which they are to be lowered in the bore.

The screens and the blind pipes are lowered into the bore one by one, starting from the bottom end. During lowering of these well pipes, the casing pipe may be withdrawn, to allow the screens and the blind pipes to hold them in position.

In unconsolidated formations, sometimes, the casings are left intact, to support the bore hole and to freely admit water into the well. In such a case, the casing pipe itself should either contain *perforations* or its lower part be replaced by a screen or a strainer.

16.25.8. Well Development. Tubewells are developed to increase their specific capacity, prevent discharge of sand, and to obtain maximum economic well life. Development means *the stabilisation of the walls of a well adjacent to the screen, by a process which removes the fine particles from the formation immediately surrounding the well screen, leaving coarser particles to contact and surround the screen.*

The *main objectives of well development* are:

(*i*) to unclog the water bearing formation

(*ii*) to increase the porosity and permeability of the water bearing formation in the vicinity of the well.

(*iii*) to stabilise the sand formation around a screened well, so that the well may yield sand free water.

Development is necessary in all gravel packed wells and other screened wells, except when the screen is made of fine wire mesh or coir or other closely knit filters, located in a highly permeable aquifer.

The basic principle in well development is to cause reversals of flow through the screen openings, that will rearrange the aquifer particles. This is essential to break down *bridging* of the groups of particles. Fig 16.57 shows how small particles can bridge between large particles across the screen openings, when the flow of water is in one direction only. Reversing the direction of flow by *surging the well* does help in removing the bridging.

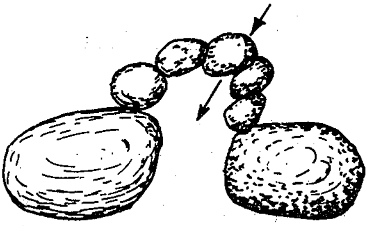


Fig. 16.57. Bridging action caused by one directional flow.

The commonly adopted **well development methods** are discussed below :

(*i*) **Well development by surging.** In this method, a plunger is worked up and down in the well, so that the water is alternately forced out into the surrounding formation and then allowed to flow back into the well. A *surge block* or *surge plunger* is the tool, which is used for this purpose.

The outflow portion of the surge cycle breaks down the bridging to loosen the fines; and the inflow portion of the surge cycle moves the fines towards the screen and into the well, from where they are removed.

(*ii*) **Well development by pumping.** In this method, a tubewell may be developed either by *over pumping* or by *rewhiding the well.*

Overpumping involves heavy pumping of the well to cause heavy drawdown. This is not a very effective method, for well development because the flow of water remains unidirectional, thus not removing the bridging of the particles.

Rewhiding involves starting and stopping of pumping intermittently to provide relatively rapid changes in the head of the well. While this may be done with any type of pump, it is most effectively done with a turbine type of pump installed with a foot valve.

(*iii*) **Well development by compressed air.** This method is most commonly adopted for developing wells, and may involve either :

(*a*) *backwashing technique,* or (*b*) *surging technique.*

A combination of both the techniques may sometimes be used for more effective development.

Backwashing technique involves forcing of the well water back into the aquifer by means of compressed air introduced into the well through the top of the casing after it has been closed. When the pressure is released, the water will flow back into the well through the screen to bring the fine particles from the area surrounding the well, thus ensuring their removal. The process is continued till no sand is brought in.

Surging technique involves the principles of both the pumping method and the surging method described earlier. The inrush of compressed air creates a powerful surge within the well, and loosens the fine material surrounding the perforations, which may then be brought into the well by continuous air injection. The operation is repeated at intervals along the screened section of the well, until sand arrival is stopped. The principle of pumping is the same as described eariler.

(*iv*) **Well development by jetting.** Jetting with water at high velocity is an effective method of well development. The method involves operating a horizontal water jet inside the well in such a way that the high velocity water stream shoots out through the screen openings. Fine particles are thus washed out of the aquifer, and the turbulence created by the jet brings these fines back into the well through the screen openings above and below the point of operation. By slowly rotating the jetting tool, and by gradually raising and lowering it, the entire surface of screen can be covered.

Use of dispersing agents in well development. Certain chemicals like tetra sodium pyrophosphate, sodium tripolyphosphate, sodium hexametaphosphate, and sodium sep-taphosphate, etc., when added to the well water, and to the water used in *backwashing* or *jetting techniques* of the well development described above, considerably helps in mud removal, thereby increasing the effectiveness of the above methods.

16.25.9. Spacing of Tubewells. Tubewells have been generally located without any scientific studies, till recently. It was thought that for Indo-Gangetic area, tubewells yielding about 45 litres/sec could be located within 2.4 sq. km., which fixed the cultivable commanded area of the tubewell to about 400 hectares. With further research now, tubewells are constructed with a command of 120 hectares, and on an average, a tubewell is located in 1.5 sq. km, and the resultant large scale pumping has not materially lowered the watertable.

16.25.10. Life of a Tubewell and Reasons for its Failure. A normal tubewell lasts for about 15 to 20 years in Northern India. It may fail due to (*a*) Incrustation, or due to (*b*) corrosion.

(*a*) **Incrustation.** The incrustation of the well pipe occurs due to the deposition of alkali salts on the inside walls of the pipe. The important salt causing incrustation is *calcium carbonate,* although calcium and magnesium sulphates and silicates may sometimes be the basic binders. The incrustation of the well pipe reduces the effective diameter of the well pipe, and hence reduces the discharge of the tubewell.

The incrustation can be reduced :

(*i*) by reducing the drawdown and hence the pumping rate ;

(*ii*) by using screens having larger area of openings (or larger diameter pipes) so as to allow some allowance for the future incrustation ;

(*iii*) by using such materials for the strainers (screens) that may easily permit the removal of incrustating material by chemical action without affecting that strainer material. In other words, the acids, etc., which are used to remove the incrustation, should not produce any effect on the strainer materials ; and (*iv*) by properly maintaining and periodically cleaning the well screens.

The incrustation can be delayed or reduced by these four methods, although it cannot be completely eliminated and thus the life of the tubewell can be increased by the above methods.

(*b*) **Corrosion.** The well pipe is gradually destroyed by corrosion due to the action of acidic water on the pipe material. When chlorides and sulphates or carbon dioxide are present in the water, the well pipe will definitely get corroded. The aquifer sand, surrounding the well pipe, finds a way out into the corroded pipe through the worn out pipe walls, thus bringing sand along with water. Hence, *corrosion results in excessive withdrawal of sand along with well water.*

Thicker pipes may be used to avoid corrosion. Stainless steel strainers will be most suitable, but are very costly. In affluent countries like U.S.A., such stainless steel screens are being progressively used, but are too expensive for the developing countries like India. An iron or steel screen can be depended upon only for a limited service life of the tubewell in most of the waters. The life of such a screen can be increased by galvanising (*i.e.*, by zinc coating) the pipe material. Other measures, which can help in reducing corrosion, and thereby increasing the life of the tubewell, are :

(*i*) by reducing the drawdown and the pumping rate ;

(*ii*) by reducing the flow velocity by increasing the percentage of the open area or the diameter of the well pipe ;

(*iii*) by using thicker pipes ;

(*iv*) by using corrosion resistant materials for the pipes ; and

(*v*) by using corrosion resistant coatings on the pipes.

16.25.11. Design of a Strainer Tubewell. The design of a strainer type tubewell essentially consists of designing the size of the tube, the size of the bore hole, the length of the strainer, and type and horse power of the pumping arrangement required to lift the water. The design considerations for these parts of a tubewell are discussed below:

(*i*) **Size of the tube.** The diameter of the tube (pipe) is decided from the considerations of permissible flow velocity through the tube. Since the strainers are installed at different levels, the velocity of water in a tube of a fixed size will not be constant but will be increasing towards the top. Hence, it is theoretically possible to reduce the size of the tube from the top towards the bottom, such that the velocity is more or less constant throughout the tube length. However, it is generally not economical to install varying size tube, and hence, a single sized or at the most a two sized tube (in case the aquifers are available at too much different depths) may be used in actual practice. The velocity through the tube may be limited between 1.5 to 4.5 m/sec. Knowing the design discharge of the tubewell and the velocity, the area of the tube and hence, its diameter can be easily calculated. The nearest available size in the market may then be used.

(*ii*) **Size of the bore hole.** Normally, the size of the bore hole should be at least 5 cm bigger than the size of the tube, so as to facilitate the lowering of the tube in the hole. If gravel packing is provided then the bore size will be decided by making provision for the gravel thickness also.

(*iii*) **Length of the strainer**. After deciding the diameter of the tube, the length of the strainer required to obtain the design discharge may be calculated for unconfined or confined aquifer cases, respectively, as given below :

(*a*) *For Unconfined Aquifer Case*; from eqn. (16.17) we have

$$Q = \frac{\pi \cdot K \cdot (d^2 - h_w^2)}{2.3 \log_{10} R/r_w}$$

If s is the drawdown or depression head, then w.r.t. to Fig. 16.13, we can easily write

$$d - h_w = s$$

Now,
$$Q = \frac{\pi . K (d - h_w)(d + h_w)}{2.3 \log_{10} \frac{R}{r_w}} = \frac{\pi K.s. (\overline{h_w + s} + h_w)}{2.3 \log_{10} \frac{R}{r_w}}$$

or
$$Q = \frac{2\pi K.s. \left(h_w + \frac{s}{2}\right)}{2.3 \log_{10} \frac{R}{r_w}} \quad ...(16.17\ a)$$

or
$$\left(h_w + \frac{s}{2}\right) = \frac{2.3\ Q \log_{10} \frac{R}{r_w}}{2\pi K.s}$$

or
$$h_w = \left[\frac{2.3\ Q \log_{10} \frac{R}{r_w}}{2\pi K.s} - \frac{s}{2}\right] \quad ...(16.17\ b)$$

h_w in this case, represents nothing but the **length of the strainer** needed. In the above equation, r_w is the radius of the well and is known by now. The value of the radius of influence (R) may be assumed between 250 to 500 m. Such a wide variation in the value of R will, at the most change the discharge up to a maximum of 12%, since the relation between Q and R is logarithmic. The suitable value of permeability coefficient (K) may be assumed. The design discharge and the depression head (*i.e.* drawdown) are also known; hence, the value of h_w, *i.e.*, the length of the strainer can be easily calculated.

(*b*) *For Confined Aquifer Case*; from eqn. (16.20) we have

$$Q = \frac{2\pi KHs}{2.3 \log_{10} \frac{R}{r_w}}$$

or
$$H = \frac{2.3\ Q \log_{10} \frac{R}{r_w}}{2\pi K.s}$$

In this equation, H represents the length of the strainer which can be easily calculated. This designed strainer length is provided in one or more aquifers, depending upon the site availabilities.

(*iv*) **Type of pumping arrangement.** Three types of pumps are generally used in order to lift water. They are

(1) *Centrifugal pumps*
(2) *Bore hole type pumps*, and
(3) *Jet pumps*.

These three types of pumps are discussed below :

(1) *Centrifugal Pumps.* A centrifugal pump lifts water from the lower level to a higher level by creating the required pressure with the help of centrifugal action. The

maximum suction head, under which the pump can practically work effectively, is about 6 to 8 m. Hence, such a pump can be used only at places, where the fluctuations in watertable plus the depression head is limited to a maximum of 8 m, as shown in Fig. 16.58. For larger values, the bore hole type pump is to be used. A centrifugal pump is generally available in two models, *i.e.* (*i*) when the pump and the motor are built together, and (*ii*) when the pump and motor are separately built and coupled together. Such centrifugal pumps are polularly known as **monoblock** centrifugal pumps.

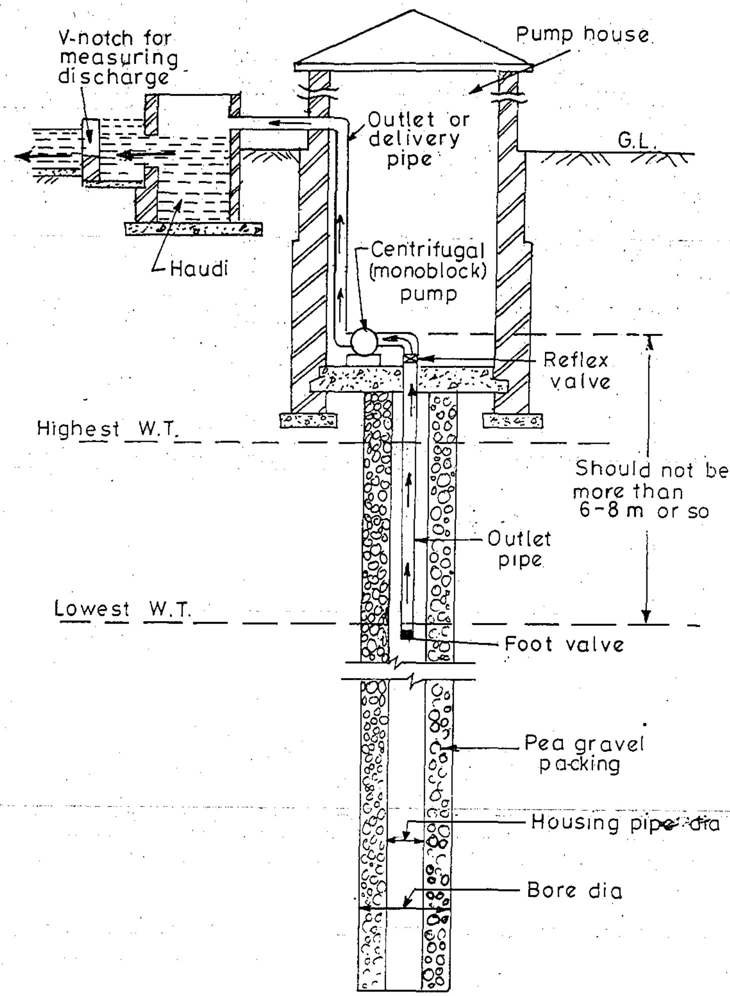


Fig. 16.58. Section of a tubewell using Monoblock Centrifugal pump.

A section of tubewell using a centrifugal pump is shown in Fig. 16.58. In this arrangement, a sump well is sometimes constructed, so as to place the pump at a required lower level, below the ground level. The pump is to be placed slightly above the highest water table, so as to avoid its submergence. The minimum water level should not be lower than the pump level by more than 6 to 8 m as shown, otherwise the pump will not work.

Hence, such an arrangement can be used only where the water is available at smaller depths from the surface, (*i.e.* generally for shallow cavity wells) and it cannot be used for those *deep tubewells* where water is generally available quite deep.

The sump well is also generally of a large size and deep enough, and has to be plugged at the bottom with concrete. This arrangement may sometimes make it costly, although this type of pump is the cheapest. Hence, such type of pumping arrangement is genrally not preferred in modern days, when the bore hole type pumps are being increasingly used.

(2) *Bore hole type Pumps.* Such pumps consist of special centrifugal pump impellers connected in series, mounted on a vertical shaft, and driven by a motor. They are of small diameter and can be lowered in the casing pipe itself. The top 20 to 30m of the bore hole and the casing pipe is generally kept wider than the remaining normal bore, so as to accommodate the pump bowl in the casing pipe.

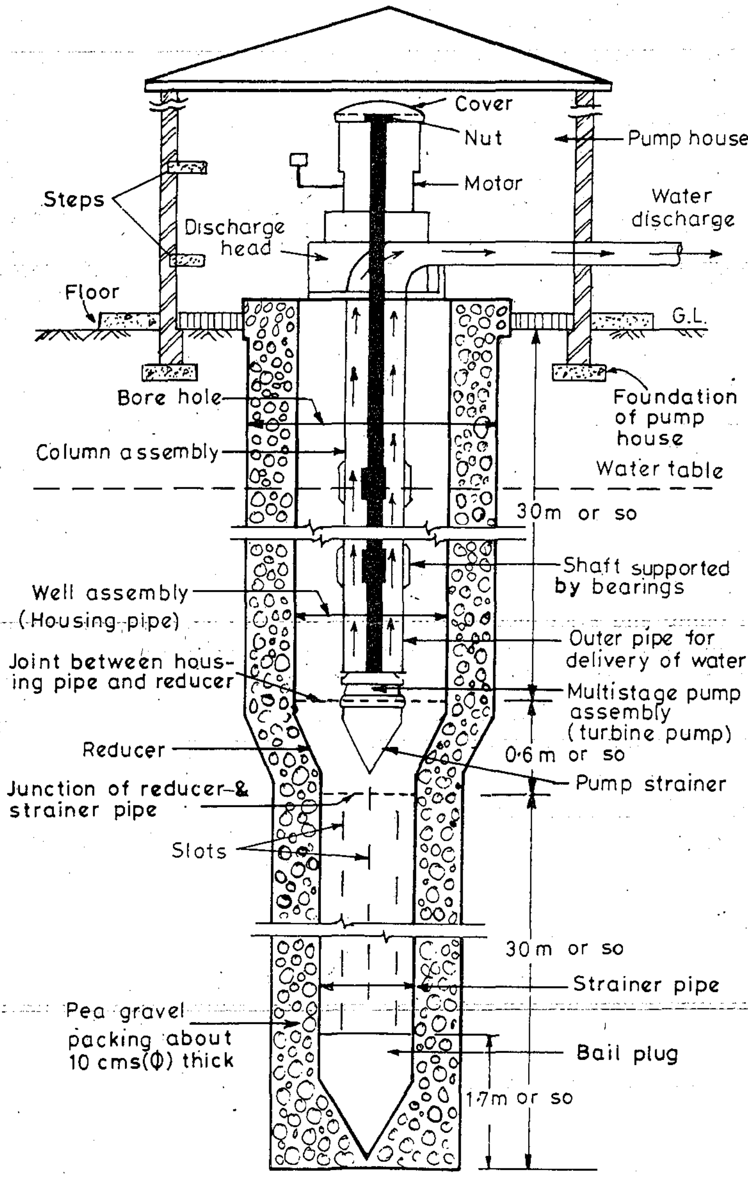


Fig. 16.59. Section of a tubewell using a turbine type of a bore hole pump.

Two types of bore hole pumps, *i.e.* (*i*) *submersible pumps* ; and (*ii*) *turbine type of pumps*, are available. In **submersible pumps,** the motor and the pump are both attached together and lowered inside the bore; whereas in a **turbine type**, the pump is driven by a direct coupled electric motor of a vertical shaft type, and, is placed at the top of the line shaft at the ground level. The necessity of constructing a sump well is thus completely avoided, which may make this arrangement a cheaper and a better alternative to the monoblock centrifugal pump, although the cost of such a pump is higher, and lowering by chain and pulley is difficult. Even among these two types of available pumps, submersible pump costs less than that of a turbine type. A section of a tubewell using turbine pump installation is shown in Fig. 16.59. A submersible pump is also shown in Fig. 16.60.

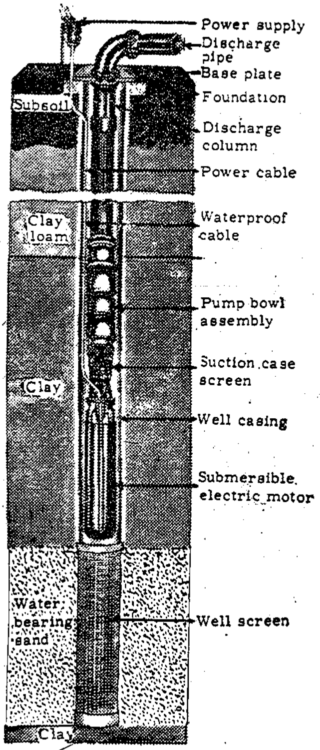
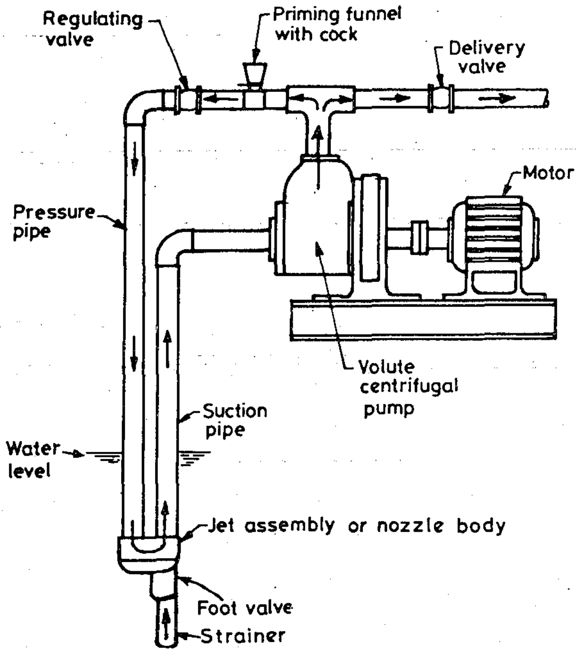


Fig. 16.60. A submersible pump.

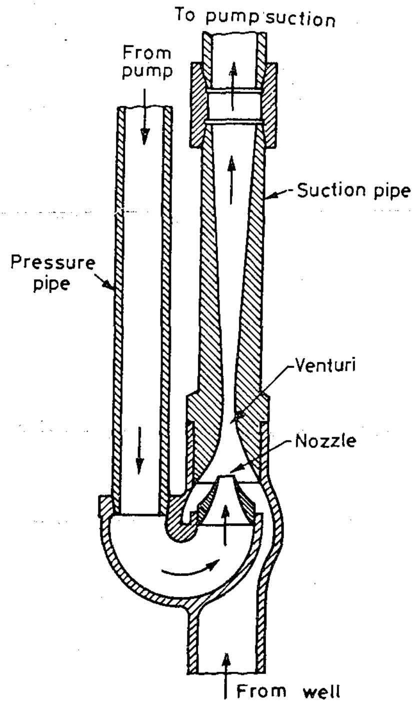
(3) *Jet Pumps*. Jet pumps are also sometimes used these days for lifting water from the smaller tubewells installed for individual domestic supplies, where the water is not available within 8 m suction lift, thereby not permitting the use of the monoblock centrifugal pumps. Jet pumps can be used for suction lift of 6 to 30 m or so. A **jet pump** consists of a combination of a centrifugal pump and a jet mechanism or ejector. In this assembly, the motor and the pump constitute a small unit like a centrifugal monoblock pump, and is placed at the ground level. A nozzle and a venturi assembly connected with the pressure pipe is joined with the main well pipe, as shown in Fig. 16.58. The pressure pipe is connected on the outlet side of the pump along with the delivery pipe. A portion of the high pressure water thus returns through the pressure pipe to activate the nozzle in the ejector. The nozzle is shaped so that it smartly and abruptly reduces the area through which the flow must pass, thereby increasing the velocity of flow. This creates a pressure area around the venturi, which draws more water from the well. The vacuum created by the impeller of the pump placed at the ground surface, draws the flow through the suction pipe. The water is pumped out through the delivery pipe at the desired pressure. The additional supply of water which is obtained from the well is discharged past the control valve, while the volume required for producing the flow is recirculated through the pressure pipe. The control valve is set to maintain the necessary pressure to produce the flow at the existing pump head.

Jet pumps cannot be started until the whole system is filled with water. No priming is however required with the installation of foot valve, as in a monoblock pump. The discharge of the jet pumps is about 20 litres per minute against the head of 15–30 m above the ground level.

Jet pumps are, however, generally not used for irrigation tubewells, because of their low efficiency, which generally is of the order of 35% as compared to 66–85% for the



(*a*). Jet pump installation.



(*b*). Sectional view of the jet assembly of the jet pump.

Fig. 16.61

other types of pumps. Moreover, jet pumps are not easily adapted to the locations where ground water levels are subject to large seasonal fluctuations, or where severe corrosion or incrustation may cause enlargement or blocking of the jet nozzles.

(*v*) **Horse power of the pump motor**

$$\text{The power of the motor} = \frac{\gamma_w \cdot QH}{\eta} \text{ Nm/s} \quad (i.e. \text{ watts})$$

$$\text{H.P. (metric)} = \frac{\text{watts}}{735} = \frac{\gamma_w \cdot QH}{735\,\eta} \qquad ...(16.47)$$

where Q = Discharge to be delivered in cumecs

H = Total lift, *i.e.* the head against which the motor has to work, in metres

γ_w = Unit weight of water in N/m^3

η = Efficiency of the pump set, and is generally taken as 65% (*i.e.* 0.65) for centrifugal or bore hole pumps.

The total head (H) against which the motor has to work consists of :

(*a*) Maximum depth of water level below the ground level.

(*b*) Maximum depression head.

(*c*) Velocity head.

(*d*) Frictional losses in the tube pipe, including the losses at bends and strainer openings.

The frictional loss in the pipe (h_f) may be calculated by using the equation

$$h_f = \frac{f' \cdot l \cdot V^2}{2gd} \quad \text{...(16.48)}$$

where f' = Coefficient of pipe friction which is generally taken to be 0.024

l = Total length of the pipe line, including the vertical as well as horizontal lengths

V = Velocity of flow in the pipe

d = Diameter of the pipe line.

The losses at the bends and those at the entry of the strainer openings are generally taken to be equal to 25 to 30% of the frictional losses in the pipe.

Knowing the total value of H, *i.e.* the head against which the motor has to work ; the horse power of the motor can be easily calculated.

Example 16.14. *Design a tubewell to deliver 33,000 gallons per hour at a depression head of 5 m. The average water level is 10 m below the ground in October and 15 m in July. The geological investigation has yielded the following results at the site of boring :*

Depth	*Type of Strata*
0 to 5 m	*Surface clay*
5 to 20 m	*Very fine sand*
20 to 30 m	*Clay with Kankar*
30 to 50 m	*Coarse sand*
50 to 60 m	*Clay*
60 to 70 m	*Medium sand*
Below 70 m	*Clay with sand stone.*

Solution.

(*i*) **Design of the well pipe**

Assume the velocity of flow in the tube = 2.5 m/sec

Discharge required = 33,000 gallons per hour = 33,000 × 4.55 litres per hour

$$= \frac{33{,}000 \times 4.55}{1{,}000} \text{ cubic metres per hour}$$

$$= \frac{33 \times 4.55}{60 \times 60} \text{ cubic metres per sec. (}i.e.\text{ cumecs)} = \mathbf{4.17 \times 10^{-2}\ cumecs}$$

Area of the tube or well pipe

$$= \frac{\text{Discharge}}{\text{Velocity}} = \frac{4.17 \times 10^{-2}}{2.5} \text{ m}^2 = 167 \text{ cm}^2$$

Diameter of tube $(d_w) = \sqrt{\frac{4}{\pi} \cdot A} = \sqrt{\frac{4}{\pi} \times 167} = 14.6 \text{ cm}$

Use 15 cm dia pipe. The actual velocity of flow in the pipe of 15 cm dia

$$= \frac{4.17 \times 10^{-2}}{\frac{\pi}{4}(0.15)^2} = \mathbf{2.36 \text{ m/sec}}$$

(*ii*) **Size of the bore hole.** The size of the bore hole should be 5 cm more than that of the pipe. Hence, use 20 cm dia bore hole.

(*iii*) **Length of the strainer**. From the geological investigation report, it is evident that the main aquifers are confined between clay strata. Hence, the discharge formula for confined stratum will be used to work out the strainer length. Using equation (16.20), we have

$$Q = \frac{2\pi KHs}{2.3 \log_{10} \frac{R}{r_w}}$$

or

$$H = \frac{2.3\, Q \log_{10} \frac{R}{r_w}}{2\pi K.s.}$$

where $Q = 4.17 \times 10^{-2}$ cumecs

r_w = Radius of the tube $= \frac{0.15}{2} = 0.075$ m

s = Depression head = 5 m (given)

R = 350 m (assumed)

K = Permeability coefficient for sand

$= 0.04 \text{ cm/sec} = 0.04 \times 10^{-2}$ m/sec

(assumed — Table 16.2)

$$\therefore \quad H = \frac{2.3 \times 4.17 \times 10^{-2} \log_{10} \frac{350}{0.075}}{2 \times 3.14 \times 0.04 \times 10^{-2} \times 5}$$

$$= \frac{2.3 \times 4.17 \times 3.669}{2 \times 3.14 \times 0.04 \times 5} = 28.01 \text{ m; say } 28 \text{ m}$$

Hence, use the length of the strainers = **28 m.**

Check for the entrance velocity (v_e)

Using eqn. (16.46) : $h = \frac{Q}{A_0 \cdot v_e}$, we get

$$28 = \frac{4.17 \times 10^{-2}}{(\pi \times 0.15 \times 15\%) \times (v_e)}$$

(assuming 15% opening area in the screens)

or $$v_e = 0.0197\,\text{m/s} = 1.97\,\text{cm/s} \le 2\,\text{cm/s (O.K.)}$$

Note : If v_e is found to be more then the safe entrance velocity of 2 to 3 cm/s, then either the dia of the screen or the length of the screen shall have to be increased. In case this is not practically feasible, then the percentage of opening area shall have to be increased from 15% to about 20 to 25% (preferably 20%).

The selected 15 cm dia screen in 19 m length can now be provided in the coarse sand aquifer (lying at depth between 30 to 50 m), and 9 m length can be provided in the medium sand aquifer (lying at depth between 60 to 70 m), so as to provide a total 28 m screen length. In both the cases, 0.5 m aquifer depths shall be left unscreened at either ends.

Note : This arrangement will lead to the screening of 95% of the aquifer depth,as against the preferred value of 90%. Even this anomaly can be corrected by using a screen of 20 cm dia, and recalculating the length of the screen.

(*iv*) **Type of pump required.** Since the fluctuations of watertable (*i.e.* 15 m– 10 m= 5 m) plus the depression head (5 m) total to be 10 m, which is more than 6 to 8, centrifugal pump is out of question. Bore hole type of pump shall, therefore, be used.

(*v*) **Horse Power of the motor**

$$\text{H.P.} = \frac{\gamma_w \cdot QH}{735\eta} \quad \text{in S.I. units}$$

where $Q = 4.17 \times 10^{-2}\,\text{m}^3/\text{s}$

$\eta = 0.65$

$\gamma_w = 9.81\,\text{kN/m}^3 = 9.81 \times 10^3\,\text{N/m}^3$

H = Total head against which the motor has to work

= Maximum depth of watertable + Depression head + Velocity head + Losses.

where, (*a*) Maximum depth of watertable = 15 m (given)

(*b*) Maximum depression head= 5 m(given)

(*c*) Velocity dead

$$= \frac{V^2}{2g} = \frac{(2.36)^2}{2 \times 9.81} = 0.29\,\text{m}$$

(*d*) Loss of head due to friction in pipe (h_f) plus losses at bends in strainer, etc. (say 0.25 h_f) where $h_f = \dfrac{f' . l . V^2}{2gd}$

where, $f' = 0.024$

l = Length of pipe line

= Vertical length of pipe including the blind pipe and strainer + Horizontal length of delivery pipe

$$= 70 \text{ m} + 10 \text{ m} = 80 \text{ m}$$

$$\therefore h_f = \frac{0.024 \times \times 80 \times (2.36)^2}{2 \times 9.81 \times 0.15} = 3.64 \text{ m}$$

Losses in strainer and bends etc.

$$= 0.25 \times h_f = 0.25 \times 3.64 = 0.91 \text{ m}$$

$$\text{Total losses} = 3.64 + 0.91 = 4.55 \text{ m}$$

$$H = 15 + 5 + 0.29 + 4.55 = \mathbf{25.84\ m}$$

Hence, $\text{H.P.} = \dfrac{\gamma_w \cdot QH}{735\eta}$

$$= \frac{(9.81 \times 10^3) \times 4.17 \times 10^{-2} \times 25.84}{735 \times 0.65} = 22.1; \ \textbf{say 23 H.P.}$$

Hence, 23 H.P. motor is required.

16.26. Ground Water Prospecting

The term 'ground-water prospecting' means *searching for the ground water.* It not only includes to find out the places where the ground water is available, but also to find out its approximate quantity and quality as well. This job can be done *by carrying out, what are called* groundwater surveys.

These **ground water surveys** or investigations are extremely important in arid regions, where ground water is scarcely available. In such regions, if such surveys are not carried out in advance, and the excavation of wells is undertaken, then everything may come out to be futile, as no sufficient and good quality water may become available for obtaining the required water supplies.

Besides the problem of conducting such surveys for obtaining water supplies, another problem which an engineer may face is to detect whether any ground water would be encountered in underground construction operations. The engineer will also have to find out means and ways to check and control that ground water and the problems created by it.

For both these purposes, investigations would have to be conducted to detect the presence of water at the given region or at the particular site, and to fairly estimate its quality or quantity, or both.

The very first indicator of the presence of groundwater in an arid region, is the presence of *plants and vegetations*, especially the plants that habitually grow in arid regions only when they can send their roots down to the watertable. Such plants are called *phreatophytes.* The type of plant, will also to certain extent, indicate the depth of the watertable. The plants, may also to some extent indicate the quality of the ground water. An idea about the different types of plants growing in a particular region, and their peculiar indications about the presence of ground water, can be gathered from the observant local inhabitants. This type of investigation is purely preliminary, and must be followed and confirmed by *geological* and other *geophysical* surveys and field investigations.

The geological studies by which groundwater may be investigated would comprise of making a detailed *geological mapping* of the area, which may give an accurate knowledge of the existing geological structures, including the lithological character, mode of origin, and petrological features of various underground rocks and strata.

In addition to these usual investigations, however, close attention must be paid to all evidences of ground water. Say for example, records of all the existing wells and bore holes must be examined and correlated. New test boreholes may also be excavated and examined, if needed. By such means, when records become available, it is possible to prepare *contour maps for the groundwater levels* over large areas of the country. These groundwater contour maps provide a means of fairly estimating the quantity of available ground water. This quantity of available ground water may also be estimated by some hydrological methods, which depend on studying the relationships between determing and using the *specific yield* factors for the rocks in question.

'Specific yield' as defined earlier, is the volume of water (expressed as percentage of the total volume of the aquifer) which will be drawn freely from the aquifer. In other words, it will be slightly less than the porosity, as some water is retained in the aquifer by molecular or capillary forces. For the measurement of specific yield, various methods have been suggested; some of which can be utilised in a laboratory on samples of material encountered in the area under investigation, and testing them for their permeability, porosity, etc. Other methods necessitate field observations and pumping tests in these bore holes, to finally calculate the specific yields.

Shock waves or seismic waves produced by artificial explosions have also been used these days, to detect the presence of ground water and its depth of occurrence. The principle underlying these seismic methods is that the velocity of the elastic waves (caused by artificial shocks) is different in moist deposits than in the dry formations of the same composition. These *seismic surveys* are, therefore conducted by firing a charge of explosive, near the ground surface, and recording the arrival of the resulting shock waves at a series of geophones, remote from the shock point. Since the velocity of the shock waves depends on the type of formation and the presence of water, it may be possible to detect and estimate the depth of the watertable from the differences in the indicated velocities recorded at several geophones.

The other important type of geophysical investigation which may be performed for groundwater exploration is called the *resistivity surveys.*

Resistivity surveys make use of the fact that water increases the *conductivity* of the rocks, and thereby decreasing their *resistivity*. Hence, if it can be established geologically, that the same rock formation is existing for a certain depth, say 100 m, and by electrical testing it is found that the resistivity is decreasing below say 60 m depth, then it can be easily concluded that the water is present below 60 m depth.

The *resistivity of rocks at various depths* can be calculated on the following principle : If electrodes (generally four) are inserted in the ground and connected in a circuit to a source of electrical energy, the current will flow from one electrode, pass through the ground, and finally leave through the other electrode. The depth to which this current penetrates in the ground, depends upon the distance between the two outer electrodes (it is generally of the order of $\frac{1}{4}$th the distance between electrodes). Thus, it is possible to send the current deeper into the ground by simply increasing the distance between the electrodes (Refer Fig. 16.62). Hence, it should be possible to determine the resistivity of the given formation-rocks, by measuring the passing current in the potentiometer circuit ; and at different depths, by repeating the experiments with different electrode

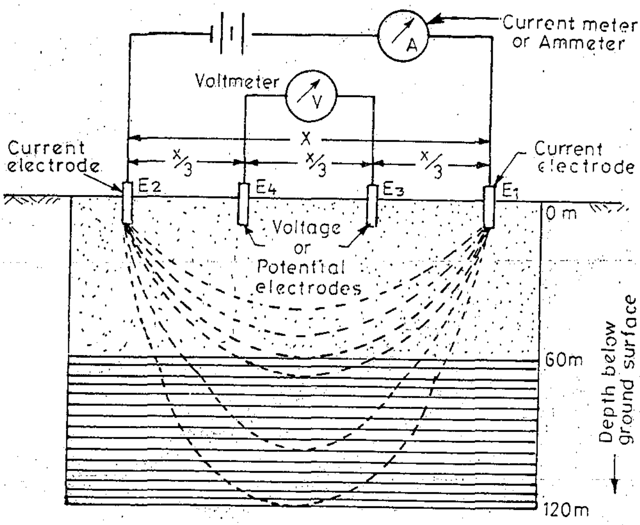


Fig. 16.62. Wenner's Resistivity method for Ground water exploration.

$$\text{Resistivity} = \frac{\text{Voltage}}{\text{Current}} \times 2\pi \left(\frac{x}{3}\right)$$

spacings. The resistivity variations with depth can then be plotted easily, and the variations studied along with geological or stratigraphical* knowledge of the existing rock formations, as pointed out above.

Say for example, in Fig. 16.63, the resistivity is decreasing below 60 m depth and up to 120 m depth; whereas let the geology of the area confirms the existence of the same type of rock (say Sandstone) up to this 120 m depth; then it easily gives the inference that water is existing between 60 m and 120 m depth.

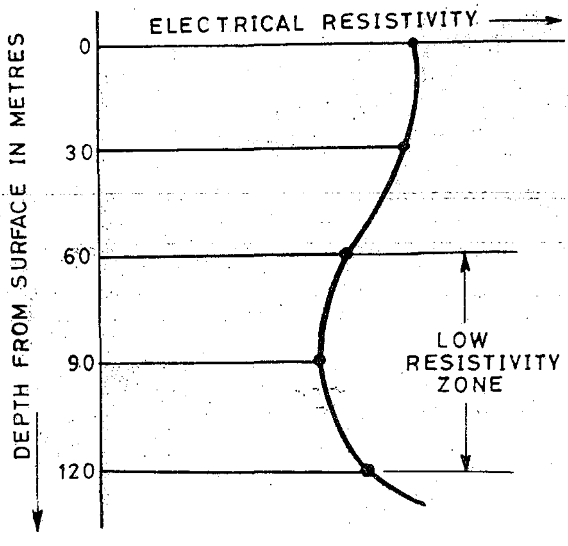


Fig. 16.63. Variation of Resistivity with depth.

Both seismic and resistivity surveys should be made and interpreted by persons who are fully trained in their work. In fact, neither of these two methods, specifically locates the groundwater, but merely indicate discontinuities which may bound an aquifer. With a few test holes as control points, however, large areas may be surveyed quite rapidly by seismic or resistivity methods. The resistivity data may also give an indication of the chemical quality of the groundwater, since dissolved salts reduce the resistivity of water.

* historical geology.

Electrical testing done in oil wells, and their recording called *electrical logs* or *resistivity logs*, are also often useful in ground water studies.

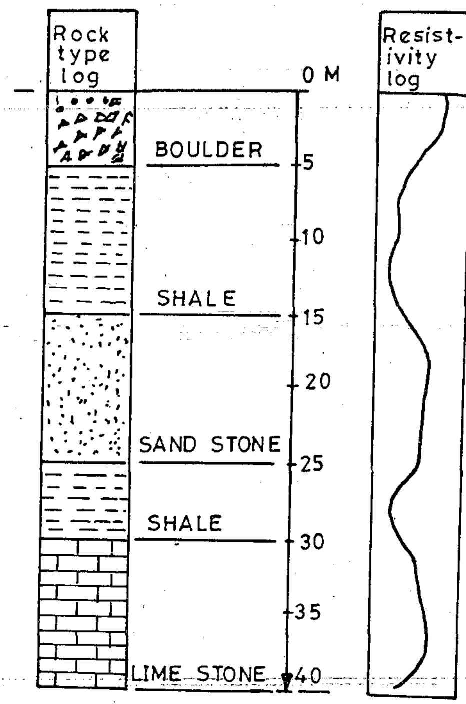
16.26.1. Logs or Recording of Bore-hole Data.

Logs may be defined as the records of the sub-surface investigations, and provide useful information regarding the nature and properties of the materials occuring at various depths below the ground surface. These records may be in the form of mere tables or graphic plots with symbolic descriptions. The data making the basis of these records may be obtained by different methods, and accordingly there are many types of bore-hole logs. A few examples of such bore hole logs are described below :

(*i*) *Geological logs* or *well logs*, as pointed out in article 16.25.6.4, are those which represent the type of the strata existing at different depths, and encountered during direct digging or boring of the wells. [Refer Fig 16.64 (*a*)]

(*ii*) *Resistivity logs* are those which indicate the values of electrical resistivity of the rocks at different depths, determined by special techniques from surface downwards in bore holes. [Refer Fig. 16.61 (*b*)].

(*iii*) *Sonic logs* are those which indicate the values of velocities of compressional waves at different depths.



(*a*) Rock log or geological log. (*b*) Resistivity log.

Fig. 16.64. Bore-hole logs.

(*iv*) *Thermal logs* are those which indicate variation in temperature with depth, as determined directly with the increasing depth.

(*v*) *Radiometric logs*, similarly, indicate the variations of radioactivity with depth.

Borehole logging plays an important role in all the major programmes of water exploration. Most of the properties, investigated for, and represented in bore hole logs, are highly sensitive to water content of the rocks, and as such, when interpreted in that light, may yield useful information.

16.27. Advantages and Disadvantages of Tube-well Irrigation Over Canal Irrigation

Advantages :

(*i*) Isolated lands which cannot be served by canals can be easily irrigated by tube-wells.

(*ii*) Cultivators can have their own private tube-wells and thus have not to depend on the government owned canal waters.

(*iii*) Canal irrigation projects require huge funds and considerable time, while wells can be constructed wherever required in a small time and with lesser funds.

(*iv*) The supply of water from a well can be started as soon as required and can be stopped at any moment, thus taking advantage of momentary rainfall.

(*v*) Tube-well channels are of shorter length and generally lined, thus resulting in lesser percolation losses.

(*vi*) Tube-well water is generally sold on a volumetric basis, which results in optimum utilisation of water at the correct time.

(*vii*) Pumping from ground water by means of tube-wells helps in lowering the ground watertable, and thus helps in reducing water logging, which on the other hand, is generally caused by canal irrigation.

(*viii*) With the help of well irrigation, more than one crop may be grown in an year.

(*ix*) The well water which is warmer in cold weather and colder in hot weather is more suitable to crops.

(*x*) The well water is more pure than the canal water, and thus the irrigation scheme may be combined with rural water supply-scheme.

(*xi*) The land acquisition is less for tube-well irrigation.

(*xii*) Unless drought continues for many years, well irrigation does not fail in droughts, while a canal supply may fail in a single drought or at the most in two or three consecutive drought years.

Disadvantages :

(*i*) If the electric supply fails (which generally does in drought years), the pumps of the tube-wells cannot be operated, and hence, the well water cannot be made available to the crops, unless diesel power is available.

(*ii*) Canal irrigation projects are generally combined with flood control and hydropower projects, thus giving added benefits.

(*iii*) The well water which is generally free from silt is not so good from manuring point of view as is the silted canal water.

(*iv*) The tube-well water proves much costlier than the canal water since the well water has to be lifted by pumps.

(*v*) The life of a tube-well is limited.

(*vi*) Frequent breakdown of power and motor parts cause large scale interruptions in the working of tube-wells. So much so that the usual running hours have been estimated to be 3,760 out of annual of 8,760.

Conclusion. Truely speaking, a combined irrigation system is the best. In other workds, we must install a large scale canal irrigation system in the country supported and supplemented by tube-wells throughout. This is what is being followed in India, especially in Punjab and U.P., where irrigation is most intensively as well as extensively practised.

PROBLEMS

1. (*a*) discuss briefly as to how the water is stored into the ground water reservoir. Briefly mention the various zones and importance of the 'zone of saturation' in this connection.

(*b*) Enumerate the different methods by which the ground water is drained and used in our country.

(*c*) What is meant by artificial recharge of ground water ? Enumerate the different methods which are used for this purpose and describe one of them briefly.

2. Define and explain any five of the following terms, as used in connection with ground water :

(*i*) Capillary fringe

(*ii*) Specific yield

(*iii*) Pellecular water

(*iv*) Field capacity

(*v*) Permeability and transmissibility

(*vi*) Aquifers and aquicludes

(*vii*) Non-artesian and artesian wells

(*viii*) Perched aquifers

(*ix*) Storage coefficient

(*x*) Specific capacity of wells

(*xi*) Well loss.

3. (*a*) Distinguish between non-equilibrium and equilibrium conditions in an aquifer from which water is withdrawn through a well. Explain when the above conditions can be expected in an aquifer. (*Engg. Services, 1970*)

(*b*) Derive a formula for discharge of well in a homogeneous artesian aquifer assuming equilibrium flow conditions. (*Engg. Services, 1970*)

(*c*) In an artesian aquifer, the drawdown is 1.5 m at a radial distance of 8 metres from a well after two hours of pumping. On the basis of Theis' non-equilibrium equation, determine the pumping time for the same drawdown (1.5 metre) at radial distance of 20 metres from the well.

[**Hint.** Follow example 16.7] [**Ans.** 12.5 hours]

4. Distinguish with sketches if necessary, the difference between an unconfined and a confined aquifer. Derive a formula for discharge of a well in a homogeneous unconfined aquifer assuming equilibrium flow condition; state the assumptions on which the formula is based. (*Engg. Services, 1969*)

5. (*a*) What is Dupuit's equation ? State the assumptions that enter in its development. Explain the Theis' formula. What is well function? (*Engg. Services, 1967*)

(*b*) Explain the following terms :

(*i*) Aquifer,

(*ii*) Aquiclude,

(*iii*) Capillary fringe,

(*iv*) Equilibrium drawdown,

(*v*) Specific yield, and

(*vi*) Perched watertable. (*Engg. Services, 1967*)

6. (*a*) Explain the following :

(*a*) Vadose water,

(*b*) Pellicular water;

(*c*) Artificial recharge;

(*d*) Non-equilibrium equation; and

(*e*) Coefficient of storage. (*Engg. Services, 1968*)

(*b*) A 30 cm dia well penetrates 20 m below the static watertable. After 24 hours of pumping at 5000 litres per minute, the water level in a test well at 100 m away is lowered by 0.5 m, and in a well at 30 m away, the drawdown is 1 m. What is the transmissibility of the aquifer?

[**Hint.** Follow example 16.5]

7. (*a*) Explain :

(*i*) Specific retention of a soil

(*ii*) Specific yield of an aquifer

(*iii*) Storage coefficient of an aquifer

(*iv*) Specific capacity of a well. (*Engg. Services, 1972*)

(*b*) Step drawdown test was carried out in a well constructed to give a yield of 2300 litres per minute. The data obtained is given below. Determine the well loss and efficiency of the well.

Yield in l.p.m.	*Drawdown in metres*
1230	3.72
1840	5.65
2460	7.62
3070	9.62
4080	13.07
6130	20.21

(*Engg. Services, 1972*)

8. (*a*) State Dupuit's assumptions for obtaining general equations governing ground water flow. Derive an expression for the confined aquifer. How can the expression be used to evaluate the aquifer permeability?

(*b*) A well penetrating an aquifer which is underlain and overlain by inpermeable layers was tested with a uniform discharge of 1000 litres/min. The steady state drawdowns measured in two observation wells which were at 1 m and 10 m radial distances from the centre of the pumped well were 13.40 m and 4.2m, respectively. Determine the hydraulic properties of the aquifer, if its saturated thickness is 10 m.

[**Ans.** Transmissibility = 0.0398 m^2/min.
Hydraulic conductivity = 0.00398 m/min.]

9. A 10 cm diameter well was pumped at a uniform rate of 500 litres/min., while observations of drawdown were made in an observation well located at a distance of 50 m from the well. The original head of water, measured from the top of the impervious layer was 25 m. The hydraulic conductivity of the aquifer was 1.83×10^{-3} m/min. Determine the drawdown at the face of the well, using Dupuit-Thiem equation, and assuming that the flow to the unconfined aquifer is under steady state. [**Ans.** 20 m]

10. Write short notes on any four of the following:

(*i*) Infiltration wells and infiltration galleries.

(*iii*) Darcy's law for measuring velocity of ground water flow.

(*iv*) Permeability and transmissibility and their relationship.

(*v*) Measuring the yield of underground water sources.

(*vi*) Surface of seepage and free surface curve.

(*viii*) Cavity formation in dug wells.

(*ix*) Wells and Tubewells

11. (*a*) Differentiate between shallow dug wells and deep dug wells. How are dug well constructed?

(*b*) Enumerate the methods which are used for determining the yield of dug wells. Discuss briefly any one of these methods.

12. What is meant by tubewells ? What are their types ? Describe the widely used type of tubewell with a neat sketch. What are the approximate values of the average yield and depth of such a tubewell ?

13. (*a*) Enumerate the different methods which are used for drilling tubewells. Discuss any one of these methods in details.

(*b*) Discuss briefly the design principles involved in the design of a strainer type of a tubewell.

(*c*) What is the average life of tubewells and what are the reasons for their failure ? What remedies will you suggest for increasing their life ?

14. Write short notes on any four of the following :

(*i*) Shallow and Deep tubewells;

(*ii*) Artificial recharge of groundwater;

(*iii*) Groundwater prospecting;

(*iv*) Development of tubewells;

(*v*) Well screens;

(*vi*) Gravel pack slotted pipe tubewells;

(*vii*) Design of gravel pack;

(*viii*) Surface and subsurface sources of water.