SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – I – GROUND WATER ENGINEERING – SCI1602

HYDROGEOLOGICALPARAMETERS

Groundwater engineering, another name for hydrogeology, is a branch of engineering which is concerned with groundwater movement and design of wells, pumps, and drains. The main concerns in groundwater engineering include groundwater contamination, conservation of supplies, and water quality

Hydrogeology (*hydro*- meaning water, and -*geology* meaning the study of the Earth) is the area of geology that deals with the distribution and movement of groundwater in the soil and rocks of the Earth's crust (commonly in aquifers). The terms **groundwater hydrology**, **geohydrology**, and **hydrogeology** are often used interchangeably.

Water is an essential requirements for all forms of the life and is considered as integral part of the living organisms life. GOD has gifted our universe with bulk amount of this valuable substance in different forms such as

1. Rivers
2. Lakes
3. Natural springs
4. Rain
5. Snow
6. Glaciers
7. Aquifers etc

During the early era apart from drinking purpose water was usually used for general usage such as agriculture, washing clothes, pots etc but With the passage of time the use of water get increased and human being started using it in different fields such as:

1. Industries
2. Preparation of food stuff
3. Medicines
4. Steam engines
5. Vehicles
6. Paper industries and so on

About 70% portion of our planet earth is consists of water while the rest 30% is consists of dry land. Apart from such a big amount of water there is also massive amount of underground water reservoirs but the main difficulty in using of this water is the difficulty to access it. Due to vast advancement of science and technology the demand for water is also increased to very high level then before it was and causing the demand for underground water usage.

The ground water reservoirs are much more pure and safe the usual water resources available at the earth's surface. Ground water constitute an integral part of the human's life and now time demands to bring it to use so that we can fulfill our fast growing demand of water. Following are the different types of ground water reservoirs and the their details.

Global distribution of water and groundwater availability in India

The total volume of water in the earth is approximately 13.6 $x10^8$ km^3. Out of which only 8.336 $x10^6$ km^3 is stored in the form of groundwater, which is approximately 0.62% of the total water of the Earth. Out of 8.336 $x10^6$ km^3 of groundwater, approximately 4.168 $x10^6$ km^3 of groundwater is in the upper 0.8 km of the earth's crust and rest 4.168 $x10^6$ km^3 is in the deep strata,

i.e. beyond 0.8 km from the ground level. Approximately 97.5% of the global water is in the oceans and 2.5% is fresh water (Fig. 2.1). Out of 2.5% of freshwater, 68.7% is in glaciers, 30.1% is groundwater, 0.8% is in permafrost and 0.4% is surface and atmospheric water. Out of the 0.4% of surface and atmospheric water, 67.4% is in freshwater lakes, 12.2% is in soil moisture, 9.5% is atmospheric water, 8.5% is in wetlands, 1.6% is in river and rest 0.8% is biological water.

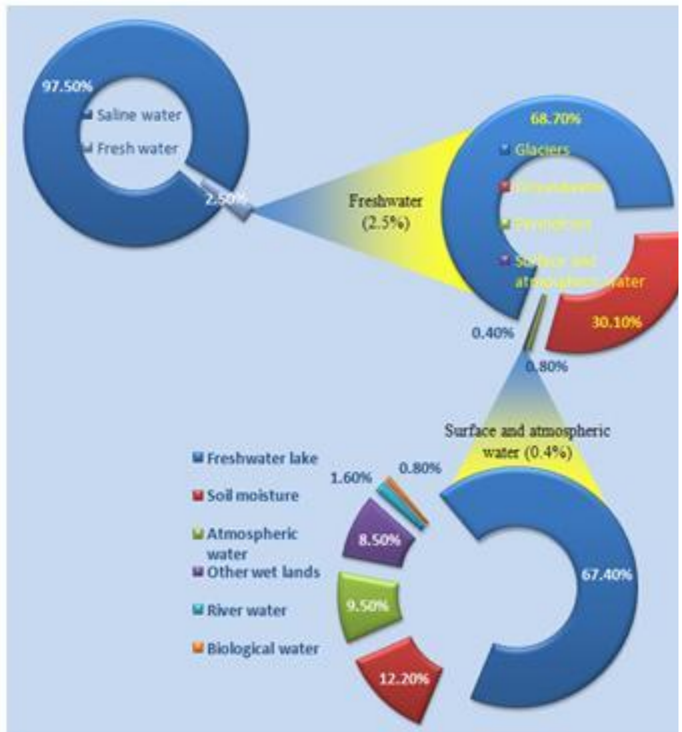


Fig. 2. 1 Global water distribution

As per ministry of water resources, Govt. of India, the total annual replenishable groundwater resource of the country is estimated as 433 billion cubic meter (BCM). Out of 433 BCM, around 34 BCM is the natural outflow. As such, the net annual groundwater resource of the country is around 399 BCM. Total annual withdrawal of groundwater is estimated as 231 BCM. Out of this 231 BCM of annually withdrawn, 213 BCM is used for irrigation and rest 18 BCM is withdrawn for domestic uses. Fig. 2.2 shows the annual groundwater resources of the country.

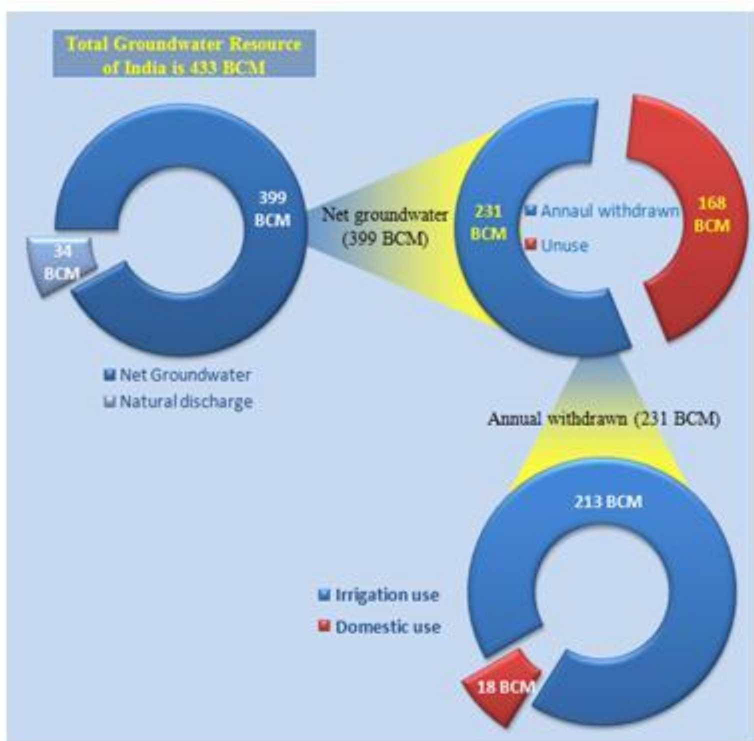


Fig. 2. 2 Groundwater resource of India

A major component of precipitation that falls on the earth surface eventually enters into the ground by the process of infiltration. The infiltrated water is stored in the pores of the underground soil strata. The water which is stored in the pores of the soil strata is known as groundwater. Therefore, the groundwater may be defined as all the water present below the earth surface and the groundwater hydrology is defined as the science of occurrence, distribution and movement of water below the earth surface. In this section of hydrology, we generally deal with the water that is stored in the voids of the soil below the earth surface and also their interaction with the water that are present above the earth surface. When all the pores of a soil matrix are filled with water, we call that the soil is in the state of saturation. We used the term *porosity* to quantify the amount of voids space available in a soil matrix. Porosity is defined as the ratio of volume of voids to the total volume of the soil matrix. The porosity is expressed as,

$$\eta = \frac{V_v}{V_T} \tag{1.1}$$

Where V_v is the volume of void and V_T is the total volume of soil solid.

Water entered into the earth surface also moves from one place to another through the pores of the underground strata. This is known as subsurface flow. The subsurface flow is three dimensional and can be estimated using Darcy's law. Detail discussion about Darcy's law is presented in Module-2 of this course. The subsurface water also comes out to the earth surface as spring, river base flow, *etc* . and also goes back to the atmosphere by the process of

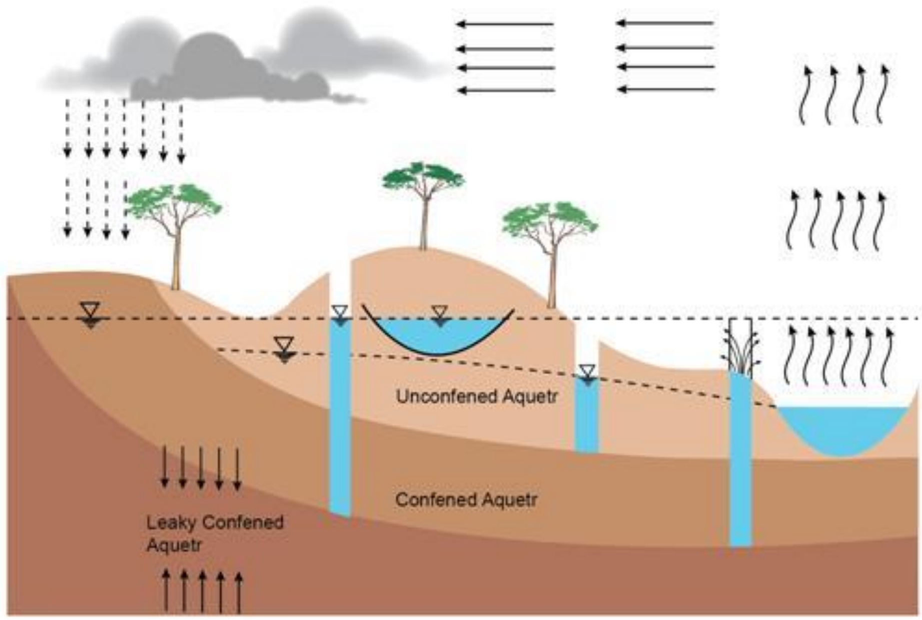


Fig.1.1 Hydro-geological cycle

evapo-transpiration. Thus this is a continuous process of recycling of water from the atmosphere down to the soil below the earth surface and back to the atmosphere again. This cycle is called hydro-geological cycle (Fig. 1.1). In this process of recycling, the water molecules spent some time under the earth surface. The average length of time spent by the water molecules under the earth surface is known as residence time of groundwater . The residence time can be calculated as,

$$t_r = \frac{V_{gr}}{q_{av}} \tag{1.2}$$

Where, t_r is the residence time for groundwater, V_{gr} is the volume of groundwater and q_{av} is the inflow or outflow at steady rate.

Hydrogeological Formations and Groundwater

The behavior of ground water in the Indian sub-continent is highly complicated due to the occurrence of diversified geological formations with considerable lithological and chronological variations, complex tectonic framework, climatological dissimilarities and various hydrochemical conditions. Studies carried out over the years have revealed that aquifer groups in alluvial / soft rocks even transcend the surface basin boundaries. Broadly two groups of rock formations have been identified depending on characteristically different hydraulics of ground water, viz. Porous formations and Fissured formations.

Porous Formations :

Porous formations have been further subdivided into Unconsolidated and Semi – consolidated formations.

Unconsolidated Formations

The areas covered by alluvial sediments of river basins, coastal and deltaic tracts constitute the unconsolidated formations. The hydrogeological environment and ground water regime conditions in the Indo-Ganga-Brahmaputra basin indicate the existence of potential aquifers having enormous fresh ground water resources.

Semi-Consolidated Formations

The semi-consolidated formations normally occur in narrow valleys or structurally faulted basins. The Gondwanas, Lathis, Tipams, Cuddalore sandstones and their equivalents are the most extensive productive aquifers. Under favourable situations, these formations give rise to free flowing wells. In select tracts of northeastern India, these water-bearing formations are quite productive. The Upper Gondwanas, which are generally arenaceous, constitute prolific aquifers.

Fissured Formations (Consolidated Formations)

The consolidated formations occupy almost two-third of the country. The consolidated formations, except vesicular volcanic rocks, have negligible primary porosity. From the hydrogeological point of view, fissured rocks are broadly classified into four types viz. Igneous and metamorphic rocks excluding volcanic and carbonate rocks, Volcanic rocks, Consolidated sedimentary rocks and Carbonate rocks.

Igneous and Metamorphic Rocks Excluding Volcanic and Carbonate Rocks

The most common rock types are granites, gneisses, charnockites, khondalites, quartzites, schists and associated phyllites, slates, etc. These rocks possess negligible primary porosity but develops secondary porosity and permeability due to fracturing and weathering. Ground water yield also depends on rock type and possibly on the grade of metamorphism.

Volcanic Rocks

The predominant types of the volcanic rocks are the basaltic lava flows of Deccan Plateau. The contrasting water bearing properties of different flow units controls ground water occurrence in Deccan Traps. The Deccan Traps have usually poor to moderate permeabilities depending on the presence of primary and secondary porespaces.

Consolidated Sedimentary Rocks excluding Carbonate rocks

Consolidated sedimentary rocks occur in Cuddapahs, Vindhyans and their equivalents. The formations consist of conglomerates, sandstones, shales, slates and quartzites. The presence of bedding planes, joints, contact zones and fractures control the ground water occurrence, movement and yield potential.

Carbonate Rocks

Limestones in the Cuddapah, Vindhyan and Bijawar group of rocks are the important carbonate rocks other than the marbles and dolomites. In carbonate rocks, the circulation of water creates solution cavities, thereby increasing the permeability of the aquifers. The solution activity leads to widely contrasting permeabilities within short distances[3].

Groundwater Potential – A Glance

Several attempts have been made to assess the ground water resources in the country. The National Commission on Agriculture (1976), assessed the total ground water of the country as 67 m. ha m, excluding soil mixture. The usable ground water resource was assessed as 35 m. ha m of which 26 m. ha m was considered as available for irrigation.The first attempt to estimate the ground water resources on scientific basis was made in 1979 when a High Level Committee, known as Ground Water over Exploitation Committee was constituted by Agriculture Refinance and Development Corporation (ARDC). Based on the norms for ground water resources computations recommended by this committee, the State Governments and the Central Ground Water Board computed the gross ground water recharge as 46.79 m. ha m and the net recharge (70% of the gross) as 32.49 m. ha m.

Norms recommended by the Ground Water Estimation Committee (1984) are currently utilized by the Central Ground Water Board and the State Ground Water Departments to compute the ground water Resources.Based on the recommendations of this committee, the annual replenishable ground water resources in the country work out to be 45.33 m. ha m. Keeping a provision of 15% (6.99 m. ha m) for drinking, industrial and other uses, the utilisable ground water resource for irrigation was computed 38.34 m. ha m per year.

The ground water resources of the country have been estimated for freshwater based on the guidelines and recommendations of the GEC-97. The total annual replenishable ground water resources of the country have been estimated as 431 billion cubic meter (BCM). Keeping 35 BCM for natural discharge, the net annual ground water availability for the entire country is

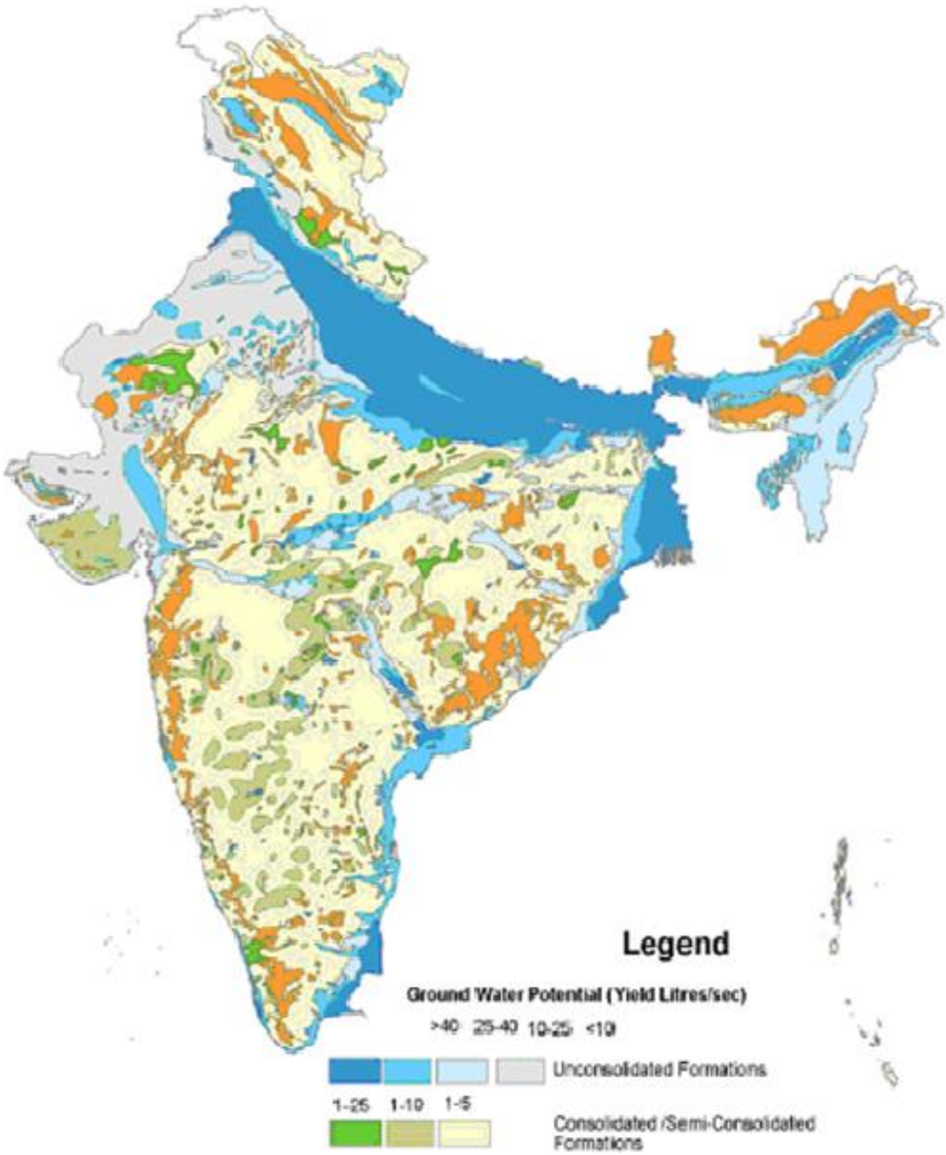


Fig 3.1 hydrogeological Map of India(Source: www.cgwb.in)

396 BCM. The annual ground water draft is 243 BCM out of which 221 BCM is for irrigation use and 22 BCM is for domestic & industrial use.

Haryana, Punjab and Rajasthan receive less than 40 cm annual rainfall and are deficient in surface water resources. As such, these states exploit more than 85 per cent of the available ground water for irrigation. Large scale exploitation of ground water is done with the help of tube wells. Gujarat, adjoining Rajasthan, also receives less rainfall and has to depend upon ground water resources. This state has developed over 55 per cent of her ground water resources. Uttar Pradesh and Bihar in the Ganga valley are rich fertile tracts where intensive irrigation is required to sustain agriculture. In the south, Tamil Nadu also has high level of 64.43 per cent of ground water development. Here, ground water is primarily used to irrigate the rice crop.

Most of the north-eastern hill states like Assam, Arunachal Pradesh, Manipur, Meghalaya, Mizoram, Nagaland and Sikkim have very low level of ground water development. Goa also receives sufficient rainfall and surface water resources are enough to meet the requirement. Therefore, the ground water resources are not much exploited. Hilly and mountainous terrain in Jammu and Kashmir and Himachal Pradesh is not much favorable for developing ground water resources [4,5].

Table 3.1 Groundwater Potential in India

Sl. No.	States / Union Territories	Annual Replenishable Ground Water Resource					Natural Discharge during non-monsoon season	Net Annual Ground Water Availability	Annual Ground Water Draft			Projected Demand for Domestic and Industrial uses up to	Ground Water Availability for future irrigation	Stage of Ground Water Development (%)
		Monsoon Season		Non-monsoon Season		Total			Irrigation	Domestic and industrial uses	Total			
		Recharge from rainfall	Recharge from other sources	Recharge from rainfall	Recharge from other sources									
	States													
1	Andhra Pradesh	16.04	8.93	4.20	7.33	36.50	3.55	32.95	13.88	1.02	14.90	2.67	17.65	45
2	Arunachal Pradesh	1.57	0.00009	0.98	0.0002	2.56	0.26	2.30	0.0008	0	0.0008	0.009	2.29	0.04
3	Assam	23.65	1.99	1.05	0.54	27.23	2.34	24.89	4.85	0.59	5.44	0.98	19.06	22
4	Bihar	19.45	3.96	3.42	2.36	29.19	1.77	27.42	9.39	1.37	10.77	2.14	16.01	39
5	Chhattisgarh	12.07	0.43	1.30	1.13	14.93	1.25	13.68	2.31	0.48	2.80	0.70	10.67	20
6	Delhi	0.13	0.06	0.02	0.09	0.30	0.02	0.28	0.20	0.28	0.48	0.57	0.00	170
7	Goa	0.22	0.01	0.01	0.04	0.29	0.02	0.27	0.04	0.03	0.07	0.04	0.19	27
8	Gujarat	10.59	2.08	0.00	3.15	15.81	0.79	15.02	10.49	0.99	11.49	1.48	3.05	76
9	Haryana	3.52	2.15	0.92	2.72	9.31	0.68	8.63	9.10	0.35	9.45	0.60	-1.07	109
10	Himachal Pradesh	0.33	0.01	0.08	0.02	0.43	0.04	0.39	0.09	0.03	0.12	0.04	0.25	30
11	Jammu & Kashmir	0.61	0.77	1.00	0.32	2.70	0.27	2.43	0.10	0.24	0.33	0.42	1.92	14
12	Jharkhand	4.26	0.14	1.00	0.18	5.58	0.33	5.25	0.70	0.38	1.06	0.56	3.99	20
13	Karnataka	8.17	4.01	1.50	2.25	15.93	0.63	15.30	9.75	0.97	10.71	1.41	6.48	70
14	Kerala	3.79	0.01	1.93	1.11	6.84	0.61	6.23	1.82	1.10	2.92	1.40	3.07	47
15	Madhya Pradesh	30.59	0.96	0.05	5.59	37.19	1.86	35.33	16.08	1.04	17.12	1.74	17.51	48
16	Maharashtra	20.15	2.51	1.94	8.36	32.96	1.75	31.21	14.24	0.85	15.09	1.51	15.10	48
17	Manipur	0.20	0.005	0.16	0.01	0.38	0.04	0.34	0.002	0.0005	0.002	0.02	0.31	0.65
18	Meghalaya	0.79	0.03	0.33	0.005	1.15	0.12	1.04	0.00	0.002	0.002	0.10	0.94	0.18
19	Mizoram	0.03	0.00	0.02	0.00	0.04	0.004	0.04	0.00	0.0004	0.0004	0.0008	0.04	0.90
20	Nagaland	0.28	0.00	0.08	0.00	0.36	0.04	0.32	0.00	0.009	0.009	0.03	0.30	3
21	Orissa	12.81	3.56	3.58	3.14	23.09	2.08	21.01	3.01	0.84	3.85	1.22	16.78	18
22	Punjab	5.98	10.91	1.36	5.54	23.78	2.33	21.44	30.34	0.83	31.16	1.00	-9.89	145
23	Rajasthan	8.76	0.62	0.26	1.92	11.56	1.18	10.38	11.60	1.39	12.99	2.72	-3.94	125
24	Sikkim	-	-	-	-	0.08	0.00	0.08	0.00	0.01	0.01	0.02	0.05	16
25	Tamil Nadu	4.91	11.96	4.53	1.67	23.07	2.31	20.76	16.77	0.88	17.65	0.91	3.08	85
26	Tripura	1.10	0.00	0.92	0.17	2.19	0.22	1.97	0.08	0.09	0.17	0.20	1.69	9
27	Uttar Pradesh	38.63	11.95	5.64	20.14	76.35	6.17	70.18	45.36	3.42	48.78	5.30	19.52	70
28	Uttaranchal	1.37	0.27	0.12	0.51	2.27	0.17	2.10	1.34	0.05	1.39	0.06	0.68	66
29	West Bengal	17.87	2.19	5.44	4.86	30.36	2.90	27.46	10.83	0.81	11.65	1.24	15.33	42
	Total States	**247.87**	**69.51**	**41.84**	**73.15**	**432.43**	**33.73**	**398.70**	**212.37**	**18.05**	**230.41**	**29.09**	**161.06**	**58**
	Union Territories													
1	Andaman & Nicobar	-	-	-	-	0.330	0.005	0.320	0.000	0.010	0.010	0.008	0.303	4
2	Chandigarh	0.016	0.001	0.005	0.001	0.023	0.002	0.020	0.000	0.000	0.000	0.000	0.020	0
3	Dadra & Nagar Haveli	0.059	0.005			0.063	0.003	0.060	0.001	0.008	0.009	0.008	0.051	14
4	Daman & Diu	0.006	0.002	0.000	0.001	0.009	0.0004	0.008	0.007	0.002	0.009	0.003	-0.002	107
5	Lakshadweep	-	-	-	-	0.012	0.009	0.004	0.000	0.002	0.002	-	-	63
6	Pondicherry	0.057	0.067	0.007	0.029	0.160	0.016	0.144	0.121	0.030	0.151	0.031	-0.008	105
	Total Union Territories	**0.138**	**0.075**	**0.012**	**0.031**	**0.597**	**0.036**	**0.556**	**0.129**	**0.052**	**0.181**	**0.050**	**0.365**	**33**
	Grand Total	**248.01**	**69.59**	**41.85**	**73.18**	**433.02**	**33.77**	**399.25**	**212.50**	**18.10**	**230.59**	**29.14**	**161.43**	**58**

Table 3.2 Status of Groundwater quality in various parts of India		
Location	**Impacts on groundwater**	**Authors**
Nalgonda district, Andhra Pradesh, India	Geo-chemical processes and temporal variation of groundwater inthis area are influenced by evaporation processes, ion exchange and dissolution of minerals	Rajesh et al (2012) [6]
Chennai, Tamil Nadu, India	The study area is always under stress due to increasingpopulation, waste water disposal and various geogenic reasons	Krishna kumar et al (2015),Packialakshmi et al,2010,2011 &2012[1,7-9]
Araniyar River Basin, Tamil Nadu	Variation in groundwater quality due to induced anthropogenic activities such as application of fertilizers and uncontrolled groundwater development.	Jasmin and Mallikarjuna,2013, [10]
Nalbari district, Assam, Northeast India	Fluoride concentration was recorded in the range of 0.02–1.56 mg/L due to hydrogeological implications	Sharma et al,2012, [11]
Chromepet, South Chennai, Tamil Nadu	The use of chemicals, such as sodium chloride, sodium sulphate, chromium sulphate etc. during the tanning processes is the major reason for the high concentration of major ions and chromium in groundwater	Brindha and Elango,2012, [12]
Coastal regions of Andhra Pradesh	Significant deterioration of groundwater quality in the coastal region due to anthropogenic and geogenic regions	SubbaRao et al, 2014, [13]
Peri urban area of Delhi	Indiscriminate usage of agrochemicala and waste water irrigation deteriorates the groundwater of peri urban Delhi	Adhikary et al, 2012, [14]
Dindigul District, Tamil Nadu, India	The quality of ground water is affected by surface contaminants sources, mineral dissolution, and evaporation.	Magesh et al, 2013, [15]
Maheshwaram watershed, near Hyderabad, India	Rapid deterioration in the groundwater quality due to increase in built-up land with unsewered sanitation and poultry farms.	Khan et al ,2011, [2]
AgastheeswaramTaluk of Kanyakumari District, Tamil Nadu, India	aquifer is impacted by saline water intrusion or some other anthropogenic activities, like intense agricultural practices and poor sewage management	Srinivas et al, 2014, [16]
Vellore and Krishnagiri districts of Tamil Nadu	Groundwater quality impacte due to Rock dissolution, Agriculture, domestic and other industrial effluence in the study area is responsible for the Cl and Na increase.	ShanmugaSundaram et al, 2015, [17]
Nagapattinam District, Tamil Nadu	Sea water intrusion in non monsoon seasons and irrigation returnflow in monsoon seasons	GnanachandraSamy et al, 2014, [18]
Nagercoil town, Tamil Nadu	aquifers are subject to contamination from sewage effluents and excess use of fertilizer	Srinivas et al 2013, [19]

	and pesticides in agriculture.	
Floodplains of upper Palar River, Tamil Nadu	The groundwater quality is highly affected owing to the seepage of tannery effluents, solid wastes and sewage.	Kuppuraj et al,2010, [20]
Panna District, Central India	Fluride contamination due to geogenic reasons	Pankaj Kumar et al,[21]
Central Gangetic plain around the Ghazipur and Ballia districts, Uttar Pradesh, northern India	Groundwater with high concentrations of geogenic arsenic occurs extensively in the Holocene alluvial aquifers	Tripathi et al, 2009, [22]

Most of the peninsular plateau area is composed of hard rocks and is not much favourable for exploiting ground water resources. Most of the states located in the peninsular plateau area have moderate level of ground water development which varies from 20 to 40 per cent. The major states of this category are Andhra Pradesh, Jharkhand, Karnataka, Kerala, Madhya Pradesh, Orissa, and Maharashtra. It is estimated that in India, 85 per cent of rural and over 50 per cent of urban water supplies depend upon ground water for meeting drinking and domestic water needs. Increasing demand for water in agriculture sector puts heavy strain on our water resources and ground water resources are over-exploited. In some districts of Punjab and Haryana, the ground water level is falling at an alarming rate of over one metre per year.

Groundwater Quality

Quality of groundwater is greatly influenced with hydro geochemical processes of groundwater with surrounding hydro geological formations. These hydrogeochemical processes are responsible for the seasonal, temporal and spatial variations of groundwater chemistry and consequently the quality [23]. The relationship between the hydro geological formations and groundwater status in terms of its quality and quantity has been expressed by several researcherslisted in Table 2. It shows the significant impacts of urbanization and anthropogenic reasons in the degradation of groundwater quality.

Subsurface Water Occurrence

Underground rivers occur only rarely in cavernous limestone. Most groundwater occurs in small pore spaces within rock and alluvium(unconsolidated sediment)

1. Groundwater accumulates over impervious material
2. Water flow through porous medium is slow (range from few centimeters to meters per day)

Porosity of Geological Material

1. Porosity is a parameter which describes the amount of open space in geologic material

2. Porosity can be stated as a fractional value (0.30) or percentage (30%) of open space (i.e. 30% of volume in the material is open space)
3. Open pore spaces occur between sediment grains
4. Open pore spaces occur in cracks or fractures in rocks
5. Open pore spaces occur in cavernous openings formed by dissolution of rock (limestone)
6. Porosity values range from 0 to 50% typically
7. Open pores can be filled with water or air or a mixture of both

Permeability of Geological Material

1. Rocks may have a high porosity but if the pore spaces are not connected, water cannot flow through rock
2. Permeability is a parameter which describes the ability of geologic material to transmit water
3. Geologic material which can transmit large quantities of water are highly permeable and called aquifers

Examples of geologic material which are typically aquifers are

1. Sand and gravel alluvium
2. Sandstone
3. Cavernous and/or fractured limestone

Geologic material which cannot transmit significant quantities of water are impermeable and called aquitards.

Examples of geologic material which are typically aquitards are:

- Clay and silt alluvium
- Shale and siltstone

Water in the Ground

Unsaturated Zone

- region of subsurface from ground surface to the water table
- Pores are partially filled with water
- Unfilled pore space contains air

Saturated Zone

- Region of subsurface in which pore spaces are saturated (completely filled) with water

Water Table

- Interface between unsaturated and saturated zone in unconfined aquifers

Capillary Fringe

- Zone above the water table where capillary forces pull water upward into pore spaces
- Same effect seen with water in straws

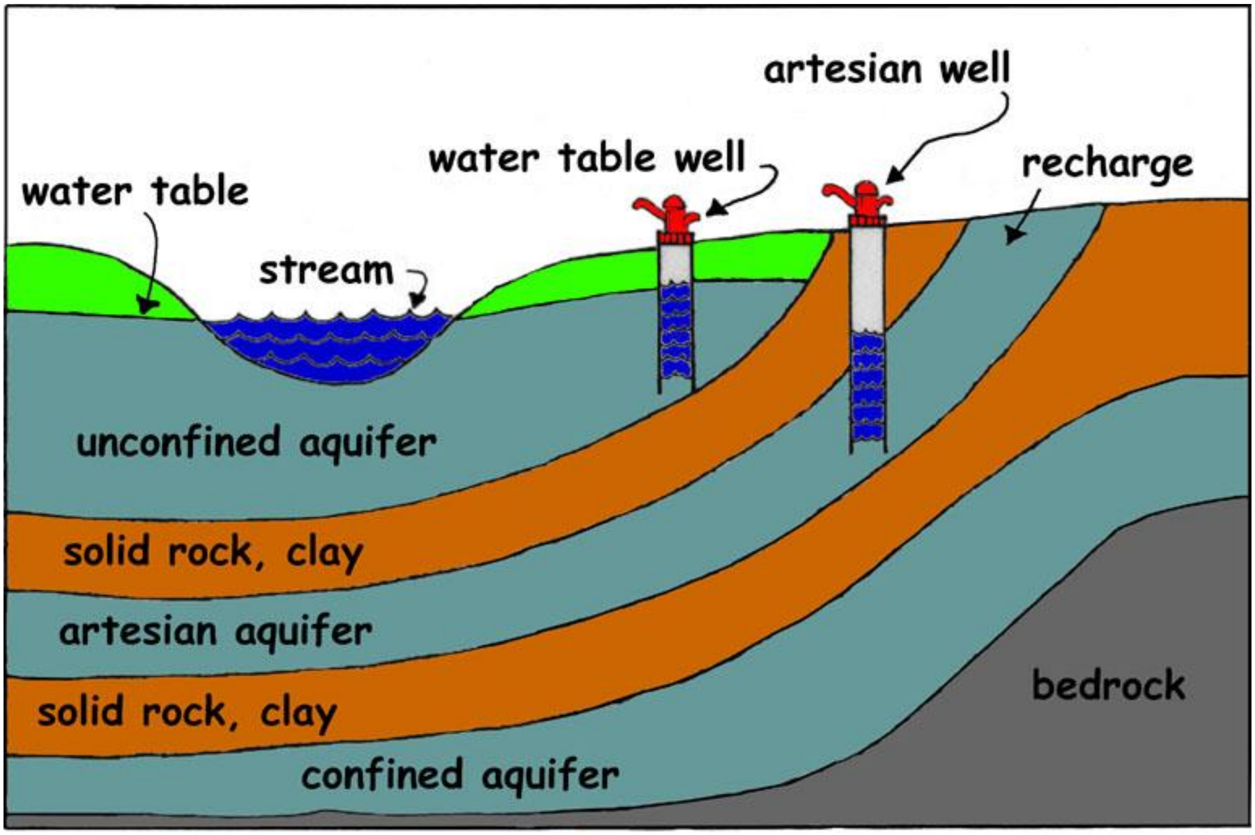
UNCONFINED AQUIFERS

- Water accumulates over an impermeable or impervious surface
- Water table can freely rise to land surface

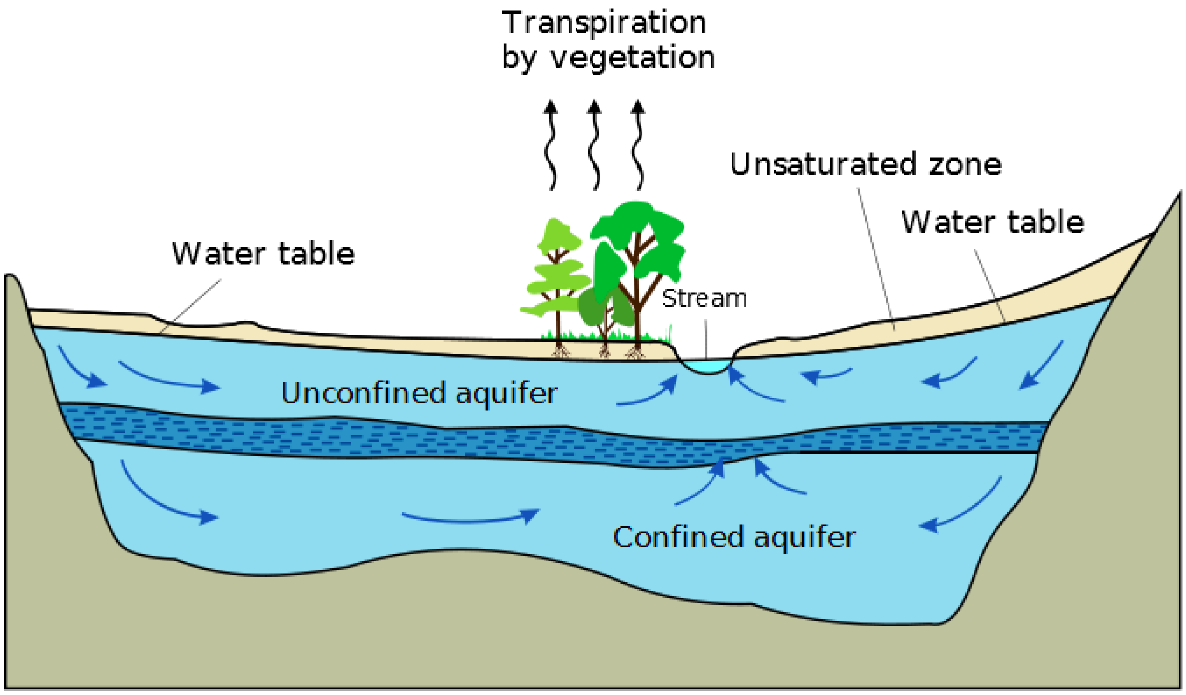
CONFINED AQUIFERS

1. Aquifer is sandwiched between 2 layers of impermeable or impervious material

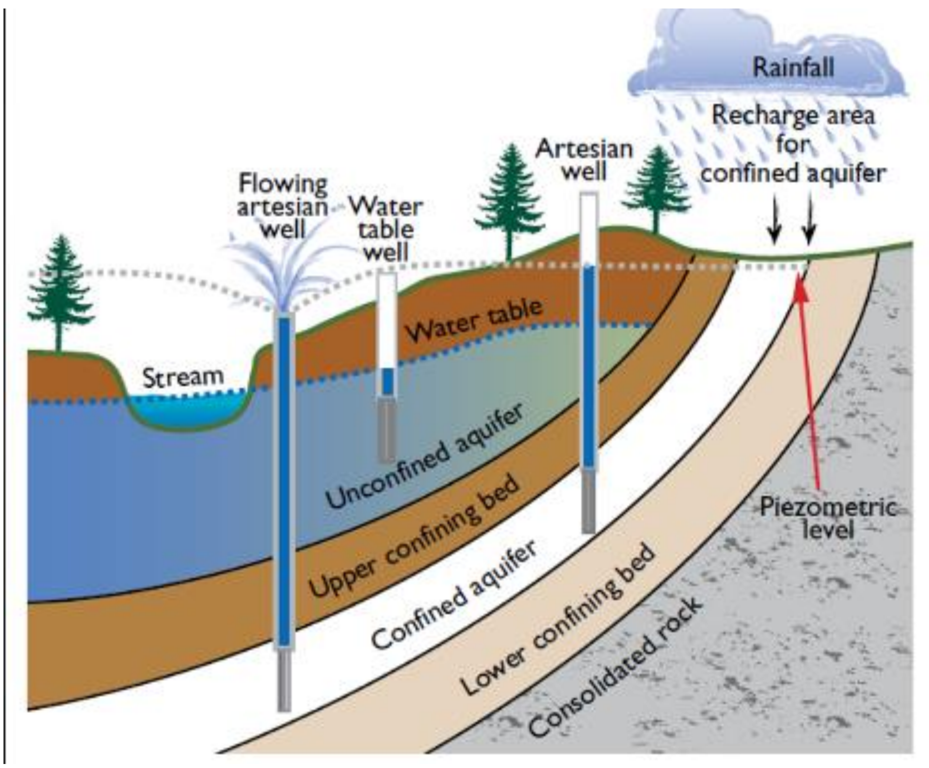
2. Water flows into aquifer from an area at surface where upper impermeable layer (confining layer) is absent
3. Groundwater in confined aquifers is under pressure
4. Wells can be drilled through the upper confining layer
5. Pressurized water will rise within the well
6. Water levels are called piezometric water level
7. Wells are called artesian wells
8. Where water levels rise above the ground surface, water freely flows out of the well (flowing artesian well)



Different types of aquifer



Confined and Unconfined aquifer



Geological and topographical controls affects the flowing and non-flowing artesian wells

Types of aquifers, i.e., (1) Unconfined Aquifer, (2) Perched Aquifer, (3) Confined Aquifer, and (4) Leaky Aquifer or Semi-Confined Aquifer.

1. Unconfined Aquifer:

An aquifer which is not overlain by any confining layer but has a confining layer at its bottom is called unconfined aquifer. It is normally exposed to the atmosphere and its upper portion is partly saturated with water. The upper surface of saturation is called water table which is under atmospheric pressure therefore this aquifer is also called phreatic aquifer.

2. Perched Aquifer:

It is a special case of an unconfined aquifer. This type of aquifer occurs when an impervious or relatively impervious layer of limited area in the form of a lens is located in the water bearing unconfined aquifer. As shown in Fig. 16.3 the water storage created above the lens is perched aquifer and its top layer is called perched water table.

3. Confined Aquifer:

It is also called artesian aquifer. It is a type of aquifer overlain as well as underlain by confining layers. The water within the aquifer is therefore held under pressure. It is sometimes called pressure aquifer also. If the aquifer has high outcrop laterally than the ground surface there will be positive hydrostatic pressure to create conditions for a flowing well. Water from such well comes to the surface without pumping. The imaginary level upto which the water will rise is called piezometric surface.

4. Leaky Aquifer:

In nature, truly confined aquifers are rare because the confining layers are not hundred per cent impervious. An aquifer which is overlain or underlain by a semi- pervious layer (aquitard) through which vertical leakage takes place due to head difference is called leaky aquifer or semi-confined aquifer.

The permeability of the semi-confining layer is usually very small as compared to the permeability of the main aquifer. Thus the water which seeps vertically through the semi-confining layer is diverted internally to proceed horizontally in the main aquifer.

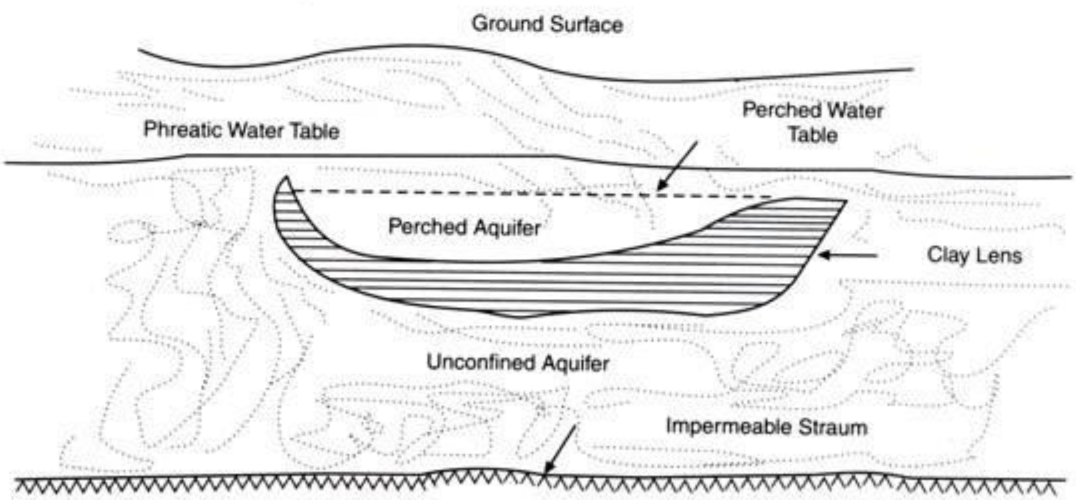


Fig. 16.3. Unconfined and perched aquifers

Figure 16.4 shows various typical groundwater structures and terminology.

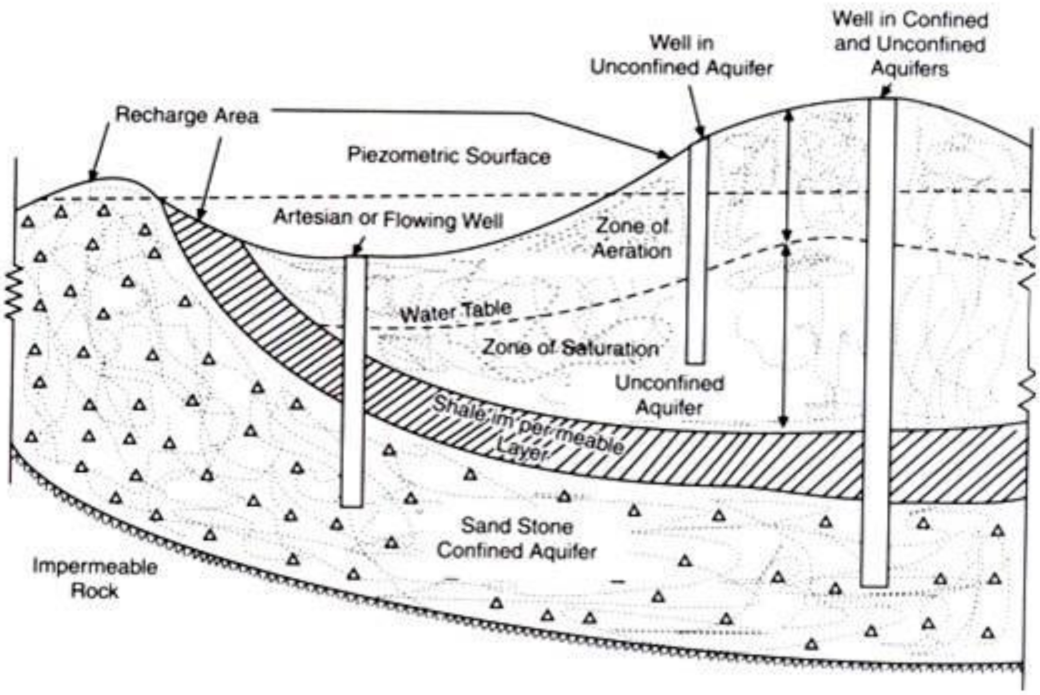


Fig. 16.4. Groundwater terminology model

Aquitard

An aquitard is an underground geological formation which contains water but significant amount of water cannot be extracted using water wells. Aquitard comprises of generally layers of clay soil with low hydraulic conductivity.

Aquifuge

It is a geological formation which is incapable to absorb or transmit water through it. Thus it is an impermeable formation.

Homogeneous and isotropic medium

A porous medium is called homogeneous when aquifer parameters are constant throughout the medium, *i.e.* the properties of the medium are independent of space (Fig. 1.7(a)). The medium will be called non-homogeneous when aquifer properties are varying with space (Fig. 1.7(b)).

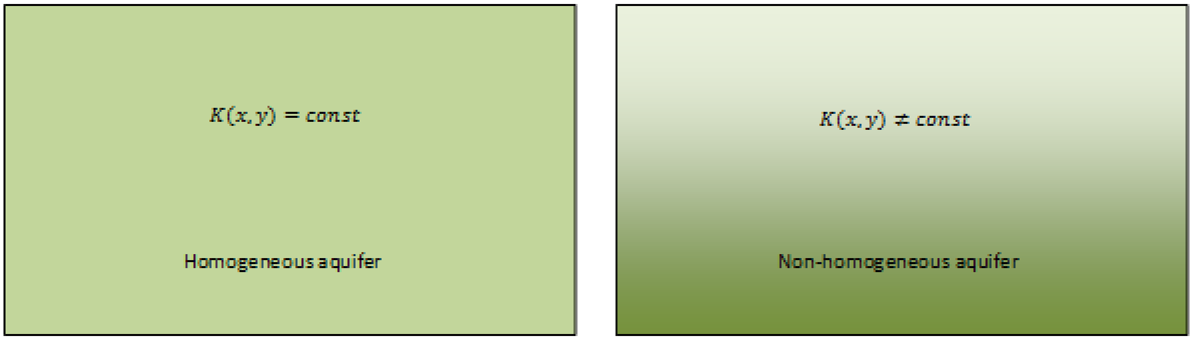


Fig. 1.7 (a) Homogeneous aquifer (b) Non-homogeneous aquifer

A porous medium will be called isotropic when medium parameters are constant in all the directions, *i.e.* the parameters are independent of direction (Fig. 1.8(a)). The medium will be called an anisotropic when the parameters are different in different directions (Fig. 1.8(b)).

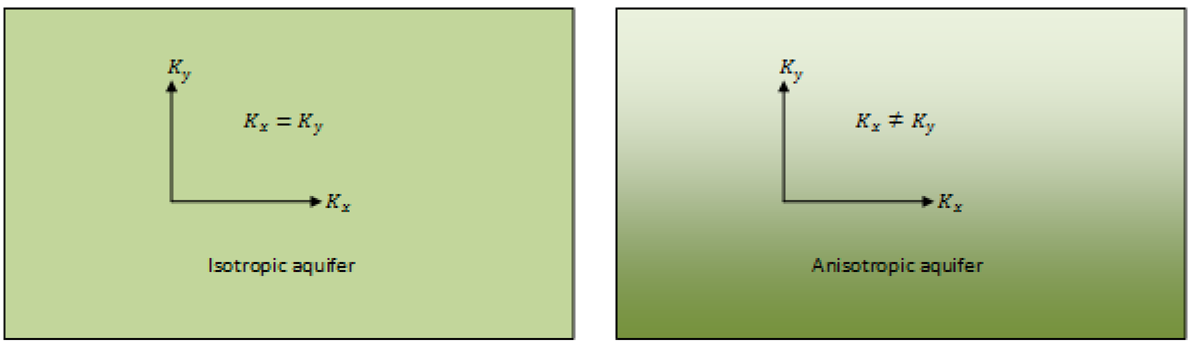


Fig. 1.8 (a) Isotropic aquifer (b) Anisotropic aquifer

The two important properties of an aquifer that are related to the storage function are the "porosity" and "specific yield". The porosity of a water bearing formation is determined by that part of its volume consisting of openings or pores. Porosity is an index of how much groundwater can be stored in a saturated medium and is usually expressed as a percentage of the bulk volume of the material.

The types of rock openings (intergranular/intragranular) that attribute porosity to rocks are described in the presentation. Porosity in alluvium i.e., gravel, sand & clay is different from porosity due to fractures in hard rock. The types of porosity such as primary/secondary are described. Molecular forces of cohesion, adhesion and capillarity that are under the influence of gravity (that is under gravity drainage) are explained.

Although the volume of water contained in a part of the aquifer is of interest to us, it is more important to know how much of this volume can actually be released by the aquifer for use. This release of water per unit area of the aquifer is generally expressed as volume per unit change in head, i.e., change in the water level. The presentation thereafter, deals with hydraulic conductivity, the property of the rock material that relates to its function allowing the flow of water.

Classification of ground water

There are various classifications of groundwater given by different researchers. However, as per the most popular classification given by Meinzer (1923), the groundwater has been divided mainly in two groups: interstitial water and internal water. The interstitial water is again subdivided into two divisions. They are vadose water present in the zone of aeration and groundwater present in the zone of saturation. The vadose water is further subdivided into three zones, *i.e.*, soil water zone, intermediate zone and capillary zone. Fig. 1.2 shows the classification of groundwater. The soil water zone is adjacent to the ground surface. The intermediate zone is between the lower edge of the soil water zone and the upper edge of the capillary zone. The capillary zone extends from the bottom edge of the intermediate zone to the upper edge of the saturated zone. The thickness of the capillary zone depends on the properties of the soil and also on the homogeneity of the soil. The depth of capillary zone is varying from few centimeters to few meters. In capillary zone, all the pores are field up with water. However, we cannot draw water by inserting a well up to that depth. This is because of the negative pressure developed at this zone due to surface tension effect. Groundwater zone starts from the bottom edge of the capillary zone. In this zone, all the pores of the soil matrix are filled with water. This zone is also known as zone of saturation. The top surface of the zone of saturation or groundwater is known as phreatic surface. This phreatic surface is also known as water table.

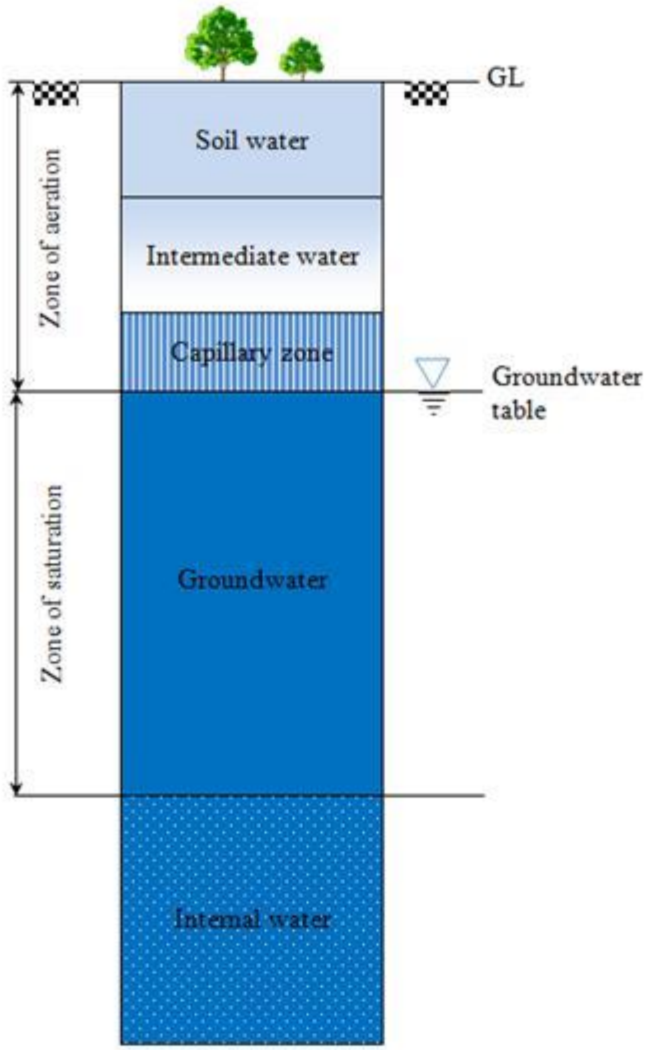


Fig. 1.2 Classification of groundwater

The degree of saturation for the soil below the water table is equal to 1, *i.e.* the soil is fully saturated. As a groundwater hydrologist, we are primarily interested for the water below the groundwater table, *i.e.* the water available in the zone of saturation . For the soil above the water table, the degree of saturation of the soil is varying between 0 and 1 . However, the degree of saturation will never be 0 due of the presence of hygroscopic water. The hygroscopic water is the water that held tightly on the surface of the soil colloidal particle. Hygroscopic water can be removed from the soil by oven drying. Fig. 1.3 shows the moisture distribution in soil column.

Aquifer Transmissivity

Consider the flow through a confined aquifer as shown in Fig. 6.1. The width of the aquifer is W. The depth of the aquifer is B. The total discharge in the *x* direction through the area WB can be written as,

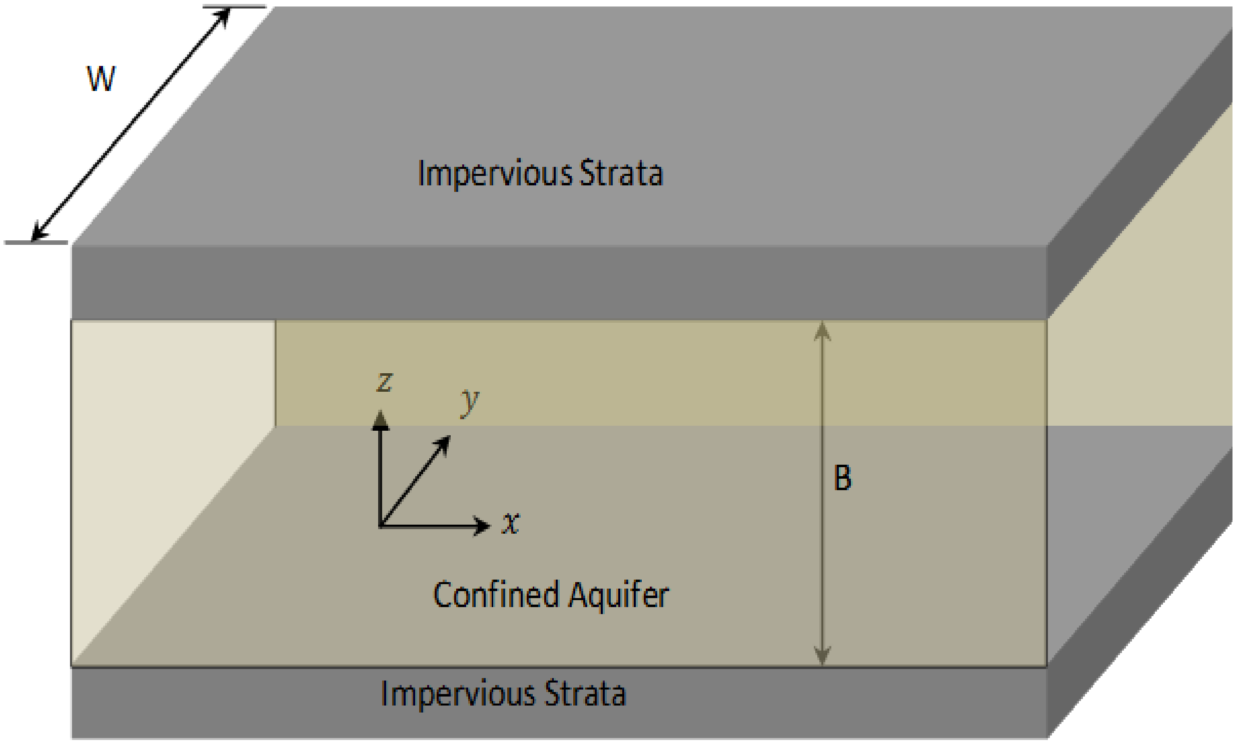


Fig.6.1 A confined Aquifer

The discharge per unit width through the first Layer may be written as

$$Q_x = WB\left(-K_{xx}\frac{\partial\varphi}{\partial x} - K_{xy}\frac{\partial\varphi}{\partial y}\right) \tag{6.1}$$

$$= -WBK_{xx}\frac{\partial\varphi}{\partial x} - WBK_{xy}\frac{\partial\varphi}{\partial y} \tag{6.2}$$

The discharge per unit width of the aquifer can be written as,

$$\acute{Q}_x = -(BK_{xx})\frac{\partial\varphi}{\partial x} - (BK_{xy})\frac{\partial\varphi}{\partial y} \tag{6.3}$$

Putting $T_{xx} = BK_{xx}$ *and* $T_{xy} = BK_{xy}$', the equation (6.3) becomes

$$\acute{Q}_x = -T_{xx}\frac{\partial\varphi}{\partial x} - T_{xy}\frac{\partial\varphi}{\partial y} \tag{6.4}$$

Similarly in the direction of y the discharge per unit width of the aquifer can be written as

$$\acute{Q}_y = -(BK_{yy})\frac{\partial\varphi}{\partial y} - (BK_{yx})\frac{\partial\varphi}{\partial x} \qquad (6.5)$$

Putting $T_{yy} = BK_{yy}$ *and* $T_{yx} = BK_{yx}'$, the equation (6.5) becomes

$$\acute{Q}_y = -T_{yy}\frac{\partial\varphi}{\partial y} - T_{yx}\frac{\partial\varphi}{\partial x}$$

Specific Storativity

Specific storativity of a porous medium is defined as the volume of water released or added into a unit volume of the aquifer under unit declination in the piezometic head (Φ). Thus it can be written as,

$$S_o = \frac{\Delta V_W}{V\Delta\phi} \qquad (7.1)$$

Where, S_0 is the specific storativity of the aquifer, ΔV_W is the amount of water release or added into the aquifer, V is the total volume of the aquifer and $\Delta\Phi$ is the change in the piezometric head.

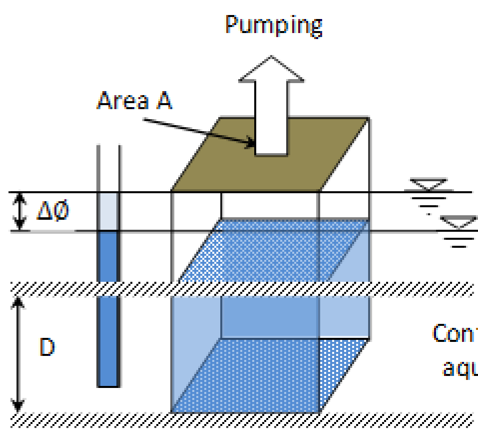


Fig. 7.1 Sketch to explain aquifer storativity of confined aquifer

Aquifer storativity

Consider a confined aquifer of horizontal area A and depth D as shown in Fig. 7.1. The initial piezometric head is at C. V_W is the amount of water withdrawn from the aquifer. As a result of pumping, the piezometric head is dropped down by an amount of $\Delta\Phi$. In this case, the equation (7.1) can be written as,

$$S_o = \frac{\Delta V_W}{AD\Delta\phi} \tag{7.2}$$

Or,

$$S_o D = \frac{\Delta V_W}{A\Delta\phi} \tag{7.3}$$

Or,

$$S_s = \frac{\Delta V_W}{A\Delta\phi} \tag{7.4}$$

Where S_s is the aquifer storativity.

Thus storativity for a confined aquifer is defined as the volume of water released from storage or added to the aquifer per unit horizontal area under unit declination or rise of peizometric head (Φ). It may be noted that like transmissivity (T) of an aquifer, the storativity is also an aquifer property. In case of confined aquifer, when Dupuit assumption of essentially horizontal flow in an aquifer is considered, the parameter T and S_s should be used. However, in the case of three dimensional flow, the hydraulic conductivity (K) and specific storativity (S_0) need to be used.

Specific yield

In case of unconfined aquifer, storativity of an aquifer can be defined as the volume of water released or added to the aquifer from a unit area under unit declination or rise in water table. In this case, the storage coefficient is called as specific yield. Fig. 7.2 shows an unconfined aquifer with horizontal area 'A'. The initial water table is at C. V_W is the amount of water withdrawn from the aquifer. As a result of pumping, the water table is drop down by an amount of Δh. In this case, the specific yield can be written as,

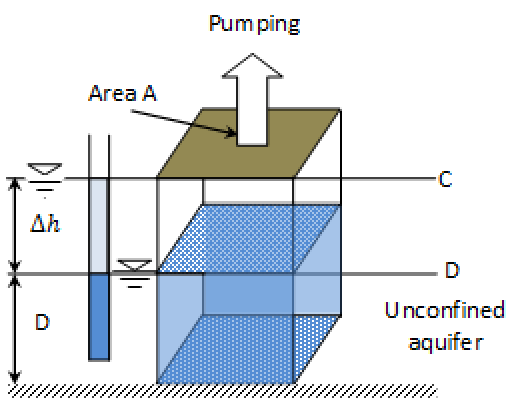


Fig.7.2 Sketch to explain aquifer storativity of unconfined aquifer

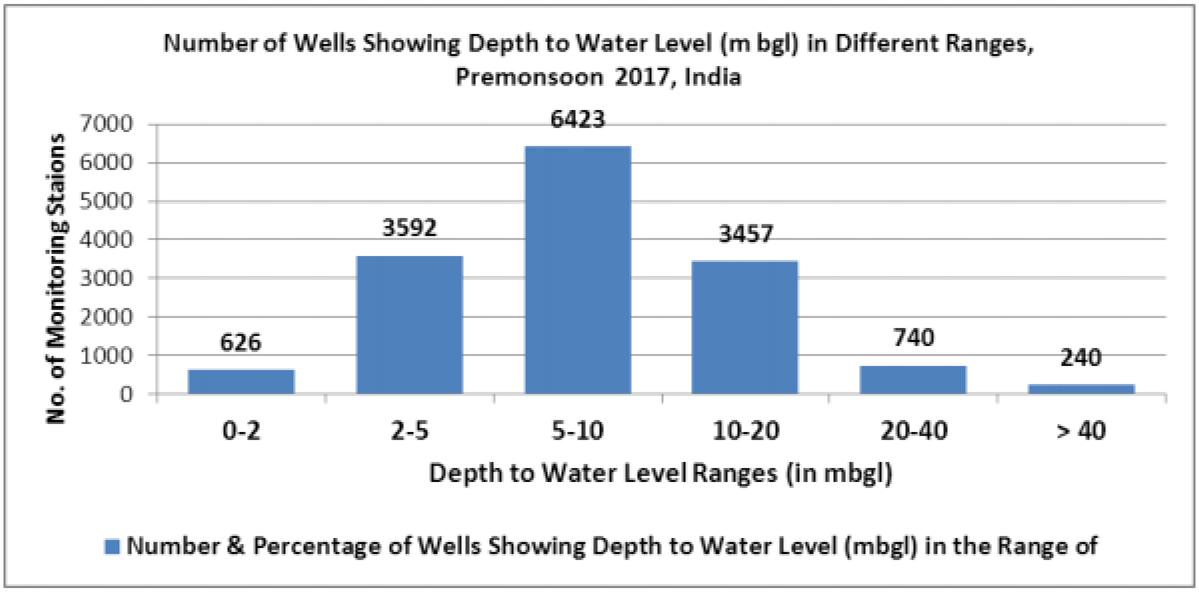
$$S_y = \frac{\Delta V_W}{A\Delta h} \tag{7.5}$$

It may be noted that a certain amount of water is always retained in the aquifer due to the capillary and hygroscopic forces which is known as specific retention (S_r). As such,

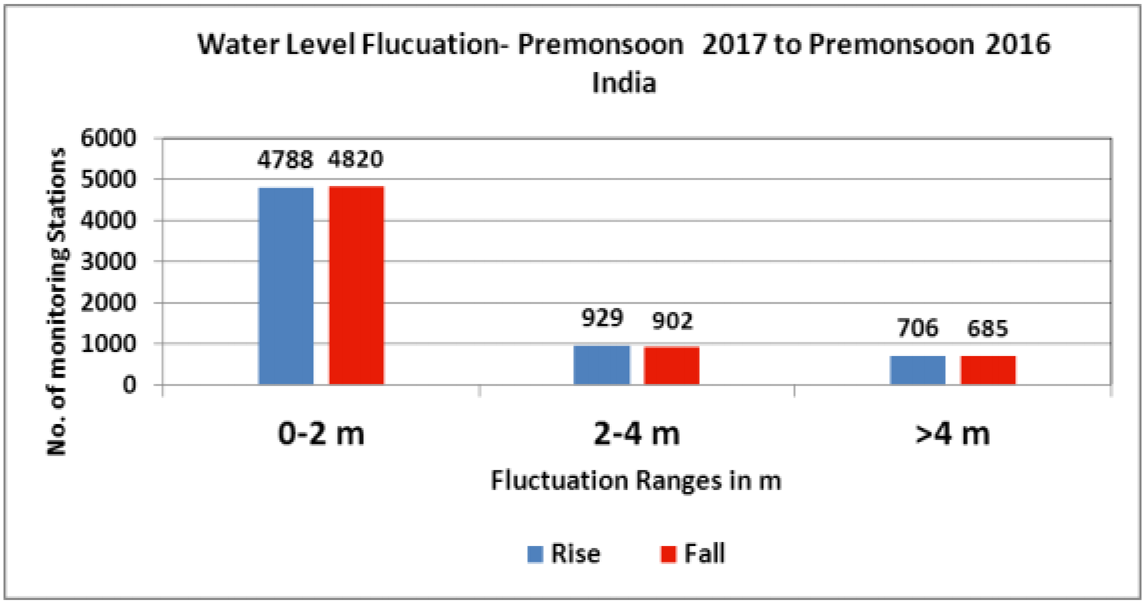
$$S_y + S_r = \eta \tag{7.6}$$

Where, η is the porosity of the porous matrix. The specific yield is therefore always less than the porosity of the porous media. Specific yield is also sometime called effective porosity.

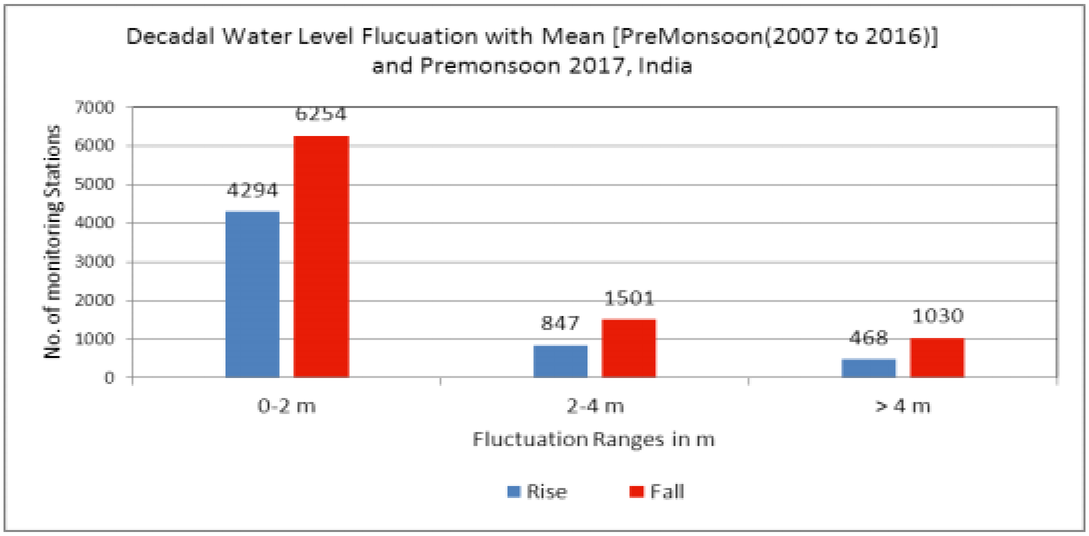
Ground Water Level Scenario in India 3.1 Ground Water Level Scenario - Premonsoon 2017 The ground water level data for Premonsoon 2017 indicates that out of the total 15078 wells analysed, 626 (4 %) wells are showing water level less than 2 m bgl (metres below ground level), 3592 (24%) wells are showing water level in the depth range of 2-5 m bgl, 6423 (43 %) wells are showing water level in the depth range of 5-10 m bgl, 3457 (23%) wells are showing water level in the depth range of 10-20 m bgl, 740 (5%) wells are showing water level in the depth range of 20-40 m bgl and the remaining 240 (2%) wells are showing water level more than 40 m bgl. The maximum depth to water level of 134.22 m bgl is observed in Bikaner district of Rajasthan whereas the minimum is less than 1 m bgl. The depth to water level map of Premonsoon 2017 for the country indicates that the general depth to water level of the country ranges from 2 to 20 m bgl. To be more specific, in major parts of the country, water level is observed to be in the range of 5 to 10 m. Very shallow water level of less than 2 m bgl is also observed locally, in isolated pockets, in few states, such as Assam, Goa and Himachal Pradesh. In major parts of north-western and western states, depth to water level is generally deeper and ranges from about 10- 40 m bgl. In parts Delhi, Chandigarh and Rajasthan, water level of more than 40 m bgl is also recorded. The peninsular part of country recorded a water level in the range of 5 to 20 m bgl. The maximum depth to water level of 134.22 m bgl is observed in Bikaner district, Rajasthan whereas the minimum is less than 1 m bgl, seen in various states.

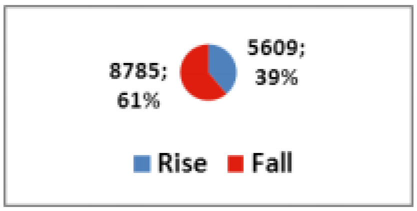


Water Level Fluctuation (Premonsoon 2017 to Premonsoon 2016) The water level fluctuation of Premonsoon 2017 to Premonsoon 2016 shows that out of 13423 wells analysed, 6423 (48%) are showing rise and 6407 (48%) are showing fall in water level. Remaining 593 (4%) stations analysed do not show any change in water level. Both rise and fall are equally predominant in the country. Both rise and fall are equally predominant. About 36% wells are showing rise in the water level in the range of less than 2 m. About 7% wells are showing rise in water level in 2-4 m range and 5% wells showing rise in water level more than 4 m range. Similarly, about 48% wells are showing decline in water level, out of which 36% wells are showing decline in water level in less than 2 m range. About 7% wells are showing decline in water level in 2-4 m range and 5% wells are showing decline in water level more than 4 m range (Fig-2 and Annexure-II). Majority of the wells showing rise/decline falls in the range of 0-2 m. A comparison of depth to water level of Premonsoon 2017 to Premonsoon 2016 is presented in the form of water level fluctuation map (Plate III) reveals that in general, there is both rise and fall in water level in almost the entire country. Rise in water level in isolated pockets is observed in the states of Assam, Bihar, Chhatishgarh, Madhya Pradesh, Gujarat, Rajasthan, Maharashtra, Telangana and Uttar Pradesh. Fall is mostly in the range of 0-2 m, although fall in the range of more than 2 m is also prevalent in all the states in small patches. Fall of more than 4 m is prominent in the states of Andhra Pradesh, Chandigarh, Delhi, Karnataka and Tamil Nadu.



Water Level Fluctuation (Premonsoon–2017 with Mean of Premonsoon (2006 - 2016) A comparison of depth to water level of Premonsoon 2017 with decadal mean of Premonsoon (2007-2016) indicates that 5609 (about 39%) of wells are showing rise in water level, out of which 30% wells are showing rise of less than 2 m. About 6% wells are showing rise in water level in the range of 2-4 m and only 3% wells are showing rise in the range of more than 4 m. 8785 (about 61%) wells are showing decline in water level, out of which 43% wells are showing decline in water in the range of 0-2 m. 11% wells are showing decline in water level in 2-4 m range and remaining 7% are in the range of more than 4 m. Decline is seen in almost all the states/UTs of the country, except few states namely Arunachal Pradesh, Bihar, Daman & Diu, Goa, Telangana, Tripura and West Bengal. Decline of more than 4 m has also been observed in pockets in the states/UTs of Andhra Pradesh, Chandigarh, Chhattisgarh, Dadra & Nagar Haveli, Daman & Diu, Delhi, Gujarat, Haryana, Karnataka, Maharashtra, Pondicherry, Punjab, Rajasthan, Tamil Nadu and Telangana. Rise in water level of more than 4 m is also observed in few states in isolated pockets such as Arunachal Pradesh, Dadra & Nagar Haveli, Rajasthan, Telangana and West Bengal. The decadal water level fluctuation map of India for Premonsoon, 2017 with the mean of Premonsoon (2007-2016) is shown in Plate-IV and frequency distribution of fluctuation ranges is shown in Fig. 3. Almost the whole country is showing decline in water level, maximum fall is observed in and around parts of Rajasthan, Haryana, Punjab, Gujarat, Telangana, and Maharashtra, A rise in water level is observed in few states but occurs sporadically.





Ground Water Development- in India

The total annual replenishable ground water resources of the country have been assessed as 433 billion cubic meter (BCM). Existing gross ground water draft as on March 2004 for all uses is 231 BCM per year. The stage of ground water development is about 58%. The development of ground water in different areas of the country has not been uniform. Highly intensive development of ground water in certain areas in the country has resulted in over exploitation leading to decline in the levels of ground water and sea water intrusion in coastal areas. There is a continuous increase in dark and over-exploited areas in the country.

As per the latest assessment of ground water resources carried out jointly by the Central Ground Water Board (CGWB) and the States, the assessment units are categorized as 'over exploited'/'critical' and 'semi-critical' based on the stage of ground water development and the long-term water level declining trend during the past decade (1995-2004). Out of 5,723 assessment units (Blocks/Mandals/Talukas) in the country, 839 units in various States have been categorized as 'over exploited', i.e., the annual ground water extraction exceeds the annual replenishable resource.

In addition, 226 units are 'critical', i.e., the stage of ground water development is above 90 per cent and less than 100 per cent of annual replenishable resource with significant decline in long term water level trend in both pre-monsoon and post-monsoon period. There are 550 semi-critical units, where the stage of ground water development is more than 70 per cent. List of these areas has been circulated to the State Pollution Control Boards and the Ministry of Environment and Forests which refer the new industries/projects falling in these areas to the Central Ground Water Authority (CGWA) for obtaining clearance for ground water withdrawal.

The CGWA has so far notified 43 over-exploited areas in the country for regulation of ground water development and management. For enforcement of the regulatory directions issued under Section 5 of Environment (Protection) Act, 1986, concerned Deputy Commissioners/District Magistrates have been authorized to take necessary action in case of violation of directives of CGWA in the notified areas. For more effective regulation of ground water development and management, Advisory Committees under the Chairmanship of District Collector/Deputy Commissioners with members drawn from various organizations have been constituted which will render advice in matters pertaining to regulation of ground water development and management.

The CGWA have also notified 65 over-exploited areas in various States, for registration of ground water abstraction structures, which showed a very steep decline in ground water levels and which required action for regulation. The CGWA has issued directions to the Chief Secretaries of all States having over-exploited blocks to take all necessary measures to promote/adopt artificial recharge to ground water/rain water harvesting. The CGWA has also decided to notify more over-exploited areas in the country in compliance of its mandate following the provisions under rule 4 of the Environment (Protection) Rules, 1986.

Determination of Aquifer Parameters

12.1 Introduction

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

12.2 Laboratory Methods

12.2.1 Grain-Size Analysis (GSA) Method

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

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

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

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

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

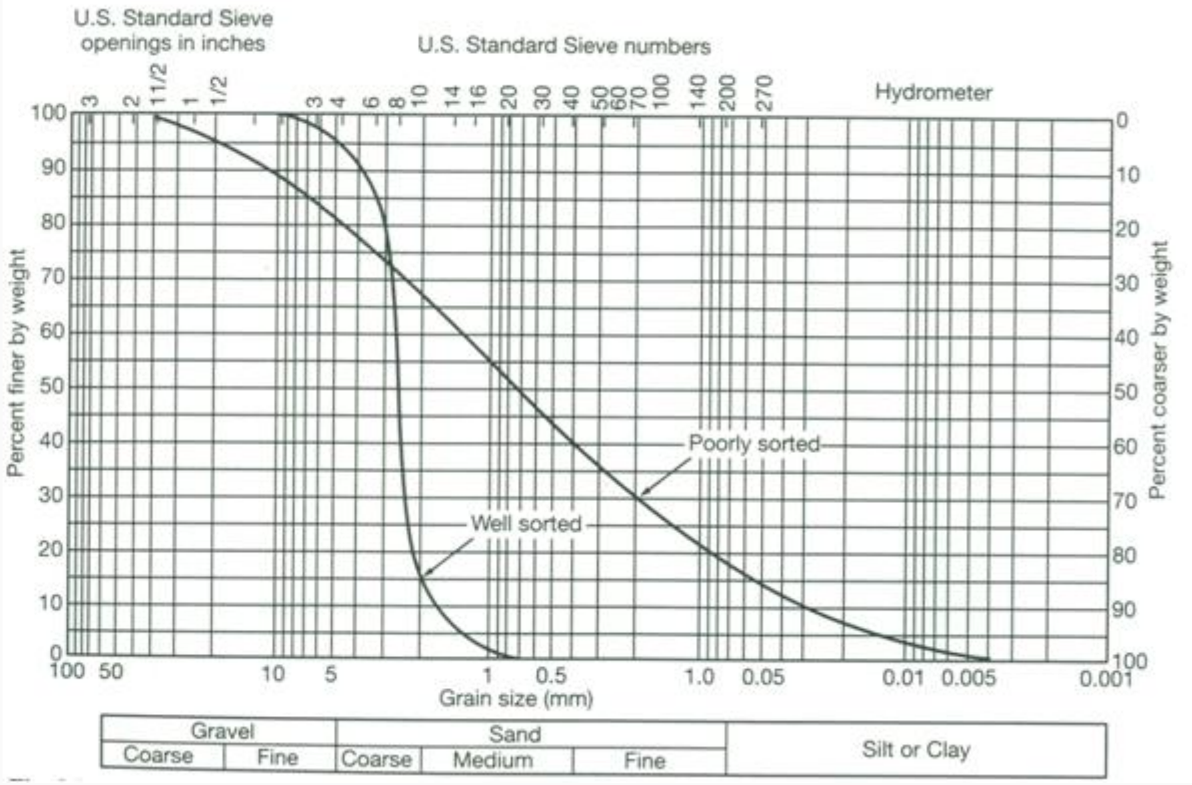


Fig. 12.1. Grain-size distribution curves for well sorted and poorly sorted samples. (Source: Brassington, 1998)

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

12.2.2 Permeameter Methods

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

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

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

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

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

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

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

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

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

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

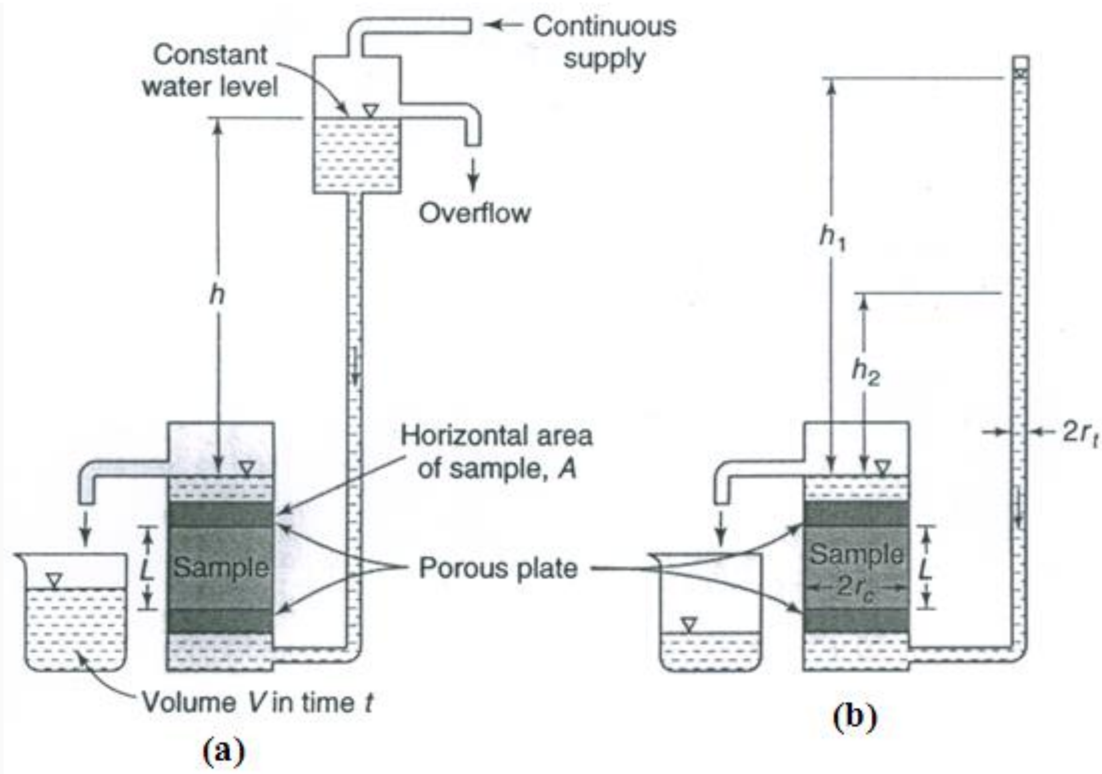


Fig. 12.2. Permeameters for measuring saturated hydraulic conductivity of geologic or soil samples: (a) Constant-head permeameter; (b) Falling-head permeameter. (Source: Mays, 2012)

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

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

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

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

12.3 Field Methods

12.3.1 Tracer Test

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

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

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

However, V_s can also be calculated as:

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

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

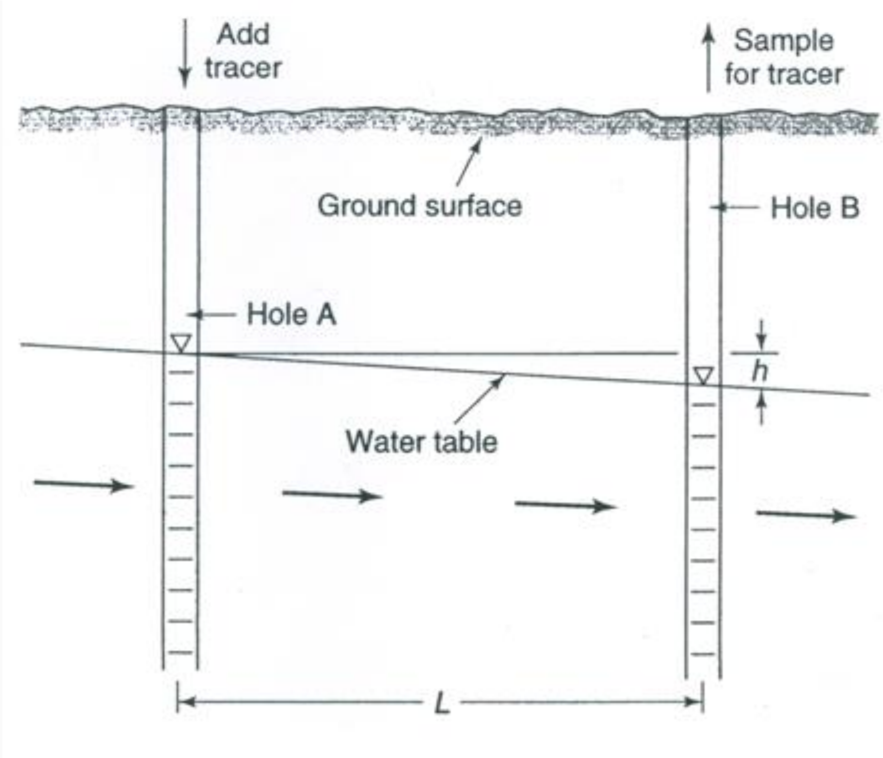


Fig. 12.3.Illustration of a tracer test in an unconfined aquifer for determining hydraulic conductivity. (Source: Mays, 2012)

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

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

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

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

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

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

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

Example Problem:

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

Solution:

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

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

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

12.3.2 Auger-Hole Method

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

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

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

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

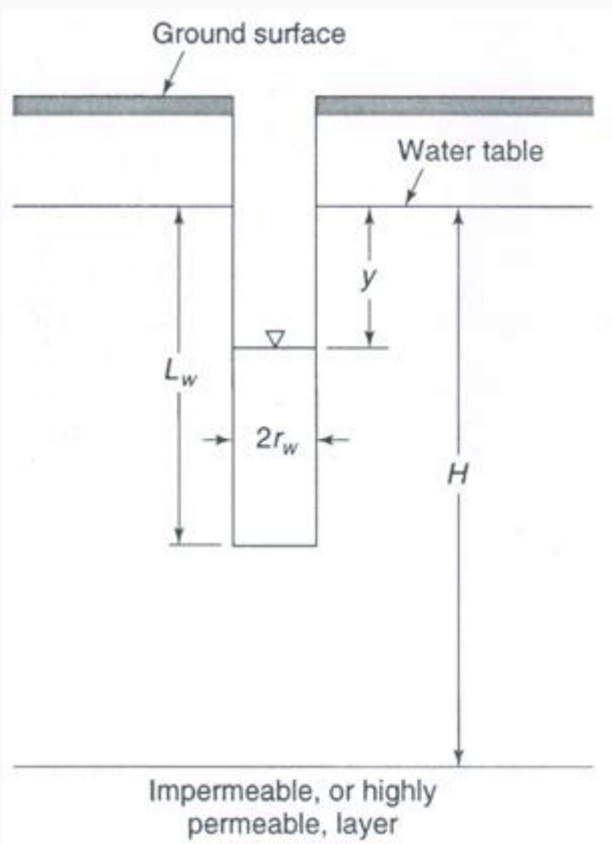


Fig. 12.4.Schematic of an auger hole and its dimensions for determining aquifer hydraulic conductivity. (Source: Mays, 2012)

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

12.3.3 Slug Test

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

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

12.3.4 Pumping Test

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

SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – II – GROUND WATER ENGINEERING – SCI1602

EVALUATIONOFAQUIFERPROPERTIES

Darcy's Experiment

In the year 1856, Henry Darcy, a French hydraulic engineer investigated the flow of water through a vertical homogeneous sand filter. Based on his experiments, he concluded that the rate flow through the porous media is proportional to the head loss and is inversely proportional to the length of the flow path. Figure 3.1 shows the setup of Darcy's experiment. As shown in the figure, the length of the vertical sand filter is L, the cross sectional area of the filter is A, the piezometric heads at top and bottom of the filter are h_1 and h_2. Thus the head loss is $(h_1 - h_2)$. The piezometric heads are measured with respect to an arbitrary datum. As per the conclusions made by Darcy, the flow rate Q is

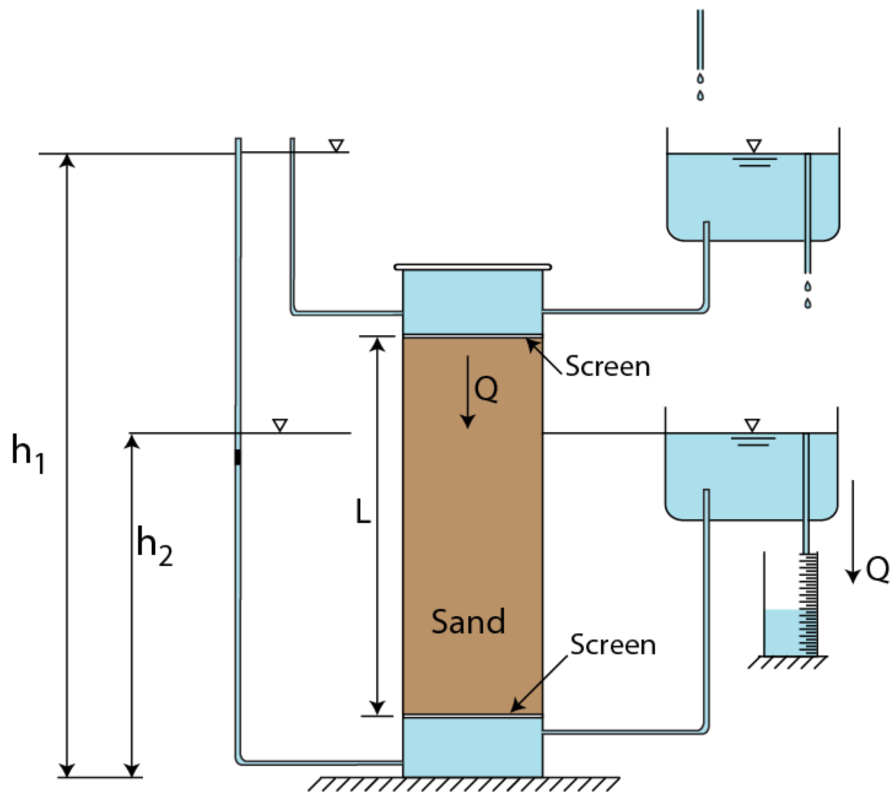


Fig. 3.1 Darcy's Experiment in vertical sand filter

- proportional to the cross sectional area *(A)* of the filter
- proportional to the difference in piezometric heads
- inversely proportional to the length *(L)* of the filter

After combining these conclusions, we have

$$Q = KA\left(\frac{h_1-h_2}{L}\right) \tag{3.1}$$

Where,

Q is the flow rate, *i.e.* the volume of water flows through the sand filter per unit time.
K is the coefficient of proportionality and is termed as hydraulic conductivity of the medium. It is a measure of the permeability of the porous medium. It is also known as coefficient of permeability.
h_1 and h_2 are the piezometric heads.
Now, defining $J = \frac{h_1-h_2}{L}$ and $q = \frac{Q}{A}$
Where J is the hydraulic gradient and q is the specific discharge, *i.e.* the discharge per unit area.

The equation 3.1 can also be written as,

$$q = KJ \tag{3.2}$$

Now consider an inclined homogeneous sand filter as shown in Fig. 3.2 In this case, the Darcy's formula can be written as,

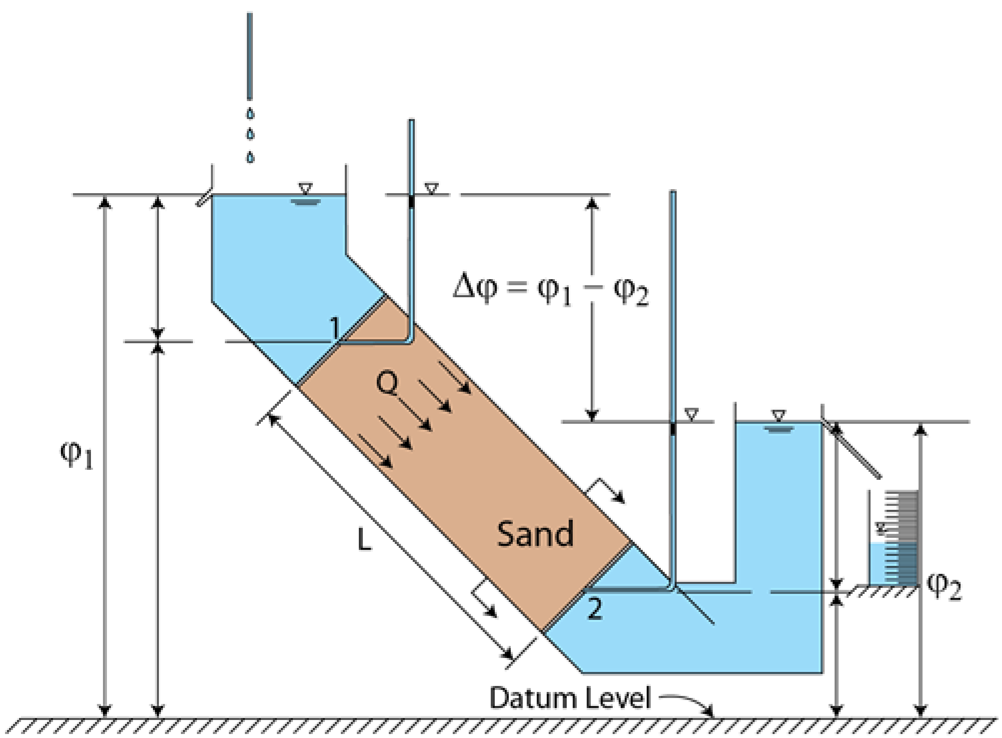


Fig. 3.2: Darcy's Experiments in inclined sand filter

$$Q = KA\left(\frac{\varphi_1 - \varphi_2}{L}\right) \tag{3.3}$$

or,

$$q = K\left(\frac{\varphi_1 - \varphi_2}{L}\right) \tag{3.4}$$

or,

$$q = KJ \tag{3.5}$$

Where, $J = \frac{\varphi_1 - \varphi_2}{L}$ and $\varphi_1 = z_1 + \frac{p_1}{\gamma}$ and $\varphi_2 = z_2 + \frac{p_2}{\gamma}$

z_1 and z_2 are the datum head or elevation head

p_1/γ and p_2/γ are the pressure head

It should be noted here that q and K have the same dimension with the velocity. The value of q will be equal to K for unit hydraulic gradient. As such for the case of isotropic medium, the hydraulic conductivity (*K*) may be defined as the specific discharge (*q*) occurs under unit hydraulic gradient (*J*= 1).The hydraulic conductivity is dependent on both porous matrix properties and fluid properties and can be expressed as

$$K = \frac{k\rho g}{\mu} = \frac{kg}{\upsilon} \tag{3.6}$$

Where, ρ is the density of the fluid, μ is the viscosity of the fluid, v is the kinematic viscosity, k is the intrinsic permeability of the soil which depends on the properties of the porous matrix.

Considering (3.6), the Darcy's Law can be written as

$$q = \left(\frac{k\rho g}{\mu}\right).J \tag{3.7}$$

It may be noted that in Darcy's law, we have neglected the kinetic energy of water. The velocity of water in case of porous medium is very low and along the flow path, the change in piezometric head is much smaller than the change in kinetic energy. Hence, kinetic energy can be neglected.

Further, it may be noted that the flow takes place from higher piezometric head to lower piezometric head and not from higher pressure to lower pressure. Only in case of horizontal flow ($z_1 = z_2$), the flow takes place from higher pressure to lower pressure. Thus incase of horizontal flow, the Darcy's formula can be written as,

$$q = K\left(\frac{P_1 - P_2}{\gamma L}\right) \tag{3.8}$$

Moreover, In case of flow through porous medium, the flow takes place only through the pores of the medium. Therefore, the cross sectional area through which the flow actually takes place is ηA. Where η is the porosity of the porous medium. As such, the average velocity of the flow can be expressed as

$$V = \frac{Q}{\eta A} = \frac{q}{\eta} = \frac{KJ}{\eta}$$

Governing equation for radial flow in an aquifer

The flow towards a well, situated in homogeneous and isotropic confined or unconfined aquifer is radially symmetric. Fig. 15.1(a) shows the cone of depression caused due to constant pumping through a single well situated at (0,0) in a confined aquifer. Fig. 15.1(b) shows the cone of impression caused due to constant recharge through the well. In case of homogeneous and isotropic medium, the cone of depression or cone of impression is radially symmetrical. The governing equation derived earlier in Cartesian coordinate system for confined and unconfined aquifer can also be derived for radial flow in an aquifer. In this lecture, we will derive the governing flow equation for confined and unconfined aquifer in polar coordinate system. The main objective of this conversion is to make the 2D flow problem a 1D flow problem. The resulting 1D problem will be simpler to solve.

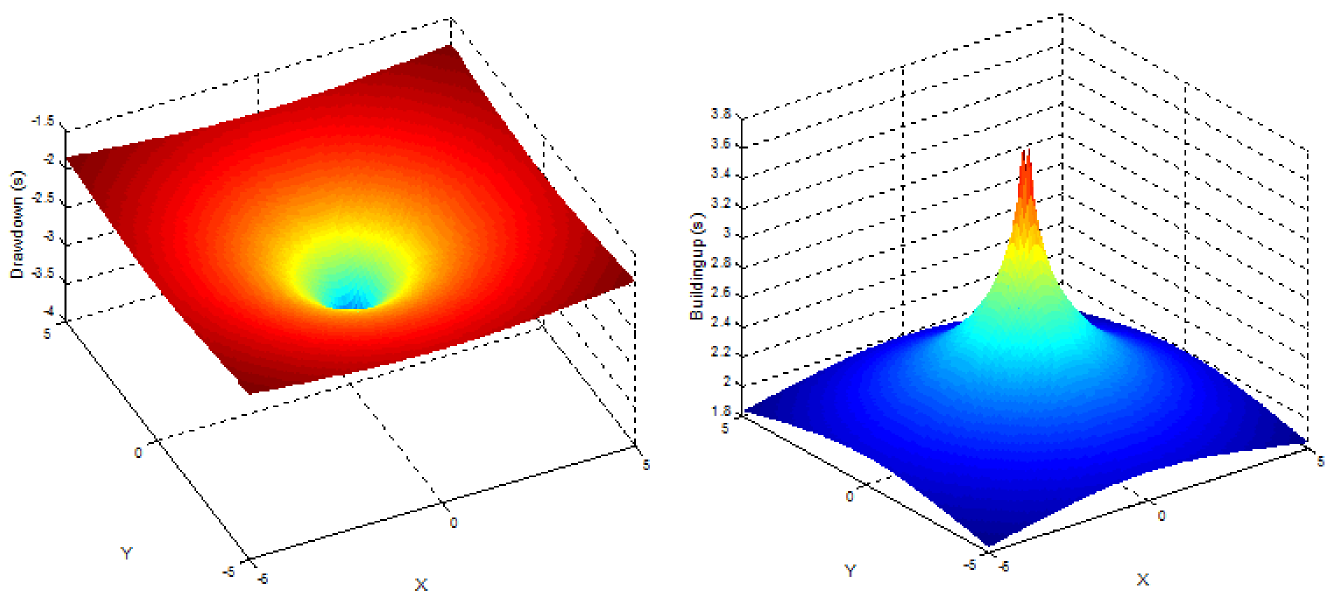
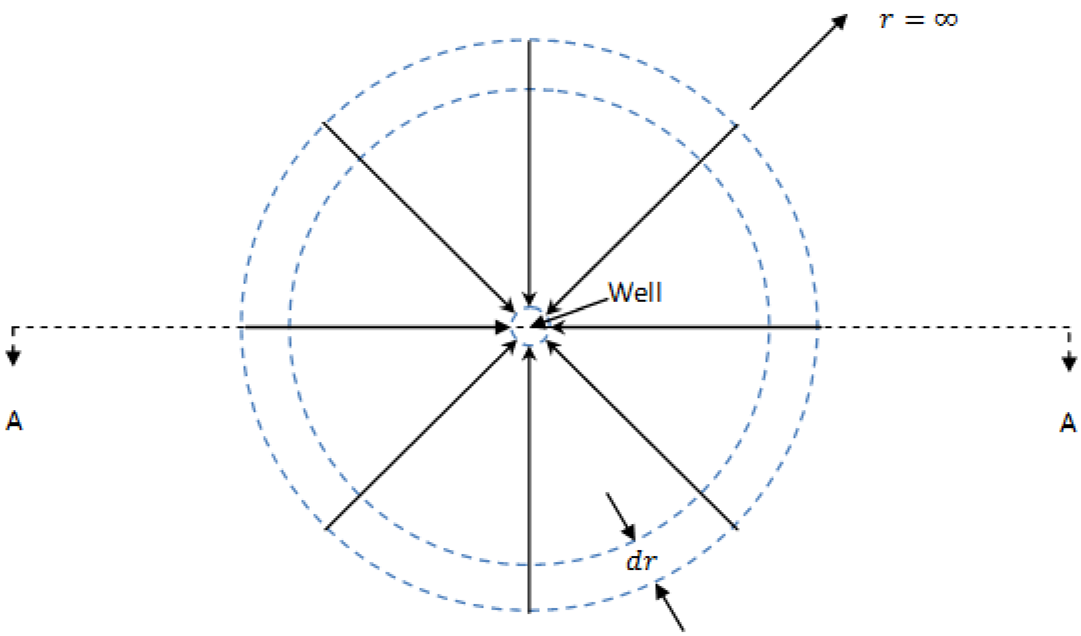
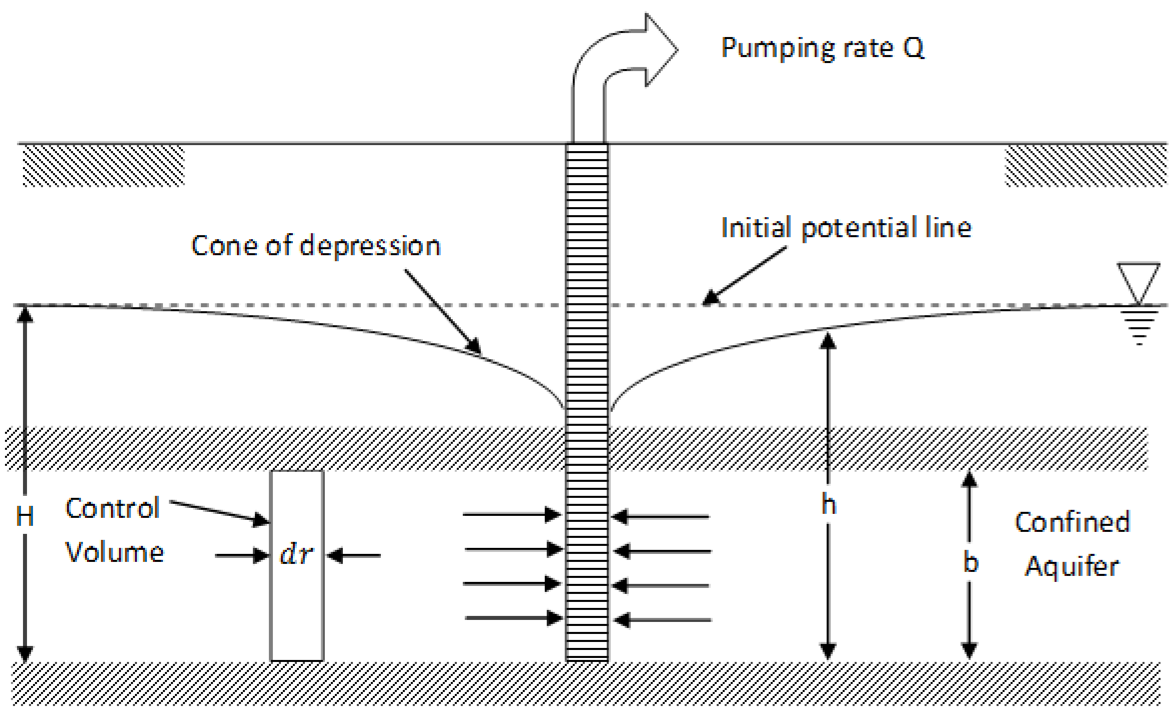


Fig. 15.1 (a) Cone of depression (b) Cone of impression

Confined aquifer



(a) Radial flow to a well



(b) Section A-A in case of confined aquifer

Fig. 15.2: A confiner aquifer

Let us consider a case of radial flow to a single well (Fig.15.2) in a confined aquifer. The Fig. 15.2 (a) shows the radial flow towards a well and a control volume of thickness dr. The Fig. 15.2 (b) shows the vertical section AA of the aquifer along with cone of depression. The aquifer is homogeneous and isotropic and have constant thickness of b. The hydraulic conductivity of the aquifer is K. The pumping rate (Q) of the aquifer is constant and the well diameter is infinitesimally small. The well is fully penetrated into the entire thickness of the confined aquifer. This is necessary to make the flow essentially horizontal. The potential head in the aquifer prior to pumping is uniform throughout the aquifer.

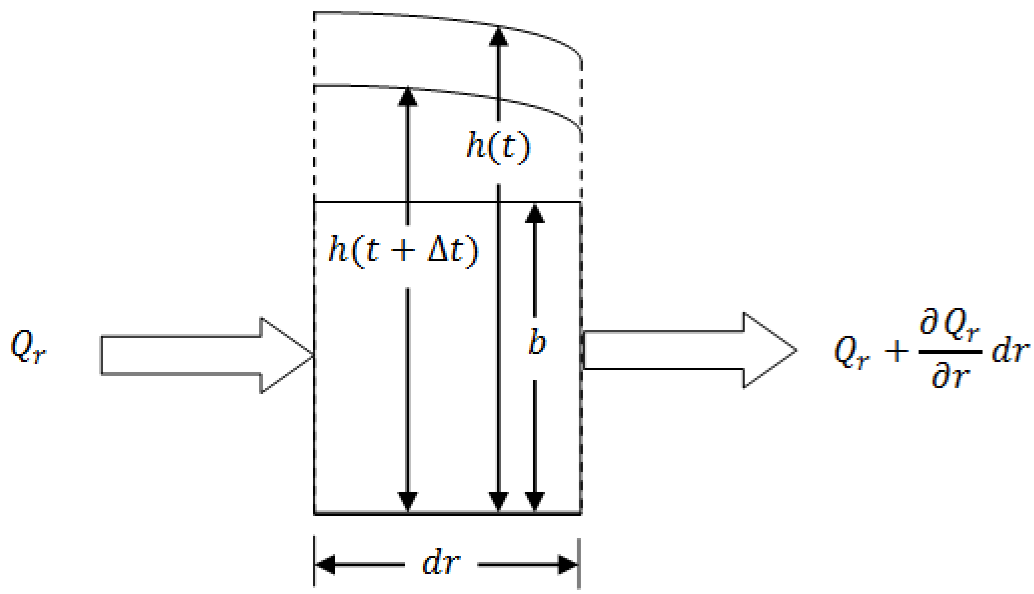


Fig. 15.3 Control volume in case of confined aquifer

Consider the control volume shown in figure 15.3.

The inflow to the control volume is Q_r

The outflow from the control volume is $Q_r + \frac{\partial Q_r}{\partial r} dr$

The net inflow to the control volume is $Q_r - \left(Q_r + \frac{\partial Q_r}{\partial r} dr\right) = -\frac{\partial Q_r}{\partial r} dr$ (15.1)

Applying principle of mass conservation on the control volume

Inflow - outflow = Time rate of change in volumetric storage

Time rate of change in volumetric storage $= \frac{\partial V}{\partial t} = V\left(\frac{\partial V}{V\partial h}\right)\frac{\partial h}{\partial t}$ (15.2)

$$\frac{\partial V}{\partial t} = VS_o\frac{\partial h}{\partial t} \quad (15.3)$$

where S_o is the specific storage $= \frac{\partial V}{V\partial h}$

Replacing V by $2\pi rdrb$, we have

$$\frac{\partial V}{\partial t} = VS_o\frac{\partial h}{\partial t} = 2\pi rdrbS_o\frac{\partial h}{\partial t} \quad (15.4)$$

$$= 2\pi rdrb\frac{S_s}{b}\frac{\partial h}{\partial t} = 2\pi rdrS_s\frac{\partial h}{\partial t} \quad (15.5)$$

Where S_s is the aquifer storativity which is equal to S_o / b

Putting (15.5) in (15.3), we have

$$-\frac{\partial Q_r}{\partial r}dr = 2\pi rdrS_s\frac{\partial h}{\partial t} \quad (15.6)$$

As per Darcy's law

$$Q_r = -KA\frac{\partial h}{\partial r} = -K2\pi rb\frac{\partial h}{\partial r} = -2\pi rT\frac{\partial h}{\partial r} \quad [\text{Putting } T = Kb] \quad (15.7)$$

Putting in equation (15.6)

$$\frac{\partial}{\partial r}\left(2\pi rT\frac{\partial h}{\partial r}\right)dr = 2\pi rdrS_s\frac{\partial h}{\partial t} \quad (15.8)$$

Simplifying,

$$\frac{1}{r}\frac{\partial}{\partial r}\left(r\frac{\partial h}{\partial r}\right) = \frac{S_s}{T}\frac{\partial h}{\partial t} \quad (15.9)$$

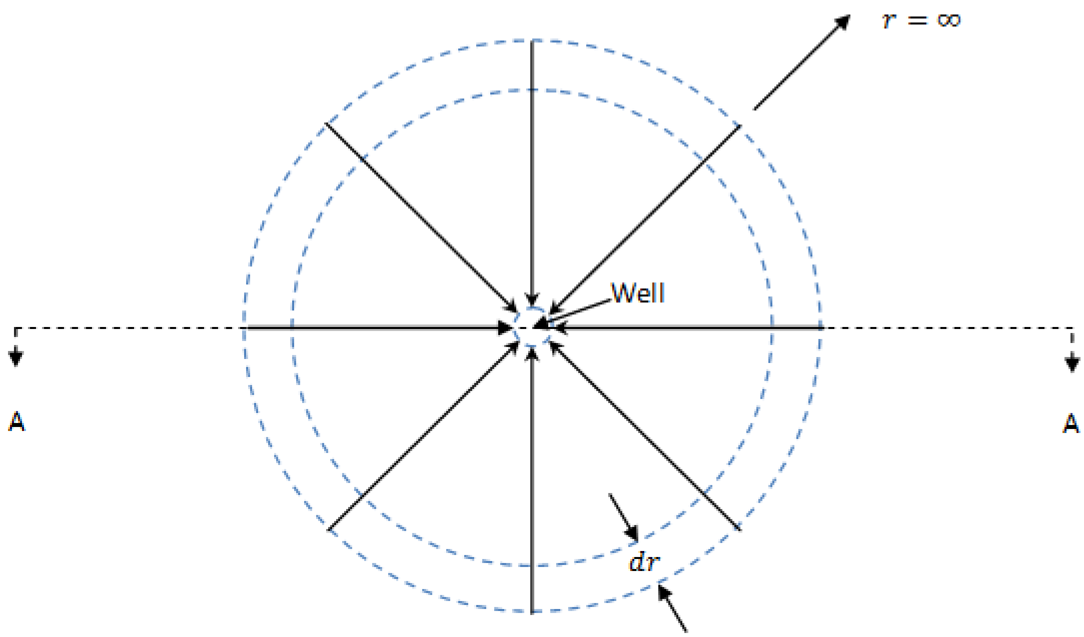
$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} = \frac{S_s}{T}\frac{\partial h}{\partial t} \quad (15.10)$$

This is the flow equation for radial flow into a well for confined homogeneous and isotropic aquifer.

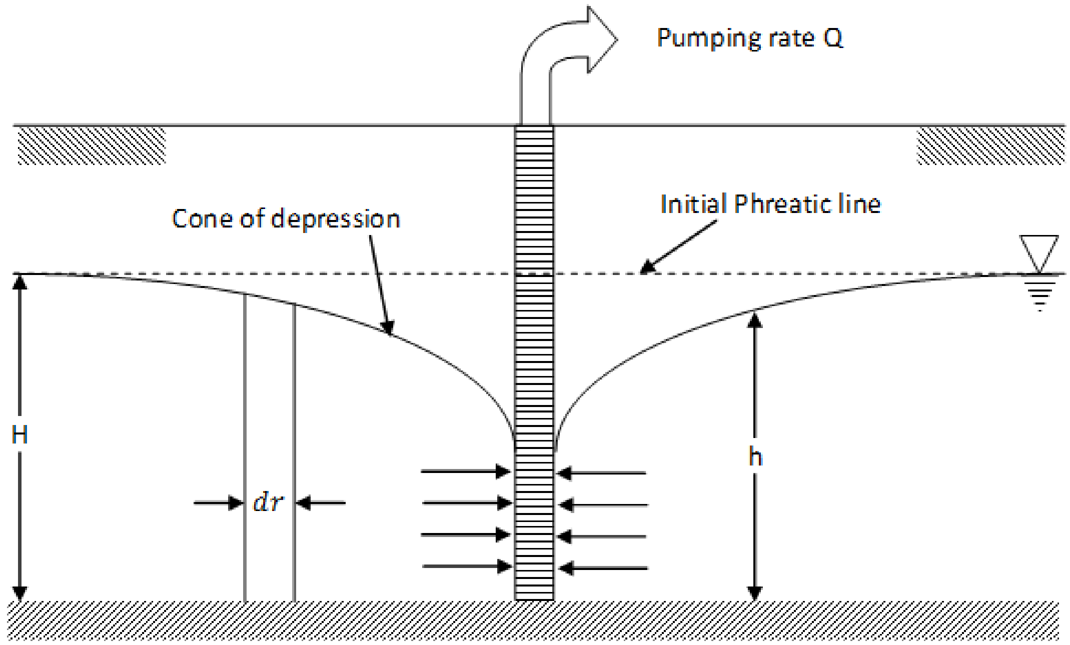
In case of steady state condition, the governing equation becomes,

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} = 0 \qquad (15.11)$$

Unconfined aquifer



- (a) Radial flow to a well



(b) Section A-A in case of unconfined aquifer

Fig. 15.4 An unconfined aquifer

Let us consider a case of radial flow to a single well (Fig. 15.4). The unconfined aquifer is homogeneous and isotropic. The hydraulic conductivity of the aquifer is K. The pumping rate (Q) of the aquifer is constant and the well diameter is infinitesimally small. The well is fully penetrated into the aquifer and hydraulic head in the aquifer prior to pumping is uniform throughout the aquifer.

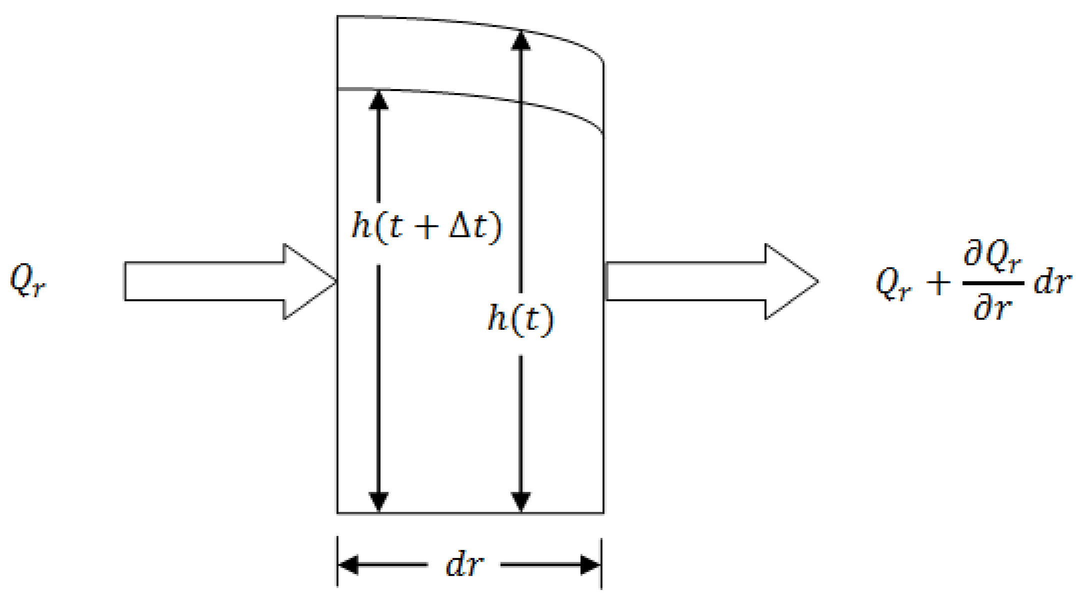


Fig. 15.5 Control volume

For the control volume shown in Fig. 15.5 above,

The inflow to the system is Q_r

The outflow from the system is $Q_r + \frac{\partial Q_r}{\partial r} dr$

The net inflow to the system is $Q_r - \left(Q_r + \frac{\partial Q_r}{\partial r} dr \right) = -\frac{\partial Q_r}{\partial r} dr$ (15.12)

Applying principle of mass conservation on the control volume

Inflow - outflow = Time rate of change in volumetric storage

Time rate of change in volumetric storage $= \frac{\partial V}{\partial t} = V \left(\frac{\partial V}{V \partial h} \right) \frac{\partial h}{\partial t}$ (15.13)

$$\frac{\partial V}{\partial t} = VS_o \frac{\partial h}{\partial t} \qquad (15.14)$$

where S_o is the specific storage $= \frac{\partial V}{V \partial h}$

Replacing V by $2\pi rdrh$, we have

$$\frac{\partial V}{\partial t} = VS_o \frac{\partial h}{\partial t} = 2\pi rdrhS_o \frac{\partial h}{\partial t} \qquad (15.15)$$

$$= 2\pi rdrh \frac{S_y}{h} \frac{\partial h}{\partial t} = 2\pi rdrS_y \frac{\partial h}{\partial t} \qquad (15.16)$$

Where S_y is the specific yield which is equal to S_o / h .

Now putting equation (15.16) in equation (15.14), we have

$$-\frac{\partial Q_r}{\partial r} dr = 2\pi rdrS_y \frac{\partial h}{\partial t} \qquad (15.17)$$

As per Darcy's law

$$Q_r = -KA \frac{\partial h}{\partial r} = -K2\pi rh \frac{\partial h}{\partial r} \qquad (15.18)$$

Putting in equation (14.17)

$$\frac{\partial}{\partial r}\left(K2\pi rh\frac{\partial h}{\partial r}\right)dr = 2\pi rdrS_y\frac{\partial h}{\partial t} \tag{15.19}$$

Simplifying, $$\frac{1}{r}\frac{\partial}{\partial r}\left(rh\frac{\partial h}{\partial r}\right) = \frac{S_y}{K}\frac{\partial h}{\partial t} \tag{15.20}$$

$$\Rightarrow \frac{1}{r}\frac{\partial}{\partial r}\left(\frac{r}{2}\frac{\partial h^2}{\partial r}\right) = \frac{S_y}{K}\frac{\partial h}{\partial t} \tag{15.21}$$

$$\Rightarrow \frac{1}{2r}\frac{\partial r}{\partial r}\frac{\partial h^2}{\partial r} + \frac{1}{r}\frac{r}{2}\frac{\partial^2 h^2}{\partial r^2} = \frac{S_y}{K}\frac{\partial h}{\partial t} \tag{15.22}$$

$$\Rightarrow \frac{1}{r}\frac{\partial h^2}{\partial r} + \frac{\partial^2 h^2}{\partial r^2} = \frac{2S_y}{K}\frac{\partial h}{\partial t} \tag{15.23}$$

This is the flow equation for radial flow into a well for unconfined homogeneous isotropic aquifer.

In case of steady state condition, the governing equation becomes,

$$\frac{1}{r}\frac{\partial h^2}{\partial r} + \frac{\partial^2 h^2}{\partial r^2} = 0 \tag{15.24}$$

Or, $$\frac{\partial}{\partial r}\left(rh\frac{\partial h}{\partial r}\right) = 0 \tag{15.25}$$

Solution unsteady flow problem of confined aquifer

flow equation for unsteady flow in confined aquifer. The equation can be written as,

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} = \frac{S_s}{T}\frac{\partial h}{\partial t} \tag{17.1}$$

Theis (1935) obtained the solution of the equation. His solution was based on the analogy between groundwater flow and heat conduction. Considering the following boundary conditions,

at $t = 0$ $\quad h = h_o$

at $t = \infty$ $\quad h = h_o$

The solution of the equation for $t \geq 0$ is

$$s(r,t) = \frac{Q}{4\pi T} W(u) \tag{17.2}$$

Where, $s(r,t)$ is the draw down at a radial distance r from, the well at time t, $u = \frac{r^2 S_s}{4Tt}$ and $W(u) = \int_u^\infty \frac{e^{-u}}{u} du$

$W(u)$ is the exponential integration and is known as well function. The well function $W(u)$ can be approximated as

$$W(u) = -0.5772 - ln(u) + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - \frac{u^4}{4.4!} + \cdots \tag{17.3}$$

Alternate analytical solution of radial flow equation

The flow equation we have derived early

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} = \frac{S_s}{T}\frac{\partial h}{\partial t} \tag{17.15}$$

Let us consider

$$u = \frac{r^2 S_s}{4Tt} \tag{17.16}$$

Thus,

$$r = \sqrt{\frac{4Ttu}{S}} \tag{17.17}$$

And

$$t = \frac{r^2 S_s}{4Tu} \tag{17.18}$$

Now we can write that

$$\frac{\partial h}{\partial r} = \frac{\partial h}{\partial u}\frac{\partial u}{\partial r} = \frac{\partial h}{\partial u}\left(\frac{2rS_s}{4Tt}\right) = \frac{\partial h}{\partial u}\left(\frac{2}{r}\frac{r^2 S_s}{4Tt}\right) = 2\left(\frac{u}{r}\right)\frac{\partial h}{\partial u} \tag{17.19}$$

Cone of Depression

- Before pumping, the water level in the well stands up to the same elevation as the water table or piezometric surface depending on the type of aquifer.
- When pumping starts, the water is removed from the aquifer surrounding the well, and in and around; the well the water table or piezometric surface is lowered and assumes the shape of an inverted cone which is known as cone of depression.
- The area of the base of this cone is known as the area of influence, because it is this area which gets affected by the pumping of the well.
- The boundary of the area of influence is known as the circle of influence. The radius of the circle of influence is known as the radius of influence.
- Further at any point the difference in elevation of the water table or piezometric surface before and after pumping is known as drawdown.
- The maximum drawdown occurs at the well and it decreases with increase in the distance from the well. The variation in drawdown with distance from the well is shown by a drawdown curve.
- The analysis of radial flow of ground water towards a well was first proposed by Dupuit (1863) and later modified by
- **The Dupuit-Theim theory is based on the following assumptions:**
- (i) The aquifer is homogeneous, isotropic, of uniform thickness and of infinite areal extent.
- (ii) The well penetrates and receives water from the entire thickness of the aquifer.
- (iii)The pumping has been continued for a sufficiently long time at a uniform rate so that an equilibrium stage or a steady flow condition has been reached.
- The coefficient of transmissibility is constant at all places and at all times.
- (v) The flow lines are radial and the flow of groundwater is horizontal.
- (vi) Flow is laminar and Darcy's law is applicable. However, the hydraulic gradient may be represented by tan 0 instead of sin 0 where 0 is the angle between the hydraulic grade line and the horizontal.
- (vi) The well is infinitely small with negligible storage and all the pumped water comes from the aquifer.

- On the basis of these assumptions the radial flow equations which relate the well discharge to drawdown for steady flow condition have been derived for wells completely penetrating a confined aquifer and an unconfined aquifer as indicated below.

Steady State Flow to Wells in Confined Aquifer (i.e., Artesian Wells or Pressure Wells):

- Figure shows a well of radius r fully penetrating a confined aquifer. Let b be the thickness of the aquifer measured between the top and bottom impervious strata, and H be the height of the initial piezometric surface measured above the impermeable strata at the bottom.
- When the well is pumped at a constant rate Q for a long time so that the water level in the well has been stabilized then the drawdown curve as shown in Fig. is developed. At this stage let h be the depth of water in the well measured above the impermeable strata at the bottom. Further let R be the radius of influence.

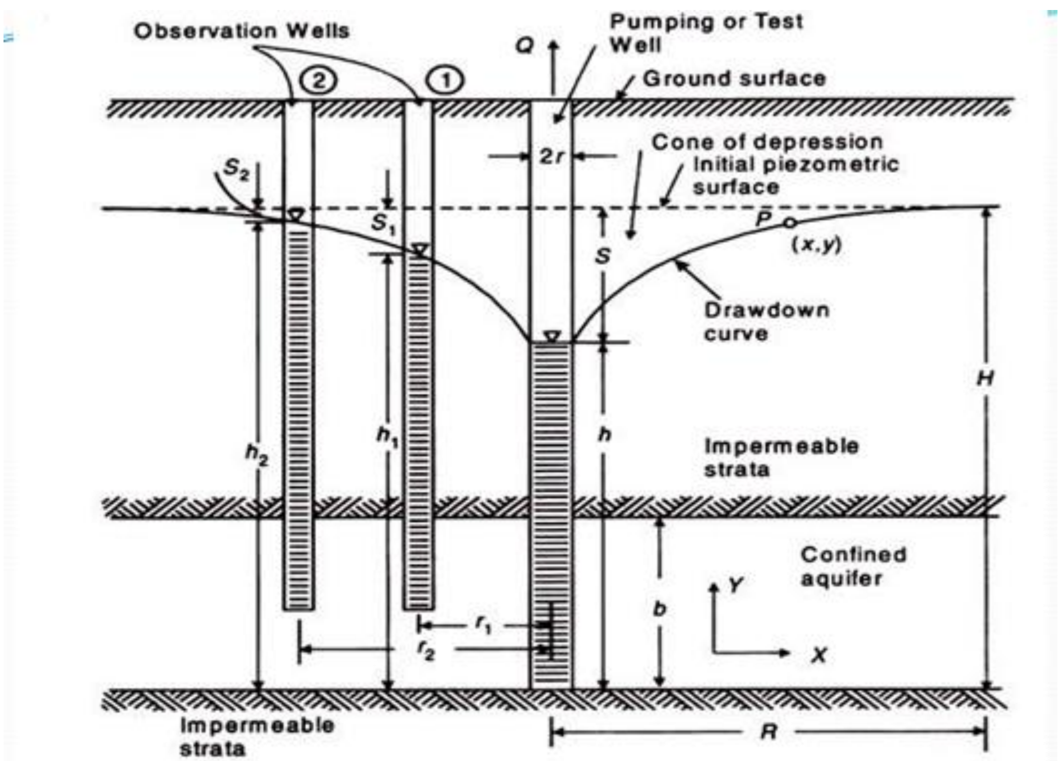


Fig Well penetrating confined aquifer

- Let (x, y) be the coordinates of any point P on the drawdown curve with respect to origin O at the centre of the well at its bottom, if a vertical cylindrical surface passing through point P and surrounding the well located at its centre is considered then the area of the portion of the cylindrical surface which is lying within the aquifer is equal to ($2\pi xb$).

- Further if (dy/dx) is the hydraulic gradient at P, then from Darcy's law the rate of flow of water through this portion of the cylindrical surface is equal to [k(dy/dx)2πxb] which by continuity is also equal to the well discharge

$$Q = k\frac{dy}{dx}(2\pi xb)$$

or $$Q\frac{dx}{x} = 2\pi kb\,dy \qquad \text{...(i)}$$

Integrating both sides of Eq. (i) between the limits, at $x = r, y = h$ at the well and at $x = R, y = H$ at the extremity of the area of influence, we get

$$Q\int_r^R \frac{dx}{x} = 2\pi kb \int_h^H dy$$

or $$Q = \frac{2\pi kb(H-h)}{\log_e(R/r)} \qquad \text{...(4.11)}$$

or $$Q = \frac{2\pi kb(H-h)}{2.303\log_{10}(R/r)} \qquad \text{...(4.11 a)}$$

or $$Q = \frac{2.73kb(H-h)}{\log_{10}(R/r)} \qquad \text{...(4.11 b)}$$

Equation 4.11 is known as equilibrium equation or Thiem equation.

If s is the drawdown at the well then since $s = (H - h)$, Eq. 4.11 may be expressed as

$$Q = \frac{2\pi kbs}{\log_e(R/r)} \qquad \text{...(4.12)}$$

or $$Q = \frac{2.73kbs}{\log_{10}(R/r)} \qquad \text{...(4.12 a)}$$

Further for a confined aquifer since the coefficient of transmissibility T = kb, Eqs. 4.11 and 4.12 become-

$$Q = \frac{2\pi T (H-h)}{\log_e (R/r)} \qquad \text{...(4.13)}$$

$$Q = \frac{2.73T (H-h)}{\log_{10} (R/r)} \qquad \text{...(4.13 a)}$$

$$Q = \frac{2\pi Ts}{\log_e (R/r)} \qquad \text{...(4.14)}$$

or

$$Q = \frac{2.73Ts}{\log_{10} (R/r)} \qquad \text{...(4.14 a)}$$

Again as indicated below the use of R can be avoided if the observation wells are available.

- As shown in Figure let there be two observation wells at radial distances r_1 and r_2 and the depth of water in them be h_1 and h_2 respectively.
- Integrating Eq. (i) between the limits, at $x = r_1$, $y = h_1$ at the observation well No. 1 and at $x = r_2$, $y = h_2$ at the observation well No. 2, the following equation may be obtained which does not involve R.

$$Q = \frac{2\pi kb(h_2 - h_1)}{\log_e(r_2/r_1)} \quad ...(4.15)$$

or

$$Q = \frac{2.73kb(h_2 - h_1)}{\log_{10}(r_2/r_1)} \quad ...(4.15\text{ a})$$

Further if s_1, and s_2 are the respective drawdowns at the two observation wells, then

$$h_2 = H - s_2$$

and

$$h_1 = H - s_1$$

Introducing these expressions in Eq. 4.15, we get

$$Q = \frac{2\pi kb(s_1 - s_2)}{\log_e(r_2/r_1)} \quad ...(4.16)$$

or

$$Q = \frac{2.73kb(s_1 - s_2)}{\log_{10}(r_2/r_1)} \quad ...(4.16\text{ a})$$

Further since $T = kb$ Eq. 4.16 may be expressed as

$$Q = \frac{2\pi T(s_1 - s_2)}{\log_e(r_2/r_1)} \quad ...(4.17)$$

or

$$Q = \frac{2.73T(s_1 - s_2)}{\log_{10}(r_2/r_1)} \quad ...(4.17\text{ a})$$

Steady State Flow to Wells in Unconfined Aquifer (i.e., Gravity Wells or Water Table Wells):

- Figure 4.29 shows a well of radius r completely penetrating an unconfined aquifer. Let H be the thickness of the aquifer measured from the impermeable strata to the initial level of the water table.

- When the well is pumped at a constant rate Q for a long time so that the water level in the well has been stabilised, i.e., an equilibrium stage or a steady flow condition has been reached, then the drawdown curve as shown in Fig. 4.29 is developed. At this stage let h be the depth of water in the well measured above the impermeable strata. Further let R be the radius of influence (or the radius of inappreciable or zero drawdown) measured from the centre of the well to a point where the drawdown is inappreciable.

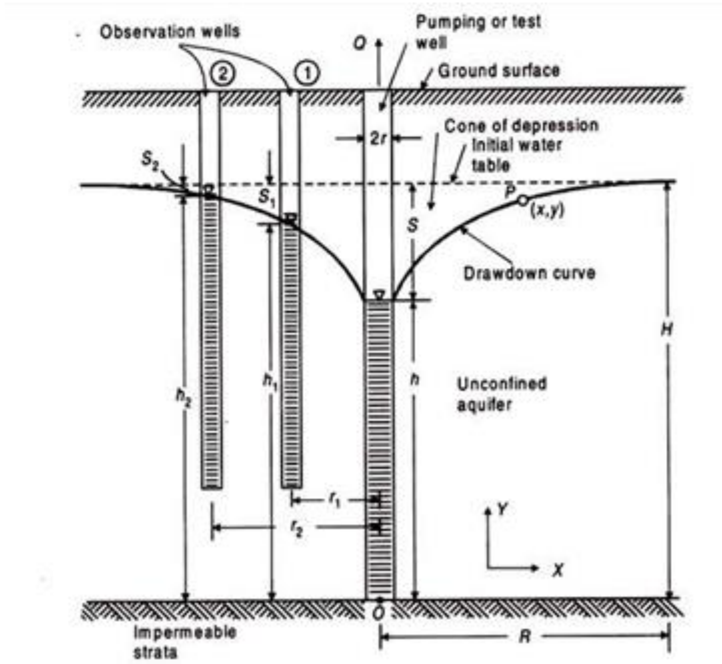


Fig Well penetrating an unconfined aquifer

- Considering the origin at a point O at the centre of the well at its bottom, let the coordinates of any point P on the drawdown curve be (x, y).
- If a vertical cylindrical surface passing through point P and surrounding the well located at its center is considered then the area of the portion of cylindrical surface which is lying within the aquifer below point P is equal to ($2\pi xy$).
- Further if (dy/dx) is the hydraulic gradient at P then from Darcy's law the rate of flow of water (or discharge)

through the cylindrical surface is equal to $\left[k\left(\frac{dy}{dx}\right)2\pi xy\right]$. By continuity this rate of flow of water is equal to the well discharge, and hence

$$Q = k\frac{dy}{dx}(2\pi xy)$$

or

$$Q\frac{dx}{x} = 2\pi ky\,dy \qquad \text{...(ii)}$$

Integrating both sides of Eq. (ii) between the limits, at $x = r$, $y = h$ at the well and at $x = R$, $y = H$ at the extremity of the area of influence, we get

$$Q\int_r^R \frac{dx}{x} = 2\pi k \int_h^H y\,dy$$

or

$$Q = \frac{\pi k\left(H^2 - h^2\right)}{\log_e(R/r)} \qquad \text{...(4.18)}$$

or

$$Q = \frac{\pi k\left(H^2 - h^2\right)}{2.303\log_{10}(R/r)} \qquad \text{...(4.18 a)}$$

or

$$Q = \frac{1.36k\left(H^2 - h^2\right)}{\log_{10}(R/r)} \qquad \text{...(4.18 b)}$$

Equation 4.18 may also be expressed in a different form as indicated below.

If s is the drawdown measured at the well then

$$s = H - h$$

$$H = s + h$$

and

$$H + h = s + 2h$$

$$\therefore \quad H^2 - h^2 = (H - h)(H + h) = s(s + 2h)$$

Introducing this expression in Eq. 4.18, we get

$$Q = \frac{\pi k s(s + 2h)}{\log_e (R/r)} \qquad ...(4.19)$$

or

$$Q = \frac{1.36 k s(s + 2h)}{\log_{10} (R/r)} \qquad ...(4.19\ a)$$

If the drawdown s is small then $(H + h) \approx 2H$ and hence

$$H^2 - h^2 = (H + h)(H - h) = 2H(H - h) = 2Hs$$

Thus from Eq. 4.18, we get

$$Q = \frac{2\pi k H s}{\log_e (R/r)} \qquad ...(4.20)$$

or

$$Q = \frac{2.73 k H s}{\log_{10} (R/r)} \qquad ...(4.20\ a)$$

Since for an unconfined aquifer the coefficient of transmissibility $T = kH$, Eq. 4.20 becomes

$$Q = \frac{2\pi Ts}{\log_e (R/r)} \qquad \text{...(4.21)}$$

or

$$Q = \frac{2.73\, Ts}{\log_{10} (R/r)} \qquad \text{...(4.21 a)}$$

Equation 4.21 is similar to the one derived for a confined aquifer.

These equations can however be used only if the radius of influence R is known. In practice, the selection of the radius of influence R is approximate and arbitrary, but the variation in Q is small for a wide range of R. The values of R in general fall in the range of 150 to 300 metres.

As shown in Fig. 4.29 let there be two observation wells at radial distances r_1 and r_2 and the depths of water in them be h_1 and h_2 respectively. Integrating both sides of Eq. (ii) between the limits at $x = r_1$ $y = h_1$ at the observation well No. 1 and at $x = r_2$, $y = h_2$ at the observation well No. 2, the following equation may be obtained which does not involve R.

$$Q = \frac{\pi k \left(h_2^2 - h_1^2\right)}{\log_e (r_2/r_1)} \qquad \text{...(4.23)}$$

or

$$Q = \frac{1.36 k \left(h_2^2 - h_1^2\right)}{\log_{10} (r_2/r_1)} \qquad \text{...(4.23 a)}$$

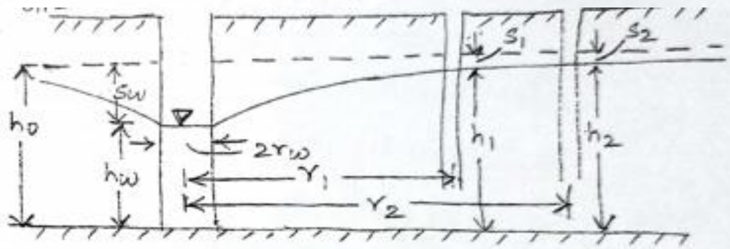
② A 30cm well penetrates 50m below the static water table. After a long period of pumping at a rate of 1800 lpm, the drawdowns in the wells at 15 and 45m from the pumped well are 1.7m and 0.8m, respectively. Determine the transmissibility of the aquifer. What is the drawdown in the pumped well.

$h_0 = 50m$, $r_w = 0.15m$

$Q = 1.8\ m^3/min.$

$r_1 = 15m$, $r_2 = 45m$

$s_1 = 1.7m$, $s_2 = 0.8m$



$\therefore h_1 = 48.3m$, $h_2 = 49.2m$.

$$Q = \frac{\pi k\,(h_2^2 - h_1^2)}{2.303 \log_{10}(r_2/r_1)} \Rightarrow 1.8 = \frac{\pi \times k \times (49.2^2 - 48.3^2)}{2.303 \log_{10}\left(\frac{45}{15}\right)}$$

$$k = 7.195 \times 10^{-3}\ m^3/min = 7.195 \times 60 \times 24$$

$$k = 10.36\ m/day., \quad T = kh_0 = 10.36 \times 50 = 518\ m^2/day$$

Assuming $r_0 = 300m$, $Q = \dfrac{\pi k\,(h_0^2 - h_w^2)}{2.303 \log_{10}\left(\frac{r_0}{r_w}\right)}$.

$$1.8 = \frac{\pi \times 7.195 \times 10^{-3} \times (h_0^2 - h_w^2)}{2.303 \times \log_{10}\left(\frac{300}{0.15}\right)} \Rightarrow h_0^2 - h_w^2 = 605m$$

$$h_w^2 = h_0^2 - 605 = 1894$$

$$h_w = 43.5m$$

$$s_w = h_0 - h_w = 6.47m$$

5. In an artesian aquifer of 8m thick, a 10cm diameter well is pumped at a constant rate of 150 lpm. The steady state drawdown observed in two wells located at 15m and 40m distance from the centre of the well are found to be 2m and 0.05m respectively. Compute T & K of the aquifer.

$Q = 150\ \text{lpm}$ (or) $0.0025\ m^3/sec.$, $b = 8m$

$r_1 = 15m$, $r_2 = 40m$, $S_1 - S_2 = 2 - 0.05$

$S_1 - S_2 = 1.95m.$

$$Q = \frac{2\pi kb(S_1 - S_2)}{2.303 \log_{10} \frac{r_2}{r_1}} \quad \text{(or)} \quad \frac{2.72\, T(S_1 - S_2)}{\log_{10}(r_2/r_1)}$$

$$0.0025 = \frac{2.72 \times T \times 1.95}{\log_{10}(40/15)}$$

$$T = 2.007 \times 10^{-4}\ m^2/sec \quad \text{(or)} \quad 17.34\ m^2/day.$$

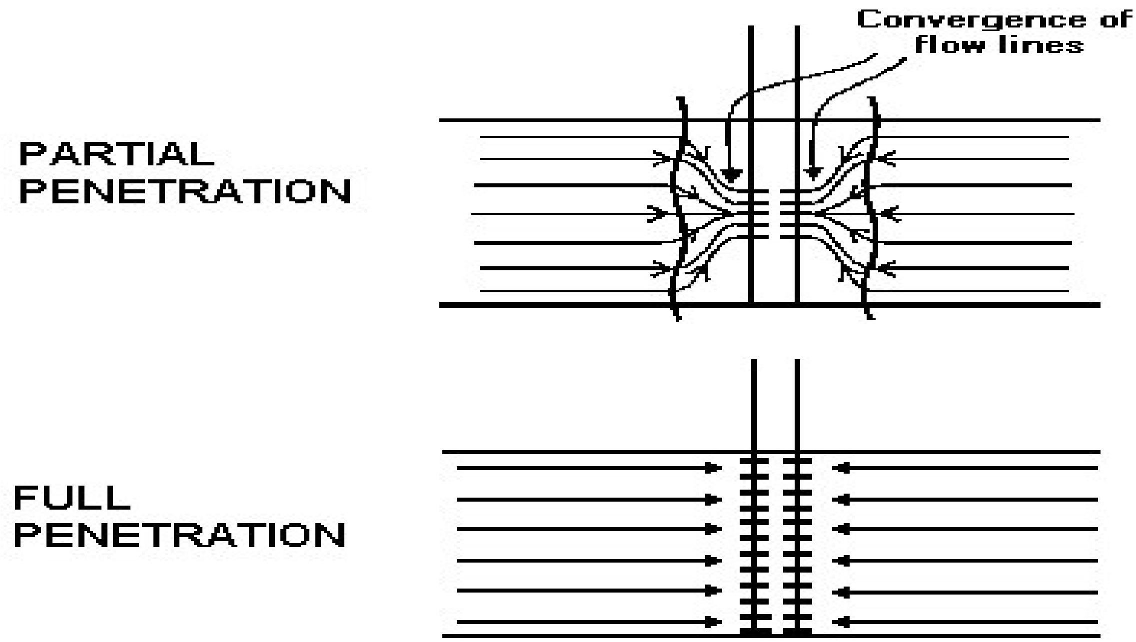
$2.007 \times 60 \times 60 \times 24$

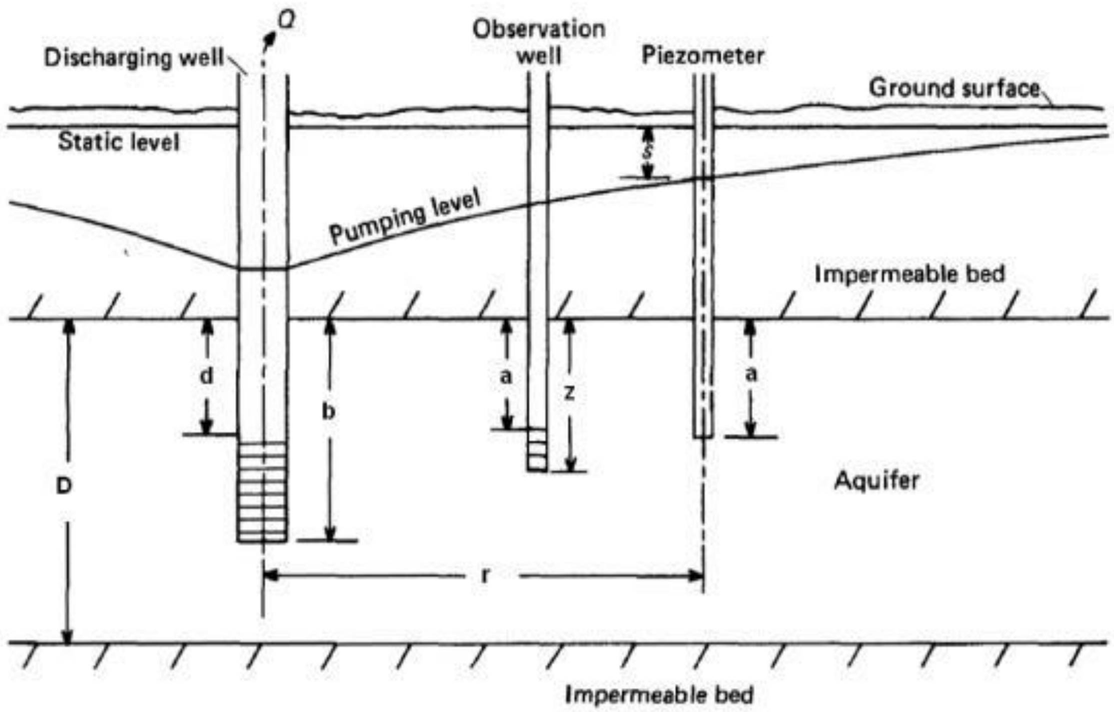
$T = K \times b \Rightarrow 17.34 = K \times 8$

$K = 2.16\ m/day$

Partially penetrating well

- Partial penetration, especially for confined aquifers, may significantly affect the associated well(s) hydraulics due to the incidence of vertical flow in the vicinity of the well.
- This induces additional head losses, affects the potential pumping rate, and the related peizometeric head distribution around the well.





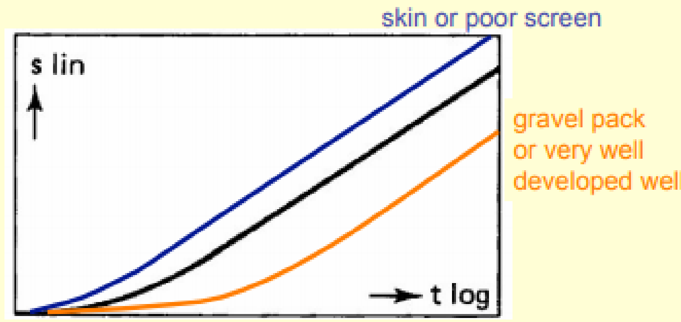
Variations from "normal" drawdown hydrographs

In-well pumping tests

- **It's best to avoid using data collected in the pumping well, data – Inappropriate or Incorrect-could damage pump or measuring equipment**
- **permeability adjacent to the well bore can be very different than in the bulk of the aquifer**
- **Bentonite clay "skin" of low permeability**
- **gravel pack**
- **well screen**

Variations from “normal” drawdown hydrographs

Skin effects



base figure from Kruseman and de Ridder (1991)

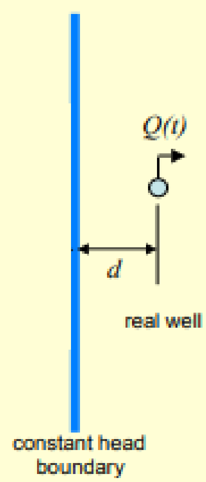
Can get T (slope is fine), but estimates of S will be off—you should never try to estimate S from a pumping well

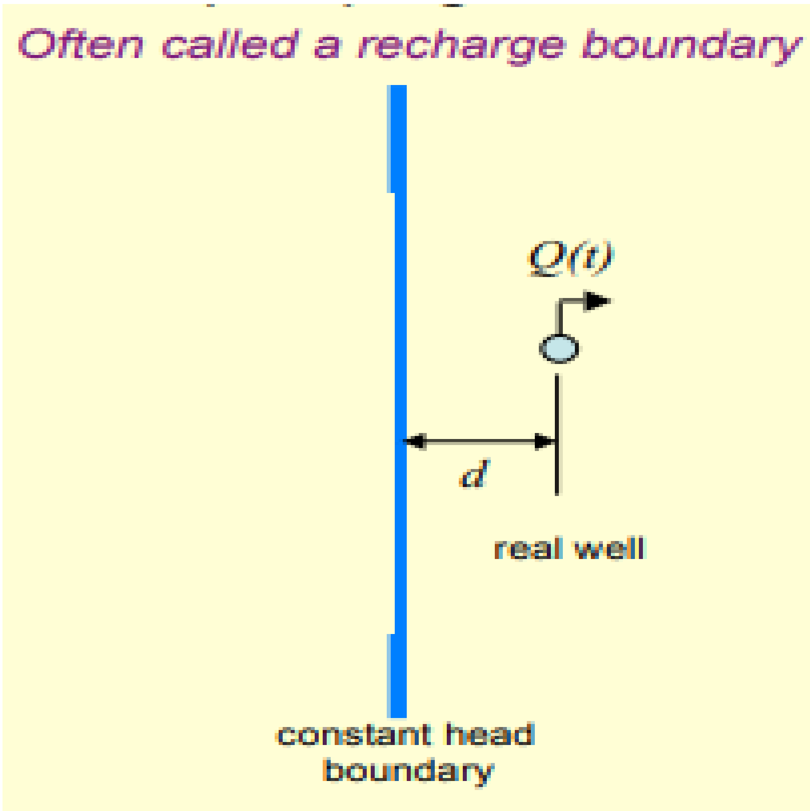
14

Image Well Theory

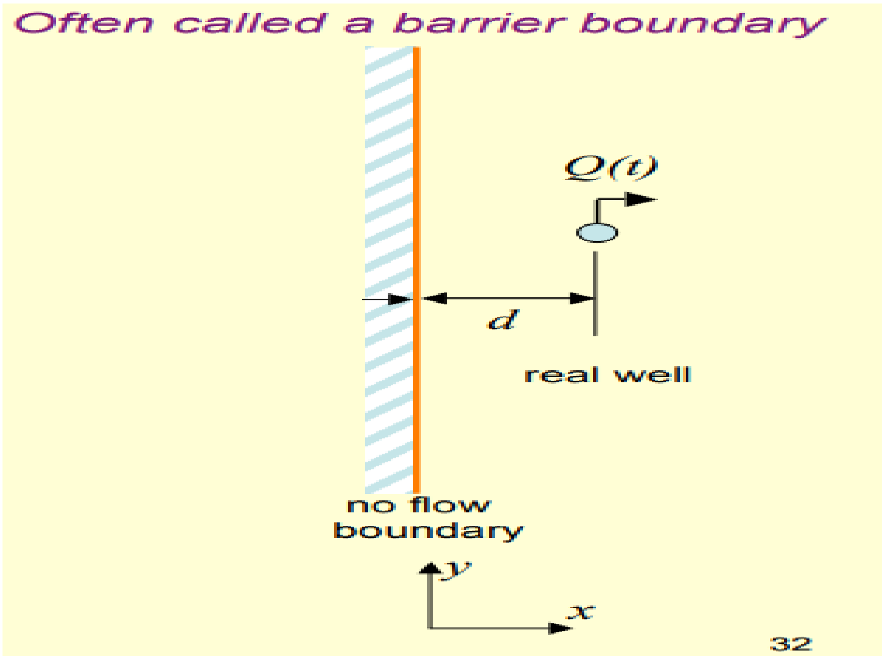
With multiple wells, not all of the wells need be real.

We can use the concept of virtual or “image” wells to superpose drawdowns in order to mimic Dirichlet and Neumann boundary conditions.

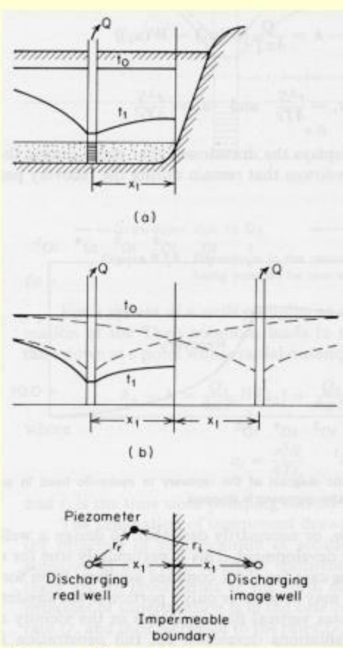




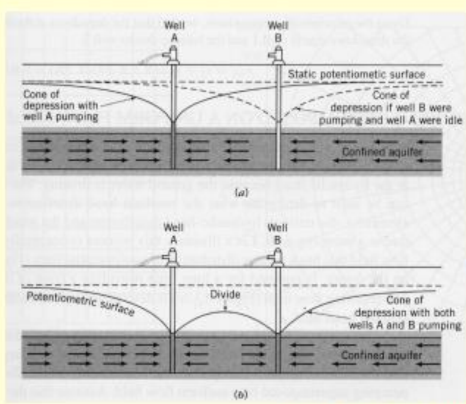
- River/stream/ coastal regions are considered as Constant head Boundary



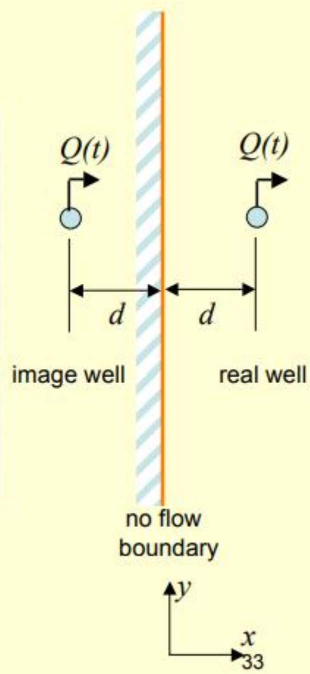
How would represent a straight line no-flux Dirichlet boundary located a distance d from the pumping well?



(Freeze and Cherry, 1979)

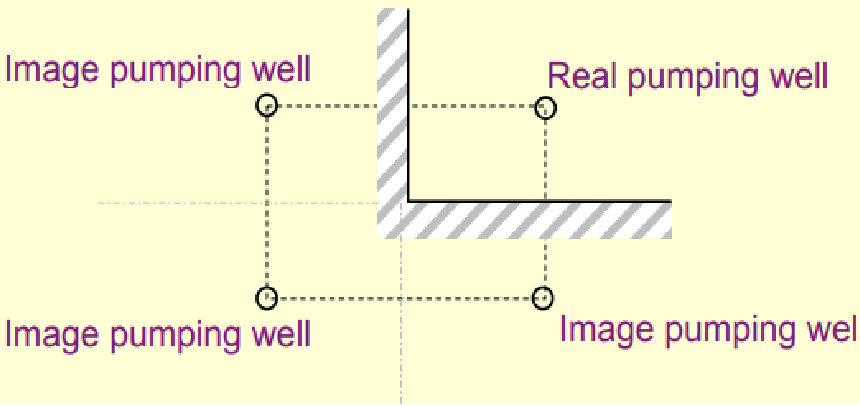


(Schwartz and Zhang, 2003)



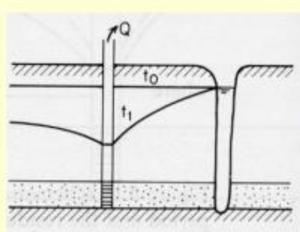
You can use image wells in other geometric patterns to mimic more complex geometric domains (see e.g., Bear, 1972)

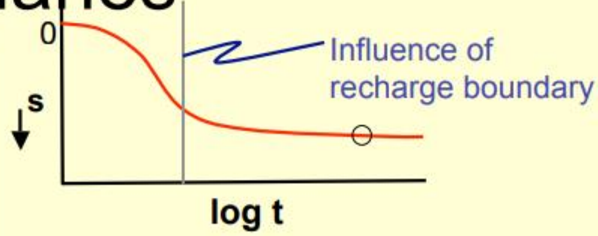
Well near a "corner":



Well Test Analysis with Boundaries

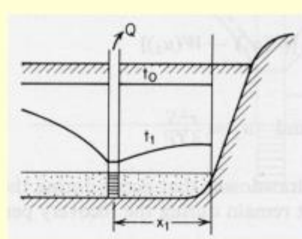
Constant head or recharge boundary

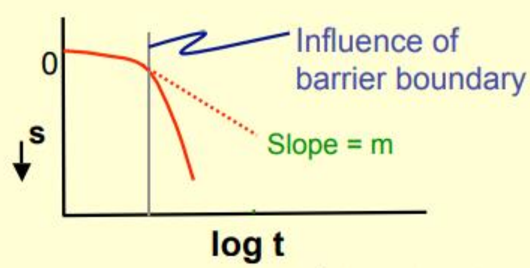




$$s = s_{real} + s_{image} = \frac{Q}{2\pi T}\ln\left(\frac{r_2}{r_1}\right)$$

Barrier or no-flow boundary





$$s = s_{real} + s_{image} = \frac{Q}{4\pi T}\left[\ln\left(\frac{2.25\,T\,t}{r_r^2 S}\right) + \ln\left(\frac{2.25\,T\,t}{r_i^2 S}\right)\right]$$

In a well when the intake of the well is less than the thickness of the well, then the well is called partially penetrated well. In case of partially penetrated well, the flow lines are not truly horizontal near the well. The flow lines are curved upward or downward near the well. However, at a distance far away from the well, the flow lines are horizontal. As a result of non-horizontal nature of the flow lines near the well, the length of the flow lines are more than the case of a fully penetrated well. Thus the drawdown in case of partially penetrating well is more than the fully penetrating well. Fig. 20.1 shows a partially penetrated well.

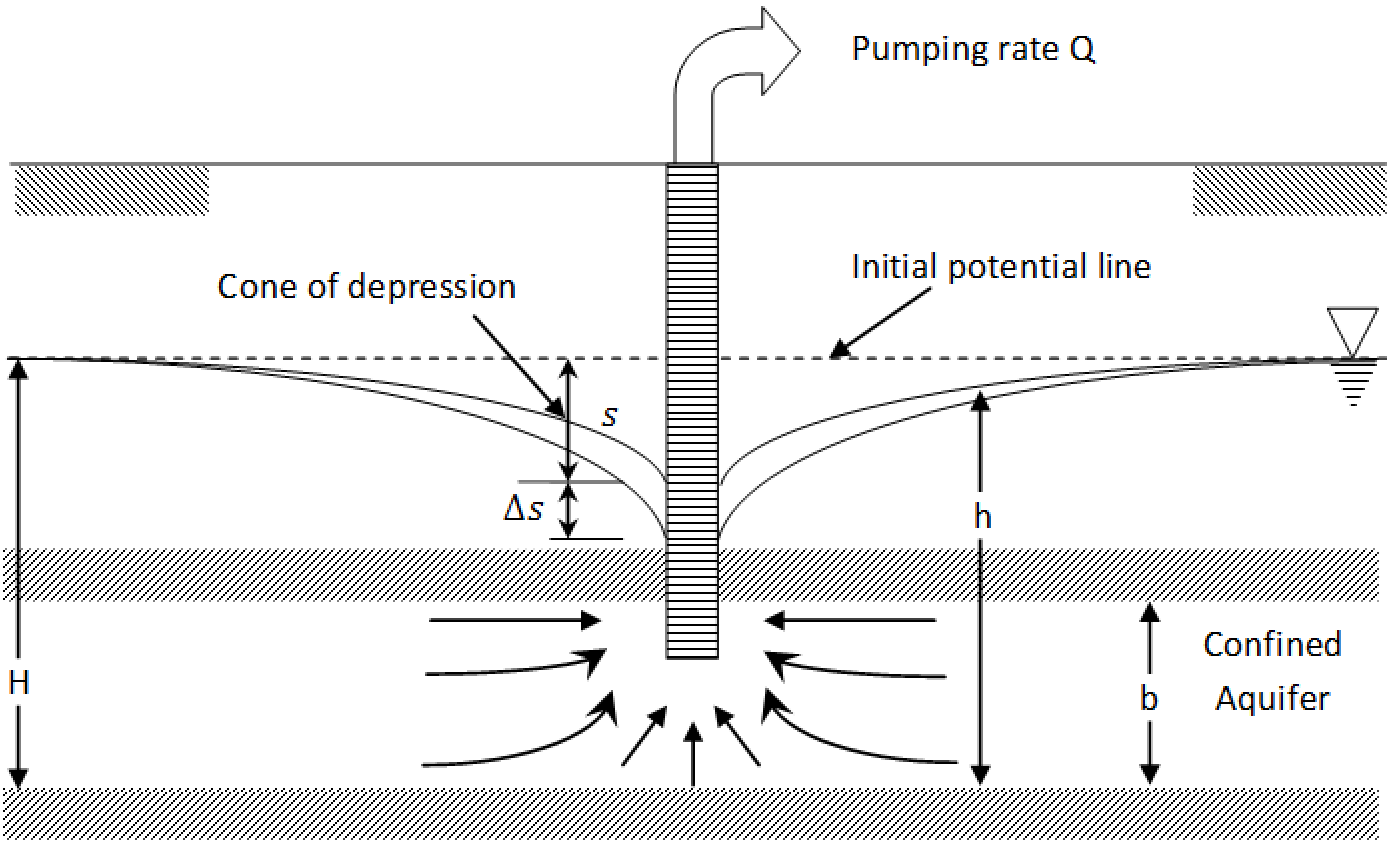


Fig. 20.1 Partially penetrated well

The drawdown of the partially penetrated well can be written as

$$s_p = s + \Delta s \tag{20.1}$$

Where, S is the drawdown of the fully penetrated well and Δ_s is the additional drawdown due to partial penetration.

For the Fig. 20.2 given below,

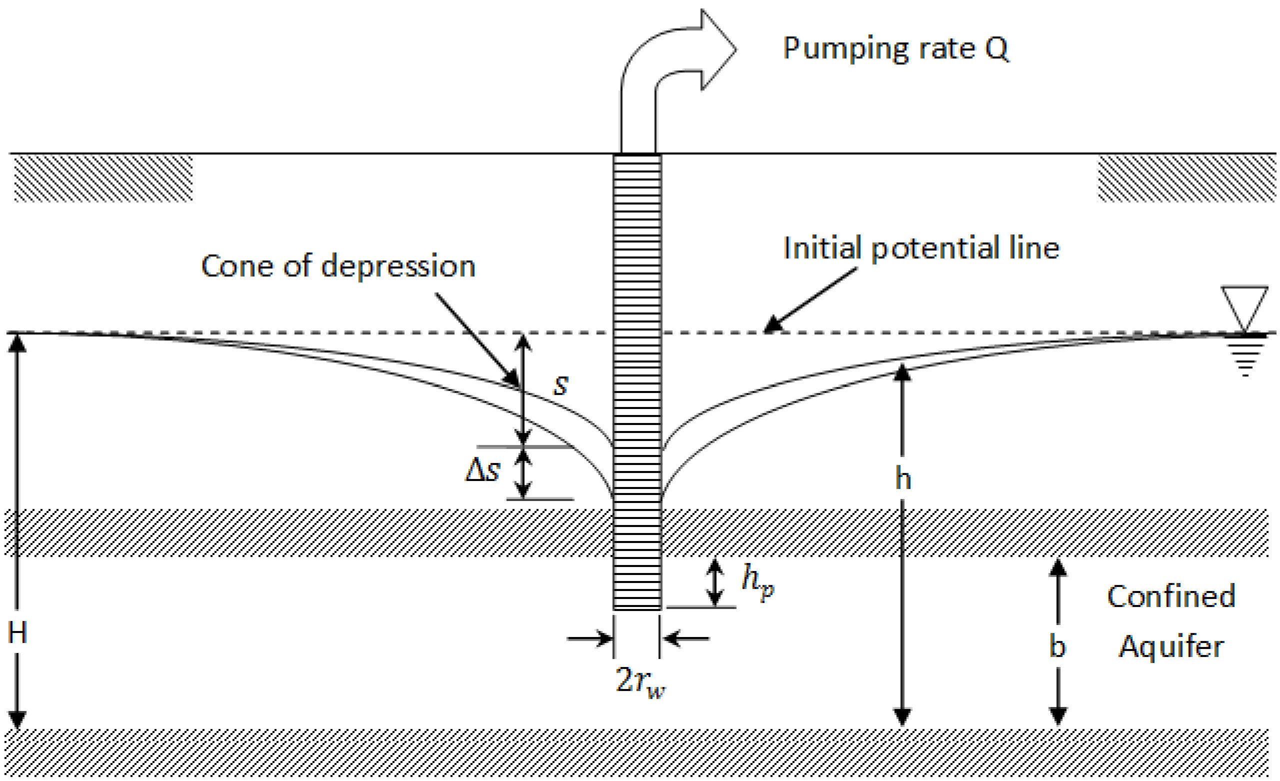


Fig. 20 .2 Partially penetrated well

Multiple well systems

In a well field, when cone of depression of one well overlaps with the cone of depression of other wells, then the actual drawdown will be more than the drawdown calculated for the individual well (Fig. 20.1). In this case, the actual drawdown can be calculated using the principle of superposition of linear system.

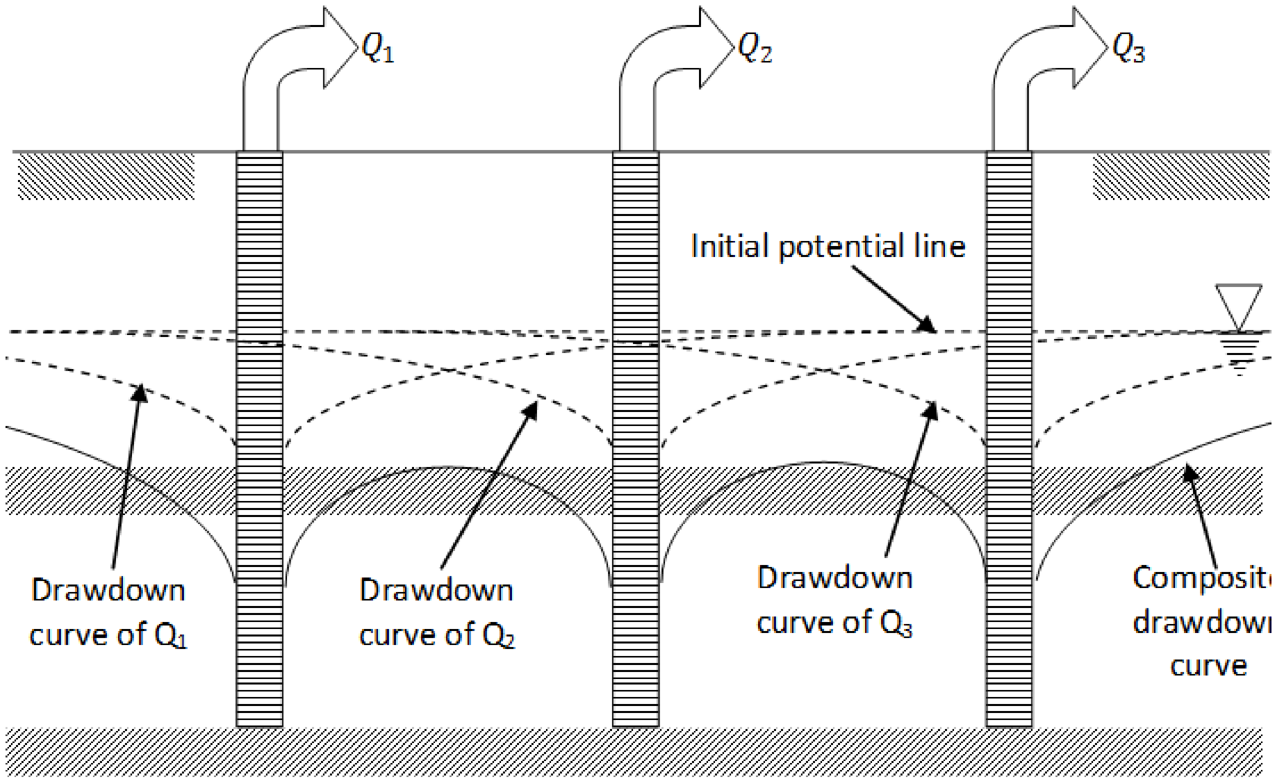


Fig. 21.1 Multiple well system

For a well field of *n* wells, the actual drawdown can be calculated as

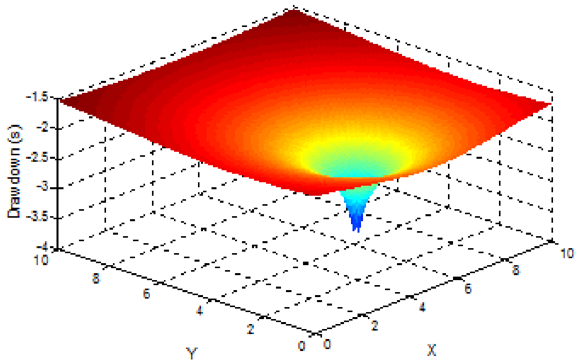
$$s_a(r,t) = s_1(r,t) + s_2(r,t) + s_3(r,t) + s_4(r,t) + s_5(r,t) + \cdots + s_n(r,t) \qquad (21.1)$$

or,

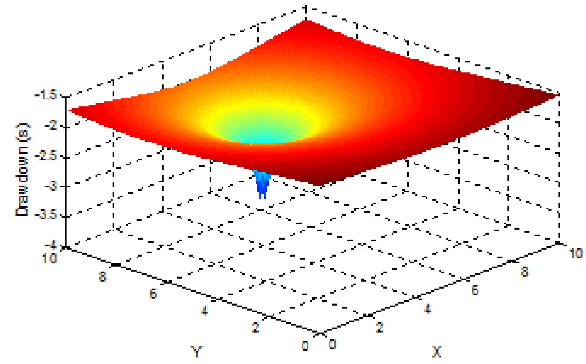
$$s_a(r,t) = \sum_{i=1}^{n} s_i(r,t)$$

Where S_a is the actual drawdown at a distance *r* at time *t* , S_i is the drawdown at that point caused by the discharge of the well *i* at time *t*, *n* is the number of wells in the well fields.

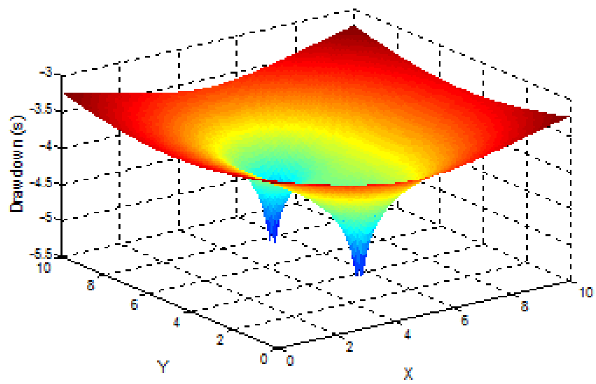
Fig. 21.2 explains the interference of cone of depression of two pumping wells. The coordinates of the two wells are (3,5) and (7,5). The individual cone of depression of the two wells are shown on Fig. 21.2 (a) and (b). The combine effect of the two wells can be obtained by adding the individual drawdown of the two wells, *i.e.* if drawdown of the first well is S_1 and the second well is S_2, the combine drawdown will be $S = S_1 + S_2$. The combine effect is shown in Fig. 21.2(c).



a) Drawdown of first well



(b) Drawdown of second well



(c) Combine drawdown

Fig. 21.2 Cone of depression of multiple wells system

Wells near aquifer boundaries

The assumption of infinite horizontal extend is no longer valid when water is pumped from a well near the aquifer boundary. Method of superposition can be used to implement the effect of aquifer boundary by adding a well at different location. The well that creates the same effect as boundary is called image well.

Well near a stream

Fig. 21.3 shows a well near a stream. In this case, the actual drawdown at the stream boundary will be zero as stream is considered as an infinite source. In order to maintain zero drawdown, an imaginary recharge well is considered at a distance equal to the distance between the pumping well and the stream boundary.

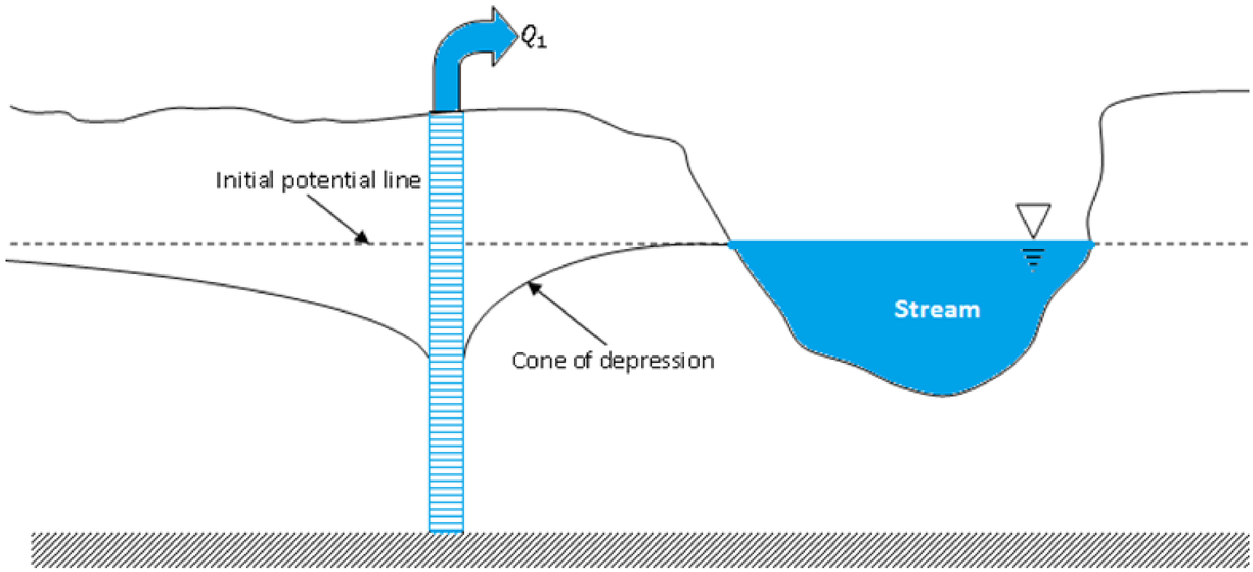


Fig. 21.3 Well near a stream

Fig. 20.4 shows an equivalent hydraulic system in an aquifer of infinite areal extend. For the equivalent hydraulic system, the time drawdown relationship for the pumping well and also for the imagery recharge well can be obtained separately. The actual drawdown can be obtained using the principle of superposition.

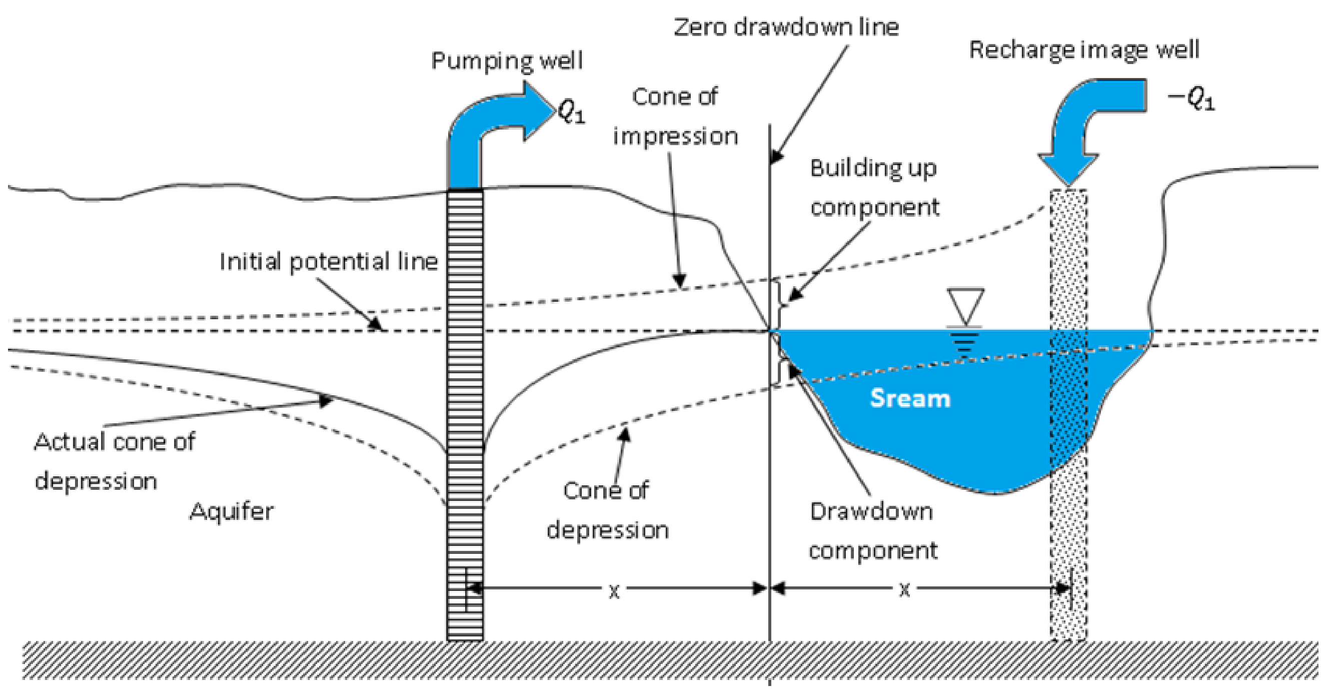


Fig. 21.4 Equivalent hydraulic system in a aquifer of infinite areal extend

Consider the Fig. 21.5 below. The pumping well is at a distance of *x* from the stream boundary. In order to calculate the actual drawdown at the observation location, an image well is

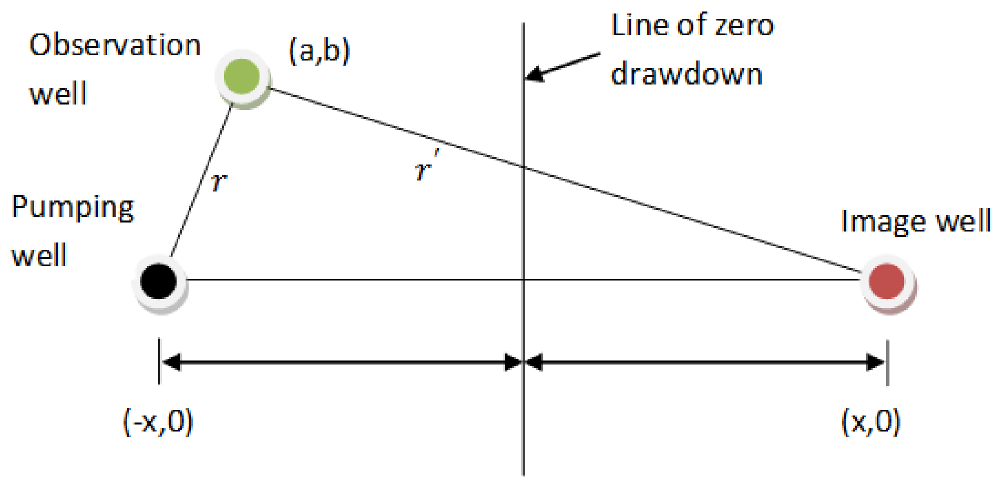


Fig. 21.5 Pumping well, Observation well and Image well

considered at a distance of *x* on the other side of the line of zero drawdown. The distance of the observation well from the pumping well is *r* and from the image well is *r*′.

For the steady state condition of a confined aquifer, the drawdown at the observation well can be obtained as

$$s(a,b) = \frac{Q}{2\pi T} ln\left(\frac{R}{r}\right) + \frac{-Q}{2\pi T} ln\left(\frac{R}{r'}\right) \tag{21.2}$$

$$\Rightarrow s(a,b) = \frac{Q}{2\pi T} ln\left(\frac{r'}{r}\right) \tag{21.3}$$

$$\Rightarrow s(a,b) = \frac{Q}{4\pi T} ln\left(\frac{(a+x)^2+b^2}{(a-x)^2+b^2}\right) \tag{21.4}$$

For the unsteady condition, the drawdown at *r* at any time *t* can be obtained as

$$s_{(r,t)} = \frac{Q}{4\pi T} W\left(\frac{r^2 S_s}{4Tt}\right) + \frac{-Q}{4\pi T} W\left(\frac{r'^2 S_s}{4Tt}\right) \tag{21.5}$$

$$s_{(r,t)} = \frac{Q}{4\pi T}\left[W\left(\frac{r^2 S_s}{4Tt}\right) - W\left(\frac{r'^2 S_s}{4Tt}\right)\right] \tag{21.6}$$

Well near an impermeable boundary

Fig. 21.6 shows a well near an impermeable boundary. In this case, the actual drawdown at the

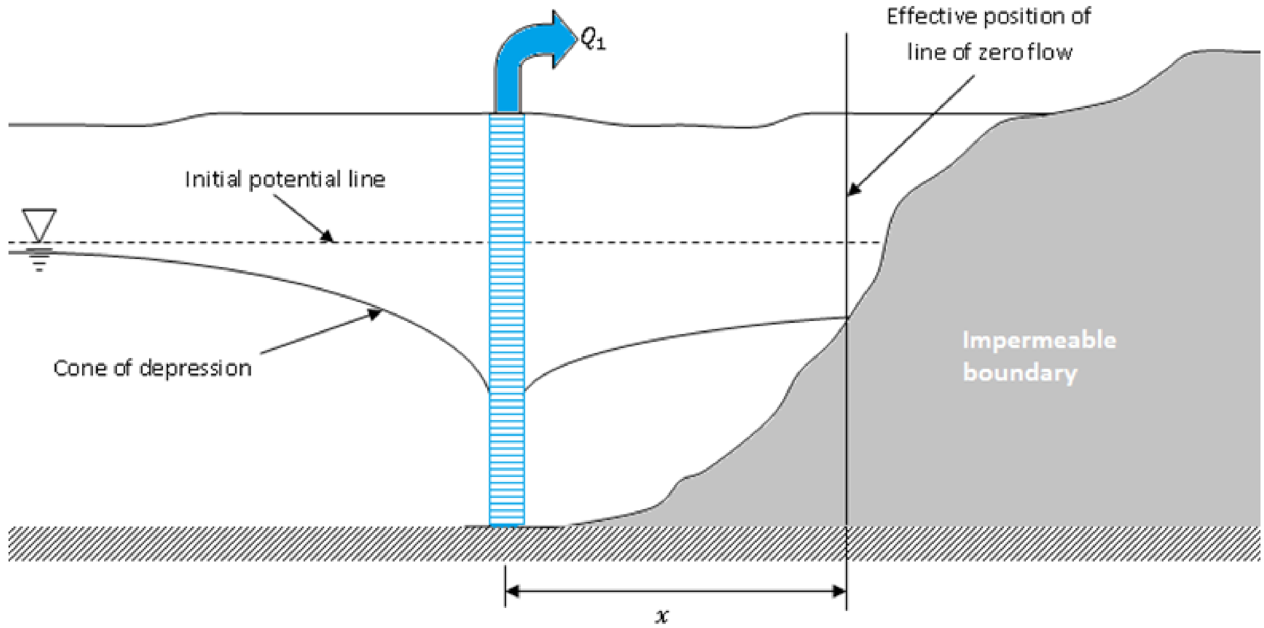


Fig. 21.6 Well near an impermeable boundary

impermeable boundary will be more than the drawdown calculated considering infinite areal extend of the aquifer medium. This problem can be solved by considering an imaginary pumping well at a distance equal to the distance between the pumping well and the image pumping well. Fig. 21.7 has shown the equivalent hydraulic system in an aquifer with infinite areal extent. For the equivalent hydraulic system, the time drawdown relationship for the pumping well and also for the imagery recharge well can be obtained separately. The actual drawdown can be obtained using the principle of superposition.

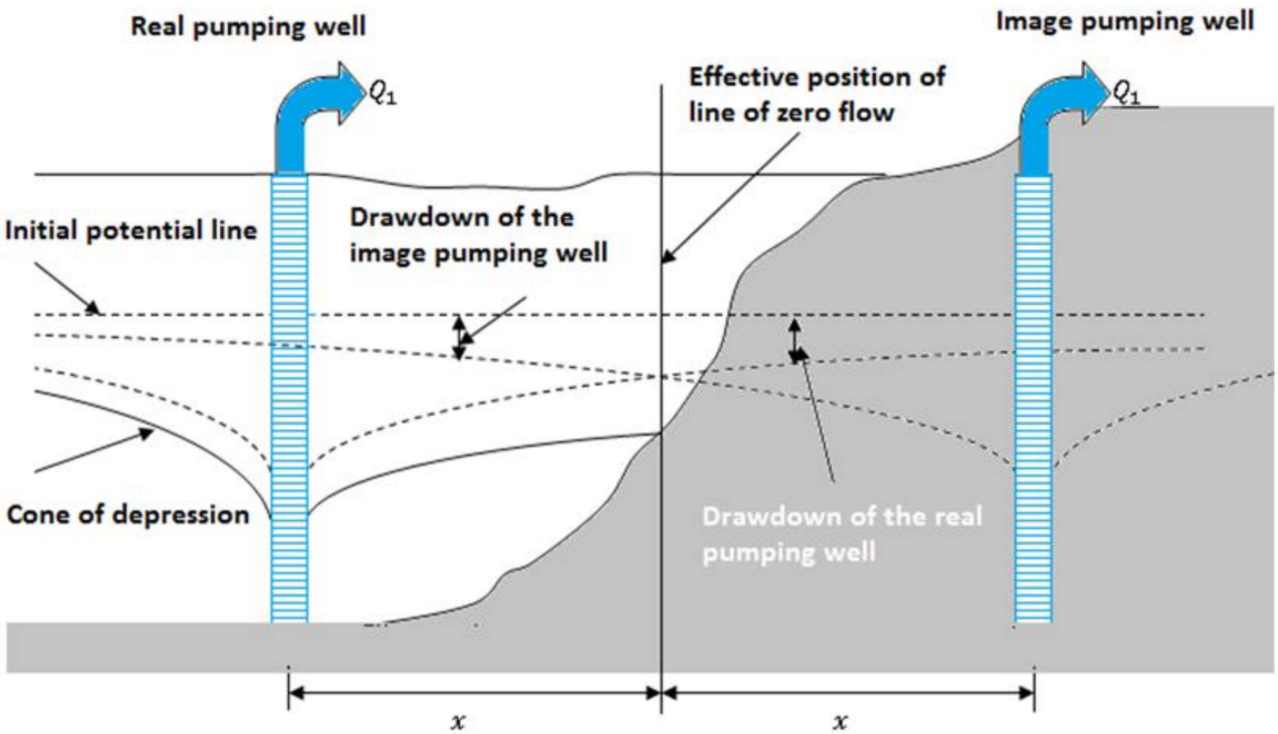


Fig. 21.7 Equivalent hydraulic system in a aquifer of infinite areal extend

Consider the Fig. 21.8 below. The pumping well is at a distance of x from the impermeable boundary. In order to calculate the actual drawdown at the observation location, an image well is considered at a distance of x on the other side of the line of zero flow. The distance of the observation well from the pumping well is r and from the image well is r' .

For the unsteady condition, the drawdown at a distance r at any time t can be obtained as,

$$s_{(r,t)} = \frac{Q}{4\pi T} W\left(\frac{r^2 S_s}{4Tt}\right) + \frac{Q}{4\pi T} W\left(\frac{r'^2 S_s}{4Tt}\right) \tag{21.7}$$

$$s_{(r,t)} = \frac{Q}{4\pi T}\left[W\left(\frac{r^2 S_s}{4Tt}\right) + W\left(\frac{r'^2 S_s}{4Tt}\right)\right] \tag{21.8}$$

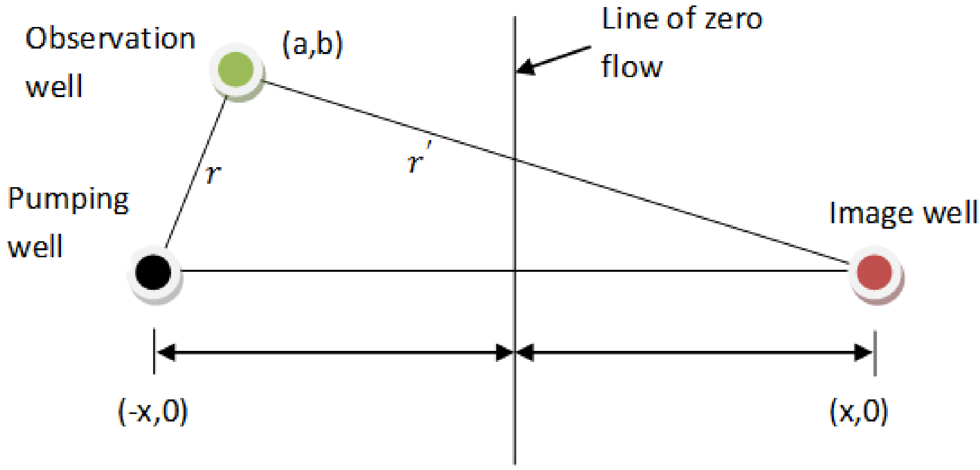


Fig. 21.8 Pumping well, observation well and image well

The transmissivity *(T)* of a confined aquifer and the hydraulic conductivity *(K)* of an unconfined aquifer can be calculated using the equation (16.14) and (16.30) respectively. These two equations were derived for steady state condition. It may be noted that it is difficult to obtain steady state pumping drawdown data as one has to continue the pumping for longer period. The unsteady flow data can be used to calculate both hydraulic conductivity ortransmissivity and storage coefficient of an aquifer. In this lecture we will mainly discuss the estimation of aquifer parameters using unsteady flow data.

Theis method – Unsteady Flow

The Theis equation can be written as

$$s = \left(\frac{Q}{4\pi T}\right) W(u) \tag{22.1}$$

Where *W(u)* is the well function and *u* is

$$u = \frac{r^2 S_s}{4Tt} \tag{22.2}$$

$$\Rightarrow \frac{r^2}{t} = \left(\frac{4T}{S}\right) u \tag{22.3}$$

In can be observed from the above relations that relation between $W(u)$ and u must be same as S and r^2/t. Using this similarity, the aquifer parameters (T,S_s) of confined aquifer can be estimated. The method for estimation of aquifer parameters can be summarized as follows.

- In a logarithmic paper plot the relationship between $W(u)$ and u. This is known as type curve.

- From the observed time drawdown data, plot the relationship between r^2/t and S on another logarithmic paper of same size.

- The observed r^2/t verses S relationship is then superimposed with the type curve in such a way that observed data fall on the segment on the type curve.

- From the two superimposed relations, the values of $W(u)$, u, S, and r^2/t are noted corresponding to a suitable convenient point.

- Now compute the aquifer parameters (T,S_s) using the equations (22.2) and (22.3).

Fig. 22.1 and Fig 22.2 show the relation between $W(u)$ and u, and r^2/t and S. The r^2/t and S is obtained from observed data.

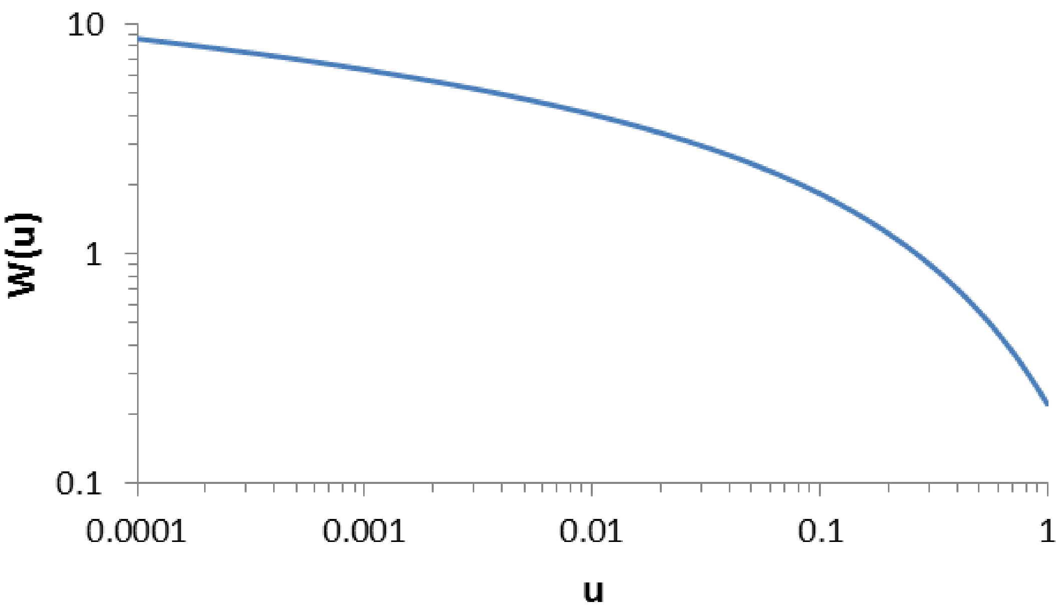


Fig.22.1 Relation between $W(u)$ and u

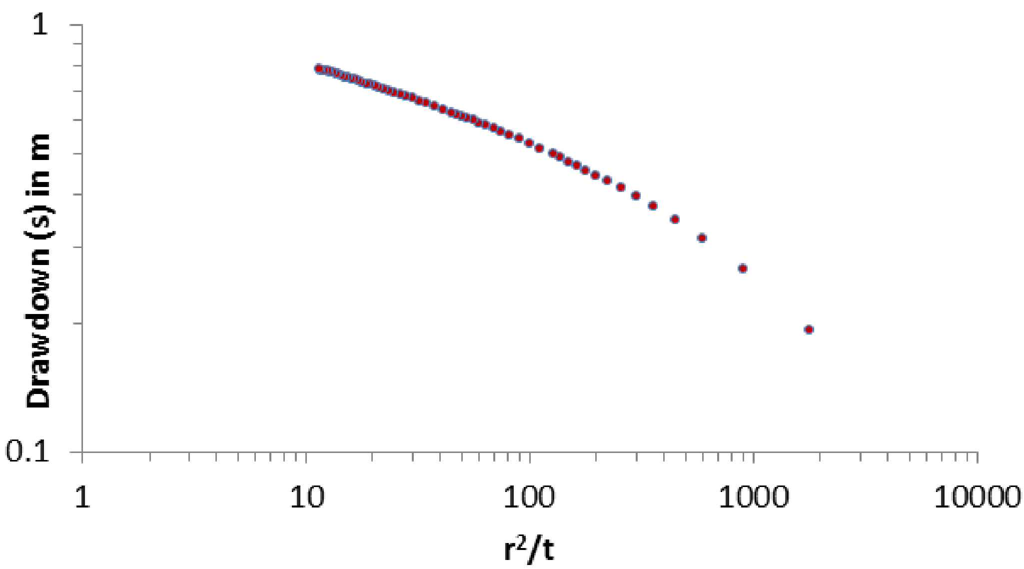


Fig 22.2 Relation between r^2/t and S

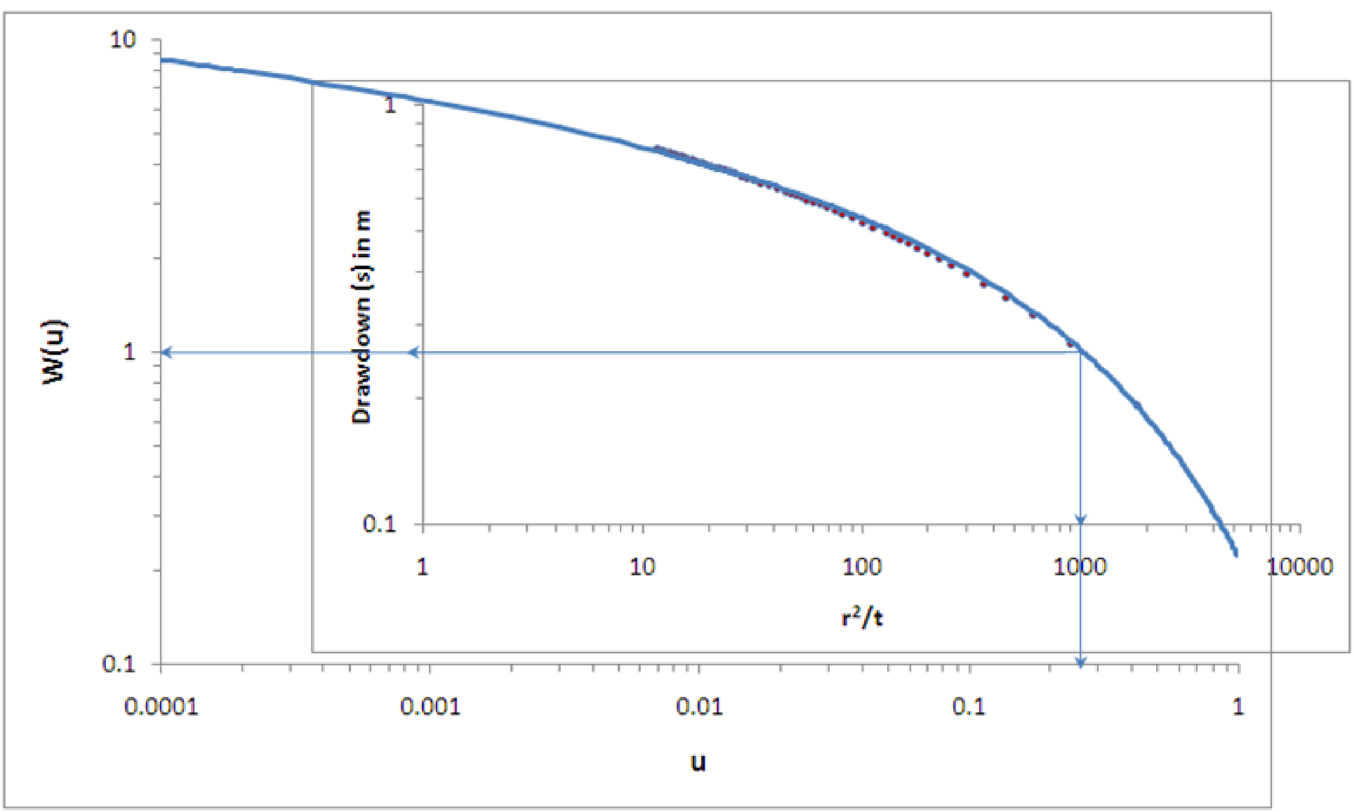


Fig 22.3 Superimposition of the relation between *W(u)* and *u*, and r^2/t and S

Fig. 22.3 show the superimposed relations between the plot *W(u)* verses *u*, *S* verses r^2/t. Considering a suitable convenient point on the superimposed curve, the values of $W(u)$, u, $S$and r^2/t can be obtained. The aquifer parameters (T,S_s) can now be computed using the equations (22.2) and (22.3).

Cooper Jacob method

The Theis equation can be written as

$$s = \left(\frac{Q}{4\pi T}\right) W(u) \tag{22.4}$$

The $W(u)$ is the well function and u is

$$u = \frac{r^2 S_s}{4Tt} \tag{22.5}$$

The $W(u)$ can be approximated as

$$W(u) = -0.5772 - ln(u) + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - \frac{u^4}{4.4!} + \cdots \tag{22.6}$$

Copper and Jacob (1946) suggested that for small value of r and large value of t, the $W(u)$ can be approximated as

$$W(u) = -0.5772 - ln(u) \tag{22.7}$$

Thus the equation (22.4) can be written as

$$s = \left(\frac{Q}{4\pi T}\right)\left(-0.5772 - ln(u)\right) \tag{22.8}$$

$$\Rightarrow s = \left(\frac{Q}{4\pi T}\right)\left(-0.5772 - ln\left(\frac{r^2 S_s}{4Tt}\right)\right) \tag{22.9}$$

$$\Rightarrow s = \left(\frac{2.30Q}{4\pi T}\right) log\left(\frac{2.25Tt}{r^2 S_s}\right) \tag{22.10}$$

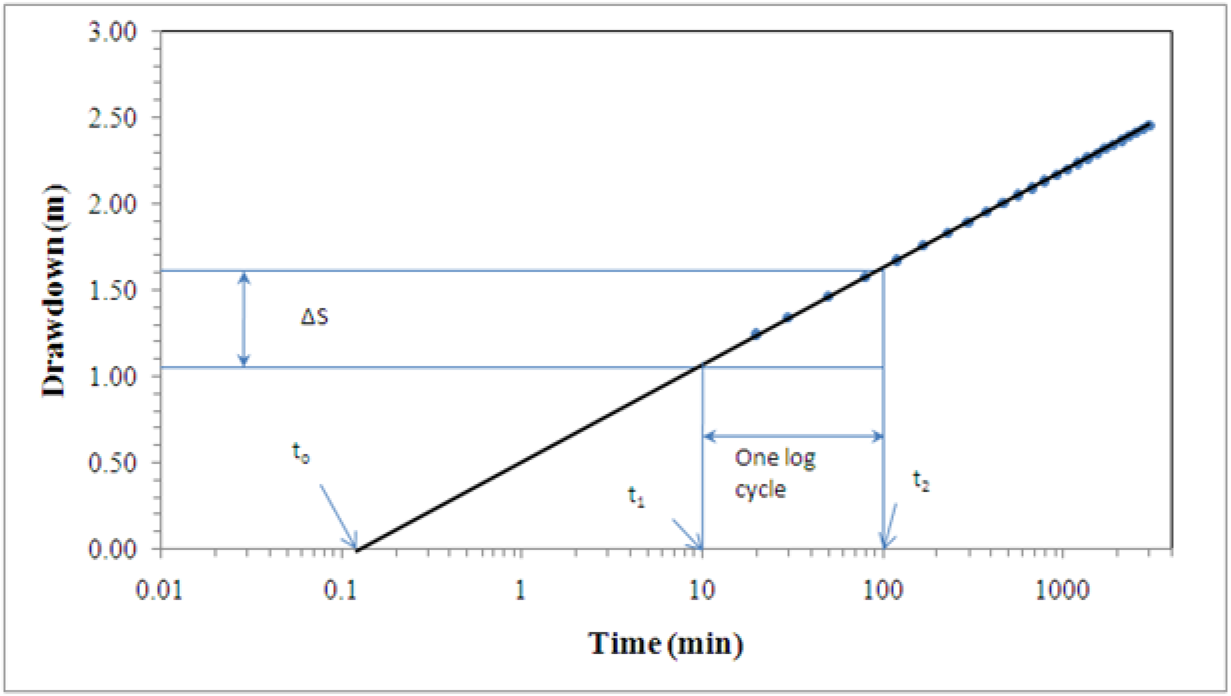


Fig 22.4 Observed time drawdown relation with time in log scale

The plot (Fig. 22.4) between time and drawdown relation is a straight line when time is plotted in logarithmic scale. Projecting the line to s = 0, the time of zero drawdown t_0 can be obtained.

Now for s = 0 at t = t_0

$$0 = \left(\frac{2.30Q}{4\pi T}\right) log \frac{2.25Tt}{r^2 S_s} \qquad (22.11)$$

Thus,

$$\frac{2.25T\,t_0}{r^2 S} = 1 \qquad (22.12)$$

$$\Rightarrow S_s = \frac{2.25T\,t_0}{r^2} \qquad (22.13)$$

Now consider one log cycle as shown in Fig. 22.4

$$\Delta s = \left(\frac{2.30Q}{4\pi T}\right) log \frac{2.25T\,t_2}{r^2 S_s} - \left(\frac{2.30Q}{4\pi T}\right) log \frac{2.25T\,t_1}{r^2 S_s} \qquad (22.14)$$

$$\Rightarrow \Delta s = \left(\frac{2.30Q}{4\pi T}\right) log\frac{t_2}{t_1} \qquad (22.15)$$

As we have considered one log cycle, $t_2 / t_1 = 10$

$$\Delta s = \left(\frac{2.30Q}{4\pi T}\right) log 10 \qquad (22.16)$$

$$\Rightarrow \Delta s = \left(\frac{2.30Q}{4\pi T}\right) \qquad (22.17)$$

Thus,

$$T = \left(\frac{2.30Q}{4\pi \Delta s}\right) \qquad (22.18)$$

It may be noted that this method is not applicable for early drawdown data as it will violate the basic assumption used in developing the method.

Video Lessons-NPTEL

1. **https://youtu.be/7RoVdqWF14M-**Mod-01 Lec-13 Unsteady Flow into Wells (Contd.)
2. **https://youtu.be/rtGOwqYjzWc-**Mod-01 Lec-14 Unsteady Radial Flow in Confined and Unconfined Aquifers

SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – III – GROUND WATER ENGINEERING – SCI1602

Geophysical exploration may be used with advantage to locate boundaries between different elements of the subsoil as these procedures are based on the fact that the gravitational, magnetic, electrical, radioactive or elastic properties of the different elements of the subsoil may be different.

- Differences in the gravitational, magnetic and radioactive properties of deposits near the surface of the earth are seldom large enough to permit the use of these properties in exploration work for civil engineering projects.
- However, the resistivity method based on the electrical properties and the seismic refraction methods based on the elastic properties of the deposits have been used widely in large civil engineering projects.

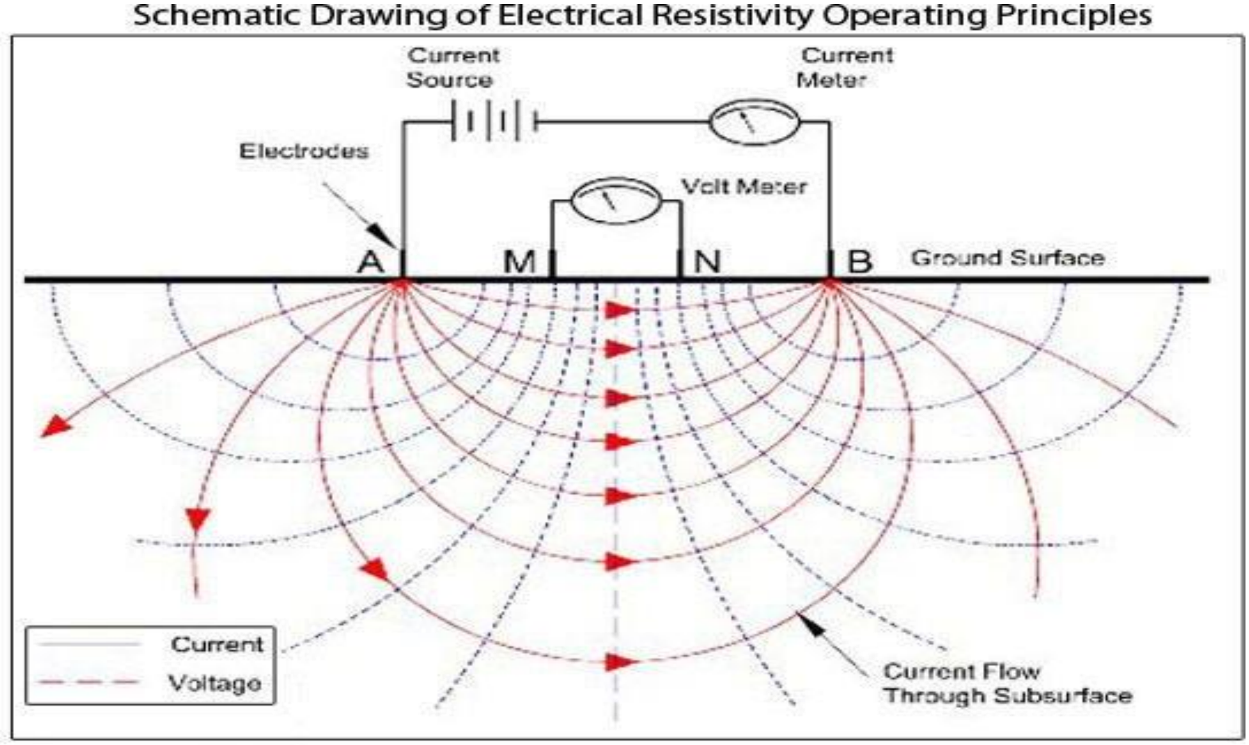
Electrical resistivity method

- Electrical resistivity method is based on the difference in the electrical conductivity or the electrical resistivity of different soils.
- Resistivity is defined as resistance in ohms between the opposite phases of a unit cube of a material.

$$\rho = RA/L$$

ρ is resistivity in ohm-cm, R is resistance in ohms, A is the cross sectional area (cm 2), L is length of the conductor (cm).

Schematic Drawing of Electrical Resistivity Operating Principles



Applications of resistivity soundings are:

- Characterize subsurface hydrogeology, Determine depth to bedrock/overburden thickness, Determine depth to groundwater, Map stratigraphy, clay aquitards, salt-water intrusion and vertical extent of certain types of soil and groundwater contamination .Estimate landfill thickness

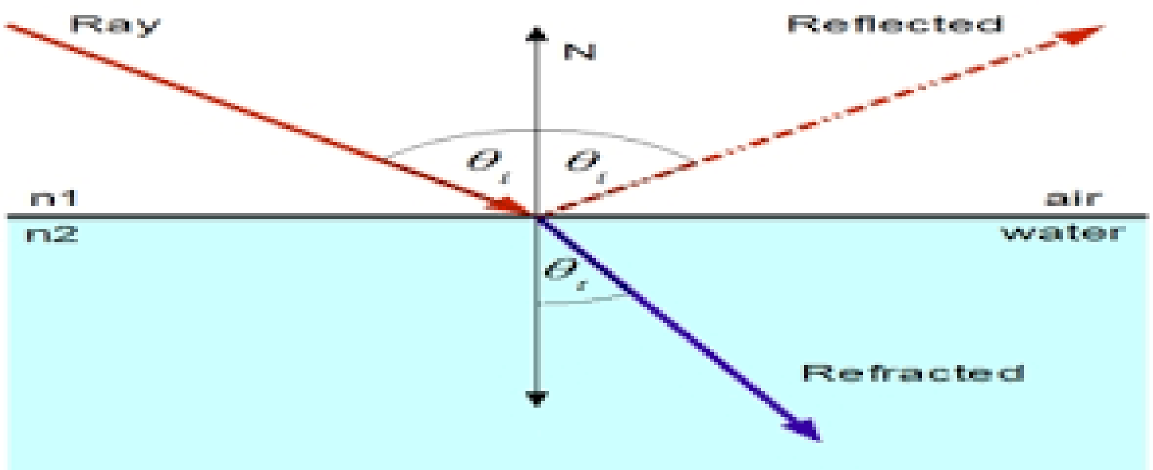
Resistivity profiling is used to:

- Map faults, Map lateral extent of conductive contaminant plumes, Locate voids, Map heavy metals soil contamination ,Delineate disposal areas ,Map paleochannels, Explore for sand and gravel ,Map archaeological sites

Seismic Method

- Seismic refraction is a geophysical method used for investigating subsurface ground conditions utilizing surface-sourced seismic waves.
- The methods depend on the fact that seismic waves have differing velocities in different types of soil (or rock): in addition, the waves are refracted when they cross the boundary between different types (or conditions) of soil or rock.
- The methods enable the general soil types and the approximate depth to strata boundaries, or to bedrock, to be determined.

Reflection and Refraction



- The seismic waves propagate downward through the ground until they are reflected or refracted off subsurface layers. Refracted waves are detected by arrays of 24 or 48 geophones spaced at regular intervals of 1 - 10 metres, depending on the desired depth penetration of the survey.
- Sources are positioned at each end of the geophone array to produce forward and reverse wave arrivals along the array. Additional sources may be used at intermediate or off-line positions for full coverage at all geophone positions.

- A **geophone** is a device that converts ground movement (velocity) into voltage, which may be recorded at a recording station. The deviation of this measured voltage from the base line is called the seismic response and is analyzed for structure of the earth.

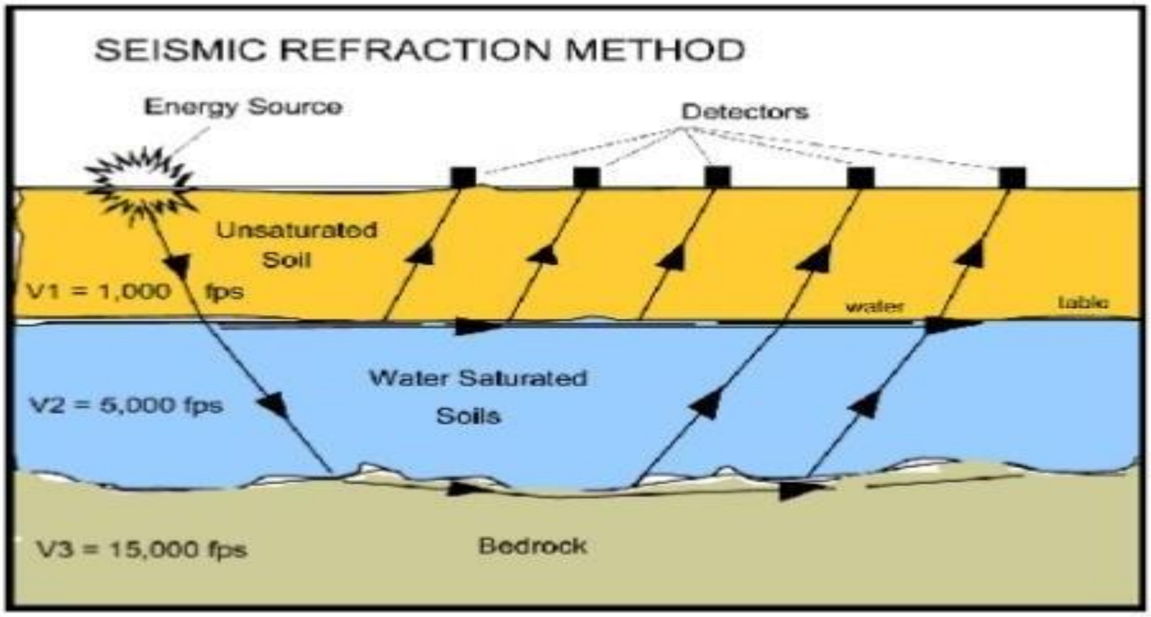
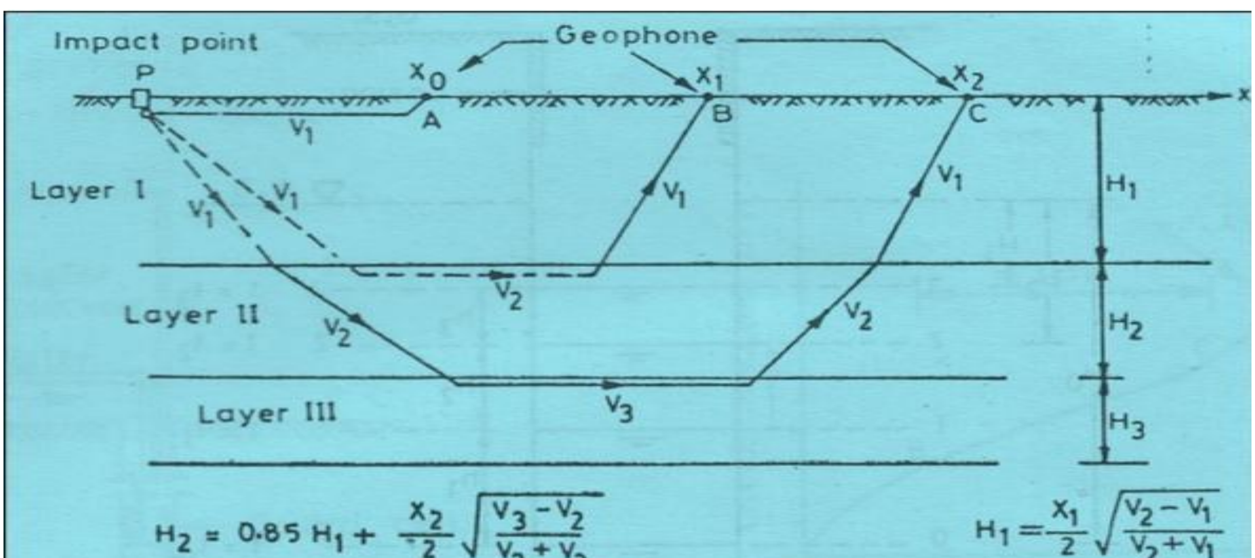
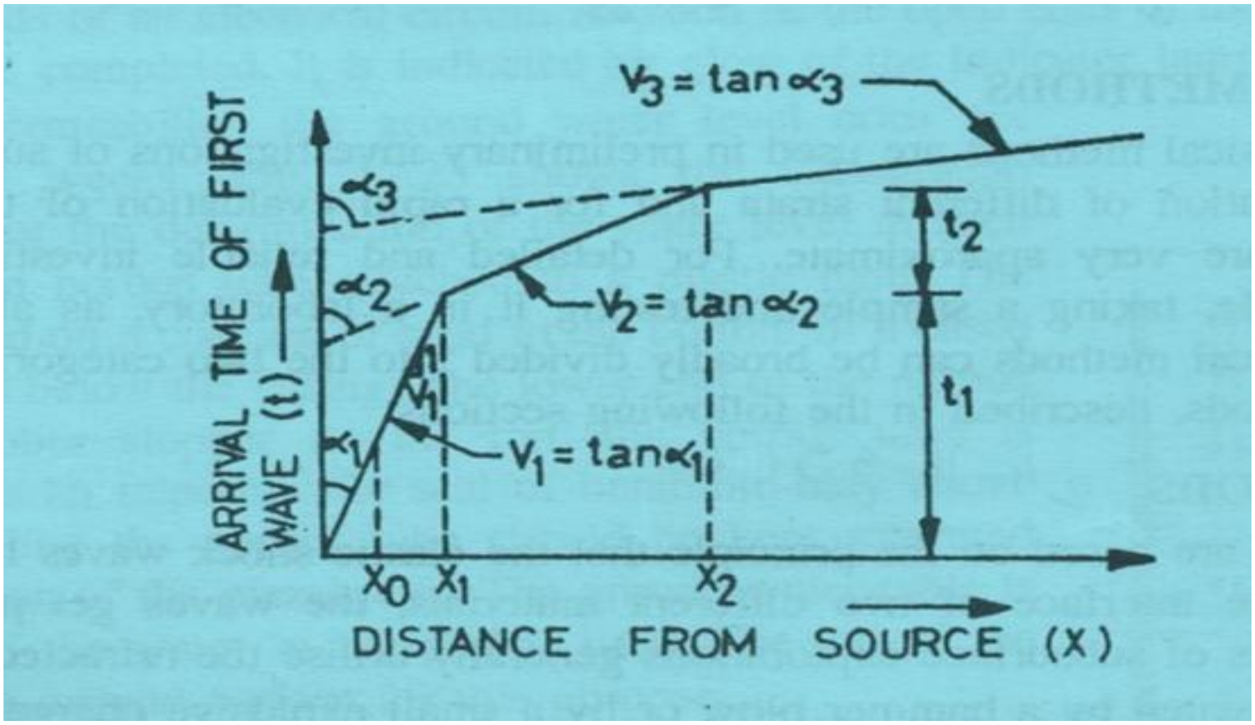


Fig 1 Seismic Refraction Method



Seismic method.



APPLICATIONS

- Measures Bedrock Depth & Overburden Thickness
- Investigates Pipeline Routes
- Locates Geological Structures
- Evaluates Sand & Gravel Deposits
- Defines Ancient Landfill Sites

Limitations in Geophysical Methods

- Resistivity method
  - Limitations
    - Valid only for strata having different electrical resistivity
    - Results are influenced by surface irregularities, wetness of strata
    - Expertise is required
    - Electrical resistivity changes gradually rather abruptly as assumed
- Seismic Method
  - Limitations
    - Cannot be used if harder surface overlies soft layer
    - Cannot be used for areas covered by concrete or asphaltic pavement
    - Cannot be used when surface is frozen
    - Requires costly equipment
    - Expertise is required

Factors Influencing Seismic Wave Velocities

- The geological factor which influence the seismic wave velocities are mainly the composition of rocks, compaction of rocks, and saturation of rocks with ground water.
- Composition
- The seismic wave velocities depend on the composition of rocks. This may be inferred from the following example

Rock type	Seismic Wave Velocity
Granite	4-6 km /sec
Basalt	5-6..5 km/sec
Sandstone	1.5 to 4 km /sec
Limestone	2.5 to 6 km/sec

Factors Influencing Seismic Wave Velocities

Compaction

- This refer to the porosity or fracturing or degree of consolidation of rock. The velocity of seismic waves in rocks is influenced considerably by this factor, the wave velocity is more in denser/ compact formations. This may be observed from the following data:

Formation	Seismic Wave Velocity
Loose sand and soil	0.1 to 0.5 km /sec
Moist Clay	1.5 to 2.5 km/sec
Sandstone	1.5 to 4 km /sec
Shale	2.1 to 4 km/sec

Factors Influencing Seismic Wave Velocities

Saturation

- The Seismic wave velocity increases with the increase of moisture content in the formation. For ex
- (I) Loose soil has a velocity of **0.1 to 0.5 km/sec**, while moist clay has a velocity of **1.5 to 2.5 km/sec**
- (ii) Dry sand has a velocity of **0.15 to 0.4 km/sec**, while wet sand has a velocity of **0.6 to 1.8 km/sec.**

Water Wells

Introduction

- Water well is an excavation or structure created in the ground by digging, driving, boring, or drilling to access groundwater in underground aquifers.
- The water in the well is drawn by a pump, or using containers, such as buckets, that are raised mechanically or by hand.
- A water well is a hole or shaft ,usually vertical, excavated in the earth for bringing ground water to the surface. But we can have also Horizontal wells called- Collector wells.

✓A water well or a borehole is a vertical capture engineered structure used to exploit the water from a water table held in the interstices or in the cracks in a rock in the sub-soil known as aquifer

Wells are drilled either for exploration or exploitation.

- **Exploration wells,** the objective is to collect information on the geology of the underlying aquifer.

Exploitation/production wells, used as pumping the required amount of water at the lowest cost, considering investment, operation, and maintenance

Exploitation wells are drilled for water supply for municipal, industrial, and irrigation purposes, and for water table control for drainage purposes.

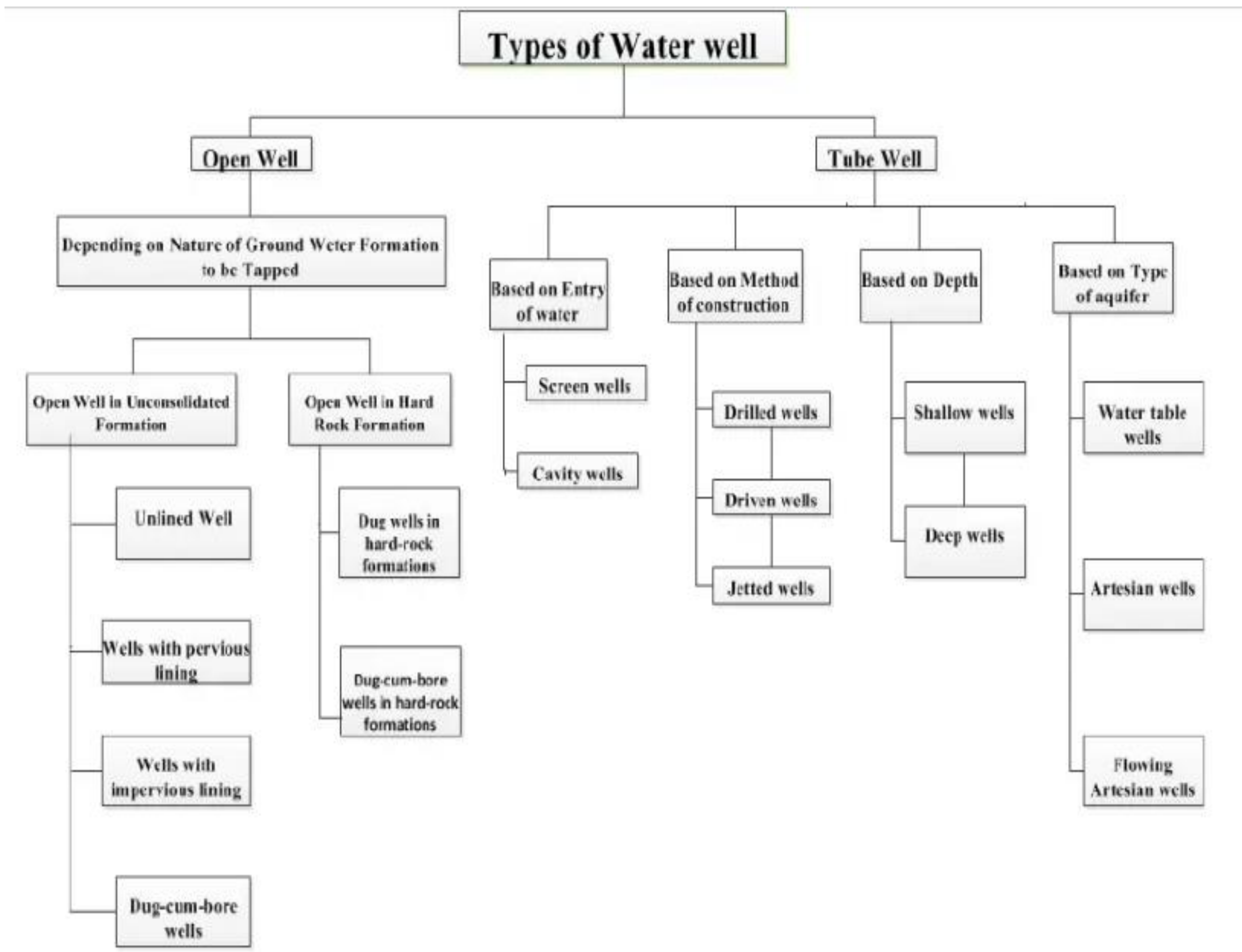
- Many methods exist for constructing wells; selection of a particular method depends on the purpose of the well, the quality of water required, depth to groundwater, geological conditions, and economic factors.

- Wells have to be designed to get the optimum quantity of water economically from a given geological formation
- Properly designed and constructed well, permits the economic withdrawal of water from a water-bearing formation. However, if improperly constructed and maintained, they can allow bacteria, pesticides, or oil products to pollute the groundwater.

- Generally, wells are designed for the purpose of irrigation, drainage, sanitation and domestic and industrial water supply.
- The design of each type of well requires particular attention, taking into account its purpose.
- Designing water well involves the *selection of proper dimensional factors* for the well structure and *choice of materials to be used* in its construction. Good design aims at an optimum combination of performance, long service life and reasonable cost.

The objective of well:

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



Open Well

- These are the wells which have comparatively large diameters and lower discharges
- Usually they have discharge of 20 m^3/hr but if constructed by efficient planning it gives discharge of 200-300 m^3/hr
- They are constructed of diameter of about 1-10 m and have depth of about 2-20m
- They are constructed by digging therefore they are also known as dug wells

Classification of Open Well based on Depth

1. **Shallow open well** : These are the wells resting on the water bearing strata and gets their supplies from the surrounding materials

2. **Deep open well** : These are the wells resting on the impervious layer known as mota layer beneath which lies water bearing pervious layer and gets their supply from this layer

Classification of Open Well Based on Type of Wall

- **Kachha wells** : These type of wells are only constructed when water table is high as these type of wells sometimes collapses

- **Wells with Impervious lining** : These are most suitable and stable type of open well. These are constructed by first digging a pit then a curb which is a circular ring with sharp bottom is inserted . Then a masonry wall up to some distance above ground is constructed , then as excavation proceeds it sinks blow and then masonry is further extended and well is constructed. As water enters from the bottom type of flow is spherical.

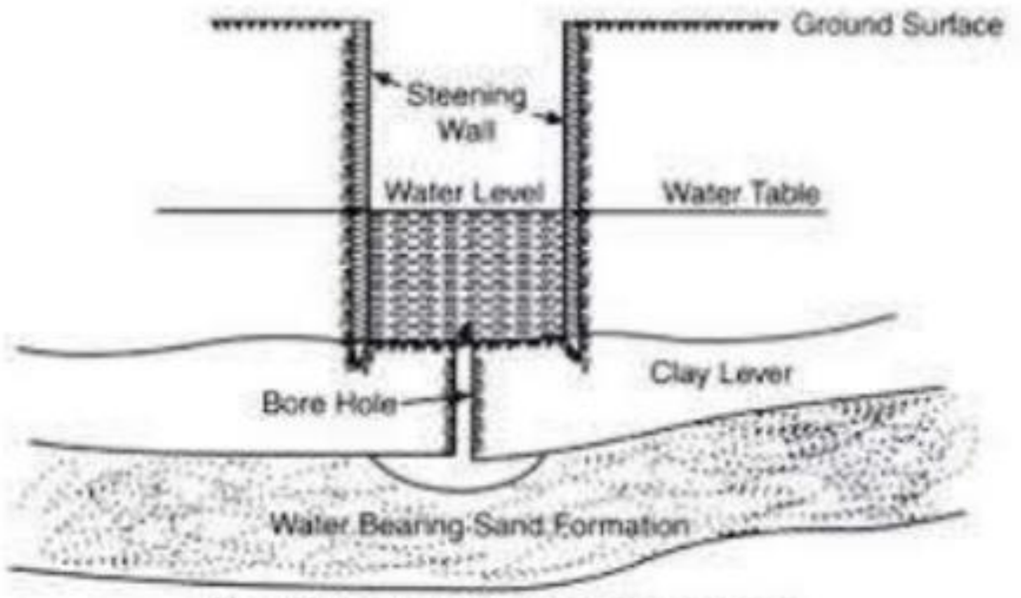


Fig. 17.2. Well with impervious lining

- **<u>Well with pervious lining</u>** : These type of wells are suitable in coarse formations these are constructed by masonry of dry bricks or stones without any binding materials. So the water supply enters from the wall of well therefore the flow is radial. Such wells are provided with bottom plug so the flow is not combination of radial and spherical.

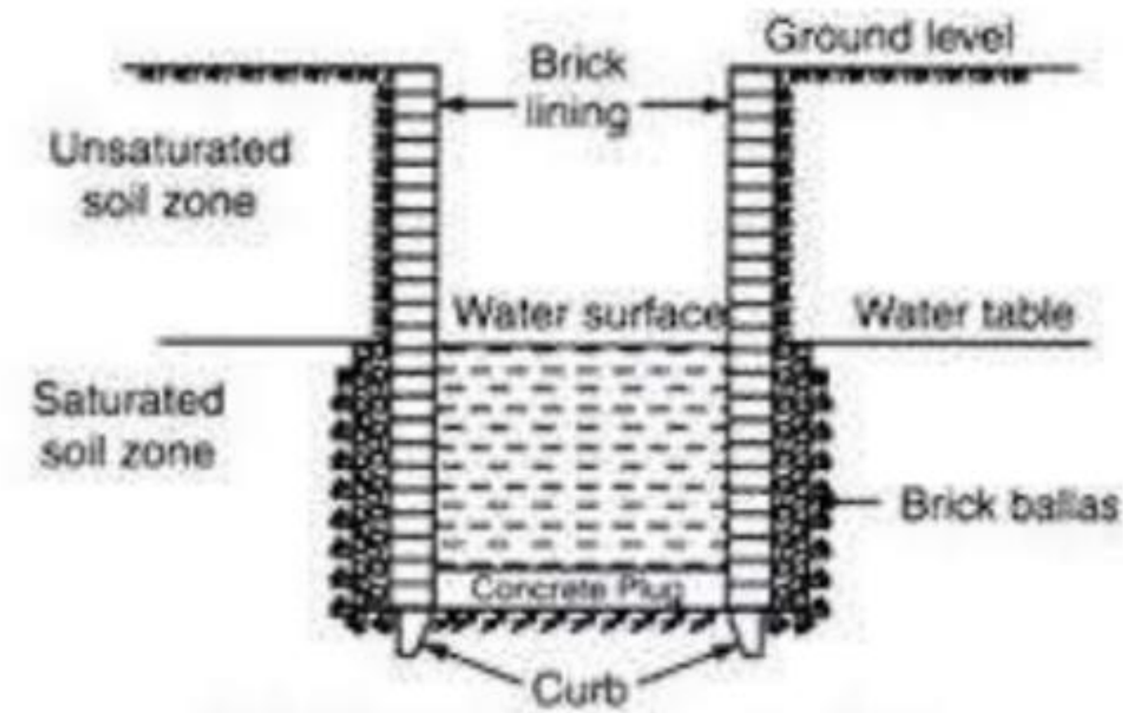


Fig. 17.1. Well with pervious lining

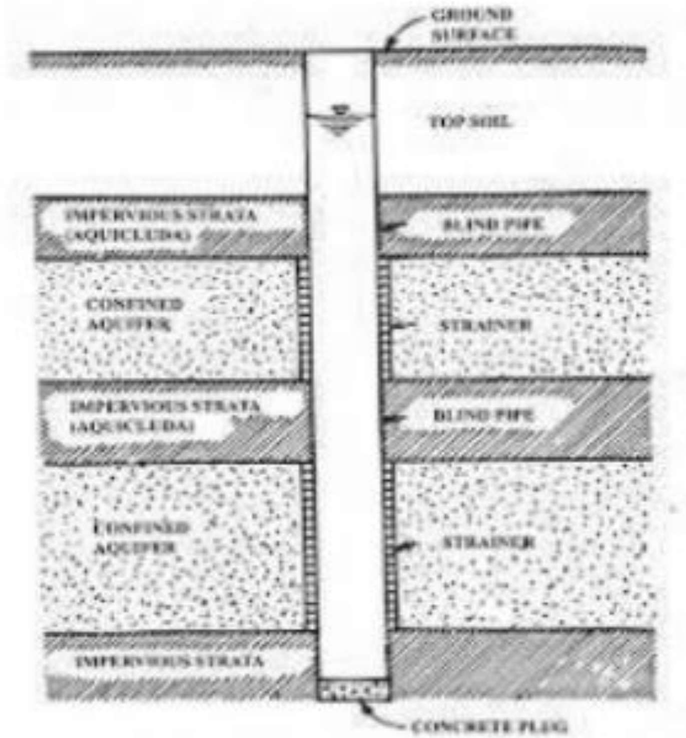
Tube walls

- A tube well is a long pipe sunk in ground intercepting one or more water bearing strata.
- As compared to open well there diameter is less about 80-600 mm.

Classification on Tube Well Based on Depth

- **Shallow tube well** : These are the tube which has depth limited to 30 meters and maximum have discharge of 20 m^3/hr
- **Deep tube well** : These are the tube wells which have maximum depth of about 600 m and may give discharge more then 800 m^3/hr

- **Strainer type tube well** : These is most commonly used tube such that in general a tube well means strainer tube well. In this type of well a strainer which a wire mesh with small openings is wrapped around the main pipe which also has large openings such that area of opening in strainer and main pipe remains same. Annual space is left between two strainer so that the open area of pipe perforations is not reduced. The type of flow is radial.



- **<u>Cavity tube well</u>** : A cavity type tube well consists of a pipe sunk in ground up to the hard clay layer . It draws water from the bottom of well . In initial stages fine sand is also pumped with water and in such manner a cavity is formed at the bottom so the water enters from the aquifer into the well through this cavity

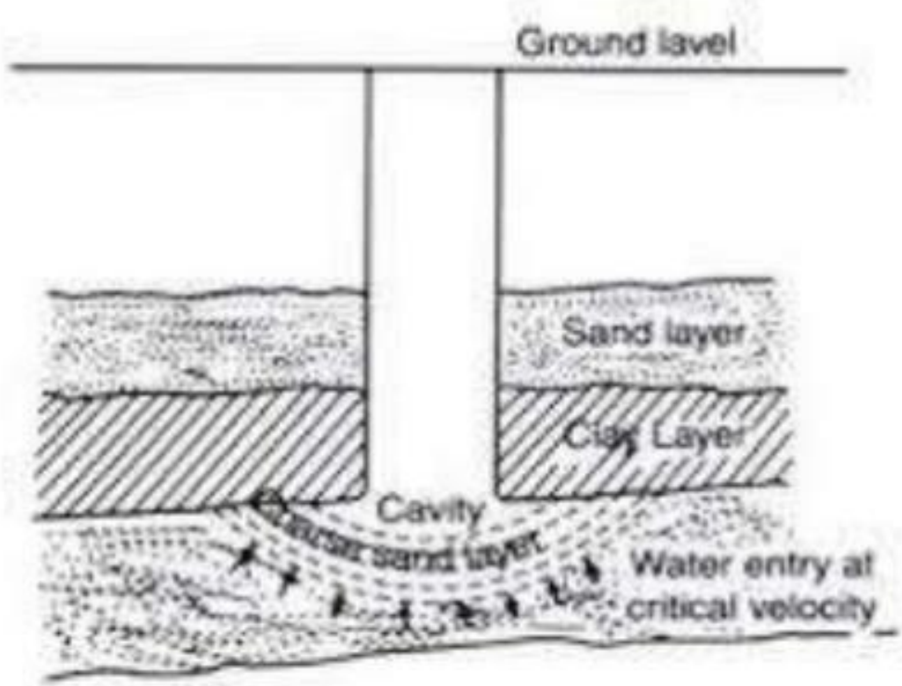
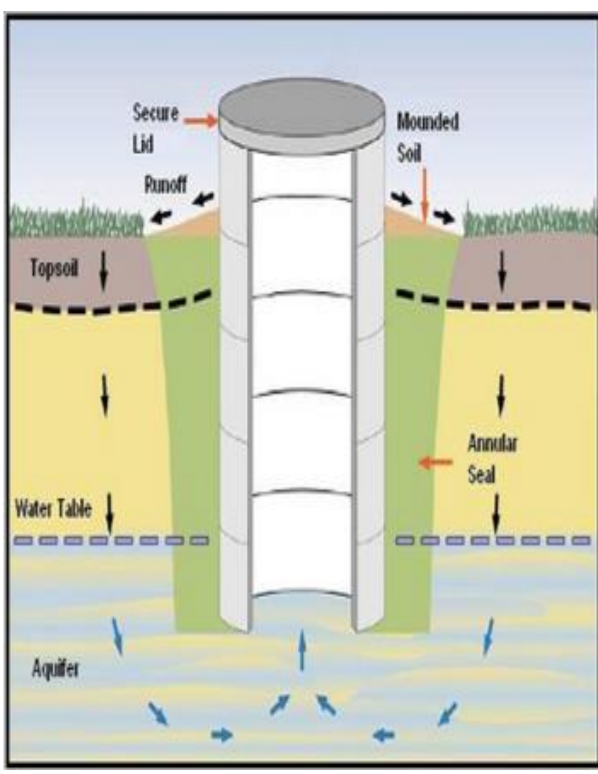


Fig. 18.2. Cavity type tubewell



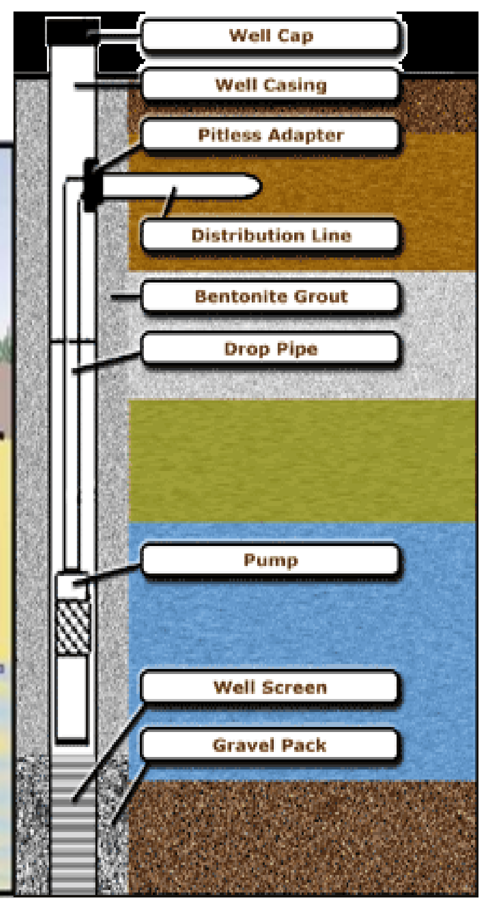
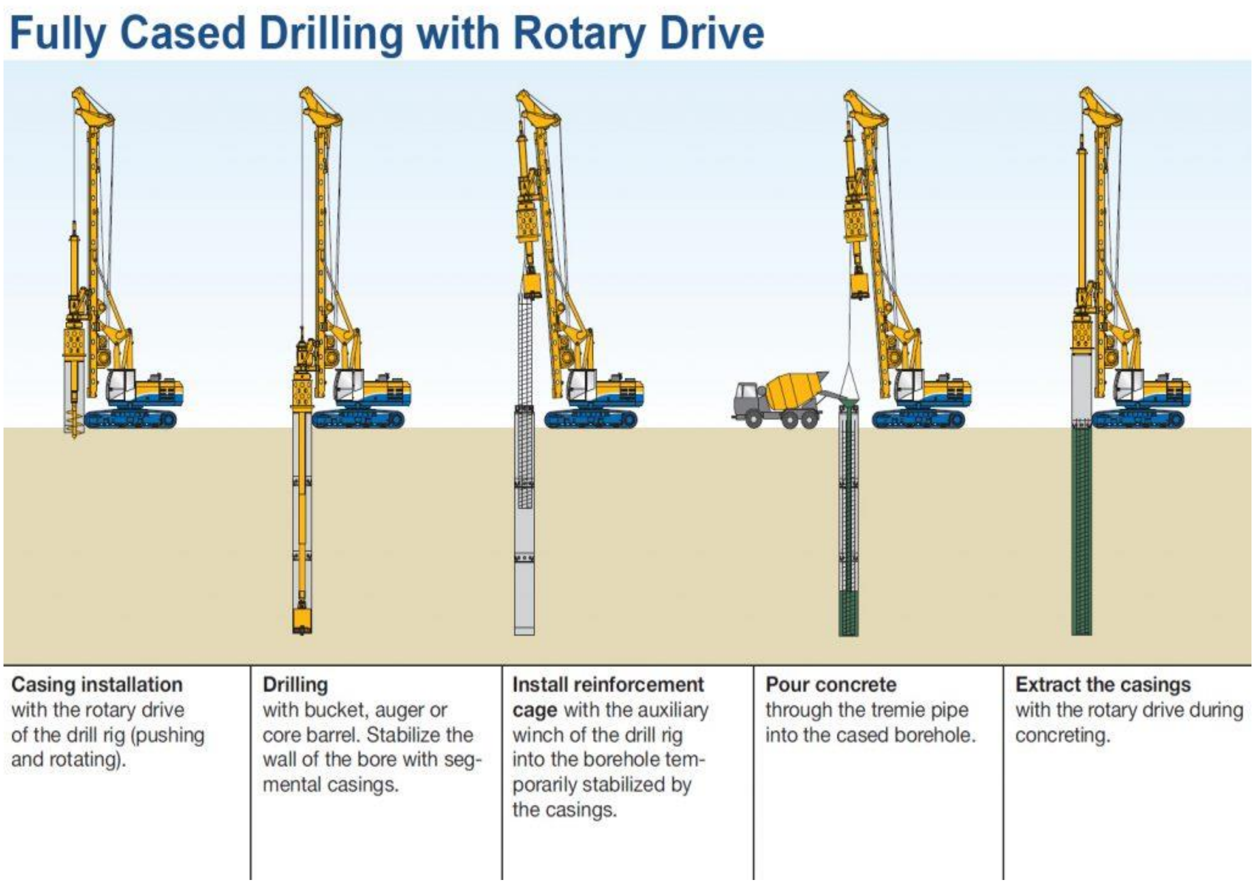
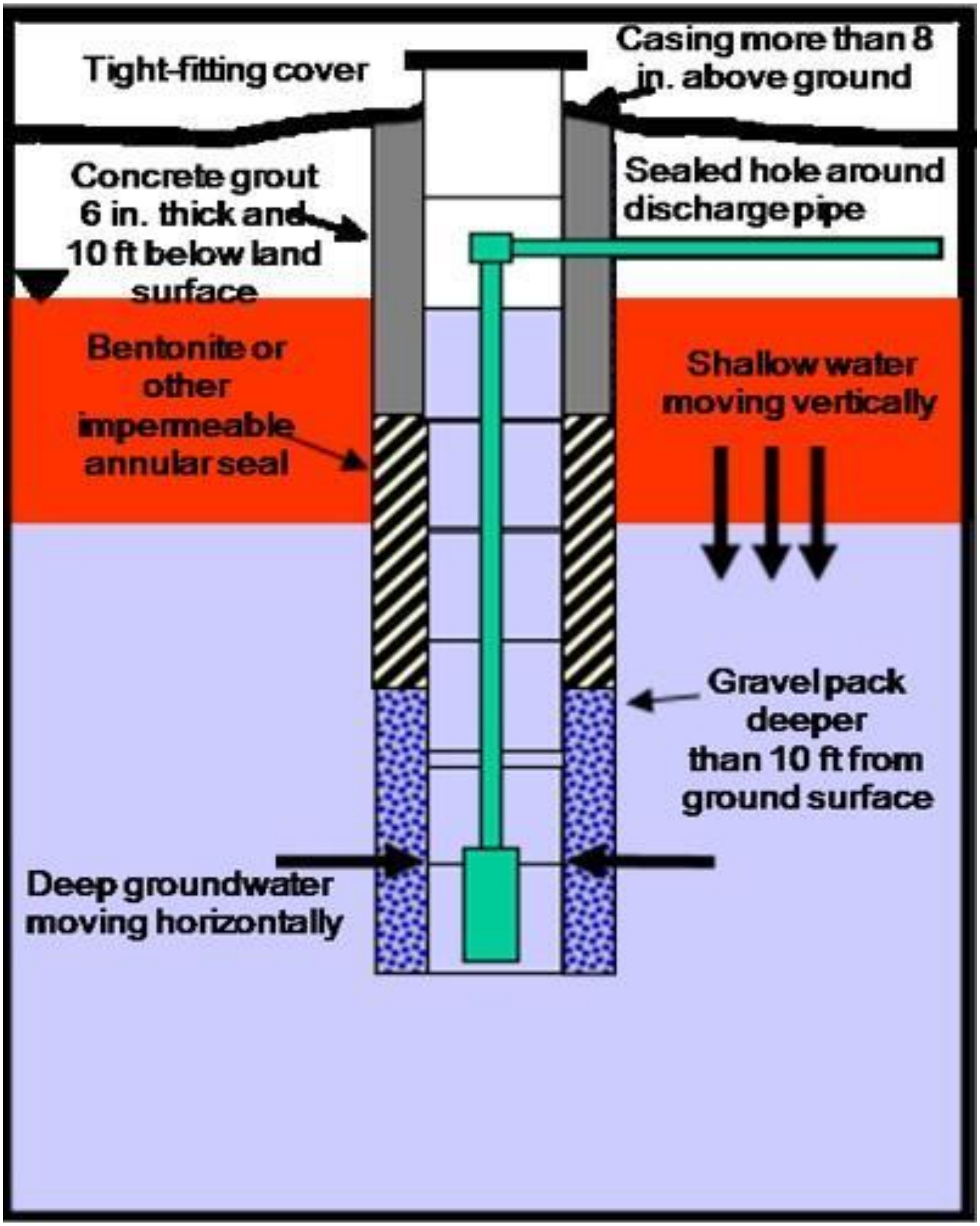
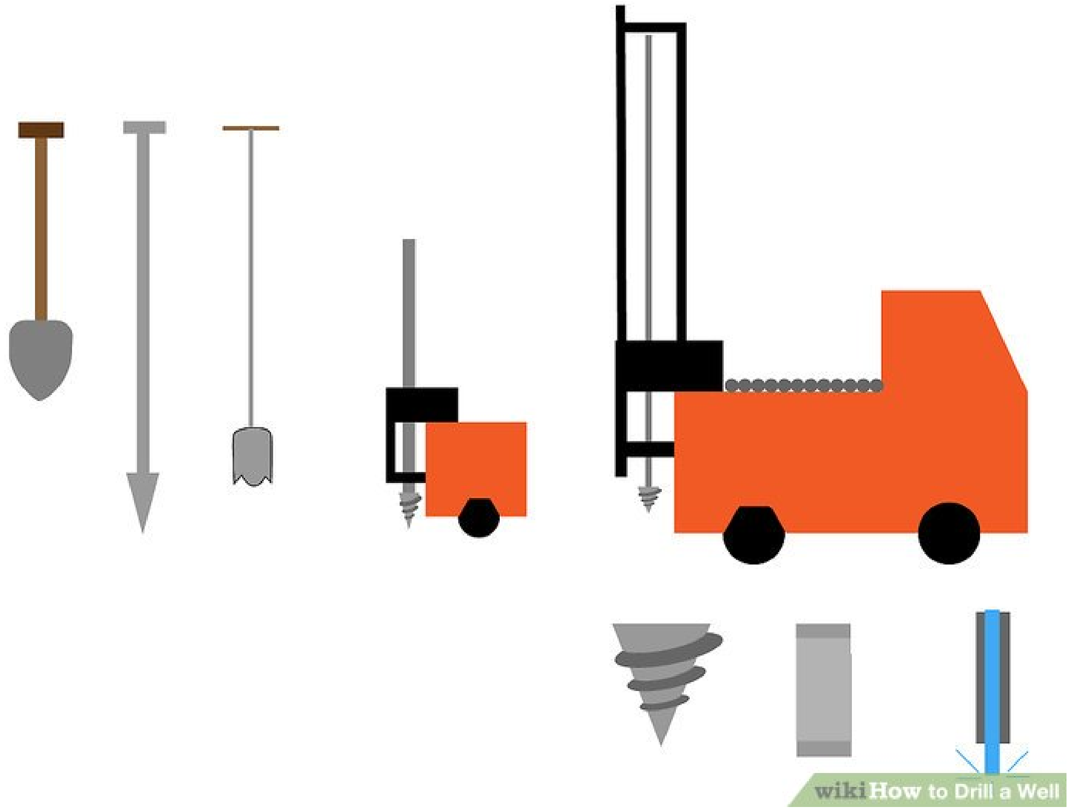
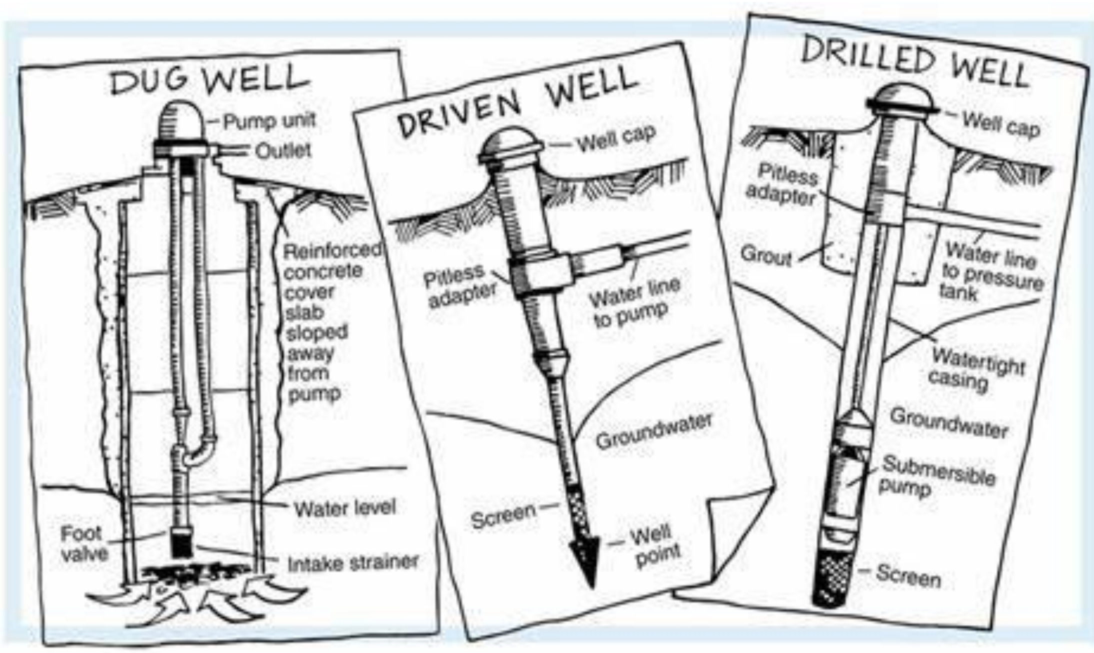


Figure Driven Well




Tight-fitting cover
Casing more than 8 in. above ground
Concrete grout 6 in. thick and 10 ft below land surface
Sealed hole around discharge pipe
Bentonite or other impermeable annular seal
Shallow water moving vertically
Gravel pack deeper than 10 ft from ground surface
Deep groundwater moving horizontally


wikiHow to Drill a Well


DUG WELL
Pump unit
Outlet
Reinforced concrete cover slab sloped away from pump
Water level
Foot valve
Intake strainer
DRIVEN WELL
Well cap
Pitless adapter
Water line to pump
Groundwater
Screen
Well point
DRILLED WELL
Well cap
Pitless adapter
Grout
Water line to pressure tank
Watertight casing
Groundwater
Submersible pump
Screen

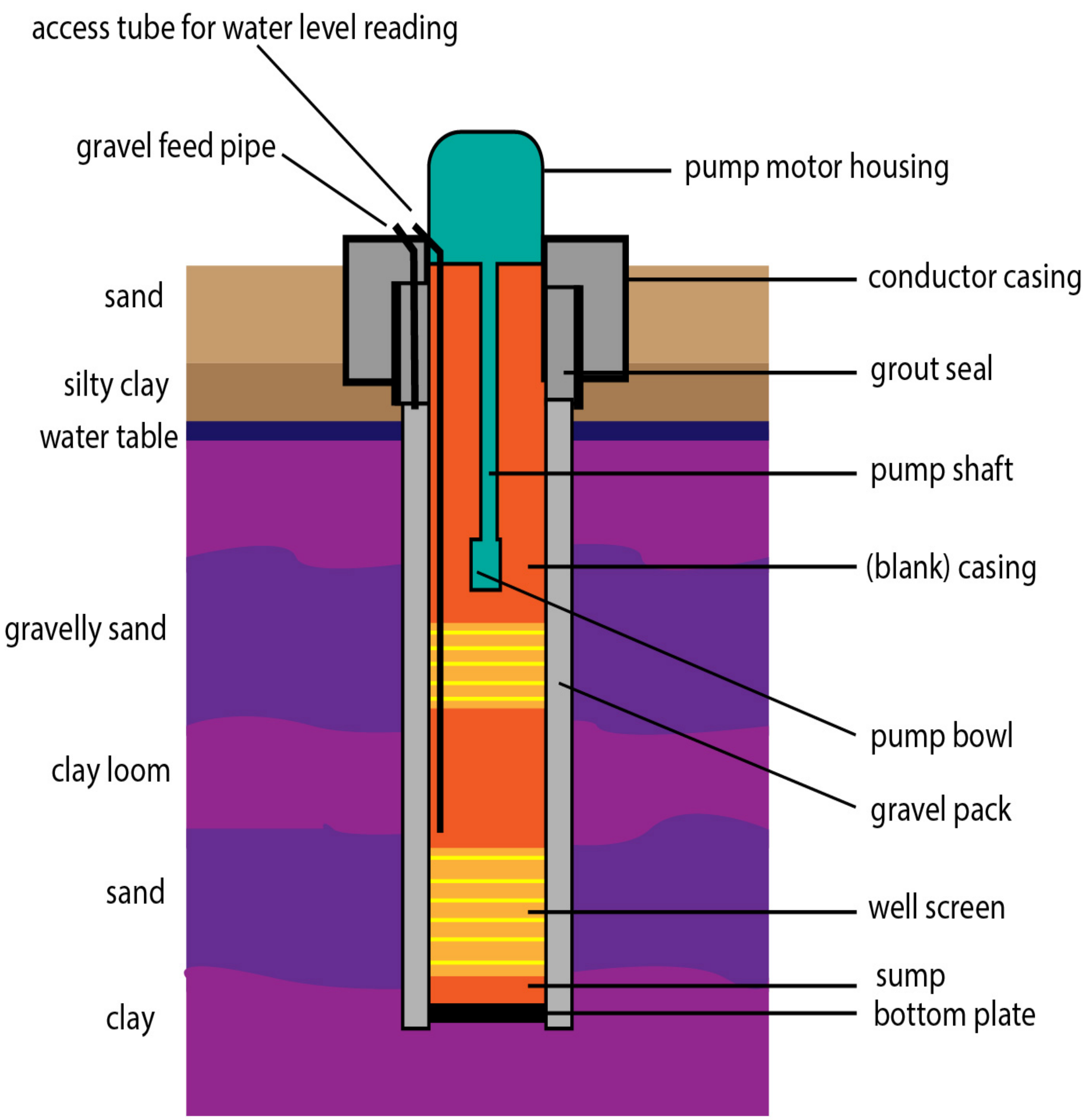
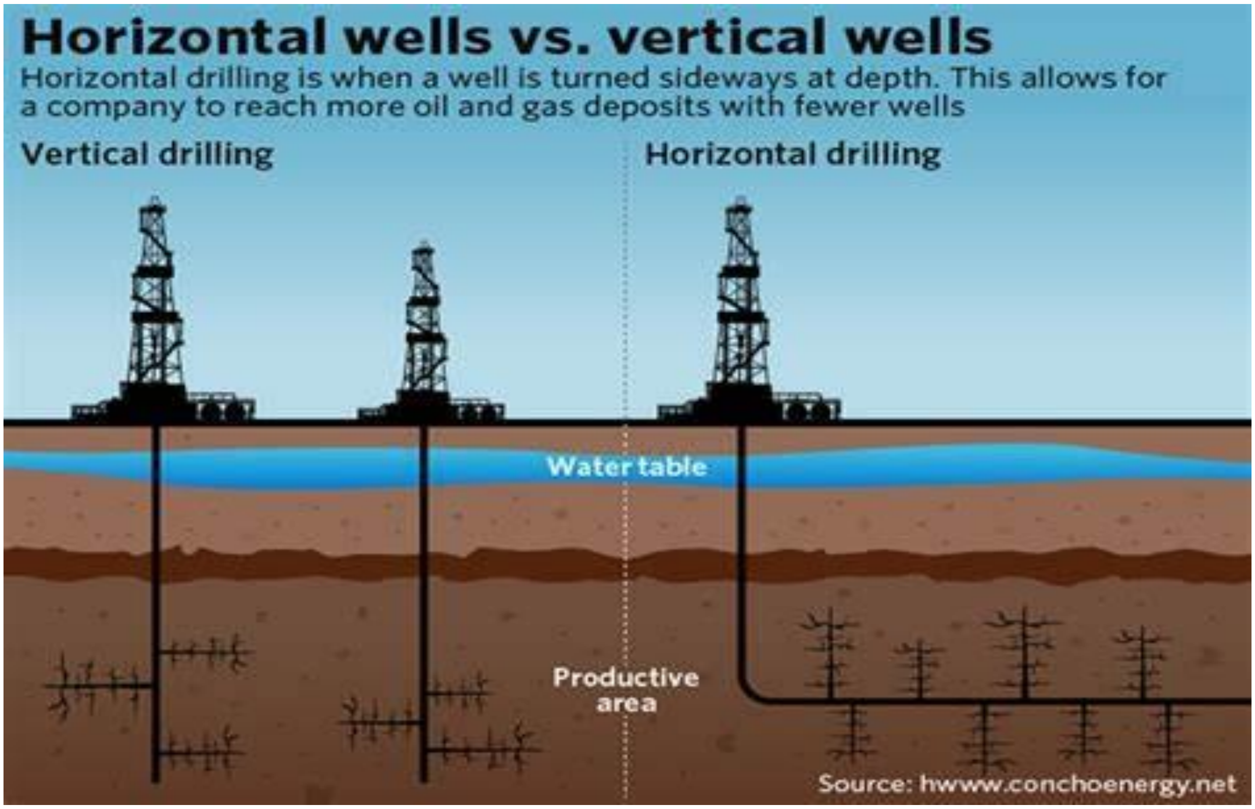


Figure1. Components of a well

Slotted PVC Pipe



Drilled wells.

Drilled wells are constructed by either cable tool (percussion) or rotary-drilling machines. Drilled wells that penetrate unconsolidated material require installation of casing and a screen to prevent inflow of sediment and collapse. They can be drilled more than 1,000 feet deep. The space around the casing must be sealed with grouting material of either neat cement or bentonite clay to prevent contamination by water draining from the surface downward around the outside of the casing.

View videos of the most common well drilling methods: air rotary, bucket auger, cable tool, down-the-hole, and reverse circulation. Videos are courtesy of Sir Sanford Fleming College.

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Driven wells. Driven wells are constructed by driving a small-diameter pipe into shallow water-bearing sand or gravel. Usually a screened well point is attached to the bottom of the casing before driving. These wells are relatively simple and economical to construct, but they can tap only shallow water and are easily contaminated from nearby surface sources because they are not sealed with grouting material. Hand-driven wells usually are only around 30 feet deep; machine-driven wells can be 50 feet deep or more.

Dug wells. Historically, dug wells were excavated by hand shovel to below the water table until incoming water exceeded the digger’s bailing rate. The well was lined with stones, bricks, tile, or other material to prevent collapse, and was covered with a cap of wood, stone, or concrete tile. Because of the type of construction, bored wells can go deeper beneath the water table than can hand-dug wells. Dug and bored wells have a large diameter and expose a large area to the aquifer. These wells are able to obtain water from less-permeable materials such as very fine sand, silt, or clay. Disadvantages of this type of well are that they are shallow and lack continuous casing and grouting, making them subject to contamination from nearby surface sources, and they go dry during periods of drought if the water table drops below the well bottom.

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References:

1. https://youtu.be/_gVV-gjP8wE-Mod-01 Lec-17 Well Completion;Well Development; Well Protection; Well Rehabilitation;

2. https://youtu.be/jcds3fbVLg0-Mod-01 Lec-18 Well Protection/Rehabilitation/Testing for yield (Contd.); Artificial Ground

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DEPARTMENT OF CIVIL ENGINEERING

UNIT – IV – GROUNDWATER ENGINEERING – SCI1602

UNIT IV GROUNDWATER QUALITY AND MOVEMENT

1.0 Groundwater geochemistry

In this section we will look at the general chemical characteristics of groundwaters, and at some of the geochemical processes that are important in the chemical evolution of water that flows through the ground – including carbonate equilibrium, oxidation-reduction reactions and adsorption-desorption processes. We will also examine some groundwater geochemical data from the Nanaimo Group in order to understand its characteristics and evolution.

Dissolved constituents in groundwater

A wide range of different elements can become dissolved in groundwater as a result of interactions with the atmosphere, the surficial environment, soil and bedrock. Groundwaters tend to have much higher concentrations of most constituents than do surface waters, and deep groundwaters that have been in contact with rock for a long time tend to have higher concentrations than shallow and or young waters. It is convenient to divide dissolved constituents into major components (the predominant cations and anions), and trace elements. Dissolved constituents are typically expressed in **mg/L** for the major components and **µg/L** for the trace elements. Some rare elements are expressed in **ng/L** (nanograms/litre). Since 1 mg is 0.001 g and 1 litre of water is very close to 1000 g, mg/L is equivalent to parts per million (ppm), while µg/L is equivalent to parts per billion (ppb). We can also express concentrations in **molality** terms (moles per litre of water). For example for a solution with 34.1 mg/L of Ca the molality of calcium is:

$$34.1/40.08 = 0.851 \text{ millimoles/litre (mM/L)}$$

It is also common to express concentrations of ions as molar **equivalents**, which is similar to molality, except that the charge on the ion is taken into consideration. If a solution has a calcium ion molality of 0.851 mM/L, it has 1.702 milliequivalents per litre (mEq/L) of Ca^{2+} because the calcium ion is divalent. A solution with 0.56 mM/L Na^+ will have 0.56 mEq/L of Na^+ because the sodium ion is monovalent.

Major components

The major dissolved components of groundwaters include the anions bicarbonate, chloride and sulphate, and the cations sodium, calcium, magnesium and potassium. These constituents are typically present at concentrations in the range of a few mg/L to several hundred mg/L.

Trace elements

All of the elements in the periodic table are present at some concentration in most water samples, but only a fraction of these are important to us. Some example concentrations (in mg/L and µg/L) for the same ten samples listed above are given in the table below. Si and F^- are the most abundant of the trace elements in these samples, followed by B, Sr, Ba and Fe. In fact the concentrations of some of the trace constituents in these samples (esp. Si) are higher than those for some of the so-called major components. Some of the values are listed as undetected (ud), indicating not that there isn't any there, but that the concentrations are below the detection limit for the analytical method used.

2.0 Groundwater geochemical processes

Water moving through the ground will react to varying degrees with the surrounding minerals (and other components), and it is these rock-water interactions that give the water its characteristic chemistry. As already noted, the silicate minerals that comprise most rocks do not react readily with most groundwaters. On the other hand, carbonate minerals do react quite readily with water, and they play an important role in the evolution of many groundwaters.

Since carbonates are present in many different types of rock, including most sedimentary rocks, and even some igneous and metamorphic rocks, carbonate chemistry is relevant to the evolution of most groundwaters.

The main mechanism for the dissolution of calcite is as follows:

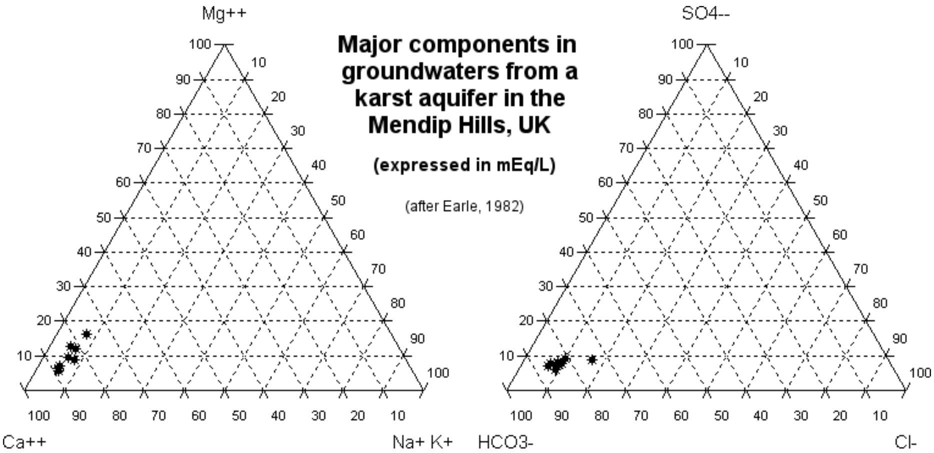
$$CaCO_3 + CO_2(g) + H_2O = Ca^{2+} + 2HCO_3^-$$

$$CO_2(g) + H_2O = H^+ + HCO_3$$

which is the reaction of carbon dioxide with water, to produce the hydrogen ions (acidic conditions) that promote the dissolution of calcite by the following reaction:

$$CaCO_3 + H^+ = Ca^{2+} + HCO_3$$

From the first reaction we can see that calcite solubility is controlled by the amount of carbon dioxide available – the more CO_2 the more calcite will dissolve. From the last reaction we can see that calcite solubility is also controlled by pH – the lower the pH (more hydrogen ions) the more calcite will dissolve. Other processes – such as oxidation of sulphide minerals, or reactions of sulphur pollutants in the air – can also produce hydrogen ions that will promote dissolution of calcite.



Oxidation-reduction reactions

Chemical reactions that involve the transfer of electrons from one ion to another are called oxidation-reduction reactions (or *redox* reactions). An example is:

$$Fe^{3+} + e^- = Fe^{2+}$$

This shows the "reduction" of ferric iron to ferrous iron. Redox reaction rates and directions are controlled by the oxidation state of the surrounding environment – for example of the water. Oxygen is the ultimate oxidant in the natural environment. Water in equilibrium with the atmosphere will be oxidizing.

Ion exchange processes

Because of their electrical charge, the ions in water have a tendency to be attracted onto solid surfaces. Such surfaces include ordinary mineral grains (eg. feldspar or quartz) but these are much less efficient than the surfaces of minerals such as iron oxides and clay minerals. Both anions and cations take part in ion exchange processes. Clays are particularly effective at adsorbing cations because their surfaces are consistently negatively charged

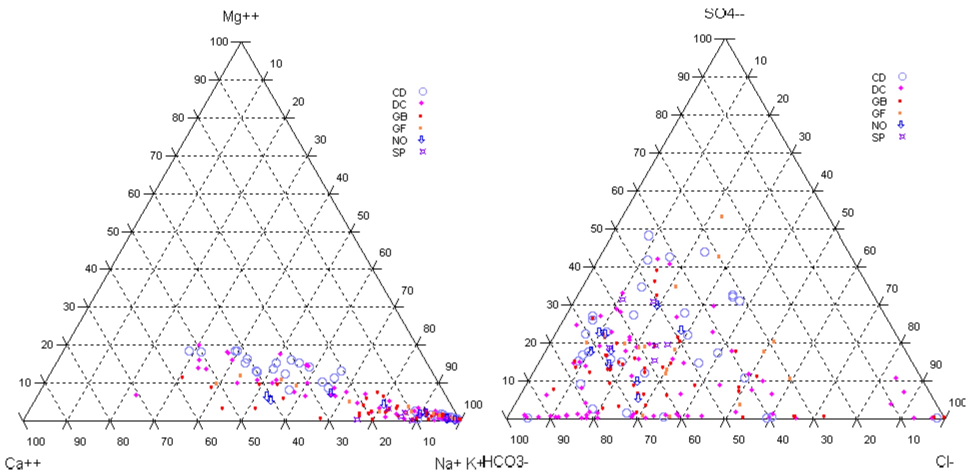
(strongly adsorbed) $\mathbf{Ca^{2+}} > \mathbf{Mg^{2+}} > \mathbf{K^{+}} > \mathbf{Na^{+}}$ (weakly adsorbed)

The ions of different elements have different tendencies to be adsorbed or desorbed[1]. The tendency for adsorption amongst the major cations in natural waters is as follows:

A water softener works because of this relationship. As the "hard" water is passed through the system calcium and magnesium ions in solution are preferentially adsorbed onto a substrate (ion-exchange resin). After some time most of the exchange sites are occupied by calcium and magnesium and the system ceases to function effectively. A NaCl brine is then passed through the system, and because of the overwhelming amount of sodium in the solution the calcium and magnesium on the exchange sites are replaced by sodium – thus "recharging" the ion exchange resin.

Ion exchange is also an important process for trace elements, especially those that behave as cations. Clay-mineral bearing rocks and sediments will naturally adsorb heavy-metal cations from contaminated water. Engineered clay barriers, such as those at the landfill, are based on this principal. As described above, other minerals, including iron-oxides, can also be effective at adsorbing trace elements.

1.2 Geochemistry of the Nanaimo Group aquifers



Reactions

- Reversible reactions reach equilibrium easily. Typically equilibrium is assumed in hydrochemistry, therefore reversibility is implied dissociation: most common type, separation of molecules into individual ions (e.g. NaCl)

- solvent (water) can directly participate in reaction (e.g. carbonation reactions)

oxidation-reduction: exchange of electrons between ions, e.g. electrons appear in the reaction equation, and one or more cations change atomic charge

Artificial Groundwater Recharge

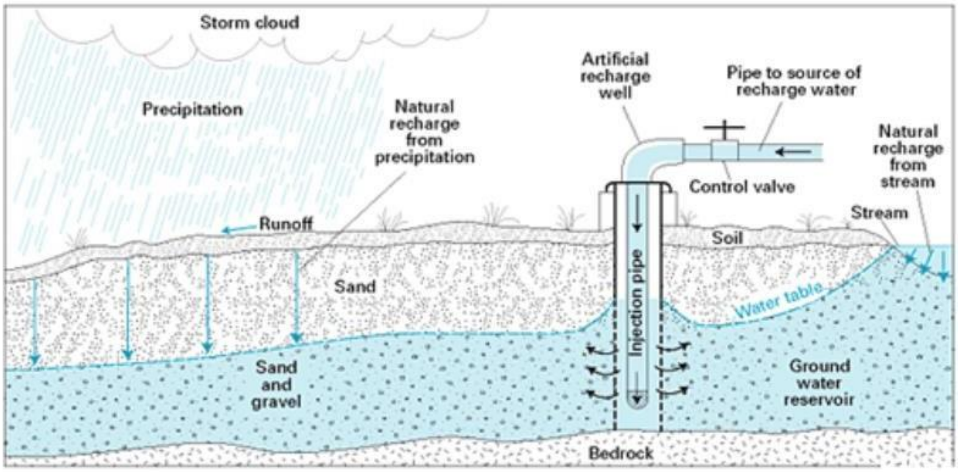
Groundwater levels are declining across the country as our withdrawals exceed the rate of aquifers to naturally replenish themselves, called recharge. One method of controlling declining water levels is by using artificial groundwater recharge. The USGS monitors wells to evaluate the effect of groundwater depletion and recharge, and provides vital information to those who depend on groundwater resources.

Artificial recharge is the practice of increasing the amount of water that enters an aquifer through human-controlled means. For example, groundwater can be artificially recharged by redirecting water across the land surface through canals, infiltration basins, or ponds; adding irrigation furrows or sprinkler systems; or simply injecting water directly into the subsurface through injection wells.

Aquifer storage and recovery is a water-storage technique applied by water-resource managers and scientists worldwide. Essentially, it involves storage of available water through wells completed into aquifers, with subsequent retrieval from these same wells during dry periods. Recovery of water stored in these wells greatly benefits environmental, agricultural, and urban uses.

Natural groundwater recharge occurs as precipitation falls on the land surface, infiltrates into soils, and moves through pore spaces down to the water table. Natural recharge also can occur as surface-water leakage from rivers, streams, lakes, and wetlands.

Artificial recharge can be done through injection of water through wells. This method often is applied to recharge deep aquifers where application of water to the land surface are not effective at recharging these aquifers.



Advantages Of Artificial Recharge

Following are the main advantages of artificially recharging the ground water aquifers.

- No large storage structures needed to store water. Structures required are small and cost-effective
- Enhance the dependable yield of wells and hand pumps
- Negligible losses as compared to losses in surface storages
- Improved water quality due to dilution of harmful chemicals/ salts
- No adverse effects like inundation of large surface areas and loss of crops
- No displacement of local population
- Reduction in cost of energy for lifting water especially where rise in ground water level is substantial
- Utilizes the surplus surface runoff which otherwise drains off.

Identification Of Areas For Recharge

The first step in planning a recharge scheme is to demarcate the area of recharge. Such an area should, as far as possible, be a micro-watershed (2,000-4,000 ha) or a mini-watershed (40-50 ha). However, localized schemes can also be taken up for the benefit of a single hamlet or a village. In either case the demarcation of area should be based on the following broad criteria: • Where ground water levels are declining due to over-exploitation • Where substantial part of the aquifer has already been desaturated i.e. regeneration of water in wells and hand pumps is slow after some water has been drawn • Where availability of water from wells and hand pumps is inadequate during the lean months • Where ground water quality is poor and there is no alternative source of water.

Sources Of Water For Recharge

Before undertaking a recharge scheme, it is important to first assess the availability of adequate water for recharge. Following are the main sources, which need to be identified and assessed for adequacy:

• Precipitation (rainfall) over the demarcated area

• Large roof areas from where rainwater can be collected and diverted for recharge

• Canals from large reservoirs from which water can be made available for recharge

• Natural streams from which surplus water canbe diverted for recharge, without violating the rights of other users

• Properly treated municipal and industrial wastewaters.

This water should be used only after ascertaining its quality “In situ” precipitation is available at every location but may or may not be adequate for the recharge purposes. In such cases water from other sources may be transmitted to the recharge site. Assessment of the available sources of water would require consideration of the following factors: • Available quantity of water • Time for which the water would be available • Quality of water and the pretreatment required • Conveyance system required to bring the water to the recharge site.

Methods Of Artificial Recharge

These can be broadly classified as: • Spreading Method - Spreading within channel - Spreading stream water through a network of ditches and furrows - Ponding over large area

(a) Along stream channel viz. Check Dams/ Nala Bunds

(b) Vast open terrain of a drainage basin viz. Percolation Tanks

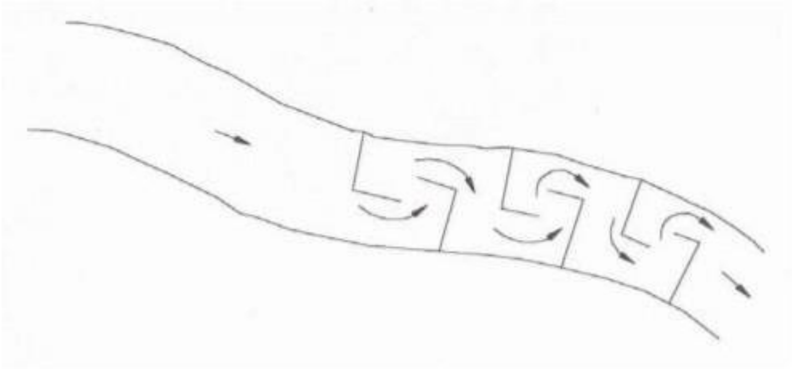
(c) Modification of village tanks as recharge structures.

• Recharge Shafts - Vertical Shafts - Lateral Shafts • Injection Wells • Induced Recharge • Improved Land and Watershed Management - Contour Bunding - Contour Trenching - Bench Terracing - Gully Plugging.

Channel Spreading

This involves constructing small ‘L’ shaped bunds within a stream channel so that water moves along a longer path thereby improving natural recharge as shown in Figure. This method is useful where a small flowing channel flows through a relatively wide valley. However this is not useful where rivers/ streams are prone to flash floods and the bunds (levees) may be destroyed

Figure 9.1 : Channel Spreading

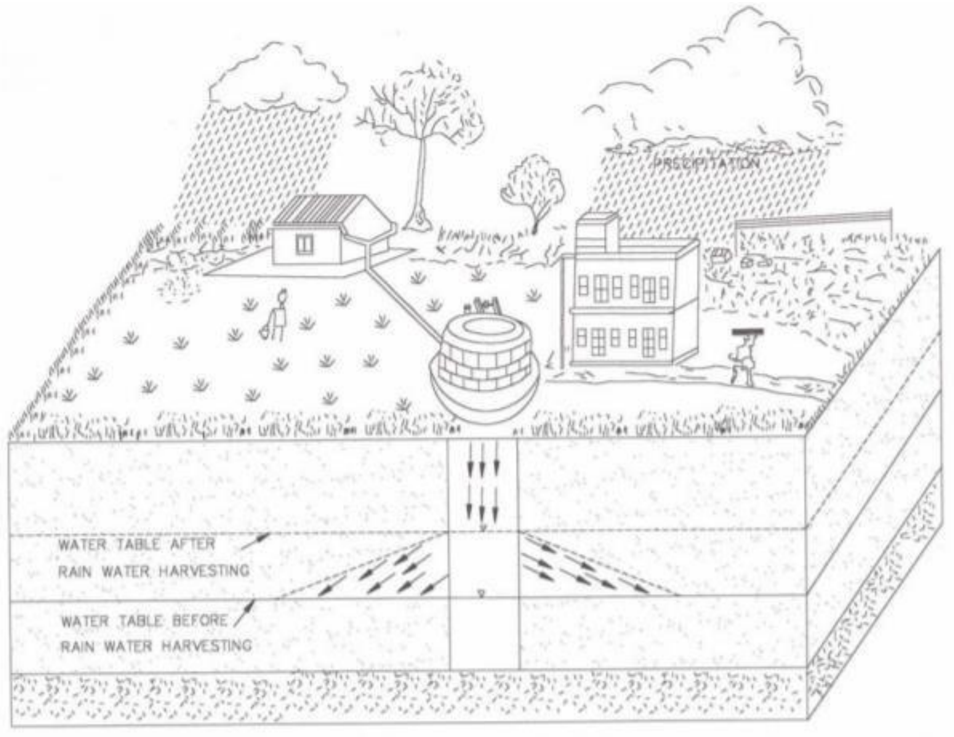


Ditch And Furrow Method

In areas with irregular topography, shallow, flat-bottomed and closely spaced ditches or furrows provide maximum water contact area for recharge water from source stream or canal. This technique requires less soil preparation than the recharge basins and is less sensitive to silting. Figure 9.2 shows a typical plan or series of ditches originating from a supply ditch and trending down the topographic slope towards the stream. Generally, three patterns of ditch and furrow system are adopted.

Recharge Of Dug Wells And Hand Pumps

In alluvial as well as hard rock areas, there are thousands of dug wells, which have either gone dry, or the water levels have declined considerably. These dug wells can be used as structures to recharge the ground water reservoir (Figure 9.3). Storm water, tank water, canal water etc. can be diverted into these structures to directly recharge the dried aquifer. By doing so the soil moisture losses during the normal process of artificial recharge, are reduced. The recharge water is guided through a pipe to the bottom of well, below the water level to avoid scouring of bottom and entrapment of air bubbles in the aquifer. The quality of source water including the silt content should be such that the quality of ground water reservoir is not deteriorated. Schematic diagrams of dug well recharge are given in Figure 9.3. In urban and rural areas, the roof top rainwater can be conserved and used for recharge of ground water. This approach requires connecting the outlet pipe from rooftop to divert the water to either existing wells/ tubewells/ borewells or specially designed wells. The urban housing complexes or institutional buildings having large roof areas can be utilised for harvesting roof top rainwater for recharge purposes (Figure 9.3).



Recharge Shaft

These are the most efficient and cost effective structures to recharge the aquifer directly. These can be constructed in areas where source of water is available either for some time or perennially. Following are the site characteristics and design guidelines: (i) To be dug manually if the strata is of non-caving nature. (ii) If the strata is caving, proper permeable lining in the form of open work, boulder lining should be provided. (iii) The diameter of shaft should normally be more than 2 m to accommodate more water and to avoid eddies in the well. (iv) In the areas where source water is having silt, the shaft should be filled with boulder, gravel and sand to form an inverted filter. The upper-most sandy layer has to be removed and cleaned periodically. A filter should also be provided before the source water enters the shaft. (v) When water is put into the recharge shaft directly through pipes, air bubbles are also sucked into the shaft through the pipe, which can choke the aquifer. The injection pipe should therefore be lowered below the water level. The main advantages of this technique are as follows: • It does not require acquisition of large piece of land as in case of percolation tanks. • There are practically no losses of water in the form of soil moisture and evaporation, which normally occur when the source water has to traverse the vadose zone. • Disused or even operational dugwells can be converted into recharge shafts, which does not involve additional investment for recharge structure. • Technology and design of the recharge shaft is simple and can be applied even where base flow is available for a limited period. • The recharge is fast and immediately delivers the benefit. In highly permeable formations, the recharge shafts are comparable to percolation tanks. The recharge shafts can be constructed in two different ways viz. vertical and lateral. The details of each are given in the following paragraphs.

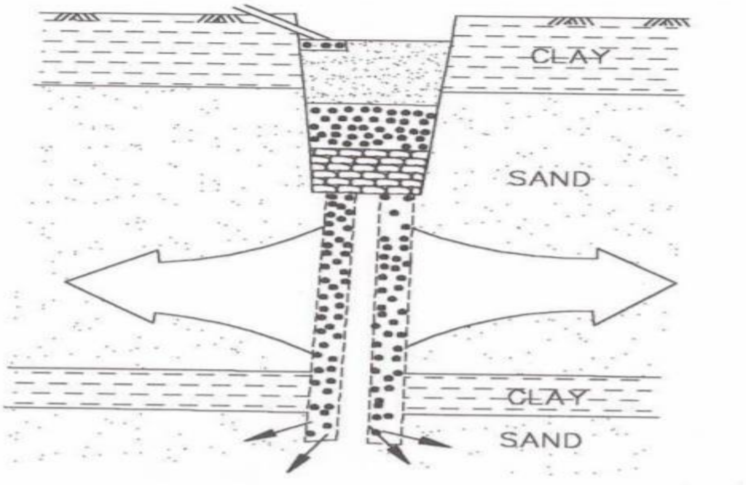
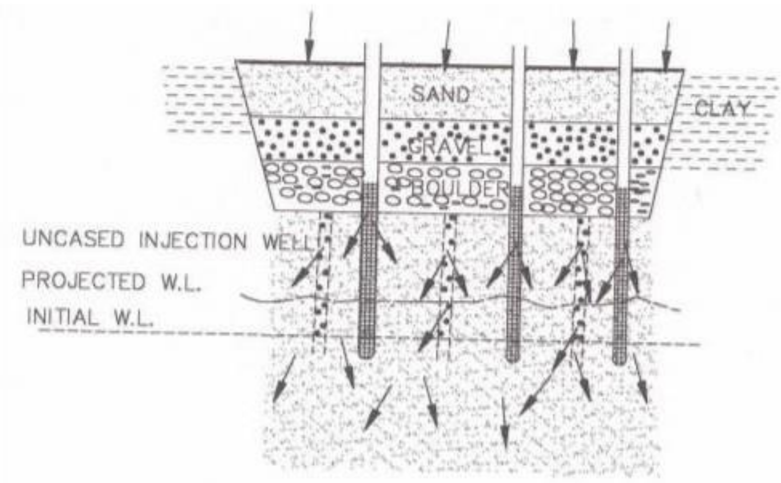


Figure 9.6 : Lateral Recharge Shaft



GROUNDWATER LEGISLATION

Recent Progress on Groundwater Legislation

By the 1990s, two events stressed the importance of groundwater management to policymakers. The first allowed local water agencies in critically overdrafted basins to develop groundwater management plans under AB 255. Secondly, in 1992 the legislature again amended the Water Code by passing the Groundwater Management Act, AB 3030. The law allows a variety of local water agencies to voluntarily develop groundwater management plans and to better coordinate the use of surface water and groundwater supplies, known as conjunctive use.

Legislation in 2001 required the California State Water Resources Control Board to create a groundwater quality monitoring program that assessed each of the state's groundwater basins and to establish a task force to increase coordination among state and federal agencies that collect groundwater contamination information.

Additional legislation in 2001 included a bill requiring local water agencies to map groundwater areas that substantially contribute to the replenishment of the groundwater basin and to submit groundwater maps to local planning agencies, as a condition of receiving state grants or loans. Another bill made the depth, soils and other information in well completion reports available to the public.

In 2002, a law was enacted that contains specific new requirements for local groundwater management plans in order for agencies to be eligible for public funding for projects. Between 2000 and 2005 the state Department of Water Resources awarded nearly $28 million in grants to local agencies to conduct 128 projects based on groundwater management plans.

Artificial recharge is a process by which the groundwater reservoir is augmented artificially. The rapid urbanization and deforestation have considerably reduced the groundwater recharge in many parts of the world. The reduction in groundwater recharge and over exploitation of groundwater due to increasing demands, the groundwater table has been depleted in many parts of the world. For example, the groundwater table in some parts of Delhi has been depleted by 20 to 30 meters in a span of 60 years. Same is the condition in other major cities in India and other parts of the world. As such there is a need to increase the groundwater recharge by some artificial means. In this lecture, we will discuss some of the methods use for artificial recharge and also the methods use in estimation of groundwater recharge.

Techniques of groundwater recharge

The artificial techniques use for groundwater recharge can be divided in two groups, i.e. direct method and indirect method. Further, the direct method can be sub grouped as surface method and sub-surface method. The main objective of the surface method is to enhance groundwater infiltration by providing more residence time with the help of structural and nonstructural measures. Some of the structural measures are contour bunding, percolation tank, check dams, etc . On the other hand, afforestation falls under the category of nonstructural measures. The induced recharge method and aquifer modification method falls under the category of indirect method.

- Direct Methods
- Surface method

1. 1. Percolation tank

In this method, series of earthen dams are constructed on suitable sites for storing of adequate quantity of surface water. The tank area should be selected in such a way that significant amount of water infiltrates through the bed of the tank and reaches the groundwater table. This method is very effective in alluvial area as well as in areas with hard rock. This method is very useful in providing continuous recharge after the monsoon.

2. Flooding

This method is suitable for relatively flat region where water can be spread as a thin layer. Water is distributed over the region using a distribution system. This method can achieve higher rate of infiltration in a region having thin vegetation cover or sand soil cover. Fig. 29.1 shows a schematic diagram of recharge basin.

Fig. 29.1

3. Stream augmentation

In this method, seepage from natural stream or river is artificially increased by putting some series of check dams across the river or stream. The placing of check dams spread the water in a larger area which eventually increases groundwater recharge. The sites for the check dams should be selected in such a way that sufficient thickness of permeable bed or weathered bed is available for quick recharging the stored water.

4. Ditch and furrow system

This method is used for uneven terrain. In this technique, a system of closely spaced flat bottom ditch or furrow is used to carry the water from the source. This system provides more opportunity to percolate the water into the ground. The spacing of the ditch depends on the permeability of the soil. For less permeable soil, more densely spaced ditch or furrow should be provided.

5. Contour bund

Contour bund is a small embankment constructed along the contour in hilly region to retain the surface runoff for longer time. This scheme is adopted for low rainfall area where internal subsurface drainage is good.

Subsurface method

1. Recharge well

Recharge wells are used to recharge water directly to the aquifer. Recharge wells are similar to pumping wells. This method is suitable to recharge single wells or multiple wells. This method is costlier than the other method as wells are required to be bored. However, sometimes abandoned tube wells can be used for recharging water into the aquifer.

2. Dug well

Dug wells can also be used to artificially recharge the groundwater. Generally, water level of dug wells depletes during the non monsoon period. Sometime the dug wells even dried up in the non-monsoon period. These dug wells can be used for recharging groundwater. The water from various sources can be collected through a distribution system and can be discharged at the dug wells.

3. Pits and shafts

Recharge pit of variable dimensions are used to recharge water to unconfined aquifer. Most of the time, especially in case of agricultural field, a layer of less permeable soil exist. Due to the existence of the less permeable

permeable strata, the surface flooding methods of recharge do not show satisfactory performance. For such type of cases, recharge pit can be excavated which are sufficiently deep to penetrate the less permeable strata. On the other hand recharge shaft is similar to the recharge pits, but the cross sectional size of the recharge shaft is much lesser than the recharge pits. Like the recharge pits, recharge shafts are also used to recharge water to unconfined aquifer whose water table is deep below the land surface and a poorly impermeable strata exist at the surface level.

- Indirect method
- Induced recharge

It is an indirect method of artificial recharge. In this method water is pumped from the aquifer hydraulically connected to the surface water sources like stream, river or lake. Due to pumping, a reverse gradient is formed and water from the surface water source enters into the aquifer and thus the aquifer is recharged . This method is good, especially when quality of the surface water is poor. The filtration of surface water through soil strata removes the impurities of the water. Thus the quality of the water receives in the wells is much better than the surface water.

Aquifer modification method

This is also an indirect method of artificial recharge. In this method, some techniques are used to change the aquifer characteristic so that aquifer can store more water and also can transmit more water. After application of these techniques, more recharge takes place under natural condition as well as under artificial condition. The most commonly used techniques are, bore blasting method, hydro-fracturing method, jacket well techniques, fracture seal cementation and pressure injection grouting, etc.

1. Bore blasting method

This method is used to increase the fracture porosity of an aquifer. Shallow bore wells are drilled in the area where fracture porosity of the aquifer is planned to increase. These bore holes are blasted with the help of explosive which creates fracture porosity in the aquifer.

2. Hydro-fracturing method

Hydro-fracturing is used to improve the yield of a bore well. In this technique, water is injected at a very high pressure to widening the existing fracture of the rock. The high pressure injection of water also helps in removing of clogging, creates interconnection between the fractures, and extends the existing length of the old fracture. The high pressure injection also creates new fracture in the rock strata. As a result of these, the water storing and transmitting capacity of the strata increases.

3. Jacket well techniques

Jacket well technique is used to increase the yield of a dug well. In this method, the effective diameter of the well is increased by drilling small diameter bores around the well in a circular pattern.

4. Fracture seal cementation and pressure injection grouting

This technique is used to control the outflow from an aquifer. Cement slurry is injected into the aquifer using mechanical means or manually near to the aquifer outlet like spring, etc. The injection of cement slurry helps in reducing the fracture porosity of the aquifer near the outlet which will eventually reduce the outflow from the aquifer.

References:

https://youtu.be/W7wfmx8t9pc-Artificialrechrge of Groundwater Methods

https://youtu.be/ztQdNJt3ZVE- Saline water Intrusion

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UNIT – V – GROUNDWATER ENGINEERING – SCI1602

GROUNWATER MANAGEMENT

Groundwater balance

Water enters into an aquifer primarily through the process of recharge from rainfall. Water may also enter into an aquifer by the processes of recharge from canal seepage (R_r), return flow from irrigation field (R_f) , leakage from overlaying and underlying aquifers, *i.e.* leaky aquifer (Q_{li}), artificial recharge (Q_r), seepage from streams and lakes (Q_{si}), inflow from the neighboring basins (Q_i). Water can come out from an aquifer by the process of withdrawal from the groundwater aquifer (Q_p), evapotranspiration from groundwater (E_t), outflow to the neighboring basins (Q_o), seepage to the streams and lakes (Q_{so}), leakage to overlaying and underlying aquifers (Q_{lo}), discharge through spring (Q_s).

Considering the various inflows and outflows as mentioned above, the groundwater balance equation can be written as:

$$R_r + R_f + Q_{li} + Q_r + Q_{si} + Q_i = Q_p + E_t + Q_o + Q_{so} + Q_{lo} + Q_s + \Delta S \qquad \textbf{(2.1)}$$

Where ΔS is the change in storage. Sometimes, from practical point of view, it may not be possible to compute all the components of the groundwater balance equation. Many times some components are lumped together to get a net response of these components.

Role of groundwater in water resources system and their management

Though groundwater is a separate section of hydrology, but in true sense it cannot be separated from the surface water. Take an example of an aquifer near a river. The river water has a very

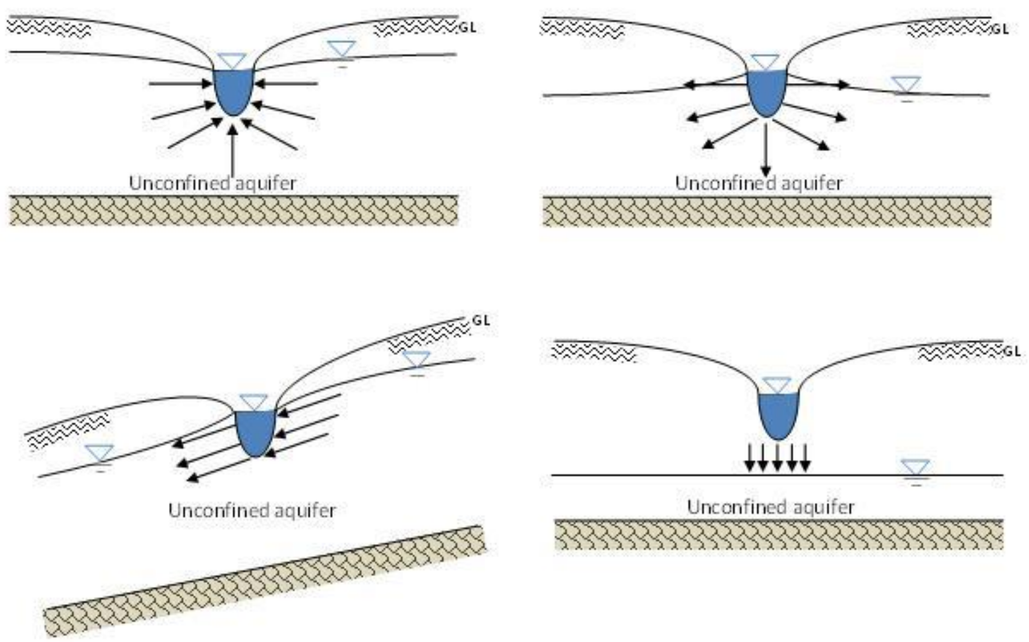


Fig. 1 Groundwater surface water interaction

strong interaction with the aquifer hydraulically connected with the river. Another example of surface water and groundwater interaction is the spring discharge. The spring discharge can be altered by controlling the groundwater level in the vicinity of the spring. These examples show that groundwater has a very strong interaction with the surface water. Therefore in management of regional water resources, it is always necessary to consider both surface water and groundwater resources of the region. As far as practicable, both the resources should be used judiciously.

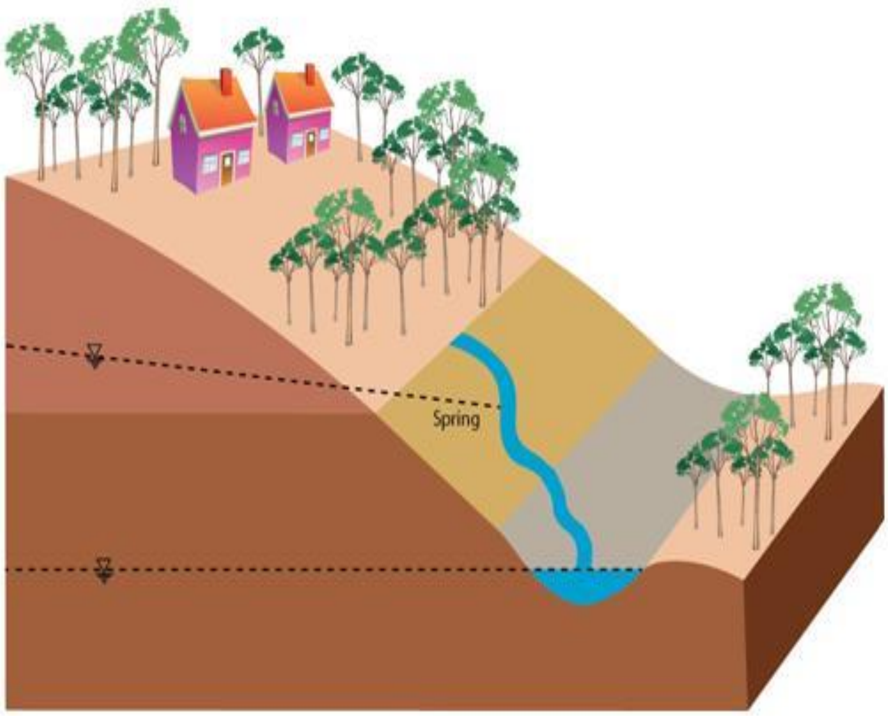


Fig. 2 Spring discharge

Estimation of the groundwater recharge

Various methods are available for estimation of the groundwater recharge. Some of the frequently used methods for estimation of groundwater recharge are,

- Groundwater balance method
- Empirical formula
- Groundwater table fluctuation method
- Zero flux plane method
- Darcy's law
- Tracer techniques

Groundwater balance method

Groundwater balance method can be used for quantitative estimation of groundwater recharge. The groundwater balance equation described earlier can be used to estimate the aquifer recharge. The equation can be written as,

Inflow to the system – Outflow from the system = Change in the storage of the system over a period of time

Putting the various inflow and outflow components, the above equation can be written as,

$$(R_r + R_c + R_i + I_s + I_b) - (E_t + P + O_s + O_b) = \frac{dS}{dt}$$

Where, R_r is the recharge from rainfall, R_c is the recharge from canal seepage, R_i is the recharge from field irrigation, l_s is the influent seepage from river, l_b is the inflow from other basin, E_t is the evapotranspiration, P is the withdrawal from the aquifer, O_s is the effluent discharge to the river, O_b is the outflow to other basin and $\frac{dS}{dt}$ is the change is storage. Knowing the other components of the groundwater balance equation, the groundwater recharge can be estimated.

Empirical formula

Many empirical equations have been developed for estimation of groundwater recharge. The empirical equation generally relates precipitation with the groundwater recharge. Chaturvedi (1973) proposed the following empirical relation based on the water level fluctuation and rainfall.

$$R = 2(P - 15)^{0.4}$$

Irrigation Research Institute, Roorkee proposed a modified version of the equation as follows,

$$R = 1.35(P - 14)^{0.4}$$

Sehgal in 1973 proposed the following empirical formula for estimation of groundwater recharge.

$$R = 2.5(P - 0.6)^{0.5}$$

Where, R and P are the groundwater recharge and precipitation measured in inch. The formula was found to be good for the areas where rainfall is between 60 to 70 cms

Groundwater table fluctuation method

The rise in water table during the rainy season can be used to estimate groundwater recharge due to rainfall. If the rise in groundwater table during the rainfall season is Δh , the groundwater recharge can be estimated as,

$$R = S_y.\Delta h + P - R_i$$

Where, R is the groundwater recharge, S_y is the specific yield, P is the groundwater abstraction during rainy season per unit area, R_i is the return flow from irrigation field during rainy season per unit area.

Zero flux plane method

The groundwater recharge can be estimated by equating changes in soil water storage below the zero flux plane to the recharge. The zero flux plane represents the plane where the vertical hydraulic gradient is equal to zero. Between two successive measurements, the rate of change in storage is assumed to be equal to the recharge rate. This method is relatively expensive as it requires soil matric potential measurements to locate the position of the zero flux plane and soil water content measurements for estimating the change in the storage.

Darcy's law

Darcy's law can be used for calculation of groundwater recharge rate. In case of unsaturated zone, the Darcy's law can be written as,

$$R = -K(\theta)\frac{\partial H}{\partial z} = -K(\theta)\frac{\partial}{\partial z}(h + z)$$

$$\Rightarrow R = -K(\theta)\frac{\partial}{\partial z}(h + z)$$

$$\Rightarrow R = -K(\theta)\left(\frac{\partial h}{\partial z} + 1\right)$$

Where, $K(\theta)$ is the unsaturated hydraulic conductivity at the ambient water content, H is the total head, h is the matric pressure head and Z is the elevation head.

Tracer techniques

Chemical or isotopic tracers can also be used for estimation of groundwater recharge. In this method, the tracer is applied as a pulse at the soil surface or at some depth below the soil surface. After some time from the application of tracer, the vertical distribution of tracer under the ground is obtained. The vertical distribution of tracer is then used for estimation of the recharge rate.

$$R = \frac{\delta z}{\delta t}\theta$$

where δz is depth of the tracer peak, δt is the time between tracer application and sampling, and θ is volumetric water content. Most commonly use tracer are, visible dyes, bromide and ^{3}H (Scanlon, *et al* 2002).

Advantages of groundwater recharge

1. Exploit the surplus surface water which otherwise drains off.

2. Enhance the reliability of yield from pumping wells.

3. Reduction of surface runoff which can eventually reduce the risk of urban flooding.

4. Reduce the risk of inundation of large surface areas and loss of crops.

5. Improvement of water quality due to the removal of harmful chemicals, suspended sediments, *etc.* in the filter layer.

6. Large storage structures are not required to store water.

7. No displacement of local people unlike the other water resources projects.

8. Reduction in cost of energy for lifting water as water table rises due to groundwater recharge.

Classification of recharge zone

Recharge zones can be classified based on the correlation coefficient between the rainfall and groundwater recharge**.**

High recharge zone	$r > 0.6$
Moderate recharge zone	$0.5 \leq r \leq 0.6$
Less recharge zone	$0.4 \leq r \leq 0.5$
Poor recharge zone	$r < 0.4$

Dynamic equilibrium in natural aquifers

As discussed earlier, an aquifer acts as a storage tank under the ground. The water enters into ground by the process of infiltration through vadose zone. Once the water reaches the aquifer , it flows in the direction of lower hydraulic head and finally comes out of the aquifer through the discharge zones. Thus the groundwater in an aquifer is in a state of continuous movement from one place to other. The movement of water depends on the nature of the subsurface geology. Many times, the average value of amount of water that enters into the ground is equivalent to the average value of the amount of water coming out of the aquifer and often reach an equilibrium state. This state of equilibrium is known as dynamic equilibrium of a natural aquifer.

Consider a case of pumping from an isolated well. The potential surface will start depleting just after the start of the pumping and will form a cone of depression. The rate of depletion depends on the aquifer storativity. Lesser the value of storativity more will be the spread of the cone of depression. The spread of cone of depression will increase with the continuation of the pumping. With the depletion of the potential surface, more and more water will move towards the well due to the increased gradient. And finally, a state will reach where the radial inflow rate to the well is equal to the pumping rate from the well. At this state, the aquifer will reach the dynamic equilibrium state.

Functions of Aquifer

Aquifer plays an important role in overall development of water resources of a region. The primary and one of the most significant uses of aquifer is the withdrawal of water for various purposes like municipal use, agricultural use, industrial use, etc. For the people living away from the river, stream and other surface water sources, the main source of water is the groundwater withdrawal from unconfined or confined aquifers. Thus one of the main functions of an aquifer is the supply of water for various needs.

Another function of an aquifer is the transmission of water from one place to another. The water withdrawal from a confined aquifer is the water recharged into the ground at a location far away from the location of withdrawal. Thus aquifer also acts as a water transmission conduit.

The groundwater is recharged into the aquifer through the process of infiltration. The rain water falls on the ground or the water carrying through a canal or the water spread over an irrigation field percolates through the unsaturated zone above the aquifer. The unsaturated zone above the aquifer acts as a filtration layer and water is purified while seeps through the zone. Thus aquifer is also acting as a filtration plant.

Aquifer can also be used to store natural gases. One of the potential applications is the storage of CO_2 in the aquifer to reduce CO_2 emission to the atmosphere. The process of storing of CO_2 in a geological formation is known as geo-sequestration.

Groundwater can also be mined similar to the process of mining of natural minerals. In general in a large time scale, the withdrawal from aquifer is nearly equal to the average natural replenishment. However, in certain situation, we may withdraw all the available water in an aquifer just like nonrenewable resources.

Management of Groundwater Resources

Groundwater management aims to achieve certain strategies for sustainable use of the resources. Some of the objectives of the groundwater management models are:

1. Maximization of total withdrawal from an aquifer.

2. Minimization of cost of per unit volume of water supplied to the consumers.

3. Maximization of net benefit of a project related to the supply of water for municipal, industrial, agricultural, *etc.* purposes.

4. Minimization of total consumption of energy of project related to the extraction and distribution of groundwater.

5. Minimization of an error function obtained from the sum of the absolute difference between actual and predicted values of certain water level.

In achieving these objectives, certain restrictions have to be satisfied for obtaining physically meaningful optimal policies. These restrictions are known as constraints. Some of the commonly applicable constraints in groundwater management models are:

1. Water level at any location in an aquifer should not go below certain value. This constraint will put restriction on the depletion of water table due to pumping of water from an aquifer.

2. Water level at any location in an aquifer should not rise above certain level. This is required to avoid water logging of an area and also to dewater an area under construction.

3. Limits on the maximum and minimum withdrawal of water from an aquifer.

4. Spring discharge should not go below certain specified value.

5. Base flow of a stream or a river should not go below certain minimum value.

6. Quality of pumped water should not deteriorate below certain specific value.

7. Limit on land subsidence.

Solution techniques of groundwater management models

A groundwater management model should include the groundwater simulation model as constraint along with other managerial constraints. Incorporation of groundwater simulation model ensures that the obtained management strategies are physically feasible. The simulation model simulates the physical processes of the aquifers. Generally embedding technique and response matrix approach (Gorelick, 1983) are used for incorporating the governing equations within the management model. The embedded optimization technique incorporates finite difference or finite element approximation of the governing equations as equality constraints within the management model, along with the other physical and managerial constraints. Some of the application of embedding technique for groundwater management problems are seen in Das, 1995; Das and Datta 1999, etc. However, this approach is not suitable for large aquifer systems. The approach may be numerically inefficient especially when applied to large aquifer systems with considerable heterogeneity. The response matrix approach is based on the principle of superposition and linearity. The performance of response matrix approach is not suitable for highly nonlinear systems (Rosenwald and Green, 1974).

As an alternative to the embedding technique and the response matrix approach, the simulation model may be incorporated with the management model as an external module. In this approach, an external simulation model is linked to the optimization model (Finney at el., 1992; Emch and Yeh, 1998; Bhattacharjya and Datta, 2009). The optimization model calls the simulation model as and when it requires any information from the simulation model. The methodology has been applied effectively for large scale groundwater management models. The main disadvantage of this approach is that numerous repetitive iterations between the simulation model and the optimizer are required to arrive at an optimal solution. The computational time can be substantially reduced by utilizing parallel processing capabilities of advanced computers. This would enable use of rigorous numerical models for simulation and its linkage to an optimization model. Also the time requirement for iterative solutions of the optimization model and the simulation model can be drastically reduced. However, this requires appropriate computer hardware and numerical simulation models specially tailored to explicit parallel processing capabilities.

An optimization technique has to be used for solving the management model. Classical optimization techniques have been applied for solving groundwater management problems. Most of the classical optimization methods use gradient search technique for finding the optimal solution. The performance of the gradient based classical optimization methods is not satisfactory when response surface is highly irregular. In such a situation, it is very likely that the solutions obtained would be local optimal solutions. One possible remedy is the use multiple solution points as initial solutions. Some people have also used global search techniques, such as Genetic Algorithm, Simulated Annealing, Differential evolution, *etc.* for solving groundwater management models.

Solution of simple groundwater management model

(a) Confined aquifer

Now we will solve a very simple groundwater management problem. Consider a case of one dimensional steady state flow in a confined aquifer. The governing equation in this case can be written as,

$$T_x \frac{\partial^2 h}{\partial x^2} + N(x) = 0$$

The finite difference form of the equation can be written as,

$$\frac{h_{i+1} - 2h_i + h_{i-1}}{(\Delta x)^2} + \frac{N_i}{T_x} = 0$$

$$h_{i-1} - 2h_i + h_{i+1} + \frac{(\Delta x)^2}{T_x} N_i = 0$$

$$h_{i-1} - 2h_i + h_{i+1} + PN_i = 0 \qquad \text{Considering } P = \frac{(\Delta x)^2}{T_x}$$

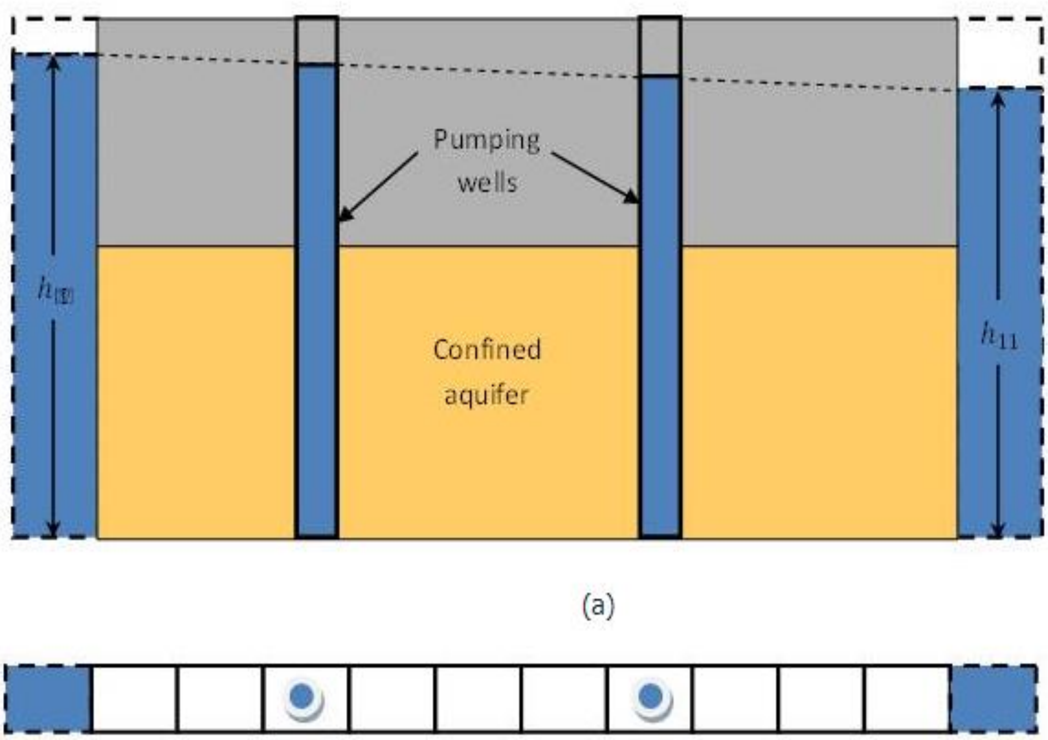


Fig. 3 Confined Aquifer

Consider the confined aquifer as shown in Fig. 1. The aquifer has been discretized to 10 blocks. Constant head boundary is considered on both upstream and downstream sides of the aquifer. Let the constant head at the upstream side be h_0 and constant head at the downstream side of the aquifer be h_{11}. Let us consider that the heads at the block centers are h_1 to h_{10} as shown in the Fig. 2. There are two pumping wells in the aquifer as shown in the Fig. Let us also consider that the main objective of the management model is to withdrawal an amount of water equal to N_{min}. The finite difference form of the governing equation at each block center has to be incorporated as constraint with the optimization problem along with the other constraints**.**

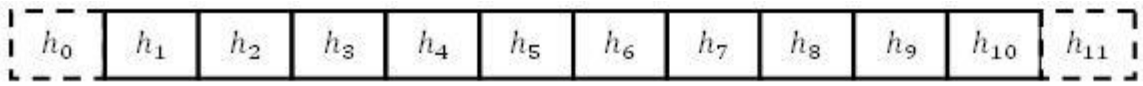


Fig. 4 Discretized aquifer

Inclusion of these constraints ensures that the solution obtained would be physically feasible. The optimization problem can be formulated as,

Maximize $f = \sum_{k=1}^{10} h_k$

Subject to

$-2h_1 + h_2 + PN_1 = h_0$

$h_1 - 2h_2 + h_3 + PN_2 = 0$

$h_2 - 2h_3 + h_4 + PN_3 = 0$

$h_3 - 2h_4 + h_5 + PN_4 = 0$

$h_4 - 2h_5 + h_6 + PN_5 = 0$

$h_5 - 2h_6 + h_7 + PN_6 = 0$

$h_6 - 2h_7 + h_8 + PN_7 = 0$

$h_7 - 2h_8 + h_9 + PN_8 = 0$

$h_8 - 2h_9 + h_{10} + PN_9 = 0$

$h_9 - 2h_{10} + PN_{10} = h_{11}$

$\sum_{k=1}^{10} N_k \geq N_{min}$

$h_k \geq 0 \quad for\ k = 1,2 \ldots 10$

$N_k \geq 0 \quad for\ k = 1,2 \ldots 10$

The objective function will try to maximize the head value of the system while satisfying all the constraints.

The constraint ensures that the minimum withdrawn is N_{min} and the constraints and are the non negativity constraints. Here the decision variables are N_1 to N_{10} and h_1 to h_{10} are the state variables. If we look at the optimization problem, the objective function and the constraints are linear in nature. As such the problem is a linear problem (LP) and can be solved using any LP solving algorithm like Simplex method. We will not discuss about any optimization algorithms here. However, students are requested to go through the NPTEL course developed by Prof. Nagesh Kumar on "Optimization Method" for details about the optimization algorithms.

The solution of the optimization problem will give the spatial distribution of pumping pattern where total pumping from all the pumping wells is equal to N_{min}.

(b) Unconfined aquifer

Now consider a case of one dimensional steady state flow in an unconfined aquifer. The governing equation in this case can be written as,

$$\frac{\partial}{\partial x}\left(T_x \frac{\partial h}{\partial x}\right) + N(x) = 0$$

Considering $T_x = Kh$

$$\frac{\partial^2 h^2}{\partial x^2} + \frac{2N(x)}{K} = 0$$

The above equation is a nonlinear equation. The equation can be made linear by considering $d = h^2$. Thus the equation can be written as,

$$\frac{\partial^2 d}{\partial x^2} + \frac{2N(x)}{K} = 0$$

The finite difference form of the equation can be written as,

$$\frac{d_{i+1} - 2d_i + d_{i-1}}{(\Delta x)^2} + \frac{2N_i}{K} = 0$$

$$d_{i-1} - 2d_i + d_{i+1} + \frac{2(\Delta x)^2}{K} N_i = 0$$

$$d_{i-1} - 2d_i + d_{i+1} + RN_i = 0 \quad \text{Considering } R = \frac{2(\Delta x)^2}{K}$$

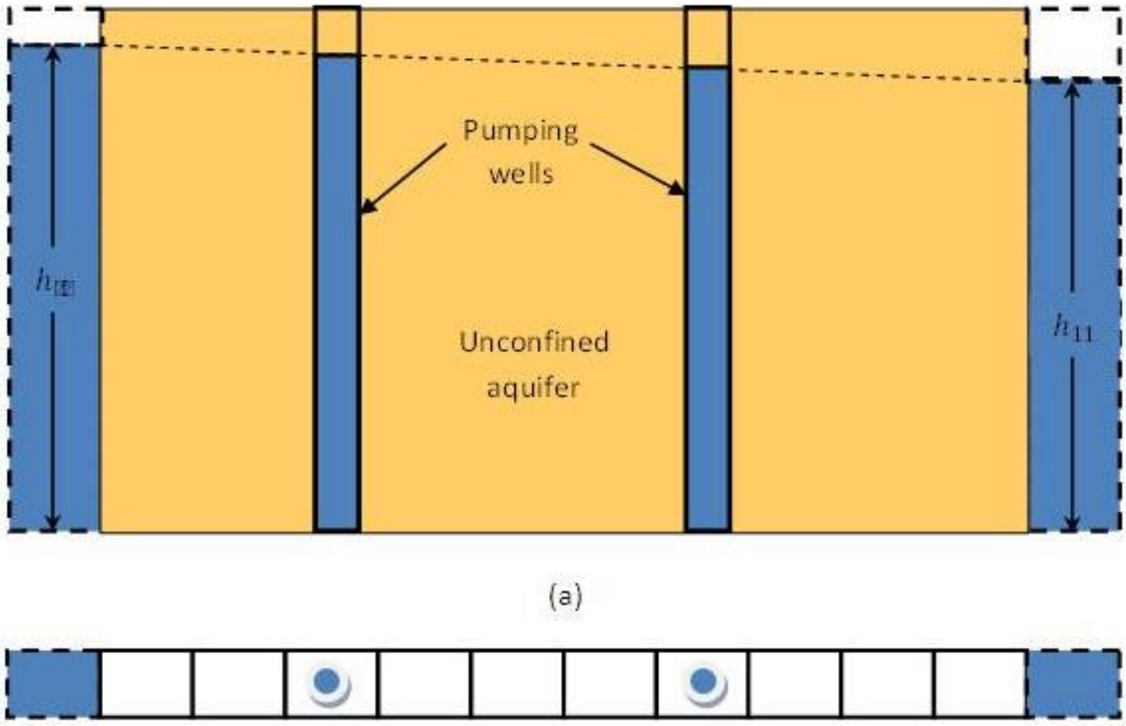


Fig. 5 Unconfined Aquifer

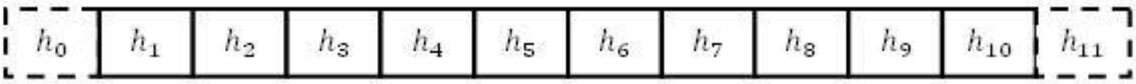


Fig. 6 Discretized aquifer

There are two pumping wells in the aquifer as shown in the Fig 3. Let us also consider that the main objective of the management model is to withdraw an amount of water equal to N_{min}. The finite difference form of the governing equation at each block centers has to be incorporated as constraints with the optimization problem along with the other constraints. Inclusion of these constraints ensures that the solution obtained would be physically feasible. The optimization problem can be formulated as, Maximize $f = \sum_{k=1}^{10} d_k$

Subject to

$$-2d_1 + d_2 + RN_1 = d_0$$

$$d_1 - 2d_2 + d_3 + RN_2 = 0$$

$$d_2 - 2d_3 + d_4 + RN_3 = 0$$

$$d_3 - 2d_4 + d_5 + RN_4 = 0$$

$$d_4 - 2d_5 + d_6 + RN_5 = 0$$

$$d_5 - 2d_6 + d_7 + RN_6 = 0$$

$$d_6 - 2d_7 + d_8 + RN_7 = 0$$

$$d_7 - 2d_8 + d_9 + RN_8 = 0$$

$$d_8 - 2d_9 + d_{10} + RN_9 = 0$$

$$d_9 - 2d_{10} + RN_{10} = d_{11}$$

$$\sum_{k=1}^{10} N_k \geq N_{min}$$

$$d_k \geq 0 \quad for\ k = 1,2 \dots 10$$

$$N_k \geq 0 \quad for\ k = 1,2 \dots 10$$

The constraint ensures that the minimum withdrawn is N_{min} and the constraints and are the non negativity constraints. Here the decision variables are N_1 to N_{10} and h_1 to h_{10} are the state variables. The solution of the optimization problem will give spatial distribution of pumping pattern where total pumping from all the wells will be equal to N_{min}.

This lecture will be dealt with the solution of more complicated groundwater management models. Consider case of transient flow in a confined aquifer. The primary design objective is to maximize the amount of pumping from the well field. At the same time, it is also necessary to see that the groundwater table does not deplete beyond certain depth. This problem can be solved using an optimization technique. In order to obtain meaningful groundwater management strategies, the aquifer simulation model is required to incorporate with the optimization model. In this lecture, we will simulate the aquifer flow process using finite difference method. In the next lecture, we will developed the management model.

Simulation of flow process using finite difference method

The transient simulation model of an aquifer gives the head values at each aquifer location after every time interval. The number of such intervals is user defined. For a particular time interval, the pumping rate is constant. The 2D governing equation for this case is,

$$\frac{\partial}{\partial x}\left[T_x \frac{\partial h}{\partial x}\right] + \frac{\partial}{\partial y}\left[T_y \frac{\partial h}{\partial y}\right] + N(x,y) = S\frac{\partial h}{\partial t} \qquad (1)$$

Finite difference form of the equation is given as :

$$A_{ij}\left(h_{i+1,j}^{n-1} + h_{i+1,j}^{n}\right) + B_{ij}\left(h_{i-1,j}^{n-1} + h_{i-1,j}^{n}\right) + C_{ij}\left(h_{i,j+1}^{n-1} + h_{i,j+1}^{n}\right) + D_{ij}\left(h_{i,j-1}^{n-1} + h_{i,j-1}^{n}\right) - \left(A_{ij} + B_{ij} + C_{ij} + D_{ij} + \frac{2S}{\Delta t}\right)h_{i,j}^{n} + 2N_{i,j}^{n} - \left(A_{ij} + B_{ij} + C_{ij} + D_{ij} + \frac{2S}{\Delta t}\right)h_{i,j}^{n-1} = 0 \qquad (2)$$

Where,

$$A_{ij} = \frac{1}{(\Delta x)^2}\left(\frac{2T_x(i+1,j) * 2T_x(i,j)}{T_x(i+1,j) + T_x(i,j)}\right) \qquad (3)$$

$$B_{ij} = \frac{1}{(\Delta x)^2}\left(\frac{2T_x(i-1,j) * 2T_x(i,j)}{T_x(i-1,j) + T_x(i,j)}\right) \qquad (4)$$

$$C_{ij} = \frac{1}{(\Delta y)^2}\left(\frac{2T_y(i+1,j) * 2T_y(i,j)}{T_y(i+1,j) + T_y(i,j)}\right) \qquad (5)$$

$$D_{ij} = \frac{1}{(\Delta y)^2}\left(\frac{2T_y(i-1,j) * 2T_y(i,j)}{T_y(i-1,j) + T_y(i,j)}\right) \qquad (6)$$

The above equation is rearranged as

$$-\left(A_{ij} + B_{ij} + C_{ij} + D_{ij} + \frac{2S}{\Delta t}\right)h_{i,j}^{n} + A_{ij}h_{i+1,j}^{n} + B_{ij}h_{i-1,j}^{n} + C_{ij}h_{i,j+1}^{n} + D_{ij}h_{i,j-1}^{n} + 2N_{i,j}^{n} = \left(A_{ij} + B_{ij} + C_{ij} + D_{ij} + \frac{2S}{\Delta t}\right)h_{i,j}^{n-1} - A_{ij}h_{i+1,j}^{n-1} - B_{ij}h_{i-1,j}^{n-1} - C_{ij}h_{i,j+1}^{n-1} - D_{ij}h_{i,j-1}^{n-1} \qquad (7)$$

Analyzing the above equation, the left hand side contains the variables involving head values at n^{th} time interval while the right hand side of the equation contains head values at (n-1)th time interval. Hence, knowing the head value at a given time interval, the head values at the next time interval can be calculated, *i.e.* h^{n+1} can be obtain using h^n, h^{n+2} can be obtain using h^{n+1}, and so on. Therefore, the head distribution at h^0 must be known for finding the solution. This is known as initial condition of the aquifer. The h^0 can be obtained from field observation. However, for the example problem considered here, the steady state head value is considered as h^0.

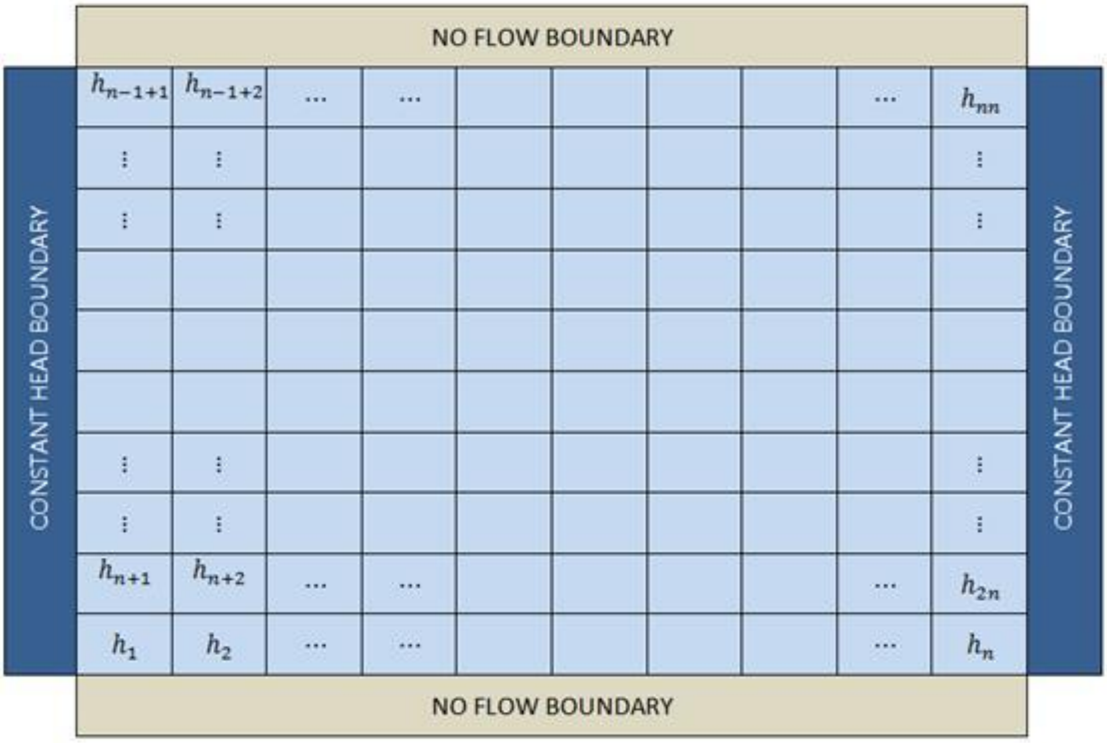


Fig. 7 Discretized aquifer

Consider the discretized aquifer shown in Fig. 24 .1. The aquifer domain has been divided in *nxn* grids. Now write the equation (24.7) in each grid center. There will be total *nxn* number of equations which is equal to the number of unknowns, *i.e.* the head value at each node center. The set of *nxn* equation can be written as,

$$\begin{bmatrix} a_{11} & \cdots & a_{1(n\times n)} \\ \vdots & \ddots & \vdots \\ a_{(n\times n)1} & \cdots & a_{(n\times n)(n\times n)} \end{bmatrix} \begin{bmatrix} h_1 \\ \vdots \\ h_{n\times n} \end{bmatrix} = \begin{bmatrix} r_1 \\ \vdots \\ r_{n\times n} \end{bmatrix} \qquad \textbf{(.8)}$$

Where a_{11} is the coefficient of h_1 in equation 1. Similarly, $a_{1(nxn)}$ is the coefficient of h_{nxn} in equation 1, $a_{(nxn)(nxn)}$ is the coefficient of $h_{(nxn)}$ in equation *(nxn)*. r_1 is the right hand side of equation 1. Similarly, r_{nxn} is the right hand side of equation nxn. The equation (24 .8) can also be written as,

$$AH = R \tag{.9}$$

where,

where,

$$A = \begin{bmatrix} a_{11} & \cdots & a_{1(n\times n)} \\ \vdots & \ddots & \vdots \\ a_{(n\times n)1} & \cdots & a_{(n\times n)(n\times n)} \end{bmatrix}$$

$$H = \begin{bmatrix} h_1 \\ \vdots \\ h_{n\times n} \end{bmatrix}$$

$$R = \begin{bmatrix} r_1 \\ \vdots \\ r_{n\times n} \end{bmatrix}$$

The equation (24 .9) can be solved as,

$$H = A^{-1}R$$

Consider the aquifer shown in Fig. 6

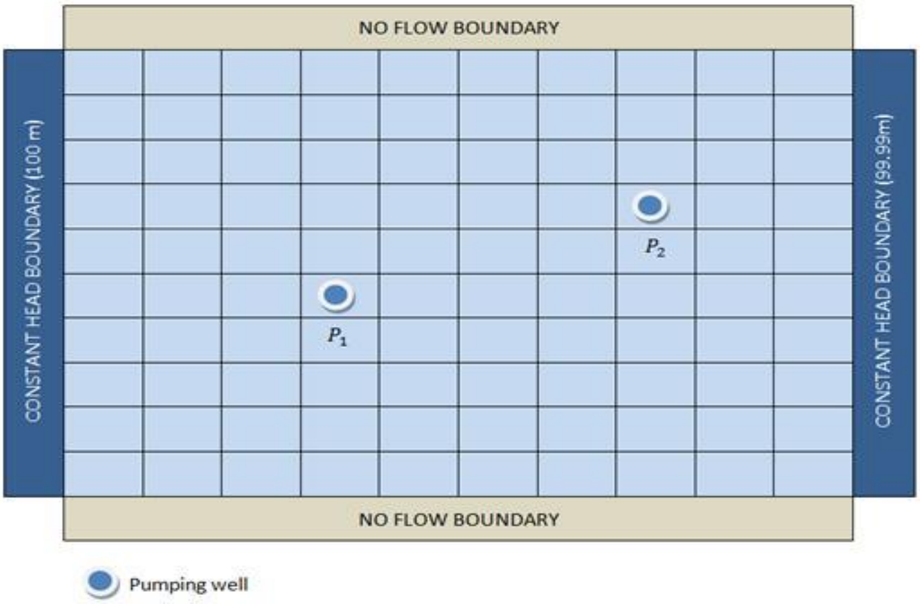


Fig. 8 Discretized aquifer with pumping wells

The confined aquifer has constant head of 100.00 m at the left side and constant head of 99.9 m at the right hand. The other two sides are bounded by impermeable layer. Thus no flow boundary

Table 1 Pumping patterns

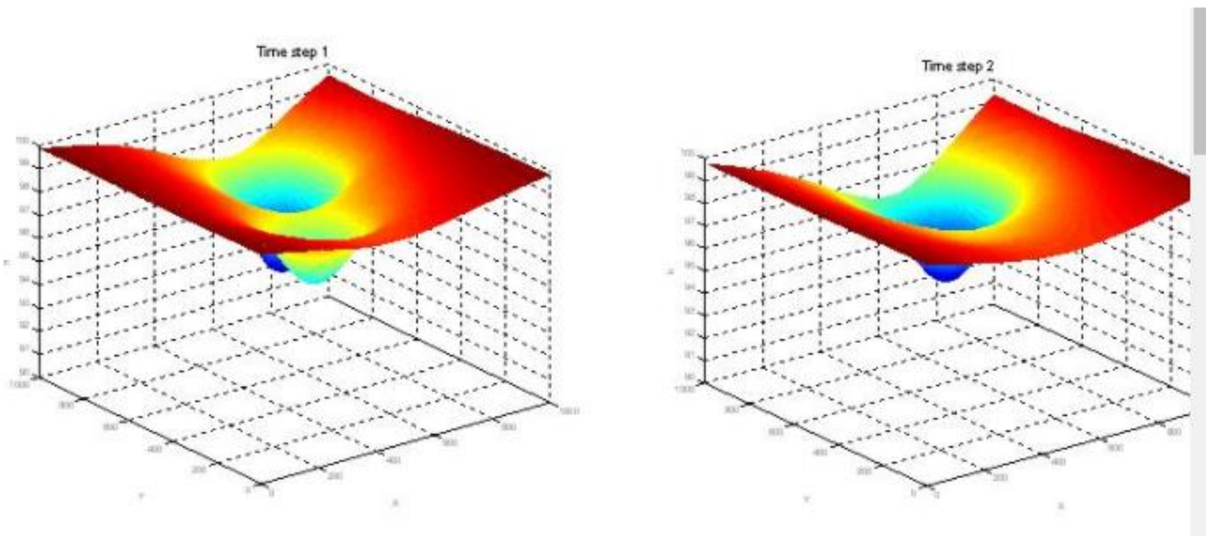
Time (hr)	Discharge (m^3/hr)	
	P_1	P_2
0-4	200	300
5-8	0	200
9-12	300	0
13-16	0	0
17-20	0	0
21-24	300	200

Table 2: Aquifer and simulation parameters

Parameters	Value
Transmissivity (T_x)	300 m^2/day
Transmissivity (T_y)	300 m^2/day
Storativity	0.001
Δx	100 m
Δy	100 m
Δt	4 h

condition is assumed on these two sides. There are two pumping wells. Pumping patterns of these two pumping wells are shown Table .1.

For the aquifer, simulation parameters shown in Table 24.2 and the solution obtained using the finite difference method is shown in Fig. 7.



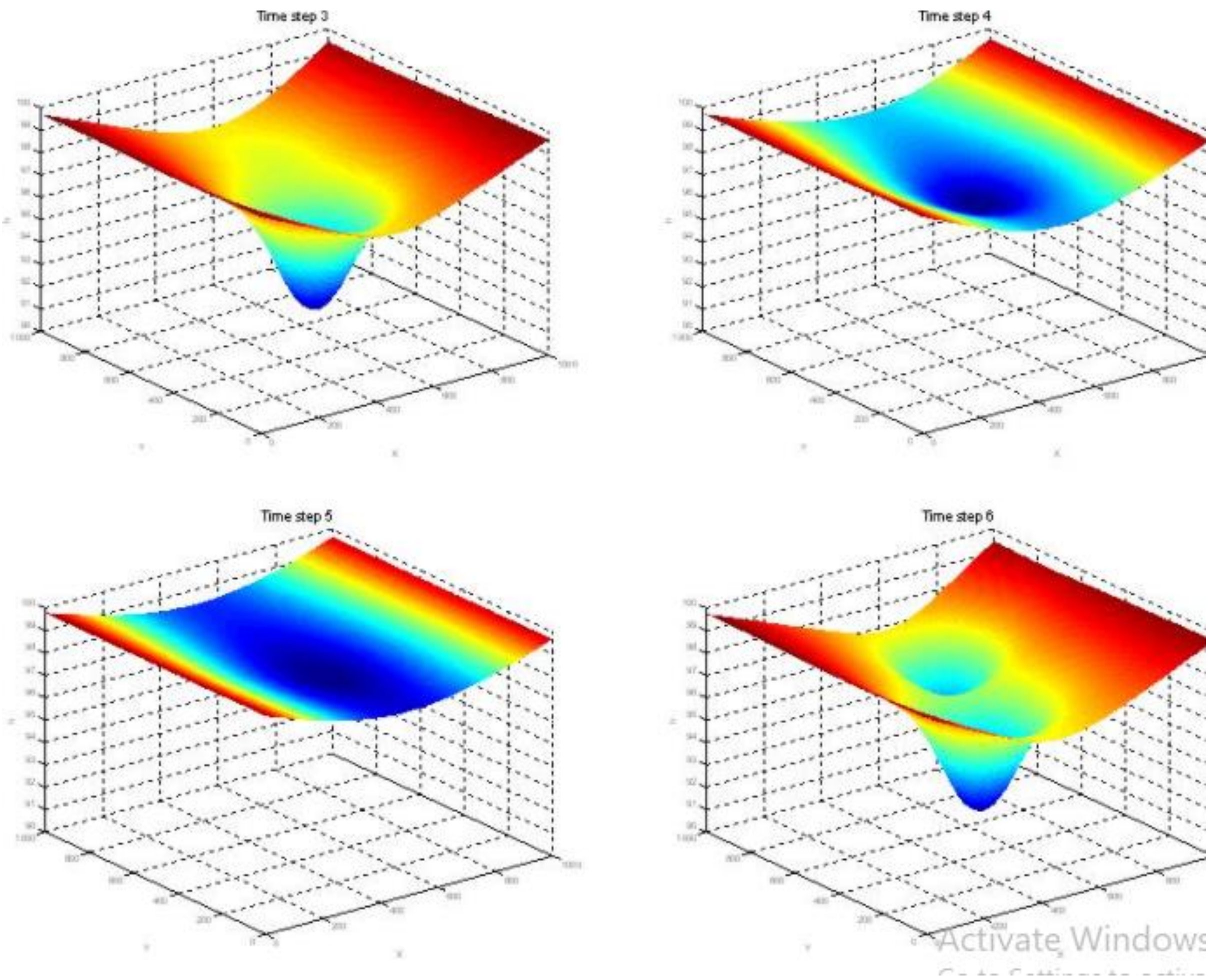


Fig. 9 Head distribution

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