GROUNDWATER, WELLS AND PUMPS

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Groundwater, Wells and Pumps

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Index

Module 1_Fundamentals of GW

Lesson 1 Introduction to Groundwater

1.1What is Groundwater?

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

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

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

1.2 Groundwater and the Water Cycle

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

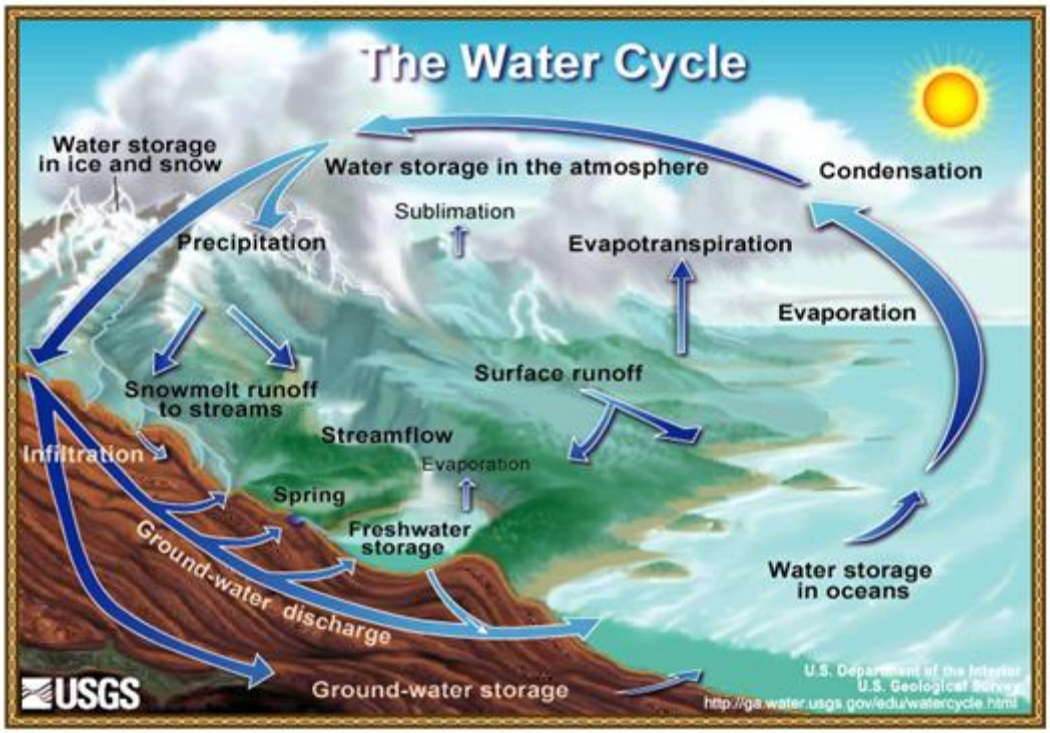


Fig. 1.1. Schematic diagram of the water cycle.

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

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

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

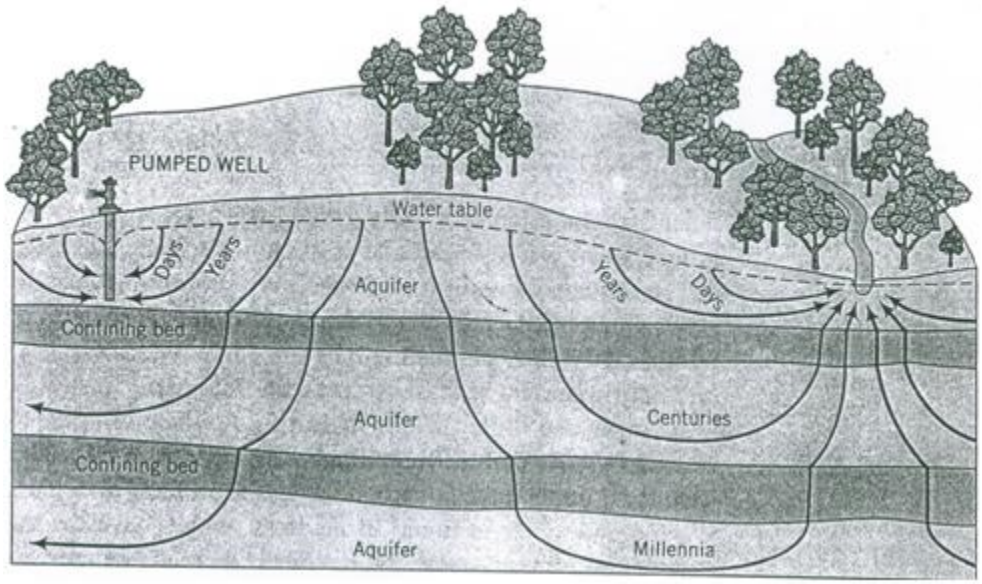


Fig. 1.2. Schematic of groundwater flow paths and residence time in multi-layered aquifer systems. (Source: Winter et al., 1998)

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

1.3 Importance of Groundwater

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

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

1.4 Groundwater Scenario: Global and Indian Perspectives

Of the 37 Mkm^3 of freshwater estimated to be present on the earth, about 22% exists as groundwater (Foster, 1998). Although groundwater is the largest available source of freshwater lying beneath the ground, its replenishment is finite and slow, and its quality can be degraded by anthropogenic activities. Historically, groundwater has been a reliable, clean and virtually unlimited water supply for much of the world population. However, with the improvement in the knowledge of hydrogeology and advances in well-drilling and pump technologies, massive groundwater withdrawal started from the 1950s in developed countries and from the 1970s in developing countries. During the past 25-30 years, more than 300 million wells have been drilled for water withdrawal in the world, and about one million wells are drilled annually in the USA alone (Zektser, 2000). Consequently, the worldwide groundwater overdraft or aquifer depletion, declining well yields, drying up of springs, streamflow depletion, and land subsidence due to over-exploitation of groundwater as well as the growing degradation of groundwater quality by natural and/or anthropogenic pollutants and by saltwater intrusion are threatening our ecosystems and even the life of our future generations (e.g., Brown, 2000; Zektser, 2000; Biswas et al., 2009). Excessive groundwater depletion currently affects major regions of North Africa, the Middle East, South and Central Asia, North China, North America, and Australia as well as localized areas throughout the world (Konikow and Kendy, 2005). The key concern is how to maintain a long-term sustainable yield from aquifers (Alley et al., 1999; Sophocleous, 2005). Global climate change and socio-economic changes are expected to complicate the use of groundwater and enhance stress on aquifer systems.

As to the groundwater scenario in India, firstly let's have a look on the rainfall characteristics of India, which has far-reaching implications for groundwater. The mean annual rainfall in India is estimated at 1,143 mm, which ranges from 11,489 mm at Mawsynram, a village in Meghalaya (wettest place on the earth) to 217 mm at Jaisalmer, a district in the Thar Desert of Rajasthan (Asawa, 1993). India is endowed with water resources only in very high rainfall regions like the eastern Gangetic plains and the Konkan-Malabar coastal strip down below the Western Ghat Mountains. Elsewhere, India's water bounty is far from plentiful (Dhawan, 1989). Such a spatial variation in the water resources is inevitable for a country of continental dimensions. What is truly striking is the temporal variation in water availability within the year as well as from one year to another!

Out of the annual precipitation of about 4000 km^3 in India, the accessible water is 1869 km^3. However, hardly 690 km^3 water is currently used, and the remaining 1179 km^3 of water directly drains into the sea –– much of it in 100 days that define the India's wet season (Aiyar, 2003). India's water problem basically stems from significant spatial and temporal variations of precipitation, mismanagement, and the fact that while nearly 70% of precipitation occurs in 100 days, the water requirement is spread over 365 days. In a number of regions, water tables have been falling at an average rate of 2 to 3 m per year due to the growing number of irrigation wells (Postel, 1993). Overuse of groundwater is reported from different parts of the country such as Tamil Nadu, Gujarat, Rajasthan, Punjab, Haryana, Orissa and West Bengal, among several other states (CGWB, 2006). A recent study based on the analysis of GRACE satellite data revealed that the groundwater resources in the states of Rajasthan, Punjab and Haryana are being depleted at a rate of 17.7 ± 4.5 km^3/year (Rodell et al., 2009). It indicated that between August 2002 to December 2008, these north-western states of India lost 109 km^3 of groundwater which is double the capacity of India's largest reservoir 'Wainganga' and almost three times the capacity of USA's largest artificial reservoir 'Lake Mead'. In addition, the growing pollution of freshwater (both surface water

and groundwater) from point and nonpoint sources and seawater intrusion into coastal aquifers of the country are posing a serious problem of human health and hygiene. Thus, increasing water scarcity and unabated water pollution threaten the sustainability of water supply and environment in India (Aiyar, 2003; Garg and Hassan, 2007). Even water is rationed in megacities such as Chennai, Bangalore, Mumbai and Delhi. Water tankers during dry periods are the burning evidence of India's severe water scarcity! Consequently, India's water security and food security are under a serious threat and the lives and livelihoods of millions are at risk.

The population of India is expected to stabilize around 1640 million by the year 2050 (UN, 1995). As a result, the gross per capita water availability will decline from about 1820 m^3/year in 2001 to as low as 1140 m^3/year in 2050. The total annual water requirement for different sectors in India was about 634 km^3 (BCM) in 2000, which will increase to 1093 km^3 (BCM) in 2025 and 1447 km^3 (BCM) in 2050 (Table 1.1). By 2050, the annual water demand in all the sectors would be more than two times the water demand in 2000. In the industry and energy sectors, the increase in water demand would be about 8 and 65 folds, respectively (Table 1.1) due to rapid growth in industrial activities and increased power demand. The water demand of 2050 is appreciably more than the current estimate of utilizable water resources potential of 1122 km^3/year (surface water = 690 km^3/year and groundwater = 432 km^3/year) through conventional development strategies (MOWR, 1999). Based on the popular Falkenmark water scarcity indicator, India is under 'water stress' conditions (freshwater availability less than 1700 m^3/person/year) today and will face 'chronic water scarcity' freshwater availability less than 1000 m^3/person/year) by 2025. Thus, water is a critical factor in determining the limits of socio-economic development of various regions and in sustaining the health of diverse ecosystems in India.

Table 1.1. Trend of annual water requirements in India (CWC, 2000)

Sl. No.	Particulars	Annual Water Requirement (km^3)		
		2000	2025	2050
1	Domestic Sector	42	73	102
2	Irrigation Sector	541	910	1072
3	Industrial Sector	8	23	63
4	Energy Sector	2	15	130
5	Other Uses	41	72	80
	Total	634	1093	1447

Recent research shows that groundwater irrigation has overtaken surface-water irrigation as the main supplier of water for India's crops. Groundwater presently sustains almost 60% of the country's irrigated area (IWMI, 2001) and the use of groundwater for irrigation has increased tremendously in the recent past. Unfortunately, well-defined policies for the sustainable use of groundwater are lacking in India. Heavy energy subsidies and even free electricity to farmers are promoting the unsustainable withdrawal of groundwater. Water conflicts, 'water lords', and water markets are gradually increasing (Jha et al., 2001). Therefore, the policy makers and water managers must rise to the challenge of finding ways to sustainably manage vital groundwater resources. It is, after all, the most 'democratic' source of water available for improving livelihoods and household food security, and reducing poverty in the country's rural areas (IWMI, 2001).

Lesson 2 Occurrence of Groundwater

2.1 Vertical Distribution of Subsurface Water

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

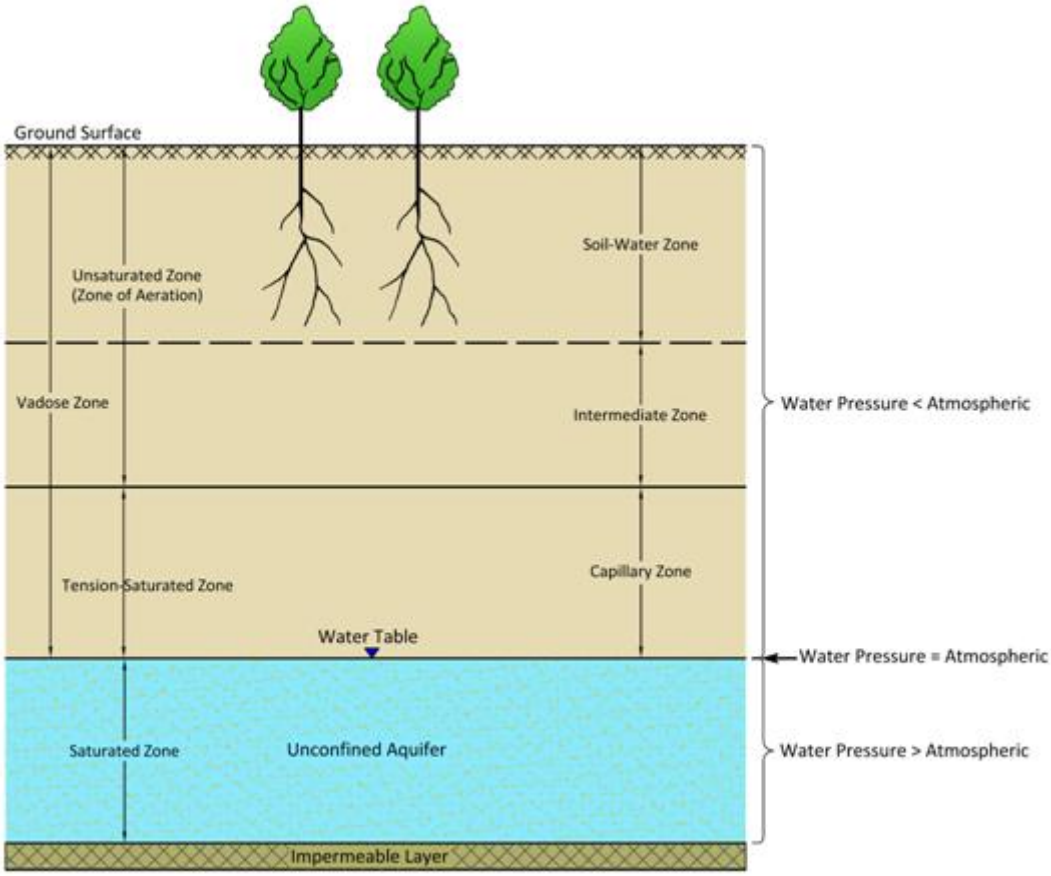


Fig. 2.1. Hydrologic zones below the ground.

(Source: Modified from Sanders, 1998)

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

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

2.2 Types of Subsurface Formations

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

2.3 Aquifer and Confining Layers

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

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

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

Aquiclude is defined as a geologic formation that can store significant amount of water but does not have the capability to transmit a significant amount of water. Clay is an ideal example of aquiclude. Aquitard is defined as a geologic formation that can store some water as well as can transmit water at a relatively low rate compared to aquifers. Although an aquitard may not yield water economically, it can hold appreciable amounts of water. Sandy clay is an ideal example of aquitard. On the other hand, aquifuge is defined as a geologic formation that can neither store nor transmit water. Solid granite is an ideal example of aquifuge. Thus, aquifuge is essentially a non-leaky confining layer, whereas aquitards and aquicludes are essentially leaky confining layers. In practice, however, aquiclude is often considered as a non-leaky confining layer because leakage through aquicludes is generally very small which can be considered practically insignificant.

2.4 Origin and Age of Groundwater

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

The water originated from magma is called magmatic water. The magmatic water available at relatively shallow depths (probably 3 to 5 km) is known as volcanic water and when it is available at deeper depths (>3 or 5 km), it is called plutonic water. Connate water is the

water present in the interstices of fossils (remains of buried animals or that have hardened into rocks) and has been out of contact with the atmosphere for considerable time of a geologic period. This water might have been derived from oceanic or freshwater sources and is highly mineralized. Juvenile water is the water that has not earlier been a part of the hydrosphere. It is derived from magma or atmosphere. Finally, metamorphic water is the water associated with rocks during metamorphism process.

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

2.5 Groundwater Regions of India

About two thirds of the total land area in India comprises consolidated formations, of which 75% is made up of crystalline rocks and consolidated sediments and the remaining 25% is trap (Raghunath, 2007). The remaining one third of the total land area comprises semi-consolidated and unconsolidated formations like alluvial tracts. Potential areas of groundwater in India are illustrated in Fig. 2.1 and their brief description is given below (Raghunath, 2007).

(1) Himalayan Highlands

All types of rocks are present in this region. Major rock types present in the Himalayan Highlands are granites, basalts, sandstones, limestones, shales, conglomerates, slates, quartzites, gneisses, schists and marbles. Favourable conditions exist with springs forming a major part of water supply in this region.

(2) Kashmir Valley

The Kashmir Valley which was a vast lake during the Pleistocene times shows a large scale development of freshwater sediments of lacustrine, fluvial and glacial origin.

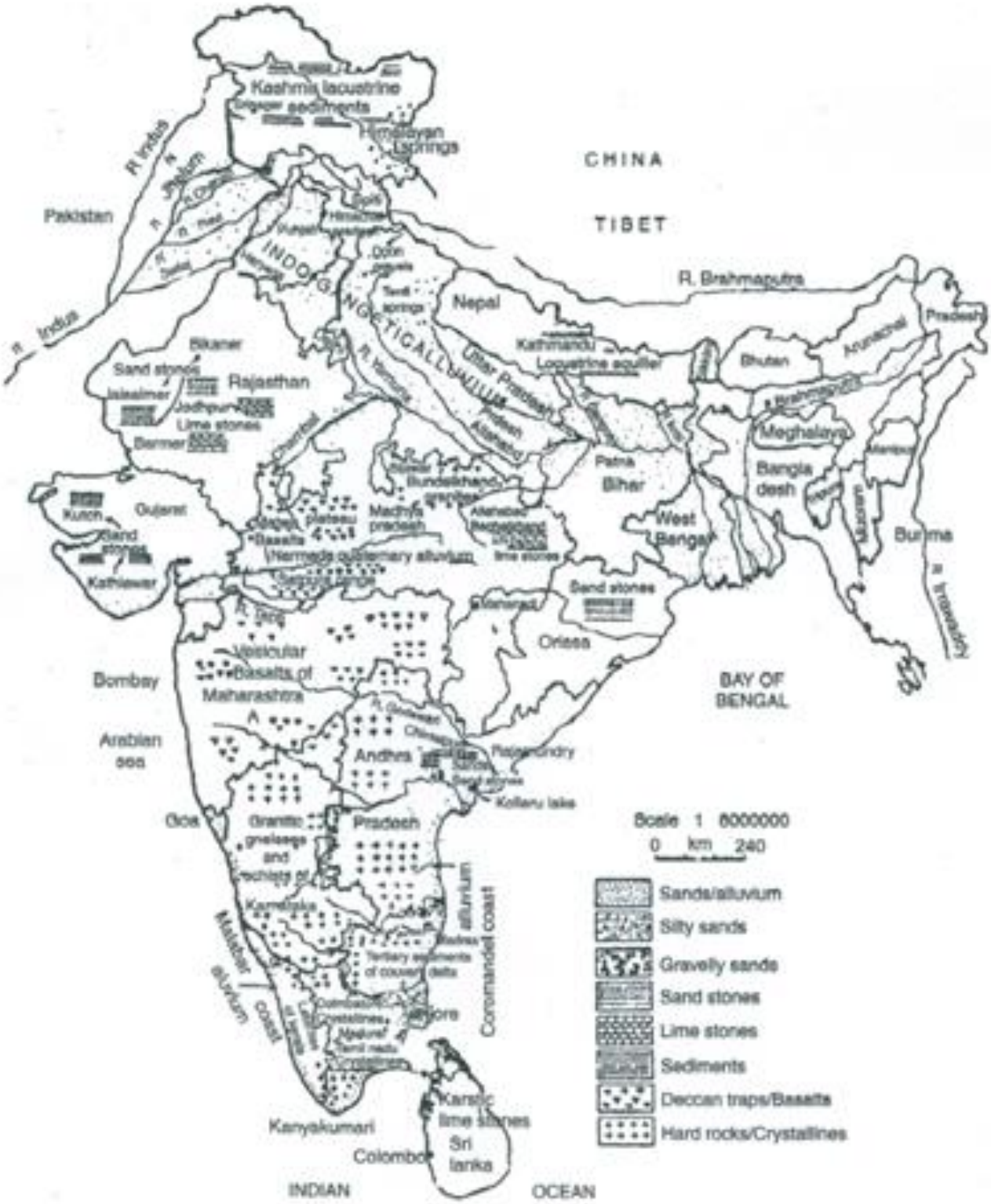


Fig. 2.1. Groundwater regions of India. (Source: Raghunath, 2007)

(3) Indo-Gangetic Alluvium

This region consists of coarse sands, gravel and boulders of variable thickness (3 to 60 m), and constitutes a vast reservoir of fresh groundwater. The exploitation of groundwater is usually done by using augers, hand boring, cable tool and rotary rigs.

(4) Coastal Alluvium (Malabar and Coromandel Coastal Areas)

The depth of aquifers varies from 15 to 150 m. Because of increasing groundwater exploitation in this region, extensive saline patches occur in Ramnad, Tirunelveli, Ongole, Nellore, and Krishna districts. In Ramanathapuram and Tirunelveli, the groundwater available in the unconfined aquifers is usually of poor quality with a chloride (Cl) concentration of >1000 ppm (mg/L) and at some places, the Cl concentration is even >3000 ppm (mg/L). However, in the west coast areas of Kerala and Karnataka, the substratum is mostly lateritic and a good yield of groundwater is expected.

Saline water influx in response to tides has been observed at places in Goa, up to a distance of 25-40 km inland. In the upper reaches, the tidal streams show cyclic fluctuations in water salinity, with the highest salinity during high tides and the lowest salinity during low tides.

(5) Cretaceous Sandstones of Kathiawar and Kutch Areas

This region has moderately potential aquifers with depths ranging from 100 to 300 m, and the yield approximately10-120 m^3/h. Groundwater is normally brackish with a TDS of 2000-5000 ppm (mg/L). In Gujarat, sandstone aquifers exist at depths of 60 to 200 m, with a yield of about 10-50 m^3/h and a TDS of 1000-2500 ppm.

(6) Mesozoic Sandstones of Lathi Region in Rajasthan (Jaisalmer, Barmer and Bikaner)

It contains moderately potential aquifers with depths ranging from 100 to 150 m, and a yield of about 45-150 m^3/h. Groundwater is generally brackish to saline, with TDS = 1000-5000 ppm, Cl = 1000-5000 ppm, EC (electrical conductivity) >3000 µmhos/cm, Na Percentage >80, and SAR (Sodium Adsorption Ratio) = 25-55.

(7) Cavernous Limestones of Vindhyan System in Borunda and Ransingaon Areas in Jodhpur District

Potential aquifers are fractured up to 150 m, and have a yield of 40-70 m^3/h. Groundwater has TDS < 2000 ppm and is potable.

(8) Doon Valley Gravels

The Doon Valley region has boulders, pebbles, gravel, sand and clay possibly of fanglomeratic and colluvial origin. Major portion of the valley is hilly having sloping ground. Hence, only the central part (approximately 388 km^2 which is about one-fifth of the total area of 2090 km^2) can be developed for groundwater exploitation. Thickness of the fill is 150-200 m and the well yield varies from 30 to 150 m^3/h. The quality of groundwater is bicarbonate to sulphate type, with TDS = 100-500 ppm, Cl >30 ppm and pH = 7.8. In the Terai zone, groundwater is available under artesian conditions at shallow depths of 3 to 50 m. Sand and gravel layers confined between silty and clayey layers constitute good confined aquifers.

(9) Quaternary Alluvium of Narmada, Purna, Tapti, Chambal and Mahanadi Rivers

This region contains lenses of sand and gravel having a thickness of 75 to 150 m. Tubewell yield varies from 20 to 150 m^3/h. Groundwater quality is good with a TDS of 100-500 ppm.

(10) Vesicular Basalts in the Deccan Trap Formations of Maharashtra and Madhya Pradesh

This region contains good aquifers, with groundwater occurring under both confined and unconfined conditions in the Satpura range and Malwa plateau. Tubewell yields in Indore, Bhopal, Raisen, Vidisha and Sagar districts range between 10 and 40 m^3/h (approximately). In central Maharashtra, the yield of the tubewells drilled in weathered basalts is about 2-10 m^3/h, while in exceptional cases the well yields are ≈ 25 m^3/h, mostly within depths of 50 to 100 m. Borewells mainly serve as a source of drinking water supply because of their low yield, and only in exceptional cases they are used for irrigation. The quality of groundwater is good (TDS < 1000 ppm) for all uses.

(11) Carbonate Rocks with Solution Cavities in Madhya Pradesh

In the Vindhyan, Cuddapah and Bijawar region, the carbonate rocks with interconnected solution cavities and caverns form good aquifers. The limestones of Raipur, Charmuria, Kajrahat (Sidhi district), Karstic areas of Chhatisgarh basin and Baghelkhand region of Madhya Pradesh yield water about 10-60 m^3/h.

(12) Dharwarian and Bundelkhand Granite Region of Madhya Pradesh

This region comprises igneous and metamorphic rocks, wherein water moves mainly through joints and openings (secondary porosity) present in these rocks. The yields of the tubewells in Tikamgarh, Chattarpur, Balaghat and Gwalior area are approximately 10-30 m^3/h; they tap mostly unconfined aquifers. The quality of groundwater in all the regions of Madhya Pradesh is normally good.

(13) Tertiary Sandstones and Quaternary Sand to Pebble Beds in the Godavari-Krishna Interstream Area

This region contains potential aquifers under artesian/confined conditions. Aquifer thickness varies from 3 to 184 m and the tubewell yields range from 20 to 120 m^3/h (approximately). The quality of groundwater in the sandstones is fresh, but that in the alluvial zone is highly saline in the vicinity of Kolleru Lake, along the coast and at depths; TDS varies from 1800 to 15000 ppm, and the Cl concentration varies from 600 to 8000 ppm, thereby making the groundwater unsuitable for any purpose.

(14) Alluvium in Palar and Kortallaiyar-Araniyar Rivers in Tamil Nadu

The alluviums of this region form potential aquifers. Groundwater is of good quality within 50 m depth, with Cl < 250 ppm and EC 750-2000 µmhos/cm.

(15) Tertiary Sediments of Cauvery Delta

In the Cauvery Delta, rocks ranging in age from Precambrian Crystallines to Quaternary Sediments are encountered. Multiple aquifer systems are quite prevalent in a sufficiently thick sedimentary basin. The deeper aquifers are generally under confined conditions and there exists hydraulic connection (vertical leakage) between overlying/underlying aquifers.

The tertiary sediments in Tanjore and South Arcot districts form extensive aquifers up to 200 m depth. Groundwater is of good quality, with Cl < 150 ppm and EC < 1500 µmhos/cm. Many of the tubewells have free flow, some of them exceeding 2 m^3/h. Although artesian wells are quite prevalent in this region, large-scale groundwater development is gradually

lowering the piezometric head and free-flow condition is progressively ceasing (i.e., flowing wells are becoming non-flowing wells).

(16) Granitic Gneisses and Schists of Karnataka

The main rock types of Karnataka are igneous and metamorphic granites, gneisses and schists of Precambrian age and basalt of the Deccan trap of Eocene-Upper Cretaceous age in the extreme northern part of the state. The well yield is very low; the borewells drilled up to depths of 30-75 m yield 5.40 m^3/h.

The well yield in the crystalline rocks depends on the presence of weathered zones, joints and fractures, of which there may be no indication at the surface. The yield of a well may be drastically different from that of another well located a few meters away. Surface resistivity survey can reveal approximate depth and extent of weathered or fractured zones which can avoid the risk of failure.

(17) Upper Gondwana Sandstones and Alluvial Tract of Orissa

Upper Gondwana sandstones and the alluvial tract of Orissa constitute potential aquifers. The average annual rainfall of the region is 142 cm and about 20% of this rainfall (about 28 cm) can be assumed as recharge to the aquifer.

In the alluvial tract where the granular aquifer material occurs within 8-10 m below the ground and also in the semi-consolidated sedimentary sandstones weathered within 5 m below the ground, open wells fitted with 2-4 kW centrifugal pumps can be installed for irrigation purposes, with a minimum spacing of 150-200 m in alluvial tracts. It is estimated that 65% of the groundwater potential of the state can be developed by the installation of open wells. Already thousands of tubewells and lakhs of open wells have been installed in this state to exploit groundwater for irrigation, drinking and industrial purposes.

(18) Quaternary Sediments in the Deltaic Tract around Digha, West Bengal

Quaternary sediments in the deltaic tract around Digha are of depth 140 m and usually yield fresh groundwater.

Finally, thermal and mineral springs are also found in some parts of India such as Maharashtra, Punjab, Bihar, Assam, in the foothills of Himalayas and Kashmir. Further details about the groundwater regions of India can be found at the website of Central Ground Water Board (CGWB), New Delhi.

Lesson 3 Aquifer and Its Properties

3.1 Introduction

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

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

3.2 Types of Aquifers

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

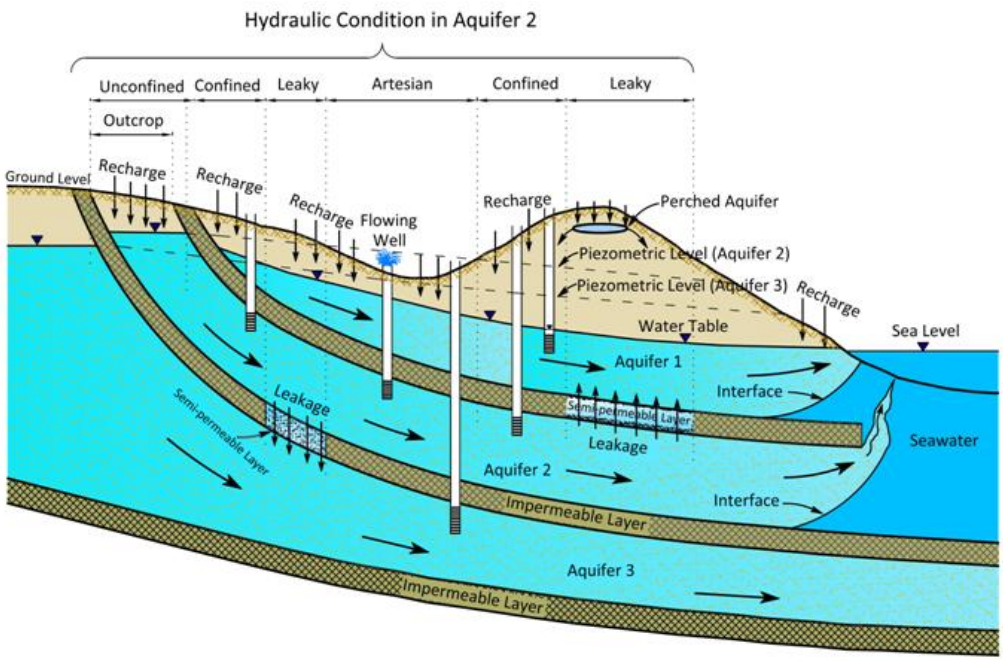


Fig. 3.1. Types of aquifers for groundwater development. (Source: Bear, 1979)

3.2.1 Unconfined Aquifers

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

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

3.2.2 Confined Aquifers

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

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

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

3.2.3 Leaky Aquifers

If an aquifer (confined aquifer or unconfined aquifer) loses or gains water through adjacent semi-permeable layers, it is called a 'leaky aquifer' (Fig. 3.1). Therefore, the terms 'leaky confined aquifer' and 'leaky unconfined aquifer' are widely used depending on whether the

leaky aquifer is confined or unconfined. However, the case of 'leaky confined aquifer' has been mostly dealt with by the groundwater experts. This is why, the term 'semi-confined aquifer' is sometimes used to denote a 'leaky aquifer'. The term 'nonleaky' is also used to describe the status of a confined or unconfined aquifer, such as 'nonleaky unconfined aquifers' and 'nonleaky confined aquifers'. In reality, ideal confined aquifers or ideal unconfined aquifers occur less frequently than do leaky aquifers.

3.2.4 Perched Aquifers

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

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

3.3 Properties of Aquifers

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

3.3.1 Porosity

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

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

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

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

3.3.2 Effective Porosity

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

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

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

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

3.3.3 Specific Retention

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

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

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

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

3.3.4 Specific Yield

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

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

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

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

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

Or,

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

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

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

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

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

11	Limestone	30	14
12	Dune Sand	45	38
13	Loess	49	18
14	Peat	92	44
15	Schist	38	26
16	Siltstone	35	12
17	Till (mainly Sand)	31	16
18	Till (mainly Silt)	34	6
19	Tuff	41	21

3.3.5 Storage Coefficient and Specific Storage

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

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

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

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

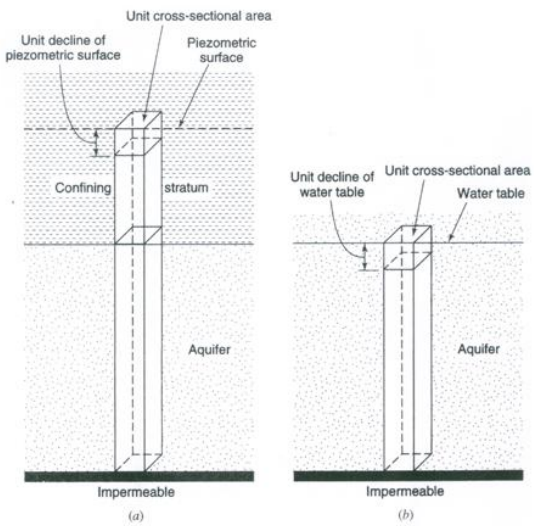


Fig. 3.2. Schematic representation of storage coefficient for: (a) Confined aquifers; (b) Unconfined aquifers. (Source: Todd, 1980)

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

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

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

The value of K_w (bulk modulus of elasticity of water) is 2.1×10^9 N/m^2, whereas the values of E_s (bulk modulus of elasticity of aquifer skeleton) for some geological materials are given in Table 3.2 (Raghunath, 2007).

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

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

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

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

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

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

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

3.3.6 Intrinsic Permeability

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

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

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

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

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

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

3.4 Water Yielding Mechanisms of Aquifers

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

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

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

3.5 Example Problems

3.5.1 Example Problem 1

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

Solution:

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

Substituting the values, we have,

$$1.5\times10^6 = 4\times10^6\times(27.5 - 25.5)\times S_y = 4\times10^6 \times 2.0\times S_y$$

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

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

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

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

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

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

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

3.5.2 Example Problem 2

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

Solution:

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

$$\Rightarrow\ 0.30 = S_y + 0.15$$

$$\therefore\ S_y = 0.30 - 0.15 = 0.15 \text{ or } 15\%,\ \textbf{Ans.}$$

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

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

= 105X10^4 m^3, Ans.

3.5.3 Example Problem 3

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

Solution:

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

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

$= (500X10^6)´(22-10)X0.0006$

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

Lesson 4 Principles of Groundwater Flow

4.1 Concept of Fluid Potential and Hydraulic Head

4.1.1 What is Fluid Potential?

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

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

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

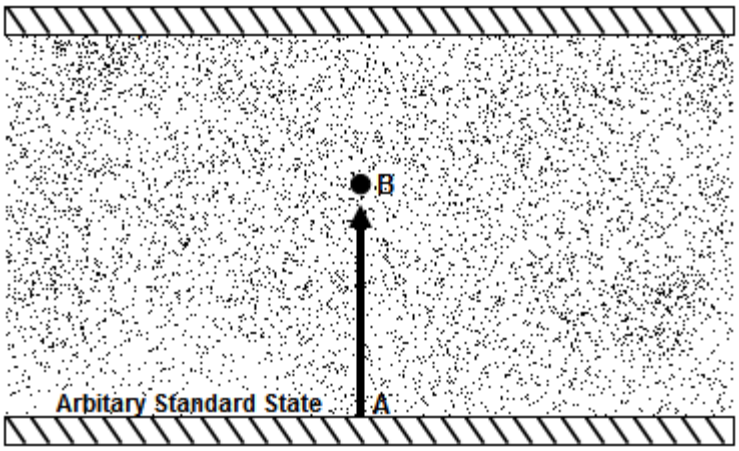


Fig. 1. Example of a subsurface flow system.

At Point A, we have:

Elevation (z) = 0,

Pressure (P) = P_o (atmospheric pressure),

Velocity (v) = 0,

Similarly, at Point B, we have:

Elevation = z,

Pressure = P,

Velocity = v,

Density (r) = r_o, and

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

Density = r, and

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

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

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

$$W_1 = mgz \qquad (4.1)$$

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

$$W_2 = \tfrac{1}{2} mv^2 \qquad (4.2)$$

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

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

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

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

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

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

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

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

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

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

4.1.2 Relation between Fluid Potential and Hydraulic Head

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

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

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

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

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

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

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

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

Or,

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

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

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

4.2 Darcy's Law

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

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

In Eqn. (4.9), the constant of proportionally (K) is known as the hydraulic conductivity of the porous medium. Sometimes K is also called saturated hydraulic conductivity of the porous medium because of the fact that the Darcy's law is strictly valid for flow through a saturated porous medium. Negative sign in Eqn. (4.9) indicates that the flow occurs in the

direction of decreasing head. The term $\frac{dh}{dl}$ is the head loss per unit length of flow and is called hydraulic gradient (i). Equation (4.9) can also be written as follows:

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

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

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

4.3 Validity of Darcy's Law

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

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

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

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

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

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

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

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

Solution: Reynolds number (R_e) is given as:

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

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

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

=0.2791 , which is less than 1.

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

4.4 Seepage Velocity

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

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

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

Equation (4.12) can also be written as:

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

Or,

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

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

4.5 Hydraulic Conductivity and Transmissivity

4.5.1 Hydraulic Conductivity

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

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

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

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

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

$$K = \frac{k \rho g}{\mu} \qquad (4.15a)$$

Or,

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

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

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

4.5.2 Transmissivity

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

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

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

According to the Darcy's law, we have:

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

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

$$\therefore \quad Q = T \times w \times i \qquad (4.17)$$

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

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

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

4.6 Leakage Factor

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

$$B = \sqrt{Tb'/K'} \qquad (4.18a)$$

Or,

$$B = \sqrt{TC} \qquad (4.18b)$$

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

The leakage factor (B) has the dimension of length, i.e., [L]. Moreover, Leakance or Leakage Coefficient is defined as $\frac{1}{C} = \frac{K'}{b'} = \frac{T}{B^2}$. The numerical values of leakance are usually given

as a measure of leakage rates (i.e., ability of the aquitard to transmit vertical flow through it). Leakance has the dimension of time inverse (e.g., day^{-1}).

Lesson 5 Governing Equations of Groundwater Flow

5.1 Introduction

The most important transmission property of geologic formations, hydraulic conductivity (*K*) usually exhibits significant variations through space within a geologic formation. It may also vary with the direction of measurement at a given location/point in a geologic formation. The first property is known as heterogeneity, while the second property is known as anisotropy. The geologic processes that produce various geological environments/settings are responsible for the prevalence of these two properties in geologic formations, including aquifers. A brief description about these two properties is given below prior to the derivation of groundwater flow equations.

5.1.1 Homogeneity and Heterogeneity

If the hydraulic conductivity (K) is independent of position (location) within a geologic formation, the formation is called homogenous. However, if K is dependent on the position within a geologic formation, the formation is called heterogeneous. If we set up a xyz coordinate system in a homogenous formation, then in a homogeneous formation, K(x, y, z) = C, C being a constant; whereas in a heterogeneous formation, K(x, y, z) ¹ C.

Many hydrogeologists and petroleum geologists have used statistical distributions to provide a quantitative description of the degree of heterogeneity in a geological formation. Presently, sufficient direct evidences exist to support the statement that the probability density function (pdf) for hydraulic conductivity values is log-normal. Therefore, for computing the average hydraulic conductivity of a heterogeneous aquifer system, the 'geometric mean' should be used instead of commonly used 'arithmetic mean'.

5.1.2 Isotropy and Anisotropy

If the hydraulic conductivity (K) is independent of the direction of measurement at a particular location in a geologic formation, the formation is said to be isotropic at the location. If the hydraulic conductivity K varies with the direction of measurement at a particular location in a geologic formation, the formation is said to be anisotropic at the location.

Let's consider a two-dimensional vertical section through an anisotropic geologic formation (e.g., 'anisotropic aquifer'). If q is the angle between the horizontal and the direction of measurement of a K-value at some point in the formation, then K = K(q). The directions in space corresponding to the q angle at which K attains its maximum and minimum values are known as the principal directions of anisotropy. They are always perpendicular to one another. In three dimensions, if a plane is taken perpendicular to one of the principal directions, the other two principal directions are the directions of maximum and minimum K in that plane.

If an coordinate system is set up in such a way that the coordinate directions coincide with the principal directions of anisotropy, the hydraulic conductivity values in the principal directions can be specified as K_x, K_y and K_z. At any point/location (x, y, z), an isotropic

formation will have $K_x = K_y = K_z$, whereas an anisotropic formation will have K_x ¹ K_y ¹ K_z. If $K_x = K_y$ ¹ K_z, as is common in horizontally bedded sedimentary formations, the formation is said to be transversely isotropic.

To fully describe the nature of the hydraulic conductivity in a geologic formation, it is necessary to use two adjectives, one dealing with heterogeneity and one with anisotropy. For example, for a homogenous and isotropic system in two dimensions: K_x (x, z) = K_z (x, z) = C for all (x, z), where C is a constant. For a homogenous and anisotropic system, K_x (x, z) = C_1 for all (x, z) and K_z (x, z) = C_2 for all (x, z) but C_1 ¹ C_2. Thus, considering heterogeneity and anisotropy, four cases can be defined for an aquifer system (Freeze and Cherry, 1979): (i) homogeneous and isotropic aquifer, (ii) homogeneous and anisotropic aquifer, (iii) heterogeneous and isotropic aquifer, and (iv) heterogeneous and isotropic aquifer.

The primary cause of anisotropy on a small scale is the orientation of clay minerals in sedimentary rocks and unconsolidated sediments. Core samples of clays and shales seldom show horizontal to vertical anisotropy (i.e., ratio of K_x to K_z) greater than 10:1; it is generally less than 3:1 (Freeze and Cherry, 1979). However, on a larger scale, the ratios of K_x to K_z usually range from 2 to 10 for alluvial formations, but the values up to 100 or more can occur where clay layers are present (Todd, 1980).

5.2 General Form of Groundwater Flow Equation

The flow of fluids through porous media is governed by the laws of physics and as such it can be described by differential equations. Since the flow through porous media is a function of several variables, it is usually described by partial differential equations in which spatial coordinates x, y and z, and time t are independent variables. For deriving groundwater flow equations, the law of conservation of mass and the law of conservation of energy are employed. The law of mass conservation is well known as the continuity principle. Thus, the governing equations of groundwater flow are derived using the continuity principle (i.e., equation of mass balance) and the Darcy law, and the derivation is presented below.

For the derivation of differential equations for groundwater flow, let us consider a small part of the aquifer called control volume having three sides of lengths Δx, Δy and Δz, respectively (Fig. 5.1). The area of the faces normal to the X- axis is $\Delta y\ \Delta z$ and the area of the faces normal to the Z-axis is $\Delta x\ \Delta y$.Considering a general case, assume that the aquifer is heterogeneous and anisotropic.

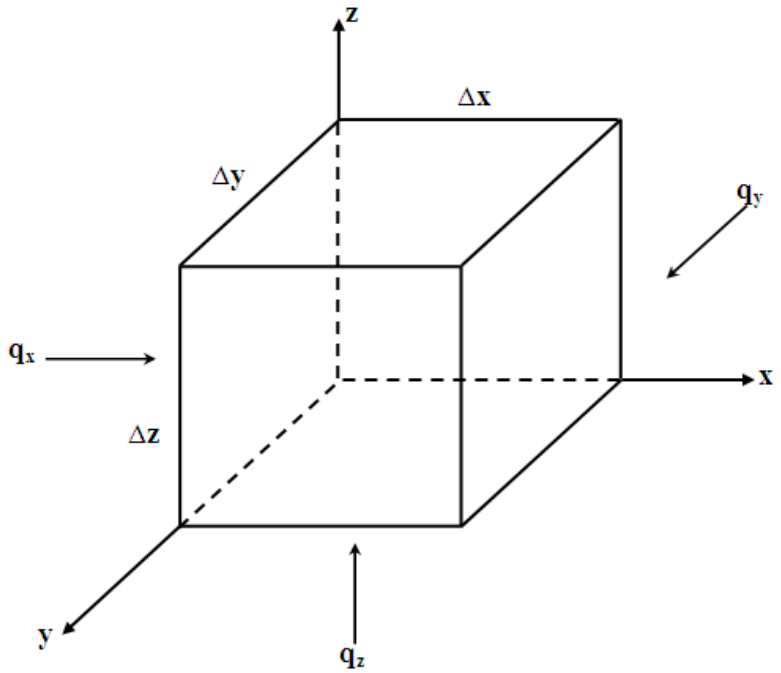


Fig. 5.1. Control volume for flow through a confined aquifer.

As we know that the actual fluid motion can be subdivided on the basis of the components of flow parallel to the three principal axes (i.e., X, Y and Z). If q is flow per unit cross-sectional area, $\rho_w q_x$ is the portion of mass flow (flux) parallel to the X-axis, and so on, where ρ_w is the fluid density (density of water in the present case).

The mass flux into the control volume is: $\rho_w q_x \Delta y \Delta z$ along the X-axis. The mass flux out of the control volume is: $\rho_w q_x \Delta y \Delta z + \frac{\partial}{\partial x}(\rho_w q_x)\Delta x \Delta y \Delta z$. Since there are flow components along all the three axes, similar terms can be determined for the other two directions. The continuity equation is given as:

Mass inflow rate – Mass outflow rate = Change of mass storage with time (5.1)

For the control volume, the term on the left hand side of Eqn. (5.1) represents net mass flux (inflow rate –outflow rate) into the control volume. Thus, the net mass flux of water in X-, Y- and Z-directions are given as:

Net flux in X-direction = $-\frac{\partial}{\partial x}(\rho_w q_x)\Delta x \Delta y \Delta z$ (5.2)

Net flux in Y-direction = $-\frac{\partial}{\partial y}(\rho_w q_y)\Delta x\,\Delta y\,\Delta z$ (5.3)

Net flux in Z-direction = $-\frac{\partial}{\partial z}(\rho_w q_z)\Delta x\,\Delta y\,\Delta z$ (5.4)

Combining Eqns. (5.2), (5.3) and (5.4) yields the sum of water inflow rate minus the sum of water outflow rate for the control volume as:

$$-\left[\frac{\partial}{\partial x}(\rho_w q_x)+\frac{\partial}{\partial y}(\rho_w q_y)+\frac{\partial}{\partial z}(\rho_w q_z)\right]\Delta x\Delta y\Delta z \qquad (5.5)$$

Now, the change in groundwater storage within the control volume is the change of water storage per unit time, which is expressed as:

$$\frac{\partial}{\partial t}(\rho_w n\,\Delta x\,\Delta y\,\Delta z) \qquad (5.6)$$

Where, n is the porosity of the aquifer.

According to Eqn. (5.1), the net rate of water inflow is equal to the change in storage. Therefore, equating Eqns. (5.5) and (5.6) and dividing both sides by $\Delta x\,\Delta y\,\Delta z$ gives:

$$-\frac{\partial}{\partial x}(\rho_w q_x)-\frac{\partial}{\partial y}(\rho_w q_y)-\frac{\partial}{\partial z}(\rho_w q_z)=\frac{\partial}{\partial t}(\rho_w n)$$

After differentiating the term on the right hand side, the above equation can be rewritten as follows:

$$-\frac{\partial}{\partial x}(\rho_w q_x) - \frac{\partial}{\partial y}(\rho_w q_y) - \frac{\partial}{\partial z}(\rho_w q_z) = n\frac{\partial \rho_w}{\partial t} + \rho_w \frac{\partial n}{\partial t} \qquad (5.7)$$

First term on the right hand side of Eqn. (5.7) denotes the mass rate of water produced by the expansion of water due to a change in density, which is controlled by the compressibility of water (β). Second term on the right hand side of Eqn. (5.7) denotes the mass rate of water produced by the compaction of the porous medium (aquifer) as reflected by the change in porosity (n), which is controlled by the compressibility of the aquifer material (α).

We know that the change in ρ_w and change in n are both produced by a change in hydraulic head (h) and that the volume of water produced by the two mechanisms for a unit decline in head is called specific storage (S_s) which is expressed as, $S_s = \rho_w g(\alpha + n\beta)$. As the mass rate of water produced (time rate of change of fluid mass storage) is $\rho_w S_s \frac{\partial h}{\partial t}$, Eqn. (5.7) can be written as:

$$-\frac{\partial}{\partial x}(\rho_w q_x) - \frac{\partial}{\partial y}(\rho_w q_y) - \frac{\partial}{\partial z}(\rho_w q_z) = \rho_w S_s \frac{\partial h}{\partial t} \qquad (5.8)$$

Expanding the left hand side by chain rule and recognizing the fact that the terms of the form are $\rho_w \frac{\partial q_x}{\partial x}$ much greater than the terms of the form $q_x \frac{\partial \rho_w}{\partial x}$, allows us to neglect the second term, i.e. $q_x \frac{\partial \rho_w}{\partial x}$, . Then, Eqn. (5.8) becomes:

$$-\rho_w \frac{\partial q_x}{\partial x} - \rho_w \frac{\partial q_y}{\partial y} - \rho_w \frac{\partial q_z}{\partial z} = \rho_w S_s \frac{\partial h}{\partial t} \qquad (5.9a)$$

Since r_w is common on both sides of Eqn. (5.9a), it can be eliminated which reduces Eqn. (5.9a) to:

$$-\frac{\partial q_x}{\partial x}-\frac{\partial q_y}{\partial y}-\frac{\partial q_z}{\partial z}=S_s\frac{\partial h}{\partial t} \qquad (5.9b)$$

From the Darcy's law, we have:

$$q_x=-K_x\frac{\partial h}{\partial x} \qquad (5.10)$$

$$q_y=-K_y\frac{\partial h}{\partial y} \qquad (5.11)$$

$$q_z=-K_z\frac{\partial h}{\partial z} \qquad (5.12)$$

Substituting Eqns. (5.10), (5.11) and (5.12) into Eqn. (5.9b) yields the main equation of groundwater flow:

$$\frac{\partial}{\partial x}\left(K_x\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_z\frac{\partial h}{\partial z}\right)=S_s\frac{\partial h}{\partial t} \qquad (5.13)$$

Where, K_x, K_y and K_z = aquifer hydraulic conductivities in the X-, Y- and Z- directions, respectively; S_s = specific storage of the aquifer; h = hydraulic head in the aquifer; and t = time.

Equation (5.13) is a general equation for transient three-dimensional flow through a heterogeneous and anisotropic saturated porous medium. In this equation, spatial coordinates x, y and z, and time t are independent variables and hydraulic head (h) is a dependent variable. The parameters of this equation are K_x, K_y, K_z and S_s. Note that Eqn. (5.13) is second order in x, y and z while first order in t. The physical meaning of the terms on the left hand side of Eqn. (5.13) is 'net outflow rate of groundwater per unit volume of the

aquifer', whereas the term on the right hand side represents 'time rate of change of groundwater volume within the unit volume of the aquifer'.

5.3 Equations for Groundwater Flow in Confined Aquifers

Since Eqn. (5.13) is a general equation of groundwater flow, it can be written in many forms that apply to a variety of different flow conditions. Some of these alternative equations are presented in subsequent sections for two extreme cases: (i) the most complex case, i.e., heterogeneous and anisotropic aquifer systems, and (ii) the simplest case, i.e., homogeneous and isotropic aquifer systems.

5.3.1 Heterogeneous and Anisotropic Confined Aquifer System

Although Eqn. (5.13) is called a general equation for groundwater flow, it essentially represents three-dimensional (3-D) transient flow through heterogeneous and anisotropic confined aquifers.

Multiplying both sides of Eqn. (5.13) by the aquifer thickness (b), we get another form of groundwater flow equation for heterogeneous and anisotropic confined aquifers:

$$\frac{\partial}{\partial x}\left(K_x b\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y b\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_z b\frac{\partial h}{\partial z}\right)=S_s b\frac{\partial h}{\partial t}$$

$$\Rightarrow \frac{\partial}{\partial x}\left(T_x\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(T_y\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(T_z\frac{\partial h}{\partial z}\right)=S\frac{\partial h}{\partial t} \qquad (5.14)$$

Where, T_x, T_y and T_z are aquifer transmissivities in X-, Y- and Z-directions, respectively; S is the storage coefficient of the aquifer; and all other variables have the same meaning defined earlier. Moreover, under steady-state flow conditions in a heterogeneous and anisotropic confined aquifer system, there is no change in aquifer storage with time, i.e. $\frac{\partial h}{\partial t}=0$, . Therefore, Eqn. (5.13) simplifies to:

$$\frac{\partial}{\partial x}\left(K_x\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_z\frac{\partial h}{\partial z}\right)=0 \qquad (5.15)$$

Moreover, for two-dimensional (2-D) transient groundwater flow in heterogeneous and anisotropic confined aquifer systems, Eqn. (5.13) can be written as:

$$\frac{\partial}{\partial x}\left(K_x\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y\frac{\partial h}{\partial y}\right)=S_s\frac{\partial h}{\partial t} \qquad (5.16)$$

and Eqn. (5.14) can be written as:

$$\frac{\partial}{\partial x}\left(T_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(T_y \frac{\partial h}{\partial y}\right) = S\frac{\partial h}{\partial t} \qquad (5.17)$$

5.3.2 Homogeneous and Isotropic Confined Aquifer System

For homogeneous and isotropic confined aquifer systems, the hydraulic conductivity (K) will not vary with space and $K_x = K_y = K_z = K$. Therefore, Eqn. (5.13) simplifies to:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \frac{S_s}{K}\frac{\partial h}{\partial t} \qquad (5.18)$$

Or,

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \frac{S}{T}\frac{\partial h}{\partial t} \qquad (5.19)$$

Eqns. (5.18) and (5.19) represent transient groundwater flow through homogeneous and isotropic confined aquifer systems. As K/S_s or T/S is called 'hydraulic diffusivity' of the aquifer system, Eqns. (5.18) and (5.19) are known as three-dimensional (3-D) diffusion equations.

For two-dimensional (2-D) groundwater flow through homogeneous and isotropic confined aquifer systems, Eqn. (5.18) simplifies to:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = \frac{S_s}{K}\frac{\partial h}{\partial t} \qquad (5.20)$$

and Eqn. (5.19) simplifies to:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = \frac{S}{T}\frac{\partial h}{\partial t} \qquad (5.21)$$

Eqns. (5.20) and (5.21) are known as two-dimensional diffusion equations.

Under steady-state flow conditions in a homogenous and isotropic confined aquifer system, there is no change in storage with time, i.e., $\frac{\partial h}{\partial t} = 0$. Therefore, Eqns. (5.18) and (5.19) can be written as:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0 \qquad (5.22)$$

Eqn. (5.22) is known as the well-known three-dimensional Laplace equation. The two-dimensional Laplace equation is given as:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0 \qquad (5.23)$$

Note that Eqns. (5.20), (5.21) and (5.23) as well as their one-dimensional forms are widely used for obtaining analytical solutions to specific field problems.

Furthermore, if a groundwater source or sink exists, Eqn. (5.13) is written as follows:

$$\frac{\partial}{\partial x}\left(K_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_y \frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(K_z \frac{\partial h}{\partial z}\right) \pm Q(x,y,z,t) = S_s \frac{\partial h}{\partial t} \qquad (5.24)$$

Where, Q is the volumetric source rate and/or sink rate per unit volume of the confined aquifer [L^3T^{-1}/L^3]. Note that if a groundwater source exists, there will be '+Q' and if a groundwater sink exists, there will be '-Q'. Groundwater sources are recharge or inflow into the aquifer and groundwater sinks are pumping or outflow from the aquifer.

Equation (5.24) can be expressed in words as:

Inflow rate – Outflow rate ± Source/Sink rate = Change of storage (5.25)

Multiplying both sides of Eqn. (5.24) by the aquifer thickness (b) yields:

$$\frac{\partial}{\partial x}\left(T_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(T_y \frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(T_z \frac{\partial h}{\partial z}\right) \pm Q(x,y,z,t) = S \frac{\partial h}{\partial t} \qquad (5.26)$$

Under steady-state flow conditions, Eqn. (5.24) can be written as:

$$\frac{\partial}{\partial x}\left(K_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_y \frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(K_z \frac{\partial h}{\partial z}\right) \pm Q(x,y,z,t) = 0 \qquad (5.27)$$

5.4 Equations for Groundwater Flow in Unconfined Aquifers

As we know that the water from unconfined aquifers is released due to draining of water from pores (pore spaces). Unlike confined aquifers, the water released due the compressibility of aquifer material and water is generally negligible (i.e., S_s0). This drainage of pores results in a decline in the position of the water table, and hence the saturated thickness of unconfined aquifers changes with time. This is in contrast with the confined aquifer wherein although the hydraulic head (piezometric level) declines, the saturated thickness of the aquifer remains constant. Thus, the ability of unconfined aquifers to transmit

water, i.e., aquifer transmissivity (T) changes with time. Such a hydraulic behavior of unconfined aquifers complicates the analysis of flow.

Since S_s0 in case of unconfined aquifers, the general equation of flow in unconfined aquifer systems is not represented by Eqn. (5.13). In principle, the location of the water table in space and time can be computed by solving the following equation:

$$\frac{\partial}{\partial x}\left(K_x \frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y \frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_z \frac{\partial h}{\partial z}\right)=0 \qquad (5.28)$$

Note that the zero appears on the right hand side of Eqn. (5.28) because S_s is zero in case of unconfined aquifers, not because the flow is steady. To solve Eqn. (5.28) is a rigorous approach. Difficulty is that the water table position is the outcome of the solution yet the water table position is required (priori) to define the flow domain in which Eqn. (5.28) applies. This problem of unconfined aquifer flow was solved by Boussinesq in 1904 who analyzed the flow through unconfined aquifers using the Dupuit-Forchheimer (D-F) assumptions.

The Dupuit-Forchheimer (D-F) assumptions for an unconfined flow system are as follows (Fig. 5.2):

(i) Flow lines are assumed to be horizontal and equipotentials are vertical at any vertical cross section of the unconfined flow system.

(ii) The hydraulic gradient is assumed to be equal to the slope of the free surface (water table) and to be invariant with depth.

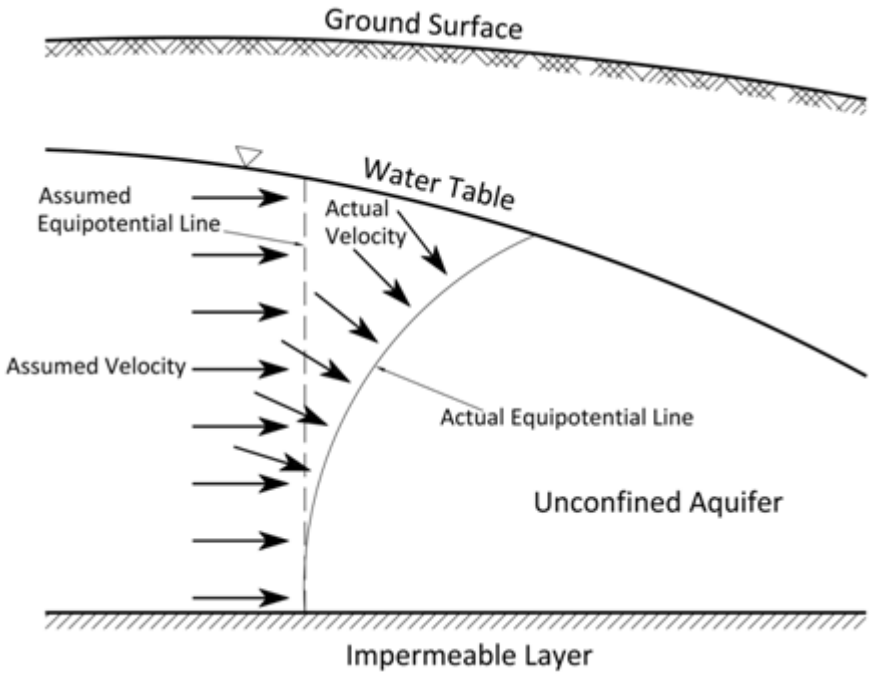


Fig. 5.2. Actual and assumed velocity distributions in unconfined aquifers.

5.4.1 Boussinesq Equation for Heterogeneous and Anisotropic Unconfined Aquifer System

Assume that the unconfined aquifer is heterogeneous and anisotropic. As the changes in density are unimportant in unconfined aquifers, mass balance is replaced by a volume

balance for unconfined flow systems. For deriving groundwater flow equation for unconfined aquifers, let's consider a small part of the unconfined aquifer (i.e., control volume) having three sides of lengths Δx, Δy and Δh, respectively (Fig. 5.3); here h is the saturated thickness which varies with a decrease or increase in the water table position. Since the D-F assumptions neglect the vertical flow in unconfined aquifers, and therefore the actual fluid motion can be subdivided into q_x and q_y on the basis of the two components of flow parallel to the X-axis and Y-axis, respectively where q denotes the flow per unit cross-sectional area of the aquifer.

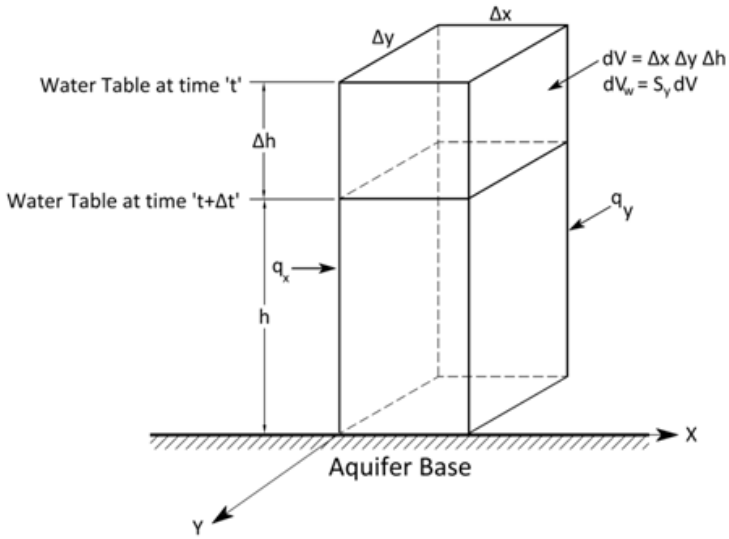


Fig. 5.3. Control volume for flow through an unconfined aquifer.

The net volumes of flow in X- and Y-directions are given as:

Net volume of flow in X-direction = $-\frac{\partial}{\partial x}(h\,q_x)\,\Delta x\,\Delta y$ (5.29)

Net volume of flow in Y-direction = $-\frac{\partial}{\partial y}(h\,q_y)\,\Delta x\,\Delta y$ (5.30)

The rate of change in groundwater storage within the control volume is given as $S_y\frac{\partial h}{\partial t}\Delta x\,\Delta y$. Now, from the continuity principle, net outflow rate is equal to the rate of change in groundwater storage within the control volume. That is,

$$-\frac{\partial}{\partial x}(hq_x)\,\Delta x\,\Delta y-\frac{\partial}{\partial y}(hq_y)\,\Delta x\,\Delta y=S_y\frac{\partial h}{\partial t}\Delta x\,\Delta y \qquad (5.31)$$

From the Darcy's law $q_x = -K_x \frac{\partial h}{\partial x}$, and $q_y = -K_y \frac{\partial h}{\partial y}$, and also Δx Δy is common on both sides which allows us to eliminate it. Thus, Eqn. (5.31) can be written as:

$$\frac{\partial}{\partial x}\left(K_x h \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_y h \frac{\partial h}{\partial y}\right) = S_y \frac{\partial h}{\partial t} \qquad (5.32)$$

Where, K_x and K_y = aquifer hydraulic conductivities in the X- and Y-directions, respectively; S_y = specific yield of the aquifer; h = hydraulic head in the aquifer; and t = time.

Equation (5.32) is known as 'nonlinear Boussinesq equation' or simply 'Boussinesq equation', which is a general equation for transient two-dimensional flow in heterogeneous and anisotropic unconfined aquifer systems. This equation is a type of differential equation that cannot be solved using calculus, except for some very specific cases.

If the drawdown in an unconfined aquifer is very small compared to its saturated thickness, the variable aquifer thickness (h) can be replaced with an average saturated thickness of the unconfined aquifer (h_{mean}) which is assumed to be constant over the aquifer. Given this assumption, the Boussinesq equation [Eqn. (5.32)] can be linearized as follows:

$$\frac{\partial}{\partial x}\left(K_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_y \frac{\partial h}{\partial y}\right) = \frac{S_y}{h_{mean}} \frac{\partial h}{\partial t} \qquad (5.33)$$

Equation (5.33) is known as 'linear Boussinesq equation', which can easily be solved using calculus. For steady-state groundwater flow in heterogeneous and anisotropic unconfined aquifer systems $\frac{\partial h}{\partial t} = 0$, (i.e., no change in aquifer storage with time). Therefore, Eqn. (5.32) simplifies to:

$$\frac{\partial}{\partial x}\left(K_x h \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_y h \frac{\partial h}{\partial y}\right) = 0 \qquad (5.34)$$

and Eqn. (5.33) simplifies to:

$$\frac{\partial}{\partial x}\left(K_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_y \frac{\partial h}{\partial y}\right) = 0 \qquad (5.35)$$

Moreover, if a groundwater source or sink exists, Eqn. (5.32) is written as:

$$\frac{\partial}{\partial x}\left(K_x h \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_y h \frac{\partial h}{\partial y}\right) \pm Q(x,y,z,t) = S_y \frac{\partial h}{\partial t} \qquad (5.36)$$

For steady-state groundwater flow conditions, Eqn. (5.36) can be written as:

$$\frac{\partial}{\partial x}\left(K_x h\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y h\frac{\partial h}{\partial y}\right)\pm Q(x,y,z,t)=0 \qquad (5.37)$$

5.4.2 Boussinesq Equation for Homogenous and Isotropic Unconfined Aquifer System

For homogeneous and isotropic unconfined aquifer systems, $K_x = K_y = K$ and the value of K will remain constant over the aquifer. Therefore, for groundwater flow through homogeneous and isotropic unconfined aquifer systems, Eqn. (5.32) reduces to:

$$\frac{\partial}{\partial x}\left(h\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(h\frac{\partial h}{\partial y}\right)=\frac{S_y}{K}\frac{\partial h}{\partial t} \qquad (5.38)$$

Note that Eqn. (5.38) is also called 'nonlinear Boussinesq equation', and it cannot be solved using calculus, except for some simple cases.

For homogeneous and isotropic unconfined aquifer systems, Eqn. (5.33) simplifies to:

$$\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}=\frac{S_y}{Kh_{mean}}\frac{\partial h}{\partial t} \qquad (5.39a)$$

Or,

$$\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}=\frac{S_y}{T}\frac{\partial h}{\partial t} \qquad (5.39b)$$

Equations (5.39a) or (5.39b) is called 'linear Boussinesq equation', which are widely used for obtaining analytical solutions, together with their one-dimensional forms. Note that Eqn. (5.39a) or Eqn. (5.39b) has the same form as that for homogeneous and isotropic confined aquifer systems [i.e., Eqn. (5.19)].

For steady-state groundwater flow conditions in homogeneous and isotropic unconfined aquifer systems, Eqn. (5.34) can be written as:

$$\frac{\partial}{\partial x}\left(h\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(h\frac{\partial h}{\partial y}\right)=0 \qquad (5.40)$$

and Eqn. (5.35) can be written as:

$$\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}=0 \qquad (5.41)$$

Equation (5.41) is the same as Eqn. (5.23), i.e., 'two-dimensional Laplace equation'.

5.5 Equations for Groundwater Flow in Leaky Confined Aquifers

The equations of confined aquifers presented in Sections 5.2 and 5.3 are based on the assumption that all flow comes from water stored in the aquifer. However, in reality,

significant flow is often generated from leakage into the confined aquifer through overlying or underlying confining layers (Fig. 5.4). If this is the case, then the confined aquifer is called 'leaky confined aquifer'. The procedures for the derivation of governing equation for flow in leaky confined aquifers are the same as discussed in Section 5.2, except for an additional term 'leakage' and the two-dimensional flow in case of leaky confined aquifers.

The leakage is considered to appear in the control volume (Fig. 5.1) as horizontal flow. This assumption is justified on the grounds that the hydraulic conductivity of the aquifer is usually much greater than that of the confining layer (aquitard). For these conditions, the law of refraction suggests that the flow in the confining layer will be almost vertical if the flow in the confined aquifer is horizontal as illustrated in Fig. 5.4. Thus, the flow in leaky confined aquifers is essentially two-dimensional in the horizontal plane. Given these flow conditions in leaky confined aquifers, the general equation of flow can be derived using the law of mass conservation and Darcy's law as presented below.

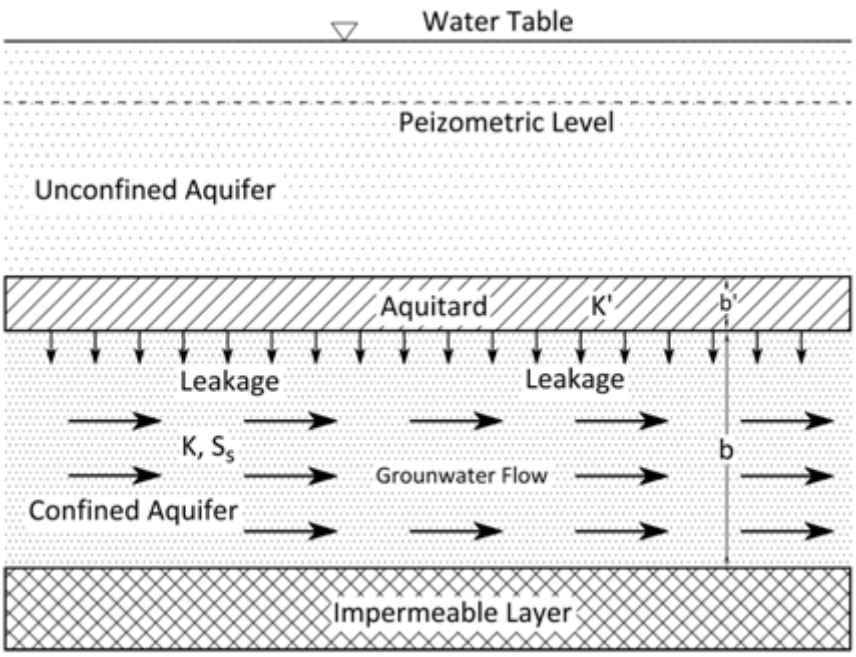


Fig. 5.4. Groundwater flow pattern in a leaky confined aquifer.

5.5.1 Heterogeneous and Anisotropic Leaky Confined Aquifer System

Let's assume a heterogeneous and anisotropic leaky confined aquifer of thickness b, which receives leakage (R_L) from an overlying unconfined aquifer (Fig. 5.4). Considering the above-mentioned flow conditions in leaky confined aquifers, the mass balance of flow for a control volume (Fig. 5.1) yields the following equation:

$$-\frac{\partial}{\partial x}(\rho_w q_x)\Delta x\Delta y-\frac{\partial}{\partial y}(\rho_w q_y)\Delta x\Delta y+\frac{\rho_w R_L}{b}\Delta x\Delta y=\rho_w S_s\frac{\partial h}{\partial t}\Delta x\Delta y \tag{5.42}$$

Dividing both sides of Eqn. (5.42) by ΔxΔy, we have:

$$-\frac{\partial}{\partial x}(\rho_w q_x)-\frac{\partial}{\partial y}(\rho_w q_y)+\frac{\rho_w R_L}{b}=\rho_w S_s\frac{\partial h}{\partial t} \qquad (5.43)$$

By assuming that the density of water does not vary spatially, i.e., ρ_w = constant over the aquifer, the density term can be taken out and thereby ρ_w gets eliminated. Thus, Eqn. (5.43) reduces to:

$$-\frac{\partial q_x}{\partial x}-\frac{\partial q_y}{\partial y}+\frac{R_L}{b}=S_s\frac{\partial h}{\partial t} \qquad (5.44)$$

From the Darcy's law $q_x=-K_x\frac{\partial h}{\partial x}$, and $q_y=-K_y\frac{\partial h}{\partial y}$. Substituting these expressions of q_x and q_y into Eqn. (5.44) yields the general equation of flow for heterogeneous and anisotropic leaky confined aquifers:

$$\frac{\partial}{\partial x}\left(K_x\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y\frac{\partial h}{\partial y}\right)+\frac{R_L}{b}=S_s\frac{\partial h}{\partial t} \qquad (5.45)$$

Multiplying both sides of Eqn. (5.45) by b (thickness of the leaky confined aquifer), we get another form of general groundwater flow equation for heterogeneous and anisotropic leaky confined aquifers:

$$\frac{\partial}{\partial x}\left(bK_x\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(bK_y\frac{\partial h}{\partial y}\right)+R_L=bS_s\frac{\partial h}{\partial t}$$

$$\Rightarrow\ \frac{\partial}{\partial x}\left(T_x\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(T_y\frac{\partial h}{\partial y}\right)+R_L=S\frac{\partial h}{\partial t} \qquad (5.46)$$

Where, K_x and K_y= hydraulic conductivities of the leaky confined aquifer in X- and Y-directions, respectively; R_L = leakage into the leaky confined aquifer; h = hydraulic head in the leaky confined aquifer; S_s = specific storage of the leaky confined aquifer; S = storage coefficient of the leaky confined aquifer; and T_x and T_y = transmissivities of the leaky confined aquifer in X- and Y- directions, respectively.

The leakage rate (R_L) can be computed from the Darcy's law as follows:

$$R_L=K'\left(\frac{h_1-h_2}{b'}\right) \qquad (5.47)$$

Where, h_1 = hydraulic head at the top of the aquitard (leaky confining layer), h_2 = hydraulic head in the aquifer just below the aquitard, = vertical hydraulic conductivity of the aquitard, and = thickness of the aquitard.

Note that if an additional source of recharge and/or a sink (e.g., pumping) occurs in case of leaky confined aquifer systems, they should also be included in Eqns. (5.45) and (5.46) along with the leakage (R_L).

5.5.2 Homogeneous and Isotropic Leaky Confined Aquifer System

For homogeneous and isotropic leaky confined aquifer systems, $K_x = K_y = K$ and the value of K will remain constant over the aquifer. Therefore, for groundwater flow through homogeneous and isotropic leaky confined aquifer systems, Eqn. (5.45) can be written as:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{R_L}{T} = \frac{S_s}{K}\frac{\partial h}{\partial t} \qquad (5.48)$$

and Eqn. (5.46) can be written as:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{R_L}{T} = \frac{S}{T}\frac{\partial h}{\partial t} \qquad (5.49)$$

5.6 Solution of Groundwater Flow Equations

Flow of water in aquifer systems (saturated porous media) can be mathematically described by the above equations according to the flow conditions under investigation. They all are partial differential equations in which the hydraulic head (h) is a dependent variable. They are solved by using a mathematical model consisting of the applicable governing flow equation, boundary conditions, and initial conditions. If the aquifer is homogenous and isotropic, and the boundaries can be described with algebraic equations, then the mathematical model can be solved by an analytical technique based on integral calculus.

However, for heterogeneous and anisotropic aquifer systems (e.g., layered aquifer systems), a numerical solution to the mathematical model is required. Numerical solutions are based on the concept that the partial differential equation can be replaced by a similar equation that can be solved using arithmetic. Similarly, the equations governing boundary and initial conditions are replaced by numerical statements of these conditions. Numerical solutions are obtained using well-known numerical techniques such as 'Finite Difference Method' (FDM), 'Finite Element Method' (FEM), 'Method of Characteristics' (MOC), 'Collocation Method' (CM), and Boundary Element Method (BEM), of which FDM and FEM are widely used by groundwater hydrologists or hydrogeologists. Numerical solutions are generally solved on computers, and the entire modeling process is known as 'numerical modeling'. Fundamentals of groundwater modeling are discussed in Chapter 22.

Lesson 6 Analysis of Steady Groundwater Flow

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

6.1 Steady Flow in Confined Aquifers

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

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

$$q = Kb\frac{dh}{dx} \qquad (6.1)$$

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

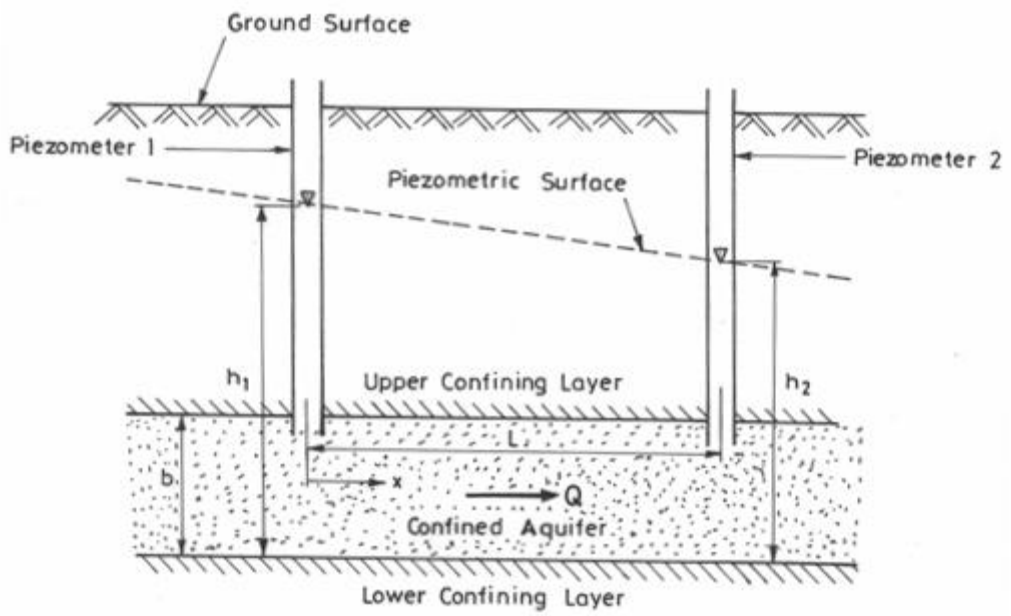


Fig. 6.1. Steady flow through a confined aquifer of uniform thickness. (Modified from Fetter, 1994)

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

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

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

6.2 Steady Flow in Unconfined Aquifers

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

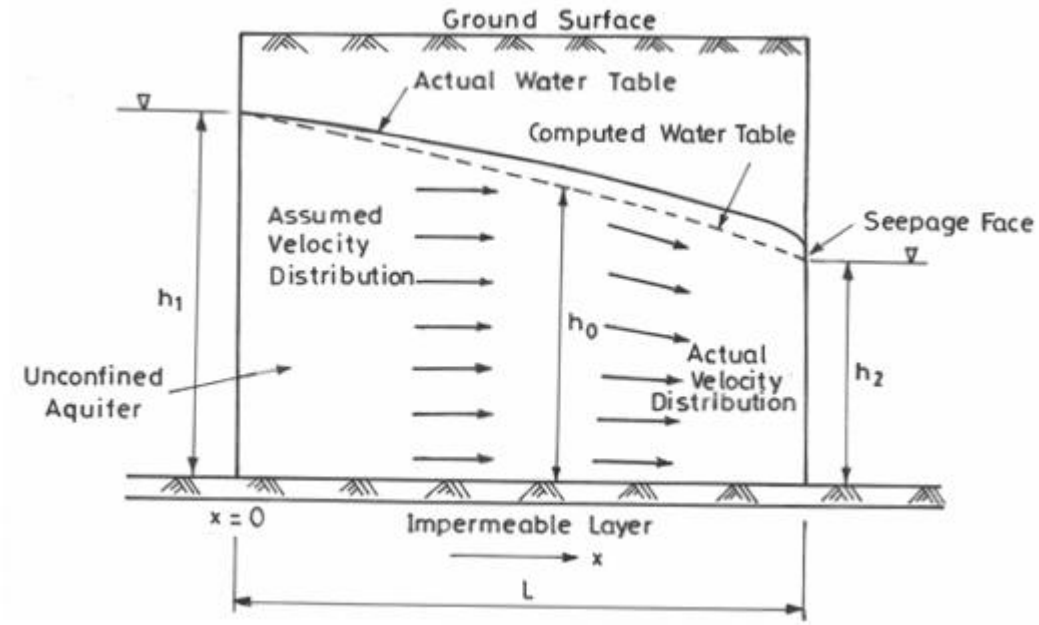


Fig. 6.2. Steady flow in an unconfined aquifer between two water bodies with vertical boundaries. (Modified from Todd, 1980)

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

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

6.2.1 Steady Unconfined Flow without Recharge or Evapotranspiration

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

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

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

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

$$\int_0^L q\,dx = -K\int_{h_1}^{h_2} h\,dh$$

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

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

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

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

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

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

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

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

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

6.2.2 Steady Unconfined Flow with Recharge or Evapotranspiration

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

$$h(x) = \left[h_1^2 - \frac{\left(h_1^2 - h_2^2\right)x}{L} + \frac{R}{K}(L - x)x\right]^{0.5} \qquad (6.6)$$

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

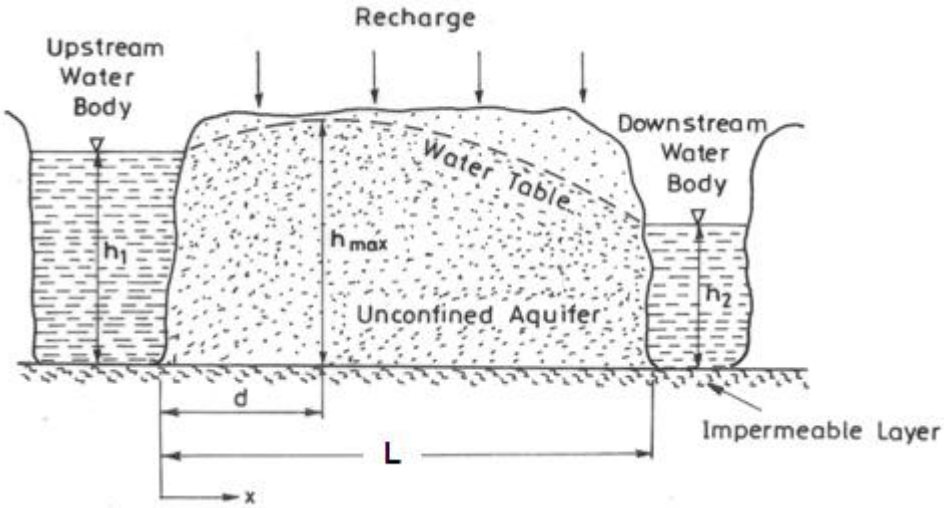


Fig. 6.3. Steady unconfined flow subject to recharge.

(Modified from Fetter, 1994)

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

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

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

Equation (6.7) is called Dupuit parabola.

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

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

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

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

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

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

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

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

6.3 Example Problems

6.3.1 Problem 1

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

Solution: From the question, we have:

Thickness of aquifer, b = 3.0 m

Difference in piezometric levels, Δh = 0.15 m

Distance between the observation wells, ΔL = 238 m

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

Effective porosity, n_e = 0.15

Width of the aquifer strip, W = 10 m

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

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

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

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

= 0.12 m^3/day, Ans.

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

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

= 0.027 m/day, Ans.

6.3.2 Problem 2

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

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

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

Solution: From the question, we have:

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

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

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

Width of the aquifer strip, W = 40 m

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

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

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

$= 1.23 \times 10^{-5}$ m²/s

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

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

$= 49.2 \times 10^{-5}$ m³/s

= 42.51 m³/day, Ans.

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

$$h_{mid} = \left[h_1^2 - \frac{(h_1^2 - h_2^2)x}{L} \right]^{0.5}$$, where x = 49.25 m

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

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

Lesson 7 Groundwater Investigation

As we know that groundwater is widely distributed beneath the earth, but its occurrence is confined to only certain geologic formations and structures. As the occurrence of groundwater cannot be seen from the earth's surface, a variety of techniques are used to explore/investigate groundwater. In view of declining groundwater levels in different parts of the world, including India, the exploration of groundwater is increasingly becoming important in both urban and rural regions of a country, especially in developing countries. This lesson deals with the scale of groundwater investigation as well as an overview of surface methods and subsurface methods of groundwater exploration.

7.1 Scale of Groundwater Investigation

Groundwater investigations can be carried out at a regional scale, local scale or site scale (Schwartz and Zhang, 2003). Regional scale investigation is the largest scale for groundwater investigations, which typically encompasses hundreds or thousands of square kilometers. It provides somewhat an overall evaluation of groundwater conditions. Local scale investigation covers an area of a few tens or hundreds of square kilometers. This type of study provides more detailed information about geology, groundwater dynamics, aquifer characteristics and water quality. On the other hand, site-scale investigation is the smallest scale for groundwater investigations, wherein a particular site is involved such as a well field, mining site, waste disposal site, industrial site, etc. Site-scale groundwater investigation provides in-depth field investigations at the site under study.

Regardless of the scale of a study, detailed planning is required to make sure that the approaches followed in the study are appropriate for the formulated objectives and that the standard procedures are adopted with utmost accuracy for field and laboratory measurements as well as for the analysis of field and laboratory data.

7.2 Surface Methods of Groundwater Exploration

7.2.1 Introduction

The exploration of groundwater can be done from the earth's surface or above-surface locations, which is known as surface investigation. Groundwater exploration can also be done using equipment/instruments extending underground, which is known as subsurface investigation. Surface investigations of groundwater usually do not provide quantitative data/information concerning aquifers or groundwater as obtained from subsurface investigations. Correct interpretation requires supplemental data from subsurface investigations to verify the findings of surface investigations. Although the surface investigations of groundwater provide an incomplete picture or qualitative information of hydrogeologic conditions below the ground, they are usually less expensive and less time consuming than the subsurface investigations (Todd, 1980).

The surface methods of groundwater exploration can be classified into two major groups (Todd, 1980): (a) geologic methods (also called 'reconnaissance methods'), and (b) geophysical methods. Geologic methods involve interpretation of geologic data or geology related data and field reconnaissance using 'Test pits and trenches', 'Adits', 'Continuous

cone penetrometer' and 'Auger'. They represent an important first step in any groundwater investigation. On the other hand, geophysical methods are 'Electric resistivity method', 'Seismic methods', 'Gravity method', 'Magnetic method', and 'Remote sensing techniques' (Todd, 1980), of which 'Electric resistivity method' is widely used for groundwater exploration. A brief description of the geologic methods and geophysical methods is provided below.

7.2.2 Geologic Methods

The occurance and movement of groundwater is mainly dependent on the geology of an area, so it is essential to study the geology as a preliminary step. The geologic methods enable to evaluate large areas for groundwater development rapidly and economically (Todd, 1980). The type of geophysical method to be conducted later can be decided only after the geologic investigations. The geologic investigation involves the collection, analysis and hydrogeologic interpretation of existing topographic maps, aerial photographs, geologic maps, well logs and other relevant data/information. This should be supplemented by geologic field reconnaissance and hydrologic data such as streamflow, springs, well yield, groundwater levels, groundwater recharge and discharge, and water quality (Todd, 1980). These field data and information indirectly/directly indicate the possibility of water-bearing formations (aquifers), their extent and continuity, interconnection of aquifers, aquifer boundaries, nature and thickness of overlying strata, presence of faults, etc. Such prior information is quite helpful in planning detailed field exploration by subsurface methods of groundwater investigation.

7.2.2.1 Test Pits and Trenches

Test pits and trenches are excavations on the ground surface for in situ examinations of near surface soil, rocks or any other geologic formations. These excavations can be done by hand tools or by power equipment like backhoes, bulldozers, scrapers, etc. The depth of the excavation depends on the field conditions, type of equipment used and the budget available.

Test pits are usually square or circular in shape with 1-3 m length or diameter, respectively. These are deeper than trenches which are about 1 to 2 m wide and may extend to any lengths. Some of the advantages of test pits and trenches are: (i) they are cost effective, (ii) information can be obtained on lateral and vertical extent of subsurface features, (iii) in situ examination is possible, and (iv) they facilitate sample collection.

7.2.2.2 Adits

Adits are horizontal or nearly horizontal excavations mainly used to drain water from mines and also serve as an entrance and ventilation. Typical dimensions of adits are 1 m × 1.5 m or 2 m × 2.5 m. They are mainly used for the exploration of rocks, their structural features such as joints, fractures, faults and shear zones. The main limitation to this method is that it is costly for small projects; generally it is not used in the soil.

7.2.2.3 Continuous Cone Penetrometer

Continuous cone penetrometer is a device consisting of a cylindrical probe with a cone shaped tip with different sensors in it. The cylindrical probe is 3 to 5 cm in diameter with its cone tip having a cross sectional area of 10 to 15 cm^2 and apex angle of 60°. A porous filter element is present above this cone tip used to measure the pore water pressure. The device is

pushed into the ground with its tip facing the surface at a controlled rate of 1.5 to 2.5 cm/s. The data is recorded by a field computer through a cable connected to the device. This is used to measure stress, sleeve friction and porewater pressure. It provides a continuous record of penetration resistance and friction.

7.2.2.4 Auger

Auger is a drilling device consisting of a rotating helical blade with an extendable steel rod and a handle. The auger is driven into the ground to remove the drilled materials. It can be hand driven or power driven. It is a cheap and fast method, but restricted only to soft unconsolidated formations. Depending on the geology, augers can be used up to a depth of 15 to 25 m. Many types of augers can be used based on the type of geologic formations available in a particular region. Hand augers are used in sand, silt and soft clay, while bucket augers are used in relatively hard grounds. Augers are cheaper, and easier to operate and maintain.

7.2.3 Geophysical Methods

Geophysical methods are scientific measurements of differences or anomalies of physical properties within the earth's crust. Electric resistivity, density, magnetism, and elasticity are the most commonly measured properties by different geophysical methods (Todd, 1980). Some of the geophysical methods are briefly described in subsequent sub-sections.

7.2.3.1 Electric Resistivity Method

Among all surface geophysical methods of groundwater exploration, the electric resistivity method has been applied most widely for groundwater investigations, even these days. Electric resistivity of a rock formation limits the amount of current passing through the formation when an electric potential is applied. If a material of resistance R has a cross-sectional area A and length L, then its resistivity can be expressed as:

$$\rho = \frac{RA}{L} \qquad (7.1)$$

The unit of resistivity is Ohm-meter (W-m). Resistivity of rock formations varies depending on the material density, porosity, pore size and shape, water content, water quality and temperature (Todd, 1980). Electric resistivity methods are based on the response of the earth to the flow of electrical current. In these methods, an electric current is introduced into the ground by two current electrodes, and the potential difference is measured between two points using potential electrodes suitably placed with respect to the current electrodes. The potential difference for unit current sent through the ground is a measure of the electrical resistance of the ground between the probes. The measured resistance is a function of the geometrical configuration of the electrodes and the electrical parameter of the ground.

The measured current (in amperes) and potential differences (in volts) yield an apparent resistivity (ρ_a) over an unspecified depth. If the spacing between electrodes is increased, a deeper penetration of electric field occurs and a different apparent resistivity is obtained (Todd, 1980). In practice, various standard electrode spacing configurations/arrangements are adopted, but mainly two types of electrode configurations known as Wenner electrode arrangement (Fig. 7.1) and Schlumberger electrode arrangement (Fig. 7.2) are most commonly used in resistivity surveys. The Wenner electrode arrangement is used almost exclusively for shallow subsurface exploration, while the Schlumberger electrode arrangement is used for both shallow and deeper subsurface investigations.

In the Wenner electrode arrangement (Fig. 7.1), A and B are current electrodes, M and N are potential electrodes, and 'a' (distance between adjacent electrodes) is called spacing or separation of the electrodes; the value of 'a' is taken as the approximate depth of resistivity measurement. In this case, the apparent resistivity (ρ_a) is given as:

$$\rho_a = 2\pi a \frac{\Delta V}{I} \qquad (7.2)$$

Where, ΔV = potential difference between the potential electrodes M and N on the earth's surface (volts), and I = direct current introduced into the earth by means of two current electrodes A and B (amperes).

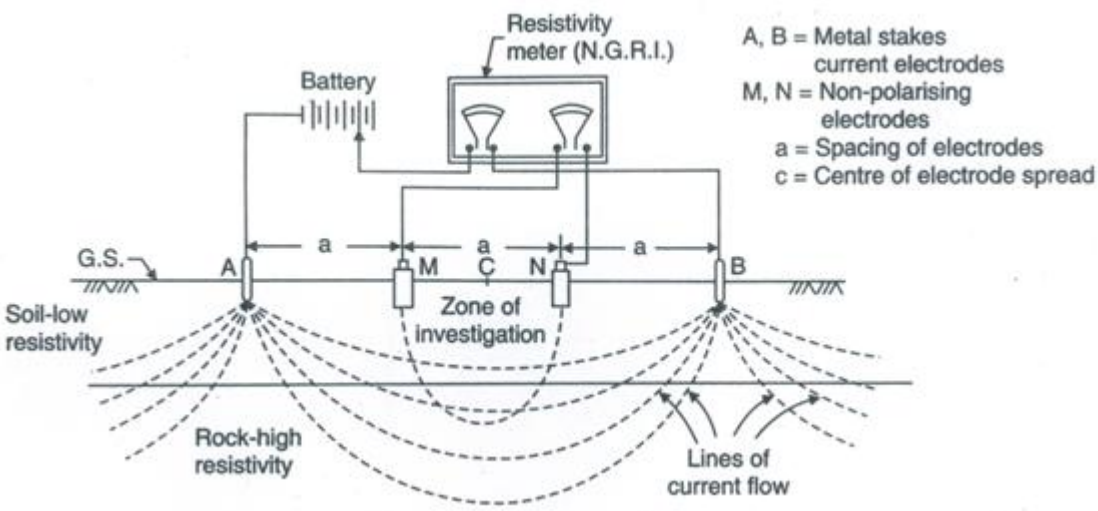


Fig. 7.1. Schematic view of the Wenner electrode arrangement.

(Source: Raghunath, 2007)

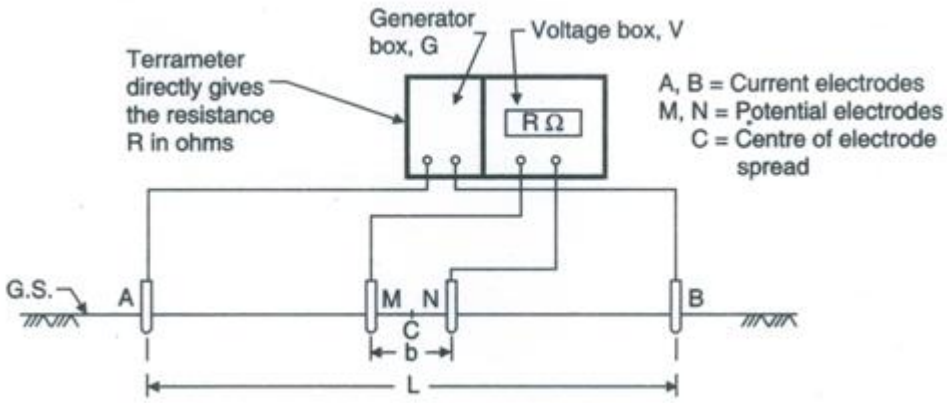


Fig. 7.2. Schematic view of the Schlumberger electrode arrangement.

(Source: Raghunath, 2007)

In the Schlumberger electrode arrangement (Fig. 7.2), the distance between the current electrodes A and B is denoted by L and that between the potential electrodes M and N is dented by l. Note that in this case, the potential electrodes are placed close together and that half of the current electrode spacing (i.e., L/2) is taken as the approximate depth of resistivity measurement. For the Schlumberger electrode arrangement, the apparent resistivity (ρ_a) is given as:

$$\rho_a = \pi \frac{\left(\frac{L}{2}\right)^2 - \left(\frac{l}{2}\right)^2}{l} \times \frac{\Delta V}{I} \qquad (7.3)$$

Theoretically, L >> l, but for practical application good results can be obtained if L ³ 5l.

Electric resistivity surveying is carried out by using an Electric Resistivity (ER) Meter. Two commonly used ER meters are: NGRI Resistivity Meter (manufactured by the National Geophysical Research Institute, Hyderabad, India) and Terrameter (manufactured by Atlas Copco ABEM AB, Sweden). Electric resistivity surveys are generally done in two ways: (i) Vertical electric sounding (VES) or sounding, and (ii) Horizontal electric profiling (HEP) or profiling. Many surveyors do both simultaneously. Vertical electric sounding is used when the zone of investigation varies vertically more than horizontally; it is frequently used for finding out suitable sites for well drilling. On other hand, in profiling, the lateral distribution of resistivity is studied by maintaining a relatively constant depth of investigation (i.e., constant electrode spacing).

When the apparent resistivity data obtained by a VES survey are plotted against the electrode spacing ('a' in case of Wenner and 'L/2' in case of Schlumberger) for various spacings at a given location, a smooth curve can be drawn through the data points. The interpretation of such resistivity-spacing curves in terms of subsurface conditions is complex

and it is usually accomplished with the help of Type Curves and computer programs. The computer programs are based on the theory of resistivity inversion and employ conventional optimization techniques (e.g., Levenberg-Marquardt technique) or non-conventional optimization techniques such as Genetic Algorithm (GA), Artificial Neural Network (ANN), and Simulated Annealing (SA).

Finally, the salient advantages of the electric resistivity method are (Todd, 1980): (i) its portable equipment and the ease of operation facilitate rapid measurements; (ii) it frequently aids in planning efficient and economic test-drilling or well-drilling programs; (iii) it is especially well adapted for locating subsurface saltwater boundaries, because the decrease in resistance due to the presence of saltwater becomes apparent on a resistivity-spacing curve; (iv) it can be used for delineating geothermal areas and estimating aquifer permeability; and (v) it can also be used for defining areas and magnitudes of polluted groundwater. However, the limitation of the resistivity method is that the factors like lateral geologic heterogeneities, buried pipelines, cables, and wire fences can disturb the electric field close to the electrodes, thereby invalidating resistivity measurements! In addition, it is not effective for determining actual resistivities below a few hundred meters, as the change in resistivity at large depths has only a slight effect on the apparent resistivity compared to that at shallow depths.

7.2.3.2 Seismic Methods

Seismic techniques involve the measurement of seismic waves travelling through the subsurface. Since seismic techniques require special equipment and trained persons for operation and data interpretation, they have been applied to a relatively limited extent for groundwater investigations. Three most commonly used seismic methods are: (i) Seismic refraction, (ii) Seismic reflection, and (iii) Seismic surface wave analysis.

Seismic refraction method involves the creation of small shock at the earth's surface either by impact of a heavy instrument or by a small explosive charge and measuring the time required for the resulting sound or shock wave to travel a known distance. Seismic waves follow the same laws of propagation as light rays and may be refracted or reflected at any interface where velocity change occurs. The changes in seismic velocities are governed by the change in elastic properties of formations. The refraction method assumes that the velocity of seismic waves increases with depth, and hence the layers must be thick enough and should have velocity contrast to be resolved. This method can provide data up to a depth of 100 m. The major limitations of seismic refraction method are that it is sensitive to acoustic noise and vibration, and cannot detect thin layers. Also, for deep measurements, it may require explosives as a source of energy.

Seismic reflection method is similar to the seismic refraction method, but field data and processing procedures are employed to maximize the energy reflected along near vertical ray paths by subsurface density contrasts. Reflected seismic energy is never a first arrival and therefore must be identified in a complex set of overlapping seismic arrivals by

collecting data from numerous shot points per geophone placement. This method can be performed in the presence of low velocity zones. The lateral resolution is high and can delineate very deep density contrast with much less shot energy compared to refraction method. This can also provide information on geologic structures thousands of meters below the surface. The limitations of this method are the high cost and longer field processing time.

Seismic surface wave analysis is a geophysical method that uses the dispersive characteristics of surface waves to determine the variation of shear wave velocity with depth. Data are acquired by measuring seismic surface wave generated by an impulsive source and received by an array of geophones. A dispersion curve is plotted from the data, with velocity of surface waves as a function of frequency. From these curves, a shear wave velocity profile is modeled for multiple locations and combined into a 2-D cross section of shear velocity. The shear wave velocity is a function of elastic properties of soil or rock and is directly related to hardness and stiffness of the material. This method can be used in water-covered areas as well.

7.2.3.3 Gravity Method

The gravity method measures the difference between the gravitational fields at two points in a series of different locations on the earth's surface and the variation is associated to the type of rock (i.e., geologic structure). Since this method is expensive and the differences in water content in subsurface strata seldom involve measurable differences in specific gravity at the surface, it has little application to groundwater exploration.

7.2.3.4 Magnetic Method

This method involves the measurement of direction, gradient or intensity of earth's magnetic field and the interpretation of variations in these quantities over an area. It uses a simple principle of balancing the force exerted by the vertical component of the earth's magnetic field on a magnet against the force of gravity. The distortion of magnetic field produced by a magnetic material in the earth's crust is called magnetic anomaly which is measured by the magnetic method and is indicative of type of rock producing it. Since magnetic contrasts are seldom associated with groundwater occurrence, the magnetic method has little relevance. However, this method can provide indirect information related to groundwater studies such as dikes that form aquifer boundaries or limits of a basaltic flow.

7.2.3.5 Remote Sensing Techniques

Remote sensing from aircraft or satellite has become an increasing valuable tool for understanding subsurface water conditions. Remote sensing techniques offer many types of investigations about an area without causing any damage to the sites. Satellite images and aerial photographs are the most commonly used remote sensing techniques.

Aerial photographs and satellite images taken at various electromagnetic wavelength ranges can provide useful information about groundwater conditions. Other non-visible portions of the electromagnetic spectrum (e.g., infrared imagery, near-infrared imagery, radar imagery, and low-frequency electromagnetic aerial survey) hold promise for a whole array of imaging techniques that can contribute to hydrogeologic investigations/surveys (Todd, 1980). Fractures and faults appear on aerial photos and satellite images as tonal variations in surface soils caused by the difference in soil moisture. The lines of springs or seeps are caused by the movement of groundwater along the fracture zones. Thus, fracture patterns and other observable surficial features obtained from remote sensing data serve as interpretive aids in groundwater studies because they can be related to the porosity and permeability of subsurface formations, and ultimately well yield (Todd, 1980; Fetter, 2000).

With the advent of powerful and high-speed personal computers and rapid development in remote sensing (RS) and geographic information system (GIS) techniques, the importance of RS technology in the fields of surface hydrology and subsurface hydrology has dramatically increased (Engman and Gurney, 1991; Jha et al., 2007). The RS technology, with its advantages of spatial, spectral and temporal availability of data covering large and inaccessible areas within a short time, has emerged as a powerful tool for the assessment, monitoring and management of groundwater resources (Jha and Peiffer 2006; Meijerink 2007; Rodell et al., 2009). In particular, the integrated use of RS, GIS and multicriteria decision analysis (MCDA) techniques has been found to be efficient and very useful for mapping and evaluating groundwater potential as well as for identifying sites suitable for artificial recharge (e.g., Jha and Peiffer, 2006; Jha et al., 2007; Jha et al., 2010; Machiwal et al., 2011).

7.3 Subsurface Methods of Groundwater Exploration

Detailed and comprehensive examination of groundwater and conditions under which it occurs can be made by subsurface investigations only. Subsurface investigations are conducted by a person or a group of persons on the earth's surface who operate the equipment/instruments extending underground through a borehole which provides direct access to subsurface formations and groundwater. Various subsurface methods of groundwater exploration can be classified into three major groups: (a) Test drilling, (b) Borehole sensing (sometimes it is also called 'television logging'), and (c) Geophysical logging.

Test drilling provides information regarding subsurface formations in a vertical line from the ground surface, whereas the borehole sensing provides more detailed information about the borehole, geologic strata, and well casing and screen. On the other hand, geophysical logging techniques provide information on physical properties of subsurface formations, groundwater quality, and well construction. A variety of geophysical logging techniques are available, of which the following are most important in groundwater hydrology (Todd, 1980; Roscoe Moss Company, 1990):

1. Resistivity logging,
2. Spontaneous potential logging,

3. Nuclear or Radioactive logging (viz., Natural-Gamma logging, Gamma-Gamma logging, and Neutron logging)
4. Temperature logging,
5. Caliper logging,
6. Fluid-Conductivity logging,
7. Fluid-Velocity logging,
8. Sonic or Acoustic logging, and
9. Casing logging.

7.3.1 Test Drilling

Drilling a small-diameter (usually 1″ or 1.5″ diameter) hole to ascertain geologic and groundwater conditions at a particular location/site is known as test drilling. Test drilling is the most reliable method to obtain information about subsurface formations at different depths, which is very useful in verifying the results of other investigation methods as well as to obtain assurance of underground conditions before well drilling. During test drilling, geologic samples are collected at regular depth intervals and the air-dried samples are subject to sieve analysis (also called 'grain-size or particle-size analysis' or sometimes 'mechanical analysis') for determining the proportion of sand, silt, clay and gravel in a given geologic sample. If such information is presented in a graphical or physical manner as a function of depth at a given site/location, it is known as a 'well log', 'borehole log' or 'geologic log' of that site/location. Fig. 7.3 shows a typical well log/borehole log at a groundwater-monitoring site consisting of unconsolidated formations. Well/borehole logs provide reliable information about subsurface conditions (i.e., variation of subsurface materials and their thickness, availability and type of aquifers, type of other layers, etc.), thereby enabling aquifers and confining layers to be delineated. They serve as a standard source of valuable information in the design and construction of production wells, monitoring wells, and foundations as well as for environmental projects and geologic studies.

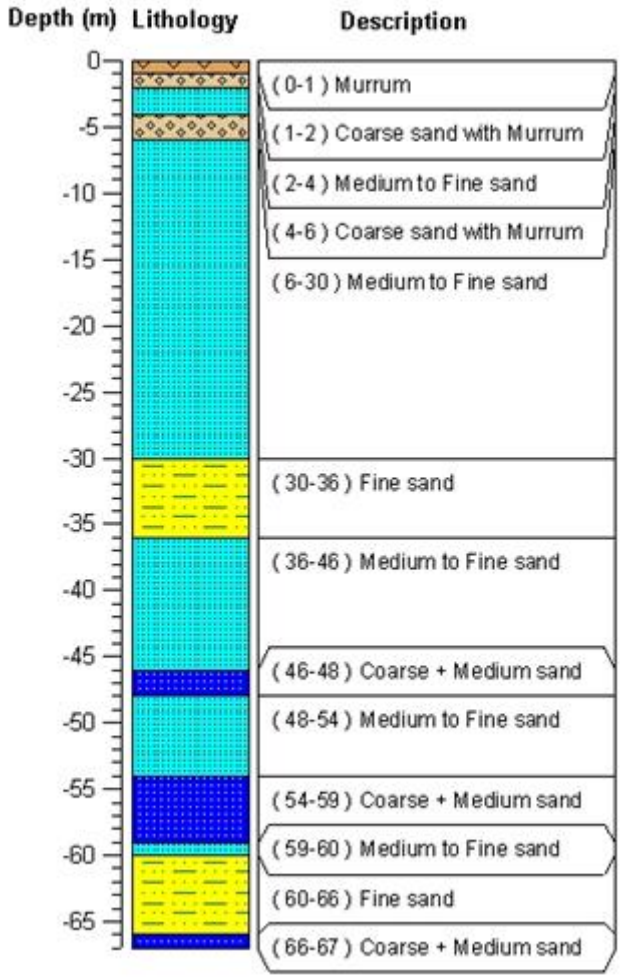


Fig. 7.3. Well log of a site prepared from geologic samples collected during observation-well drilling in an unconsolidated formation.

If the test drilling proves fruitful, it is re-drilled to a larger diameter to form a pumping well (also called 'production well'). The test holes created by test drilling also serve as observation wells (also called 'monitoring wells') for measuring groundwater levels, taking groundwater samples, or for conducting pumping tests. Although any well-drilling method can be used for test drilling, the 'cable-tool method' and 'hydraulic rotary method' (described in Lesson 18) are most common in unconsolidated subsurface formations (Todd, 1980). In fact, the choice of drilling method for test drilling depends on the type of information required, type of material encountered, drilling depth, and the location of investigation.

7.3.2 Borehole Sensing

A borehole sensing or television logging is a convenient technique with increasing use for investigating boreholes (uncased or cased). Specially designed wide-angle cameras, typically less than 7 cm in diameter (Todd, 1980), are equipped with lights and when lowered into a borehole (uncased or cased), provide continuous visual inspection of the borehole which can be preserved in electronic storage devices. Borehole sensing has a variety of applications such as locating changes in geologic strata, pinpointing large pore spaces, inspecting the condition of well casing and screen, checking for debris in wells, locating zones of sand entrance, and searching for lost drilling tools (Todd, 1980).

7.3.3 Geophysical Logging

This involves lowering sensing devices in a borehole (cased or uncased) and recording a physical parameter that may be interpreted in terms of subsurface formation characteristics; groundwater quantity, quality and movement; or physical structure of the borehole (Todd, 1980). Table 7.1 summarizes the types of information that can be obtained by various geophysical logging techniques. The details of various geophysical logging techniques can be found in Todd (1980), Roscoe Moss Company (1990), and Fetter (2000).

Table 7.1. Summary of logging applications to groundwater hydrology (Source: Todd, 1980)

Sl. No.	Type of Information	Possible Logging Techniques
1	Lithology and straitigraphic correlation of aquifers and associated rocks	Resistivity, sonic, or caliper logs made in open holes; radiation logs made in open or cased holes
2	Total porosity or bulk density	Calibrated sonic logs in open holes; calibrated neutron or gamma-gamma logs in open or cased holes
3	Effective porosity or true resistivity	Calibrated long-normal resistivity logs
4	Clay or shale content	Natural gamma logs
5	Permeability	Under some conditions long-normal resistivity logs
6	Secondary permeability-fractures, solution openings	Caliper, sonic, or television logs
7	Specific yield of unconfined aquifers	Calibrated neutron logs
8	Grain size	Possible relation to formation factor derived from resistivity logs
9	Location of water level or saturated zones	Resistivity, temperature, or fluid conductivity logs; neutron or gamma-gamma logs in open or cased holes
10	Moisture content	Calibrated neutron logs
11	Infiltration	Time interval neutron logs
12	Dispersion, dilution, and movement of waste	Fluid conductivity or temperature logs; natural gamma logs for some radioactive wastes
13	Sources and movement of water in a well	Fluid velocity or temperature logs

14	Chemical and physical characteristics of water, including salinity, temperature, density and viscosity	Calibrated fluid conductivity or temperature logs; resistivity logs
15	Construction of existing wells, diameter and position of casing, perforations, screens	Gamma-gamma, caliper, casing, or television logs
16	Guide to screen setting	All logs providing data on the lithology, water-bearing characteristics, and correlation and thickness of aquifers
17	Cementing	Caliper, temperature, or gamma-gamma logs; acoustic logs for cement bond
18	Casing corrosion	Under some conditions caliper, casing, or television logs
19	Casing leaks and/or plugged screen	Fluid velocity logs

At the end, it is worth mentioning that apart from the above-mentioned subsurface investigation techniques, there are some other important subsurface investigation methods which can provide important information about the hydrogeologic conditions and the dynamics of groundwater in a basin. These methods are: tracer tests for groundwater flow; groundwater-level monitoring for flow directions and aquifer conditions; pumping tests for aquifer parameters, well parameters, well yield and well evaluation; groundwater-level fluctuation measurements for analyzing spatio-temporal changes in groundwater storage, groundwater behavior and surface water-groundwater interaction; and groundwater sampling for water-quality assessment.

Module 2_Well Hydraulics

Lesson 8 Introduction to Water Wells

8.1Introduction

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

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

8.2 Functions of Wells

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

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

(2) To control seawater intrusion.

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

(4) To lower water table for construction projects.

(5) To relieve pressure under dams.

(6) To drain agricultural land or urban land.

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

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

8.3 Classification of Water Wells

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

8.3.1 Open Wells

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

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

8.3.1.1 Unlined and Lined Open Wells

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

where aquifer and subsoil conditions are favorable and when the rate of withdrawal is not excessive (Michael et al., 2008).

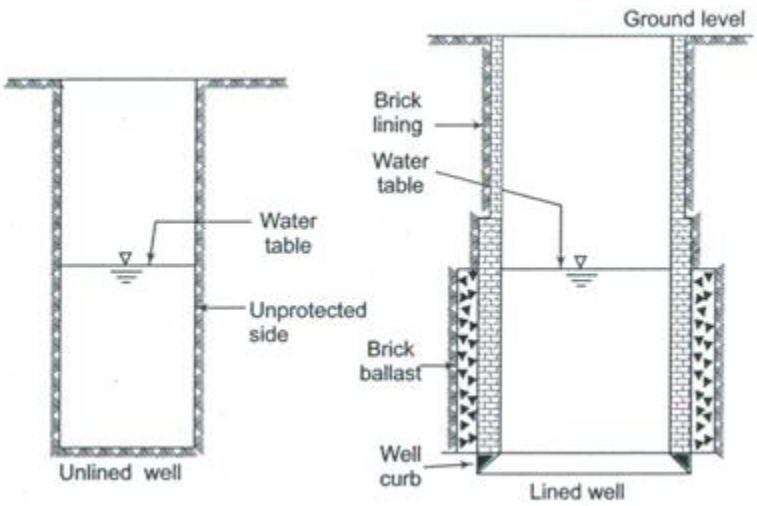


Fig. 8.1. Unlined open well and an open well lined with pervious lining.

(Source: Michael et al., 2008)

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

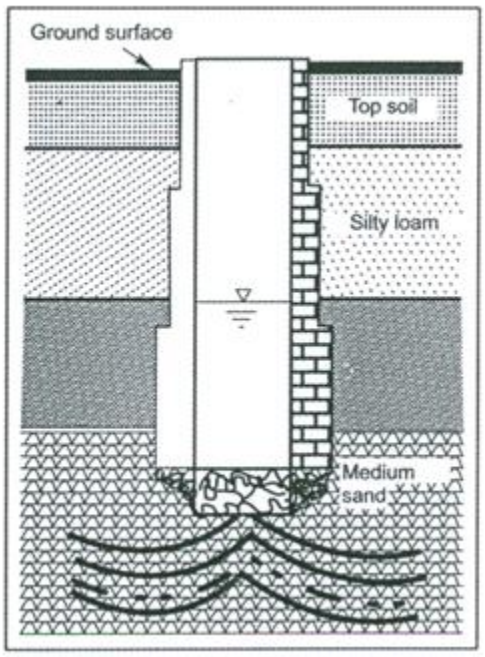


Fig. 8.2. Open well lined with permanent masonry lining.

(Source: Michael et al., 2008)

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

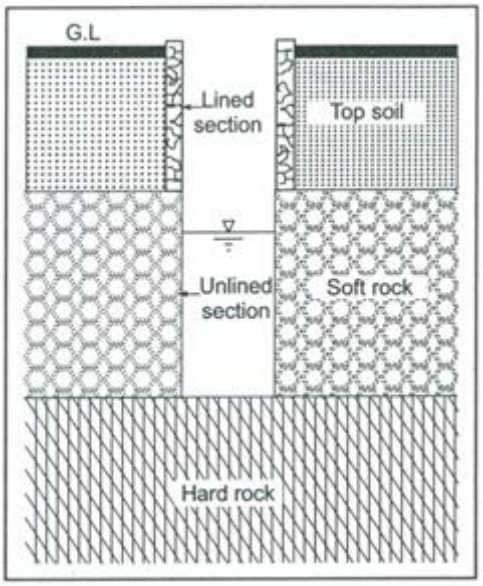


Fig. 8.3. Open well in a hard-rock formation. (Source: Michael et al., 2008)

8.3.1.2 Dug-cum-Bore Wells

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

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

8.3.2 Tubewells

Tubewells are wells consisting of pipes ranging in size from 6 to 45 cm in diameter and sunk into an aquifer (Sarma, 2009). Tubewells are constructed by installing a pipe below the ground surface passing through different geological formations comprising water-bearing and non-water-bearing strata. Blind pipes (casing pipes) are placed in the non-water-bearing

layers and well screens are placed in the water-bearing layers (Fig. 8.5). Several tubewells have been and are being installed worldwide for meeting water demands in domestic, agricultural and industrial sectors. The type of tubewell to be constructed

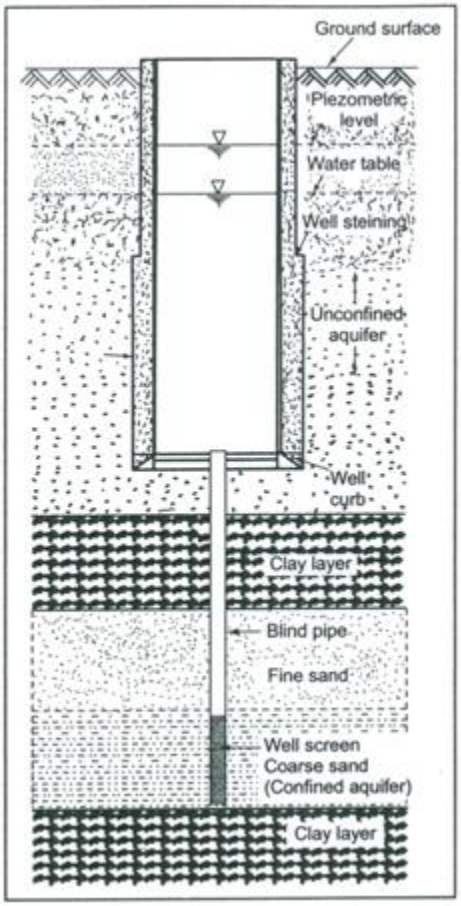


Fig. 8.4. Dug-cum-bore well with a screened vertical bore.

(Source: Michael et al., 2008)

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

Tubewells are also classified based on the depth, method of construction, entry of water into the wells and the type/nature of the aquifer (Michael et al., 2008; Sarma, 2009). As mentioned above, based on the depth of the well, tubewells are classified as shallow tubewells or deep tubewells. Shallow tubewells are of low capacity and their average depth is normally less than 35 m. They mostly tap one aquifer. Deep tubewells are of high capacity and their depth usually ranges from 60 to 300 m (Michael et al., 2008). They often tap two or more aquifers. Based on the method of construction, tubewells are classified as bored tubewells, drilled tubewells, driven tubewells and jetted tubewells; they are described in Lesson 17. Tubewells in unconsolidated formations generally consist of blind pipes, strainers and gravel pack (if necessary). However, tubewells in hard-rock formations are known as borewells, because the borehole remains stable for most of its depth and the tube is placed only in the upper weathered soil zone (Fig. 8.6). No strainer/screen or gravel pack is required for borewells.

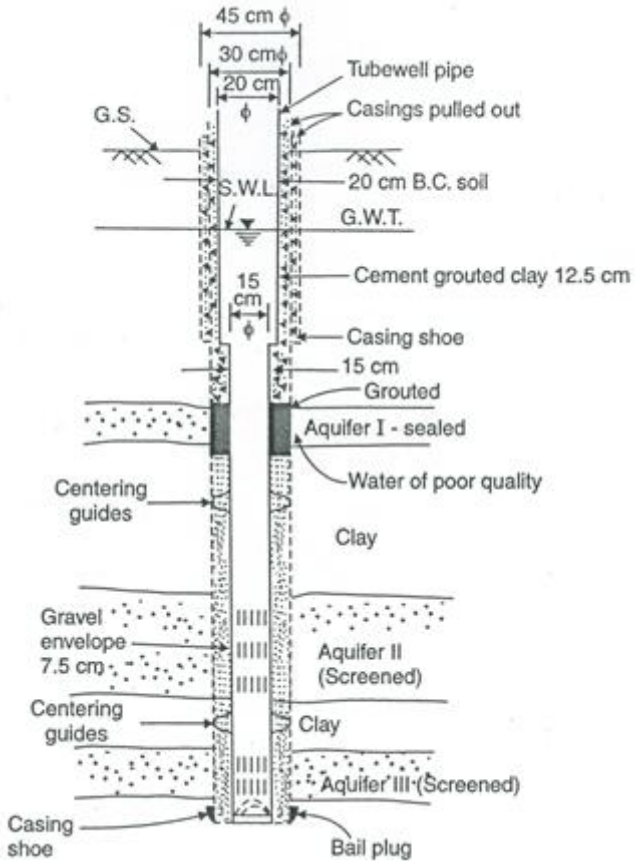


Fig. 8.5. A typical tubewell in an unconsolidated formation.

(Source: Raghunath, 2007)

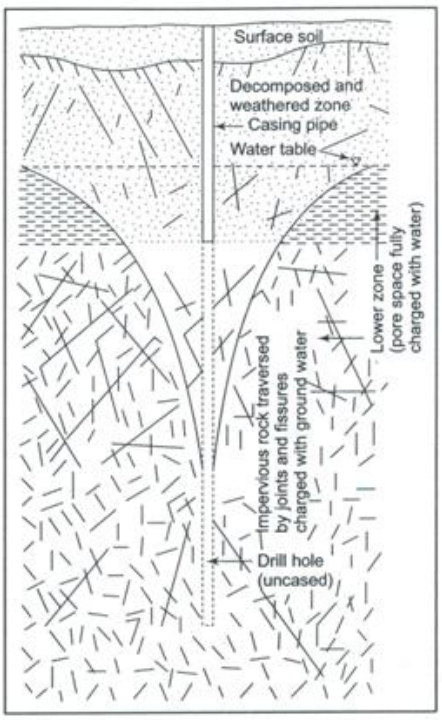


Fig. 8.6. Schematic of a borewell tapping fissured zone in a hard-rock area. (Source: Michael et al., 2008)

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

8.3.3 Cavity Wells

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

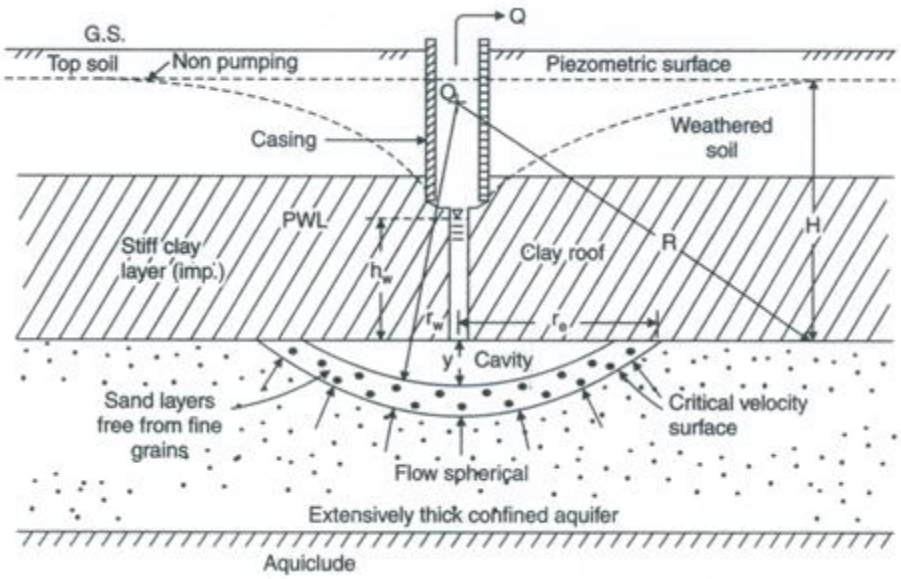


Fig. 8.7. Schematic of a cavity well. (Source: Raghunath, 2007)

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

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

8.3.4 Filter Points

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

8.4 Advantages and Disadvantages of Open Wells and Tubewells

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

8.4.1 Advantages of Open Wells

1. Storage capacity of water is available in the well itself.
2. They do not require sophisticated equipment and skilled persons for constructions.
3. They can be easily operated by installing an ordinary centrifugal pump or by using a manual water-lifting device.
4. They can be revitalized by deepening by vertical boring or by blasting at the bottom, or by creating horizontal or inclined bores on the sides of the well to intercept water-bearing formations.

8.4.2 Disadvantages of Open Wells

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

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

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

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

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

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

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

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

8.4.3 Advantages of Tubewells

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

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

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

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

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

(6) They usually provide good-quality groundwater.

(7) They are relatively less vulnerable to contamination.

8.4.4 Disadvantages of Tubewells

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

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

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

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

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

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

8.5 Selection of Well Site and Type of Well

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

(1) Topography;

(2) Climate;

(3) Vegetation;

(4) Geology;

(5) Porosity, permeability and alteration of rocks;

(6) Joints and faults in rocks;

(7) Folded strata;

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

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

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

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

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

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

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

Lesson 9 General Procedure for Analyzing Groundwater Flow Towards Wells

9.1 Introduction

Analytical solutions of the equations governing groundwater flow for a particular combination of the characteristics of the aquifer and the well can be obtained on the basis of suitable assumptions concerning type of flow and aquifer properties. The aquifer properties include transmissivity, storage coefficient, nature of the boundaries of the flow domain, homogeneity and isotropy as well as the depth and areal extent of the aquifer. Despite the simplifying assumptions inherent in the analytical solutions, they provide pragmatic answers to the groundwater problems if applied judiciously.

When a production well tapping an aquifer of infinite areal extent is pumped at a constant rate, the radius of influence of the well expands radially from the pumping well. In practice, flow towards a pumping well is mostly unsteady (transient). A number of specific situations of unsteady flow towards groundwater wells are discussed in Sarma (2009). For each flow situation, the governing equation, appropriate boundary conditions and initial condition are defined, and then solutions to the governing flow equations are presented. Note that for deriving analytical solutions, several simplfying assumptions are made, of which the basic assumptions required to be made for analyzing either steady or unsteady groundwater flow to wells are described in the subsequent section.

In this lesson, a general procedure for the analysis of groundwater flow to wells is demonstrated, which has been adapted from Sarma (2009). The analysis of steady groundwater flow to wells is discussed in Lesson 10 and that of transient groundwater flow to wells is discussed in Lesson 11.

9.2 Basic Assumptions for Analyzing Groundwater Flow to Wells

The following are the basic assumptions made for analyzing steady or transient groundwater flow to wells:

(1) The aquifer is bounded on the bottom by a confining layer.

(2) The groundwater level of the aquifer is horizontal and not changing with time prior to the start of pumping.

(3) All changes in the position of the groundwater level are due to the effect of the pumping well alone.

(4) All geologic formations are horizontal and of infinite horizontal extent.

(5) The aquifer is homogeneous and isotropic.

(6) All flow is radial towards the well.

(7) Groundwater flow is horizontal.

(8) Darcy's law is valid.

(9) The pumping well and the observation wells are fully penetrating.

(10) The pumping well has an infinitesimal diameter (i.e., negligible storage) and is 100% efficient (i.e., no well losses).

(11) Groundwater has a constant density and viscosity.

9.3 General Procedure

A critical examination of the procedures adopted in different cases for analyzing groundwater flow to wells reveals that there exists certain commonality in the procedures. For example, for the cases of transient groundwater flow to wells, by taking Laplace transform of the governing partial differential equation (p.d.e.) and the associated boundary and initial conditions, the equation can be reduced to the p.d.e. in terms of space coordinates only. Further, in the case of groundwater flow towards partially penetrating wells, two of the boundaries of the flow region are parallel to each other. Such problems can be simplified by using Finite Fourier transforms and thereby reducing the p.d.e. to a Bessel differential equation in one space variable (Sarma, 2009). The associated initial and boundary conditions are also transformed accordingly. Thereafter, the Bessel differential equation in the transformed domain subject to the corresponding transformed boundary conditions can be solved. By taking inverse transforms, first the inverse Finite Fourier transforms and then the inverse Laplace transform for transient flow cases, the solution to the original problem can be obtained. Thus, for solving the governing p.d.e. for most cases of groundwater flow to wells, a general procedure can be adopted. Various steps involved in the general procedure are as foolows (Sarma, 2009):

Step 1: If the groundwater flow equation describes transient flow condition, take Laplace transform for the governing p.d.e. as well as the associated initial and boundary conditions.

Step 2: If the resulting p.d.e. involves parallel boundaries, take an appropriate Finite Fourier transform for the p.d.e. and the boundary conditions obtained in Step 1. Note that if the parallel boundaries constitute barrier boundaries (impermeable or no-flow boundaries), the Finite Fourier cosine transform is to be used. However, for the case of parallel boundaries constituting recharge boundaries, the Finite Fourier sine transform is to be used.

Step 3: The resulting Bessel differential equation of zero order is solved using the standard solution procedure and the properties of Bessel functions. It is worth mentioning that the differential equation obtained for all the cases is the modified Bessel differential equation of order zero for which standard solutions are available in the literature.

Step 4: The solution of the Bessel differential equation, subject to the associated boundary conditions, which are also transformed through Finite Fourier transforms, is then inverse transformed using the appropriate inverse Fourier transform (sine or cosine as the case may be).

Step 5: The resulting expression of the solution for the drawdown is transformed by using the inverse Laplace transform to obtain the solution of the original p.d.e.

The above procedure, which involves only five steps, can be applied to many cases of groundwater flow to wells. This generic procedure is demonstrated in this lesson using two examples of transient groundwater flow to wells in a leaky confined aquifer.

9.4 Illustrative Examples

9.4.1 Example 1: Transient Groundwater Flow to a Partially Penetrating Well in Leaky Confined Aquifers

Occurrence of leakage flow from or into the aquifer being pumped is a function of the potential difference between the aquifer and the aquifer on the other side of the aquitard through which leakage takes place. Thus, in the case of leaky aquifers, while there is a scope for recharge to the aquifer being pumped, the flow continues to be transient unless the rate of recharge is equal to the rate of pumping.

Considering radial symmetry of flow towards wells and the basic assumptions mentioned in Section 9.2, except for the assumption 'the pumping well is fully penetrating' and referring to Fig. 9.1, the governing p.d.e. for the case of transient flow of groundwater towards a partially penetrating pumping well in a homogeneous and isotropic leaky confined aquifer of infinite aerial extent can be written as:

$$\frac{\partial^2 s(r,z,t)}{\partial r^2}+\left(\frac{1}{r}\right)\frac{\partial s(r,z,t)}{\partial r}+\frac{\partial^2 s(r,z,t)}{\partial z^2}-\left(\frac{1}{B^2}\right)s(r,z,t)=\left(\frac{S}{T}\right)\frac{\partial s(r,z,t)}{\partial t} \qquad (9.1)$$

Euation (9.1) is subject to the following initial and boundary conditions:

Initial Condition (I.C.): $s(r,z,0)=s(\infty,z,t)$

Boundary Conditions (B.C.): $\frac{\partial s}{\partial r}(r,0,t)=\frac{\partial s}{\partial r}(r,b,t)=0$

and

$$(d-\ell)r\left.\frac{\partial s(r,z,t)}{\partial r}\right|_{r=r_w}=\begin{cases}-\left(\frac{Q}{2\pi K}\right) & \ell\le z\le d\\ 0 & \text{Otherwise}\end{cases} \qquad (9.2)$$

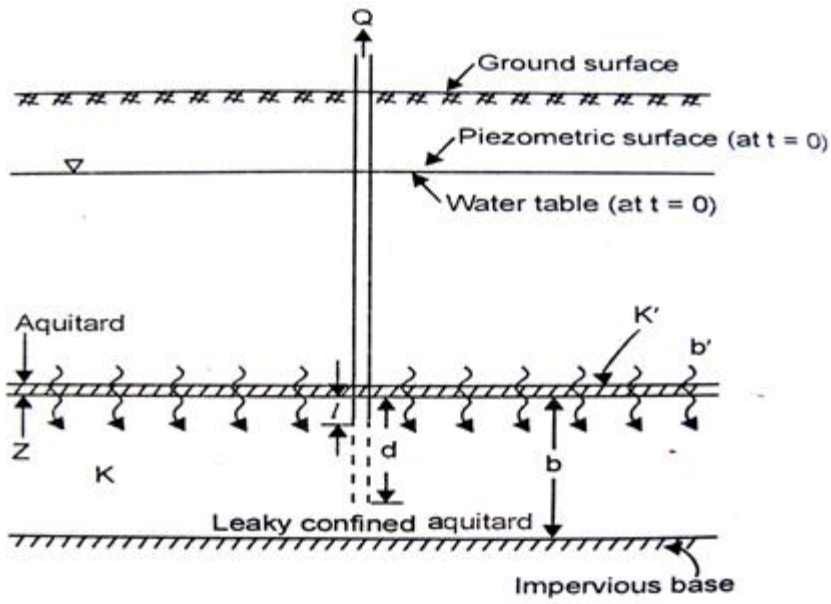


Fig. 9.1. Transient flow towards a partially penetrating well in a leaky confined aquifer. (Source: Sarma, 2009)

Where, s = drawdown in the leaky confined aquifer, r = radial distance from the pumping well, Q = constant pumping rate, r_w = radius of the pumping well, B = leakge factor, T = transmissivity of the leaky confined aquifer, S = storage coefficient of the leaky confined aquifer, b = thickness of the leaky confined aquifer, K = hydraulic conductivity of the leaky confined aquifer, K′ = hydraulic conductivity of the aquitard (leaky confining layer), b′ = thickness of the aquitard, and the meaning of other symbols are as illustrated in Fig. 9.1.

Step 1: Since the p.d.e. [Eqn. (9.1)] describes transient flow, applying the first step of taking Laplace transform of the p.d.e. and the associated initial and boundary conditions [Eqn. (9.2)] leads to:

$$\frac{\partial^2 \bar{s}(r,z)}{\partial r^2} + \frac{1}{r}\frac{\partial \bar{s}(r,z)}{\partial r} + \frac{\partial^2 \bar{s}(r,z)}{\partial z^2} - \left(\frac{1}{B^2} + p\frac{S}{T}\right)\bar{s}(r,z) = 0 \tag{9.3}$$

Subject to transformed initial condition and boundary conditions given as:

$$\bar{s}(r,z) = s(\infty,z) = 0$$

$$\frac{\partial \bar{s}}{\partial r}(r,0) = \frac{\partial \bar{s}}{\partial r}(r,b) = 0$$

$$(d-\ell)\,r\left.\frac{\partial \bar{s}(r,z)}{\partial r}\right|_{r=r_w} = \begin{cases} -(1/p)(Q/2\pi K); & \ell \le z \le d \\ 0 & \text{Otherwise} \end{cases} \qquad (9.4)$$

Step 2: Equation (9.3) is a p.d.e. in two space dimensions. Further the flow is such that the flow domain (Fig. 9.1) has two parallel no-flow boundaries (at z = 0, and z = b). Hence, taking the Finite Fourier cosine transform for the p.d.e. [Eqn. (9.3)] and the associated boundary conditions [Eqn. (9.4)] leads to:

$$\frac{\partial^2 \bar{s}_c(r,n)}{\partial r^2} + \left(\frac{1}{r}\right)\frac{\partial \bar{s}_c(r,n)}{\partial r} - \left[\left(\frac{n\pi}{b}\right)^2 \bar{s}_c(r,n) + (-1)^n s_c(r,b) - s_c(r,0)\right]$$
$$= \left[\left(\frac{1}{B^2}\right) + p\,\frac{S}{T}\right]\bar{s}_c(r,n)$$

Or,

$$\frac{\partial^2 \bar{s}_c(r,n)}{\partial r^2} + \left(\frac{1}{r}\right)\frac{\partial \bar{s}_c(r,n)}{\partial r} - \lambda^2\,\bar{s}_c(r,n) = 0 \qquad (9.5)$$

In which, $\lambda^2 = \left[\left(\frac{n\pi}{b}\right)^2 + \left(\frac{1}{B^2}\right) + p\,\frac{S}{T}\right]$.

Equation (9.5) is subject to:

$$\bar{s}_c(\infty,n) = 0$$

and

$$(d-\ell)\,r\left.\frac{\partial \bar{s}(r,n)}{\partial r}\right|_{r=r_w} = \begin{cases} -(1/p)(Q/2\pi K)\int_{\ell}^{d}\cos(n\pi z/b)\,dz; & \ell \le z \le d \\ 0 & \text{Otherwise} \end{cases} \qquad (9.6)$$

Step 3: Equation (9.5) is a modified Bessel differential equation of order zero, for which the general solution is given as:

$$\bar{s}_c(r,n) = C_1 K_0(\lambda r) + C_2 I_0(\lambda r)$$

The value of the constants C_1 and C_2 can be evaluated using Eqn. (9.6) and the properties of the modified Bessel functions K_0 and I_0. Thus, $C_2 = 0$ and

$$C_1 = \frac{1}{(d-\ell)}\left(\frac{1}{p}\right)\left(\frac{Q}{2\pi K}\right)\int_{\ell}^{d}\cos\left(\frac{n\pi z}{b}\right)dz; \quad \ell \le z \le d$$

Therefore,

$$\bar{s}_c(r,n) = \left[\frac{1}{(d-\ell)}\left(\frac{Q}{2\pi K}\right)\left(\frac{1}{p}\right)\left(\frac{b}{n\pi}\right)\left[\sin\left(\frac{n\pi d}{b}\right) - \sin\left(\frac{n\pi \ell}{b}\right)\right]\left[K_0(\lambda r)\right]\right] \tag{9.7}$$

Step 4: Taking inverse finite cosine Fourier transform of Eqn. (9.7) leads to:

$$\bar{s}(r,z,p) = \left(\frac{1}{b}\right)\left[s_c(r,n)\right]\Big|_{n=0} + \left(\frac{2}{b}\right)\sum_{n=1}^{\infty}\frac{1}{n}\left[s_c(r,n)\cos\left(\frac{n\pi z}{b}\right)\right]$$

The first term on the R.H.S. (with n = 0) is equal to:

$$\left(\frac{Q}{2\pi Kb}\right)\left(\frac{1}{(d-\ell)}\left(\frac{1}{p}\right)\right)\left\{\sin\left[\frac{(n\pi d/b)}{(n\pi/b)}\right] - \sin\left[\frac{(n\pi \ell/b)}{(n\pi/b)}\right]\right\}\left[K_0(\lambda^* r)\right]$$

$$\left(\frac{1}{p}\right)\left(\frac{Q}{2\pi Kb}\right)\left[K_0(\lambda^* r)\right]$$

In which,

$$\lambda^* = \left(\frac{1}{B^2} + p\frac{S}{T}\right)^{1/2} \tag{9.8}$$

and the second term on the R.H.S. is equal to:

$$\left(\frac{2}{b}\right)\sum_{n=1}^{\infty}\left[\frac{1}{n}\left(\frac{1}{p}\right)\left(\frac{Q}{2\pi K}\right)\left(\frac{b}{n\pi}\right)\left(\frac{1}{(d-\ell)}\right)\right]$$
$$\times\left[\sin\left(\frac{n\pi d}{b}\right) - \sin\left(\frac{n\pi \ell}{b}\right)\right]\left[\cos\left(\frac{n\pi z}{b}\right)\right]\left[K_0(\lambda^* r)\right]$$

Therefore,

$$s(r,z,p)=\left[\left(\frac{1}{p}\right)\left(\frac{Q}{2\pi Kb}\right)\left[K_0(\lambda^* r)\right]+\left(\frac{2}{b}\right)\sum_{n=1}^{\infty}\left\{\left(\frac{Q}{2\pi Kb}\right)\right.\right.$$
$$\left.\left.\left(\frac{1}{p}\right)\left[\frac{1}{(d-\ell)}\right]\left(\frac{b}{n\pi}\right)\right\}\right]\frac{1}{n}\times\left[\sin\left(\frac{n\pi d}{b}\right)-\sin\left(\frac{n\pi \ell}{b}\right)\right]\times$$
$$\left[\cos\left(\frac{n\pi z}{b}\right)\right]\left[K_0(\lambda^* r)\right]$$

(9.9)

Step 5: Taking inverse Laplace transform of Eqn. (9.9), the first term becomes:

$$LT^{-1}\left\{\left[(1/p)(Q/2\pi Kb)\right]\left[K_0(\lambda^* r)\right]\right\}$$

$$=\left(\frac{Q}{2\pi Kb}\right)LT^{-1}\left\{\left(\frac{1}{p}\right)K_0\left(\frac{1}{B^2}+\frac{pS}{T}\right)^{1/2}r\right\}$$

$$=\left(\frac{Q}{2\pi Kb}\right)LT^{-1}\left\{\left(\frac{1}{p}\right)K_0\left[\left(\frac{r^2S}{T}\right)^{1/2}\left(\frac{T}{SB^2}\right)+p\right]^{1/2}\right\}$$

$$=\left(\frac{Q}{4\pi Kb}\right)\left[W\left[\left(\frac{r^2S}{4Tt}\right),\frac{r}{B}\right]\right]$$

(A)

and

$$LT^{-1}\left\{\left(\frac{2}{b}\right)\sum_{n=1}^{\infty}\left(\frac{1}{p}\right)\frac{1}{n}\left(\frac{Q}{2\pi K}\right)\left[\frac{1}{(d-\ell)}\right]\left(\frac{b}{n\pi}\right)\right\}$$
$$.\left[\sin\left(\frac{n\pi d}{b}\right)-\sin\left(\frac{n\pi \ell}{b}\right)\right]\left[\cos\left(\frac{n\pi z}{b}\right)\right]\left[K_0(\lambda^* r)\right]$$

$$=\left[\left(\frac{Q}{2\pi Kb}\right)\left(\frac{2b}{\pi(d-\ell)}\right)\right]\left[\sum_{n=1}^{n}\left(\frac{1}{n}\right)\left[\sin\left(\frac{n\pi d}{b}\right)-\sin\left(\frac{n\pi \ell}{b}\right)\right]\right.$$
$$\left.\left[\cos\left(\frac{n\pi z}{b}\right)\right]*LT^{-1}\left\{\left(\frac{1}{p}\right)\left(\frac{r^2S}{T}\right)^{1/2}\left[\left(\frac{T}{S}\right)\left(\frac{n\pi}{b}\right)^2+\left(\frac{T}{S}\right)\left(\frac{1}{B^2}\right)\right]^{1/2}\right\}\right]$$

$$= \left[\left(\frac{Q}{4\pi Kb}\right)\left(\frac{2b}{\pi(d-\ell)}\right)\right]\left\{\sum_{n=1}^{n}\left(\frac{1}{n}\right)\left[\sin\left(\frac{n\pi d}{b}\right)-\sin\left(\frac{n\pi \ell}{b}\right)\right]\right.$$

$$\left.\left[\cos\left(\frac{n\pi z}{b}\right)\right]\right\} * \left\{W\left(r^2\left(\frac{S}{4Tt}\right)\right), \left[\left(\frac{n\pi}{b}\right)^2+\left(\frac{1}{B}\right)^2\right]^{1/2}, r\right\} \qquad \text{(B)}$$

Hence, s(r,z,t) = (A) + (B)

$$= \left(\frac{Q}{4\pi Kb}\right) W\left(u, \frac{r}{B}\right) + \left\{\left(\frac{Q}{4\pi Kb}\right)\left[\frac{2b}{\pi(d-\ell)}\right]\sum\left(\frac{1}{n}\right)\left[\sin\left(\frac{n\pi d}{b}\right)\right.\right.$$

$$\left.\left.-\sin\left(\frac{n\pi \ell}{b}\right)\right].\left[\cos\left(\frac{n\pi z}{b}\right)\right]\right\} W\left\{u, \left[\left(\frac{n\pi}{b}\right)^2+\left(\frac{1}{B}\right)^2\right]^{1/2}.r\right\}$$

(9.10)

Thus, in this example, due to the nature of the groundwater flow condition, all the five steps were used to obtain the expression for drawdown around the pumping well. However, in many cases, all the five steps many not be required. Nevertheless, by following the sequence and using the relevant steps, the corresponding solution can be obtained. One such case is presented below as Example 2.

9.4.2 Example 2: Transient Flow to a Fully Penetrating Well in Leaky Confined Aquifers

The partial differential equation governing transient flow towards a pumping well fully penetrating a homogeneous and isotropic leaky confined aquifer of infinite aerial extent (Fig. 9.2) written in the radial coordinate system in terms of drawdown (s) is given as:

$$\frac{\partial^2 s(r,t)}{\partial r^2}+\frac{1}{r}\frac{\partial^2 s(r,t)}{\partial r^2}-\frac{s(r,t)}{B^2}-=\frac{S}{T}\frac{\partial s(r,t)}{\partial t} \qquad (9.11)$$

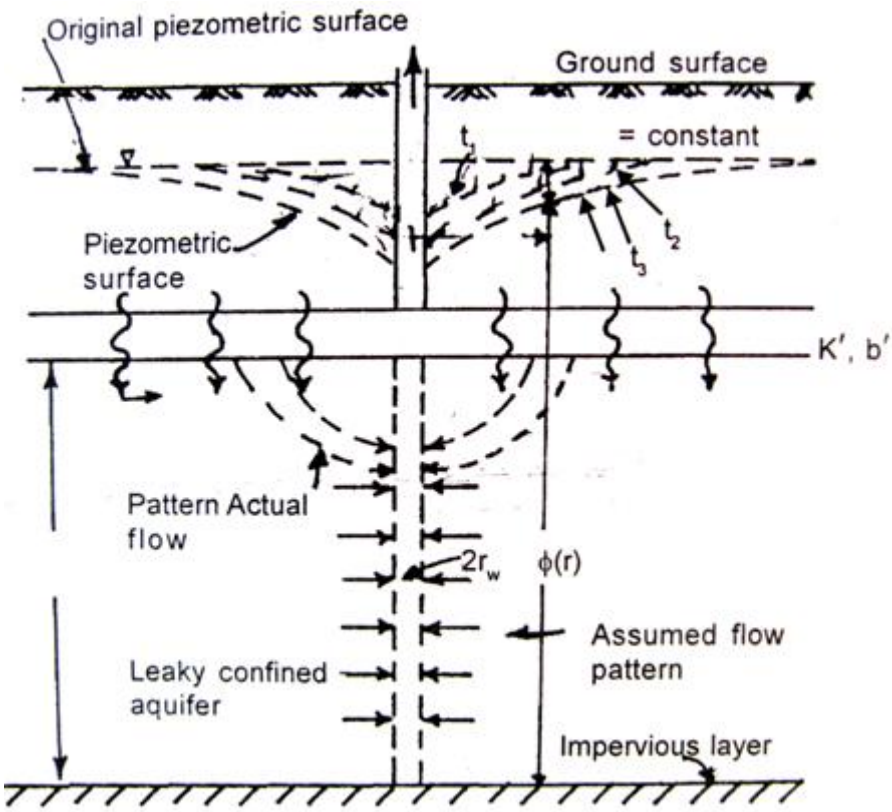


Fig. 9.2. Transient flow towards a fully penetrating well in a leaky confined aquifer. (Source: Sarma, 2009)

Equation (9.11) is subject to the following initial and boundary conditions:

Initial condition: s (r, 0) = 0

Boundary conditions: s (¥, t) = 0

$$\left.\frac{r_w\,\partial s(r,t)}{\partial r}\right|_{r=r_w} = -\left(\frac{Q}{2\pi T}\right)$$

and

Step 1: Taking Laplace transformation of the p.d.e. and the associated initial and boundary conditions leads to:

$$\frac{\partial^2 \bar{s}(r,p)}{\partial r^2} + \frac{1}{r}\frac{\partial \bar{s}(r,p)}{\partial r} - \lambda^2 \bar{s}(r,p) = 0 \qquad (9.12)$$

Subject to:

$$\bar{s}(\infty,p)=0 \quad \text{and} \quad r\frac{\partial \bar{s}}{\partial r}(r,p)\bigg|_{r=r_w}=-\left(\frac{1}{p}\right)\left(\frac{Q}{2\pi T}\right) \tag{9.13}$$

Where,

$$\lambda^2=\left[p\frac{S}{T}+\frac{1}{B^2}\right].$$

Step 2: This step is not required in this case because the p.d.e. is a function of s and r only.

Step 3: Equation (9.12) is a modified Bessel differential equation of zero order and of first kind, the solution of which is familiar.

Step 4: Evaluation of Eqn. (9.12) in terms of the boundary conditions leads to:

$$\bar{s}(r,p)=\left(\frac{1}{p}\right)\frac{Q}{2\pi T}K_0(\lambda r) \tag{9.14}$$

Step 5: Taking the inverse Laplace transform of Eqn. (9.14) leads to the solution of the original problem as follows:

$$s(r,t)=LT^{-1}\left\{\left(\frac{Q}{2\pi T}\right)\left(\frac{1}{p}\right)K_0\left[r\left(\frac{pS}{T}+\frac{1}{B^2}\right)^{1/2}\right]\right\}$$

Or,

$$s(r,t)=\frac{Q}{4\pi T}W\left(u,\frac{r}{B}\right) \tag{9.15}$$

Where,

$$u=\left(\frac{r^2S}{4Tt}\right).$$

Equation (9.15) is known as Hantush-Jacob formula, which is described in Lesson 11. This equation is limited to the conditions that the leakage derived from the storage of the aquitard is negligible and the r/B ratio is small, preferably less than 0.1. It can be seen that Eqn. (9.15) reduces readily to the solution of the case of non-leaky confined aquifer (i.e., Theis equation), if B is set to a very large value (¥).

Note thatt in this example, only four steps are required to obtain the solution. For the further details on the derivation of analytical solutions to various types of well hydraulics problems, interested readers are referred to Sarma (2009) and Batu (1998).

Lesson 10 Steady Groundwater Flow to Wells

10.1 Introduction

When a well is pumped, water flows into the well from the surrounding aquifer because of difference in hydraulic heads at the well and in the aquifer caused by pumping. Before pumping, water level in the well stands at a height theoretically equal to the static water pressure in the saturated layer around the well. This water level is known as 'static water level' or 'pre-pumping water level' (Fig. 10.1). When pumping starts, water is removed from the aquifer surrounding the well and the water level in the well ['piezometric level' in case of confined aquifers (Fig. 10.1a) and 'water table' in case of unconfined aquifers (Fig. 10.1b)] starts lowering. The water level in the well at any instant during pumping is known as 'pumping water level'.

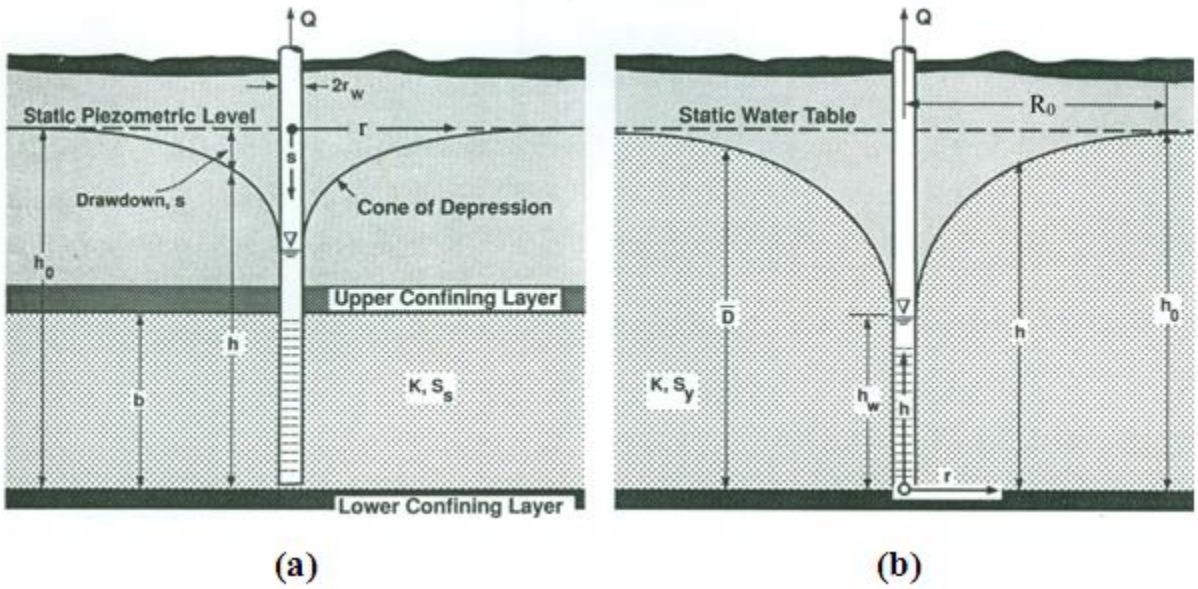


Fig. 10.1. Drawdown pattern in: (a) Confined aquifer; (b) Unconfined aquifer. (Source: Roscoe Moss Company, 1990)

The difference between the static water level and the pumping water level at any instant is called 'drawdown', which is a function of pumping rate, pumping duration and distance from the pumping well. Drawdown is always maximum at the pumping well and it decreases with an increase in the distance from pumping well (Fig. 10.1). The rate of pumping from an aquifer significantly affects the hydraulic gradient in the aquifer. The faster the well is pumped, the steeper the hydraulic gradient will be in the vicinity of the well.

A drawdown curve at a given time shows the variation of drawdown with distance from the pumping well. In three dimensions, the drawdown curve takes the shape of an inverted cone centered on the pumping well, which is known as cone of depression. The outer limit of the cone of depression defines the area of influence of the well. The boundary of the area of influence is called circle of influence and the radius of the circle of influence is called radius

of influence. Thus, the radius of influence (R_0) is the distance from a pumping well to the edge of the cone of depression (Fig. 10.1) where drawdown is zero.

As more and more groundwater is pumped through the well, the more water comes from aquifer storage. As a result, the radius of influence increases until when the rate of pumping (discharge) becomes equal to the rate of flow into the well from the area around the well. At this instant of time, a steady flow condition exists in the aquifer and the cone of depression gets stabilized (i.e., it does not change with pumping time). This equilibrium condition changes when the pumping rate is increased or decreased. Note that under steady-state conditions, the entire pumped water is assumed to be coming from external sources beyond the radius of influence. In contrast, under unsteady-state (transient-flow) conditions, either entire pumped water is assumed to be coming from the aquifer storage within the radius of influence or the pumped water is assumed to be coming partly from the aquifer storage within the radius of influence and partly from external sources beyond the radius of influence depending on field conditions.

In this lesson, the main equations for analyzing steady flow to fully penetrating wells in confined and unconfined aquifers are derived and their applications are discussed. In addition, the concept of partial penetration and the equations for computing steady drawdown in partially penetrating wells installed in confined and unconfined aquifers are presented.

10.2 Steady Flow to Fully Penetrating Wells

10.2.1 Basic Assumptions for Analyzing Flow to Wells

The following are the basic assumptions made for analyzing flow to wells:

1. The aquifer is bounded on the bottom by a confining layer.
2. The groundwater level of the aquifer is horizontal and not changing with time prior to the start of pumping.
3. All changes in the position of the groundwater level are due to the effect of the pumping well alone.
4. All geologic formations are horizontal and of infinite horizontal extent.
5. The aquifer is homogeneous and isotropic.
6. All flow is radial towards the well.
7. Groundwater flow is horizontal.
8. Darcy's law is valid.
9. The pumping well and the observation wells are fully penetrating.
10. The pumping well has an infinitesimal diameter (i.e., negligible storage) and is 100% efficient (i.e., no well losses).
11. Groundwater has a constant density and viscosity.

10.2.2 Radial Symmetry

The flow towards a well is termed radial flow. Radial flow can be thought of as flow along the spokes of a wagon wheel towards the hub. Besides the above-mentioned basic assumptions, it is also assumed that flow to wells is radially symmetric. This means that the values of aquifer transmissivity (T) and storage coefficient (S) do not depend on the direction

of flow in the aquifer. Although solutions can be found for cases in which the value of the horizontal hydraulic conductivity is different from that of the vertical hydraulic conductivity, it usually assumed that the aquifer has radial symmetry.

10.2.3 Steady-State Condition

As mentioned above, if pumping is continued for a long time, the groundwater level may reach a state of equilibrium, i.e., no change in drawdown with time. When the equilibrium (steady-state) condition is achieved, the cone of depression stops growing because it reaches a recharge boundary which contributes all flow to the well. The hydraulic gradient of the cone of depression causes water to flow at a constant rate from the recharge boundary to the well. Such a hydraulic condition in the aquifer is known as a steady flow condition. The assumption of radial symmetry means that the recharge boundary has an unlikely circular geometry entered about the pumping well! In this lesson, our focus is to analyze the steady flow to pumping wells, and hence it is also assumed that flow towards the well is under steady-state conditions.

10.2.4 Steady Radial Flow in Confined Aquifers

For analyzing steady radial flow in a confined aquifer, apart from the above assumptions, the following additional assumptions are necessary:

- The aquifer is bounded on the top and bottom by impervious confining layers (i.e., there is no leakage through confining layers).
- The well is pumped at a constant rate.

Figure 10.2 shows a pumping well fully penetrating a confined aquifer and is subjected to pumping. Groundwater level under equilibrium conditions is also depicted. Under equilibrium (steady-state) conditions, the rate of pumping (Q) is equal to the rate that the aquifer transmits water to the well. This problem was first solved by G. Thiem in 1906, which is presented below.

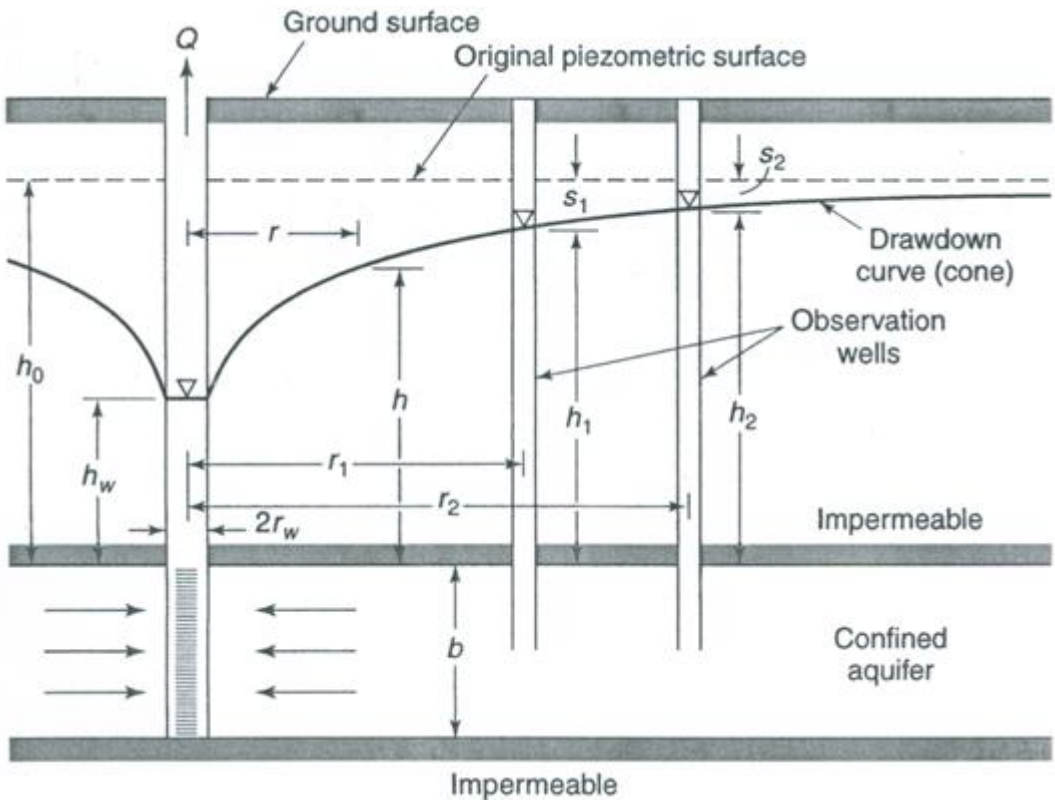


Fig. 10.2. Steady flow to a pumping well in a confined aquifer.

(Source: Todd, 1980)

From the Darcy's law, the flow of water through a circular section of the aquifer towards the pumping well is given as:

$$Q = (2\pi rb) \times K \times \frac{dh}{dr} \tag{10.1}$$

Where, Q = constant rate of pumping from the well, r = radial distance from the circular section to the pumping well, b = thickness of the confined aquifer, K = hydraulic conductivity of the confined aquifer, and dh/dr = hydraulic gradient.

Since transmissivity (T) is the product of aquifer thickness (b) and hydraulic conductivity (K), Eqn. (10.1) can be expressed as:

$$Q = 2\pi rT \times \left(\frac{dh}{dr}\right) \tag{10.2}$$

Eqn. (10.2) can be rearranged as follows:

$$dh = \frac{Q}{2\pi T} \times \frac{dr}{r} \qquad (10.3)$$

Let's consider that two observation wells are installed in the aquifer at distances r_1 and r_2 from the pumping well, respectively with hydraulic heads h_1 and h_2 (Fig. 10.2). Now, integrating Eqn. (10.3) with these boundary conditions, we have:

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

$$\Rightarrow \quad h_2 - h_1 = \frac{Q}{2\pi T} \times \ln\left(\frac{r_2}{r_1}\right) \qquad (10.4)$$

$$\therefore \quad Q = 2\pi T \times \frac{h_2 - h_1}{\ln\left(\frac{r_2}{r_1}\right)} \qquad (10.5)$$

In practice, instead of the hydraulic head (h), drawdown (s) is measured, and hence after expressing h_1 and h_2 as drawdowns, Eqn. (10.5) can be written as:

$$Q = 2\pi T \times \frac{(s_1 - s_2)}{\ln\left(\frac{r_2}{r_1}\right)} \qquad (10.6)$$

Where, s_1 and s_2 are steady drawdowns in the two observation wells located respectively at r_1 and r_2 distances from the pumping well.

Eqn. (10.5) or (10.6) is known as the Thiem equation or equilibrium equation for confined aquifers. It is evident from the Thiem equation that the drawdown varies with the logarithm of the distance from the pumping well. Thiem equation can be used to compute constant

pumping rate (Q), steady drawdown (s_1 or s_2) or distance of the point of observation from the pumping well (r_1 or r_2) if the aquifer parameter T and other variables are known. Additionally, this equation can also be used to calculate aquifer parameter T (or, K if the aquifer thickness is given) if other variables are known. Note that Eqn. (10.5) or (10.6) does not contain aquifer storage parameter (S); this is due to the fact that under steady-state conditions, there is no change in hydraulic head with time which means water is not coming from the aquifer storage.

Furthermore, if we consider that the first observation well is located at a distance r_w (radius of the pumping well) where hydraulic head is h_w and instead of the second observation well at r_2, we consider that $r_2 = R_0$ (radius of influence) where drawdown (s_2) is zero and hence hydraulic head is h_0 (static or pre-pumping groundwater level), the Thiem equation can be expressed as follows:

Eqn. (10.5) becomes

$$Q = 2\pi T \times \frac{h_0 - h_w}{\ln\left(\frac{R_0}{r_w}\right)} \qquad (10.7)$$

and Eqn. (10.6) becomes

$$Q = 2\pi \mathrm{T} \times \frac{s_w}{\ln\left(\frac{R_0}{r_w}\right)} \qquad (10.8)$$

Where, s_w denotes the drawdown in the aquifer at a distance r_w (i.e., at the well face). Since the pumping well is assumed to be 100% efficient (ideal condition), well losses can be neglected which enables us to take the drawdown in a pumping well equal to the drawdown at the well face (s_w).

The above form of the Thiem equation [Eqn. (10.7) or (10.8)] is practically very useful because unlike the earlier form of this equation [Eqn. (10.5) or (10.6)], it requires only one observation well's data. Also, Eqn. (10.7) or (10.8) facilitates to determine radius of influence (R_0) of the well.

Finally, it is worth mentioning that drawdown changes gradually with time and equilibrium (steady-state) condition rarely exists under real field conditions. Therefore, when the difference in drawdowns (s_1-s_2) becomes essentially constant while both values are still increasing, it is assumed to be quasi-steady- state condition. Thus, the Thiem equation generally gives good results after only a few days of pumping.

10.2.5 Steady Radial Flow in Unconfined Aquifers

As we know that the analysis of flow in unconfined aquifers is more complicated than that in confined aquifers. Thiem also derived an equation for steady radial flow in unconfined aquifers which is discussed in this section. Besides the basic assumptions and the assumptions of radial symmetry and steady-state condition mentioned above, the following additional assumptions are made in this case:

(1) The aquifer is unconfined and underlain by a horizontal confining layer.

(2) The well is pumped at a constant rate.

(3) The Dupuit-Forchheimer assumptions are valid.

Using the Darcy's law, the radial flow in the unconfined aquifer (Fig. 10.3) can be described as:

$$Q = (2\pi rh) \times K \times \frac{dh}{dr} \qquad (10.9)$$

Where, Q = Constant rate of pumping, r = radial distance from the circular section to the pumping well, h = saturated thickness of the unconfined aquifer, K = hydraulic conductivity of the unconfined aquifer, and dh/dr = hydraulic gradient.

Arranging Eqn. (10.9), we have:

$$hdh = \frac{Q}{2\pi K} \times \frac{dr}{r} \qquad (10.10)$$

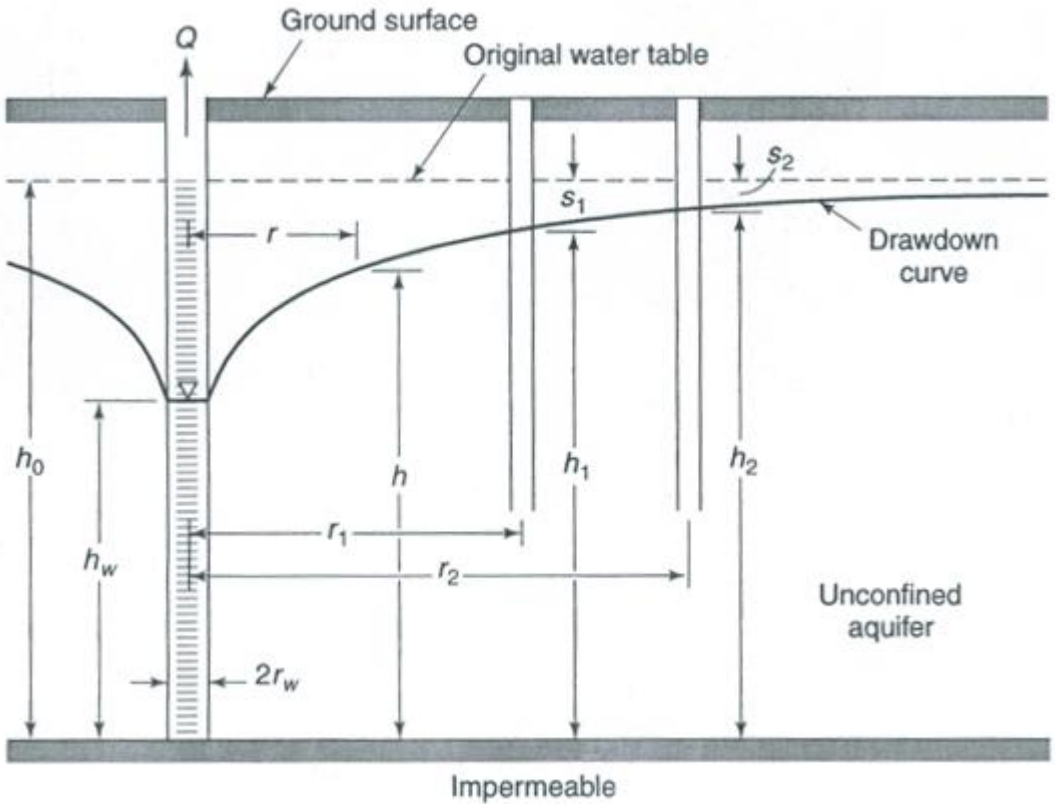


Fig. 10.3. Steady flow to a pumping well in an unconfined aquifer.

(Source: Todd, 1980)

Let's consider that two observation wells are located in the unconfined aquifer at distances r_1 and r_2, with hydraulic heads h_1 and h_2, respectively (Fig. 10.3). Now, Eqn. (10.10) can be integrated with these boundary conditions as:

$$\int_{h_1}^{h_2} h\,dh = \frac{Q}{2\pi K}\int_{r_1}^{r_2}\frac{dr}{r} \qquad (10.11)$$

$$\Rightarrow \quad h_2^2 - h_1^2 = \frac{Q}{\pi K}\times \ln\left(\frac{r_2}{r_1}\right)$$

$$\therefore \quad Q = \pi K \times \frac{h_2^2 - h_1^2}{\ln\left(\frac{r_2}{r_1}\right)} \qquad (10.12)$$

Like the Thiem equation for confined aquifers, Eqn. (10.12) can also written in the form of drawdowns as follows:

$$Q = \pi K \times \frac{(h_1 + h_2) \times (s_1 - s_2)}{\ln\left(\frac{r_2}{r_1}\right)} \qquad (10.13)$$

Where, s_1 and s_2 are steady drawdowns in the two observation wells located respectively at r_1 and r_2 distances from the pumping well.

Eqn. (10.12) or (10.13) is called Thiem equation for unconfined aquifers or Dupuit's equation. Like the Thiem equation for confined aquifers, this equation [Eqn. (10.12) or (10.13)] can also be used to compute Q, steady drawdowns (s_1 or s_2), distance of the point of observation from the pumping well (r_1 or r_2), or K depending on the known variables and parameters.

Note that Eqn. (10.12) or (10.13) fails to describe accurately the drawdown curve near the pumping well because the large vertical flow components contradict the Dupuit-Forchheimer assumptions. However, the estimates of K for given hydraulic heads are good. In practice, the values of drawdowns should be small compared to the saturated thickness of the unconfined aquifer so as to ensure reliable use of Eqn. (10.12) or (10.3). To assess this condition, the ratio $\frac{s_{max}}{h_0}$ (where s_{max} is the maximum drawdown in the unconfined aquifer and h_0 is the initial saturated thickness of the unconfined aquifer) is used. Two cases can arise, which are described below.

Case A: If $\frac{s_{max}}{h_0}$ is less than or equal to 0.02, the drawdown in the unconfined aquifer can be considered very small compared to its thickness. In this case, Eqn. (10.12) or (10.13) can be used reliably. Alternatively, even the original Thiem equation [Eqn. (10.5), (10.6), (10.7) or (10.8)] can be used without significant errors. While applying Eqn. (10.5), (10.6), (10.7) or (10.8) to the unconfined aquifer problems, T is replaced with Kh_0 and the remaining variables/parameters are treated in the same manner as in case of confined aquifers.

Case B: If $\frac{s_{max}}{h_0}$ is greater than 0.02, the drawdown in the unconfined aquifer is considered appreciable compared to its thickness. In this case also, the original Thiem equation [Eqn. (10.6) or (10.8)] can be used, but the observed drawdowns of the unconfined aquifer need to be corrected so that they can be representative for the equivalent confined aquifer. The following formula is used to correct the drawdowns of the unconfined aquifer:

$$s' = s - \frac{s^2}{2h_0} \qquad (10.14)$$

Where, s′ = corrected drawdown (drawdown for the equivalent confined aquifer), s = drawdown observed in the unconfined aquifer, and h_0 = initial saturated thickness of the unconfined aquifer.

Thus, using the corrected drawdowns, the original Thiem equation [Eqn. (10.6) or (10.8)] can be reliably applied to solve unconfined aquifer problems. Note that in this case, if one wishes to apply the Dupuit's equation for unconfined aquifers [i.e., Eqn. (10.13)], h_1 and h_2 of Eqn. (10.13) have to be replaced with (h_0-s_1) and (h_0-s_2), respectively. Thus, Eqn. (10.13) becomes:

$$Q = \pi K \times \frac{\left[(s_1 - \frac{s_1^2}{2h_0}) - (s_2 - \frac{s_2^2}{2h_0}) \right] \times 2h_0}{\ln\left(\frac{r_2}{r_1}\right)} \qquad (10.15)$$

It is clear that Eqn. (10.15) is similar to Eqn. (10.6), if T of Eqn. (10.6) is replaced with Kh_0 and s_1 and s_2 of Eqn. (10.6) are replaced with '$s_1 - \frac{s_1^2}{2h_0}$' and '$s_2 - \frac{s_2^2}{2h_0}$', respectively according to Eqn. (10.14). Thus, the use of the original Thiem equation [Eqn. (10.6) or (10.8)] for solving unconfined aquifer problems even in the situation when drawdowns are considerably large is correct and justified provided that the observed drawdowns are corrected according to Eqn. (10.14).

Moreover, if the drawdown in the unconfined aquifer is to be calculated/predicted using the original Thiem equation [Eqn. (10.6) or (10.8)], firstly drawdown at a given location is calculated assuming that the aquifer is confined. Thereafter, the calculated drawdown () is corrected using the following equation to obtain the actual drawdown (s) in the unconfined aquifer:

$$s = h_0 - \left(h_0^2 - 2s'h_0\right)^{0.5} \qquad (10.16)$$

10.3 Steady Flow to Partially Penetrating Wells

10.3.1 What is a Partial Penetrating Well?

A well (pumping well or observation well) having screen length less than the aquifer thickness is known as a partially penetrating well. The flow pattern to such wells significantly differs from the flow pattern around the fully penetrating wells.

In many cases, the open hole or the well screen of a pumping/observation well does not extend from the top to the bottom of an aquifer. In all of the cases considered earlier, it was assumed that the well penetrates the entire saturated thickness of the aquifer. This causes flow in the aquifer to be essentially horizontal. However, the flow towards a partially penetrating well is three dimensional because of vertical flow components near the well (Fig. 10.4).

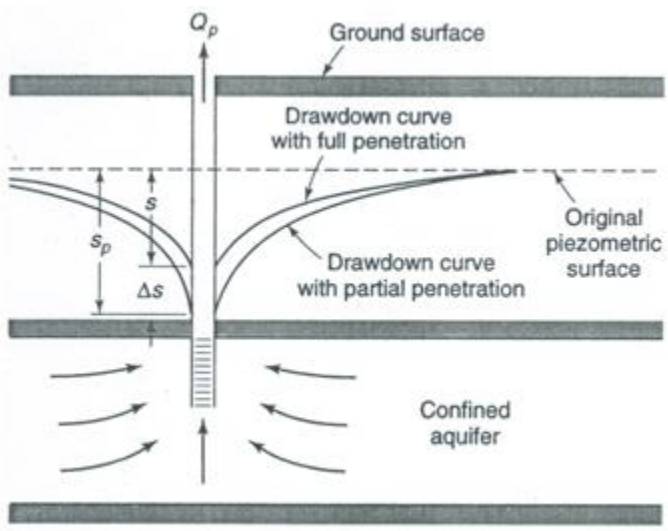


Fig. 10.4. Effect of a partially penetrating well on drawdown.

(Source: Todd, 1980)

10.3.2 Effect of Partially Penetrating Wells

It is obvious from Fig. 10.4 that the average length of flow lines into a partially penetrating well exceeds that into a fully penetrating well. Therefore, a greater resistance to flow is encountered, which results in increased drawdowns. In addition, if the aquifer is anisotropic, the values of the vertical hydraulic conductivity (K_v) and the horizontal hydraulic conductivity (K_h) are important. This will affect both the amount of water pumped from the well and the potential field caused by drawdown. For practical purposes, the following relationships exist between two similar wells - one partially penetrating and another fully penetrating the same aquifer:

If $Q_p = Q$, then $s_p > s$ (10.17)

and if $s_p = s$, then $Q_p < Q$ (10.18)

Where, Q = well discharge, s = well drawdown, and the subscript 'p' refers to the partial penetrating well.

Detailed methods for analyzing the effects of partial penetration on well flow for steady and transient conditions in confined, leaky confined, unconfined, and anisotropic aquifer systems are presented by Hantush (1966), Hantush and Thomas (1966) and others. The calculation of steady drawdowns due to the pumping of partially penetrating wells in homogeneous and isotropic confined and unconfined aquifer systems is discussed in the subsequent section.

10.3.3 Partially Penetrating Well in Confined Aquifers

The drawdown (s_p) at the well face of a partially penetrating well in a confined aquifer is expressed as (Todd, 1980):

$$s_p = s + \Delta s \qquad (10.19)$$

Where, s = drawdown with full penetration, and Ds = additional drawdown due to the partial penetration. Note that here the value of s can be calculated using the Thiem equation described above.

For steady-state flow towards a well in a confined aquifer with the screen starting from the top of the aquifer as shown in Fig. 10.5(a), the additional drawdown due to the partial penetration (Ds) can be given as (Todd, 1980):

$$\Delta s = \frac{Q_p}{2\pi T} \times \left(\frac{1-p}{p}\right) \times \ln\left[\frac{(1-p)h_s}{r_w}\right] \qquad (10.20)$$

Where, T = aquifer transmissivity, p = penetration fraction (i.e., $p = h_s/b$), h_s = screen length, and r_w = radius of the well.

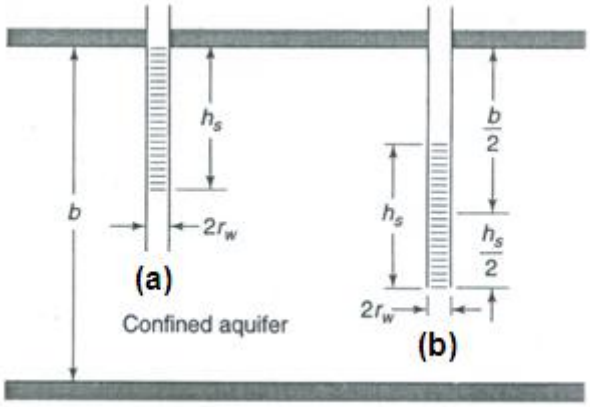


Fig. 10.5. Two basic configurations of well screen. (Source: Todd, 1980)

Equation (10.20) is valid for p > 0.20. Note that the increase in drawdown is the same whether the screen of a partially penetrating well starts from the top or from the bottom of the aquifer (Todd, 1980).

On the other hand, for the case of a well installed in a confined aquifer with the screen located in the center of the aquifer thickness [Fig. 10.5(b)], the value of Ds is given as (Todd, 1980):

$$\Delta s = \frac{Q_p}{2\pi T} \times \left(\frac{1-p}{p}\right) \times \ln\left[\frac{(1-p)h_s}{2r_w}\right] \qquad (10.21)$$

10.3.4 Partially Penetrating Well in Unconfined Aquifers

The drawdown (s_p) at the well face of a partially penetrating well in an unconfined aquifer can be expressed as (Todd, 1980):

$$s_p^2 = s^2 + \Delta s 2h_0 \qquad (10.22)$$

Where, h_0 = initial saturated thickness of the unconfined aquifer. In this case also, the value of s can be calculated using the Thiem equation described above.

For a partially penetrating well in an unconfined aquifer with the well screen location as shown in Fig. 10.5(a), Eqn. (10.20) can be modified as follows (Todd, 1980):

$$\Delta s2h_0 = \frac{Q_p}{\pi K} \times \left(\frac{1-p}{p}\right) \times \ln\left[\frac{(1-p)h_s}{r_w}\right] \qquad (10.23)$$

Where, h_0 = initial saturated thickness of the unconfined aquifer, and K = T/h_0.

Similarly, Eqn. (10.21) can be modified for a partially penetrating well in an unconfined aquifer with the screen located in the center of the aquifer thickness as follows (Todd, 1980):

$$\Delta s2h_0 = \frac{Q_p}{\pi K} \times \left(\frac{1-p}{p}\right) \times \ln\left[\frac{(1-p)h_s}{2r_w}\right] \qquad (10.24)$$

Given the values of s and $Ds2h_0$, the drawdown (s_p) at the well face of a partially penetrating well in an unconfined aquifer can be calculated using Eqn. (10.22).

10.3.5 Tips for Handling Partial Penetration Problem

Although the evaluation of the effects of partial penetration is complicated except for the simplest cases, common field situations often reduce the practical importance of partial penetration. For example, any well with 85% or more open or screened hole in the saturated aquifer thickness may be practically considered as fully penetrating (Todd, 1980). If an observation well fully penetrates an anisotropic aquifer, or if a partially penetrating observation well is located at a distance greater than from the pumping well, the effect of a partially penetrating pumping well is negligible (Hantush, 1964). However, if the pumping well is partially penetrating, and the observation wells are also partially penetrating and located closer to the pumping well than, then the drawdown formula is different and highly complex. Hantush (1961, 1964) analyzed the drawdown behavior in such observation wells and provided analytical solutions for some specific cases. Further, the effects of partial penetration of a well pumping from an unconfined aquifer are discussed in Neuman (1974).

Moreover, for the many alluvial aquifer systems having pronounced anisotropy, the vertical flow component becomes very small. Hence, a partially penetrating pumping well can be approximated as a fully penetrating well in a confined or leaky confined aquifer with a saturated thickness equal to the length of the well screen (Todd, 1980).

10.4 Steady Flow to Cavity Wells

Cavity well is a special type of tubewell constructed in the confined aquifer, which has no strainer. It draws water through the cavity formed in the aquifer just below the upper confining layer of the confined aquifer (Fig. 10.6). It is not very deep and requires a thick clay layer (or, rock) just above the aquifer layer to form a strong and reliable roof above the cavity. The hydraulics of cavity wells under steady-state conditions was developed by Mishra et al. (1970) and is described in Michael and Khepar (1999).

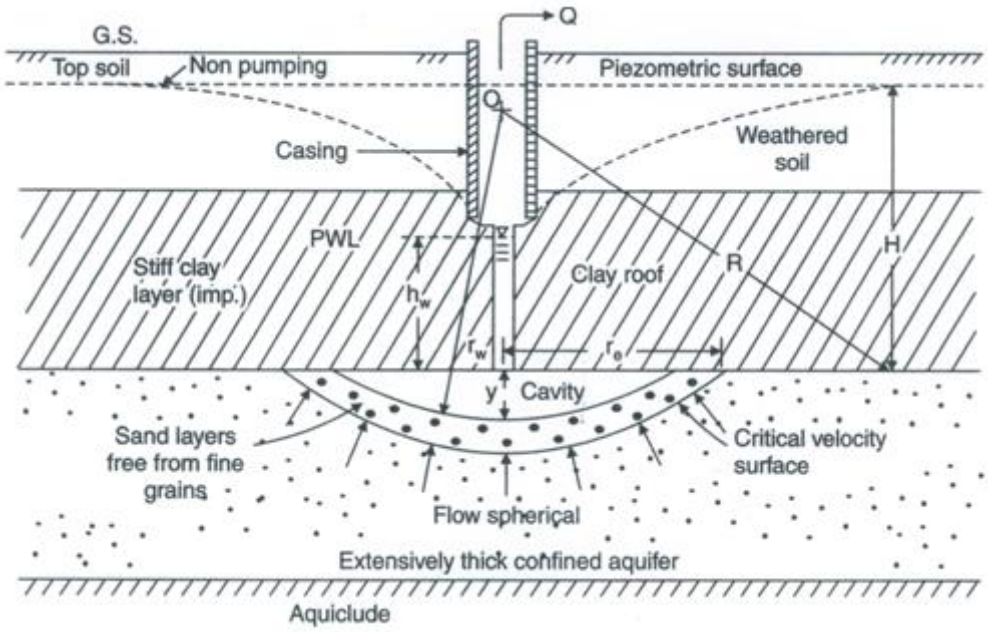


Fig. 10.6. Schematic diagram of a cavity well subject to pumping.

(Source: Raghunath, 2007)

Assuming the cavity to be segment of a sphere of radius r_w resting on the top of a homogeneous and isotropic confined aquifer of infinite areal extent and extensive thickness, the discharge of the cavity well under steady flow conditions is given as (Raghunath, 2007):

$$Q = \frac{2\pi K y (H - h_w)}{1 - \frac{r_w}{R}} \qquad (10.25)$$

Where, Q = constant rate of pumping (well discharge), K = hydraulic conductivity of the confined aquifer, y = depth of the cavity at the center, H = initial/pre-pumping hydraulic head (or, hydraulic head at radius of influence), R = radius of influence of the cavity well, and h_w = hydraulic head at a distance r_w (radius of the cavity).

Further, the width of the cavity (r_e) is given as (Raghunath, 2007):

$$r_e = \sqrt{(2r_w - y)y} \qquad (10.26)$$

Lesson 11 Unsteady Groundwater Flow to Wells

11.1 Introduction

This lesson focuses on the analysis of unsteady (transient) groundwater flow to pumping wells. The steady-state (equilibrium) flow condition mentioned in Lesson 10 is very difficult to achieve under field conditions; it is a rare and transitory incidence. As a result, flow to pumping wells is mostly unsteady (transient). The analysis of unsteady groundwater flow to pumping wells is more complicated than that of the steady groundwater flow to pumping wells. As discussed in Lesson 10, the groundwater flow to pumping wells occurs in radial directions and it is assumed to be radially symmetric. For writing governing equations for axisymmetric groundwater flow to pumping wells, polar coordinates are preferred. It can be shown that the three-dimensional transient groundwater flow equation for homogeneous and isotropic confined aquifer systems [Eqn. (5.19) of Lesson 5] can be expressed in the polar coordinate form as follows:

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} = \frac{S}{T}\frac{\partial h}{\partial t} \qquad (11.1)$$

Where, h = hydraulic head, r = radial distance from the pumping well, S = storage coefficient of the aquifer, T = transmissivity of the aquifer, and t = time.

For steady groundwater flow to pumping wells in a homogeneous and isotropic confined aquifer system, Eqn. (11.1) reduces to the Laplace equation in the polar coordinate system:

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} = 0 \qquad (11.2)$$

Moreover, for transient axisymmetric groundwater flow to pumping wells in a homogeneous and isotropic leaky confined aquifer system, Eqn. (5.49) of Lesson 5 can be expressed in the polar coordinate form as:

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} + \frac{R_L}{T} = \frac{S}{T}\frac{\partial h}{\partial t} \qquad (11.3)$$

For steady axisymmetric groundwater flow to pumping wells in a homogeneous and isotropic leaky confined aquifer system, Eqn. (11.3) reduces to:

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} + \frac{R_L}{T} = 0 \qquad (11.4)$$

Where, R_L is the leakage rate, which can be determined by the Darcy's law as:

$$R_L = K'\left(\frac{h_1 - h_2}{b'}\right) \quad (11.5)$$

Where, h_1 = hydraulic head at the top of the aquitard (leaky confining layer), h_2 = hydraulic head in the aquifer just below the aquitard, K¢ = vertical hydraulic conductivity of the aquitard, and b¢ = thickness of the aquitard.

Solutions of the above transient radial flow equations for a variety of boundary conditions have yielded a number of useful equations. These solutions have been obtained by using Laplace transforms, finite Fourier transforms, Bessel functions, and error functions (e.g., Sarma, 2009; Batu, 1998). These solutions can be used to calculate transient drawdown in a pumping well and/or nearby observation wells or discharge of the pumping well, if the aquifer parameters and the remaining variables are known. Conversely, if aquifer parameters are unknown, pumping test (described in Lesson 13) can be conducted and the observed time-drawdown data or time-recovery data can be analyzed to determine hydraulic parameters of different aquifer systems.

11.2 Solution of Unsteady Flow to Wells in Confined Aquifers

11.2.1 Theis Equation

Theis (1935) was the pioneer in solving Eqn. (11.1) and obtained an analytical solution for the transient flow of groundwater to pumping wells tapping a confined aquifer (Fig. 11.1). For solving Eqn. (11.1), Theis made the following assumptions in addition to the basic assumptions mentioned in Lesson 10:

(1) The aquifer is confined (i.e., it is bounded on the top and bottom by confining layers).

(2) Groundwater flow to the pumping well is under unsteady-state condition.

(3) There is no source of recharge to the aquifer. That is, all the pumped water comes from the aquifer storage.

(4) The aquifer is compressible and the water is released instantaneously from the aquifer storage with the decline in head due to pumping.

(5) The well is pumped at a constant rate.

The analytical solution was found by Theis based on the analogy between groundwater flow and heat conduction in solids and considering following initial condition and two boundary conditions:

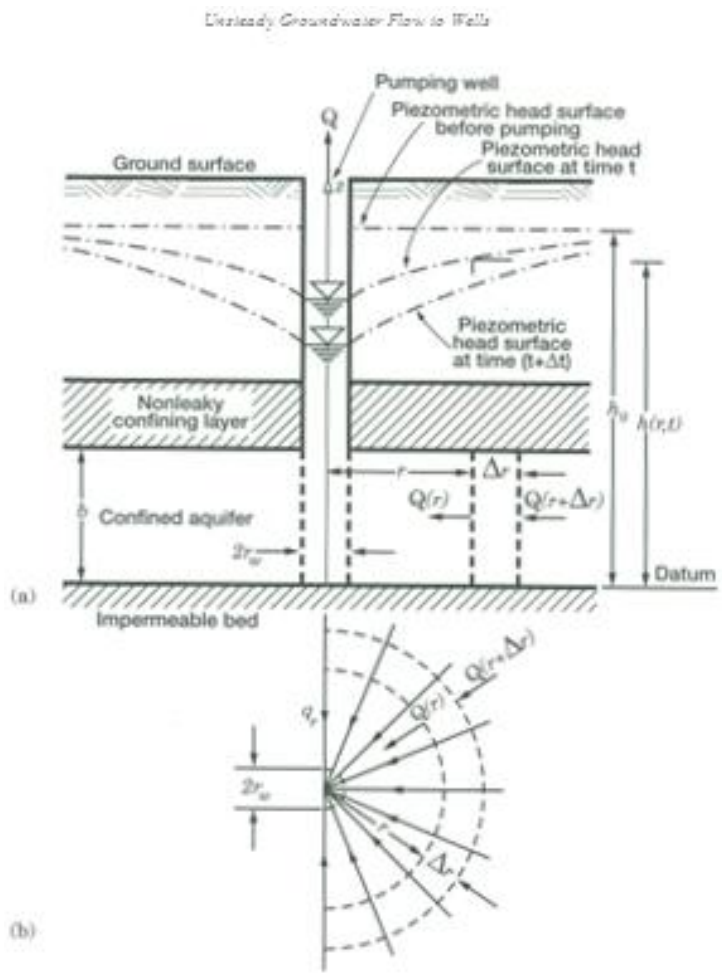


Fig. 11.1. Unsteady flow to a fully penetrating well in a confined aquifer: (a) Vertical cross section; (b) Plan. (Source: Batu, 1998)

Initial condition: h(r,0) = h_0 for all r (i.e., the hydraulic head before pumping at any distance r from the pumping well is equal to the initial hydraulic head).

Boundary conditions: (i) $h(\infty,t) = h_o$ for all t (t >0). That is, the hydraulic head at an infinite radial distance for all time is constant at h_o (initial hydraulic head).

(ii) $\lim_{r\to 0}\left(r\frac{\partial h}{\partial r}\right) = \frac{Q}{2\pi T}$ (i.e., the rate of groundwater withdrawal is constant at the pumping well).

Given the above initial and boundary conditions, the ultimate solution is expressed as follows:

$$s(r,t) = \frac{Q}{4\pi T}\int_u^\infty \frac{e^{-u}}{u}du \qquad (11.6)$$

The dimensionless variable u appearing in Eqn. (11.6) is defined as:

$$u = \frac{r^2 S}{4Tt} \qquad (11.7)$$

Where, h = hydraulic head at distance r from the pumping well, h_o = hydraulic head just before the start of pumping (t = 0), Q = constant pumping rate, s(r,t) = $h_o - h$ = drawdown at a distance r from the pumping well at time t, S = storage coefficient of the aquifer, T = transmissivity of the aquifer, and t = time since pumping started.

The exponential integral $\int_u^\infty \frac{e^{-u}}{u}$, is known as well function for confined aquifers or Theis well function and is generally denoted by W(u). Thus, Eqn. (11.6) can also be written as follows:

$$s(r,t) = \frac{Q}{4\pi T} W(u) \qquad (11.8)$$

Eqn. (11.6) or (11.8) is called Theis equation or Non-equilibrium equation. As W(u) is a complicated function, its calculation is not straightforward. Therefore, the values of W(u) for different values of u have been presented in a tabular form, which can be found in standard books on groundwater hydrology or hydrogeology.

11.2.2 Cooper-Jacob Equation

H. H. Cooper and C. E. Jacob in 1946 suggested a simple but approximate method to compute Theis well function [W(u)] by replacing W(u) with an infinite series as follows:

$$W(u) = -0.577216 - \ln(u) + u - \frac{u^2}{2 \times 2!} + \frac{u^3}{3 \times 3!} - \frac{u^4}{4 \times 4!} + \ldots \qquad (11.9)$$

Cooper and Jacob (1946) observed that after the pumping well is running for some time, the value of u becomes small and the higher-power terms (involving u^2, u^3, u^4 and so on) of Eqn. (11.9) become so small that they can be easily ignored. Thus, Cooper and Jacob (1946) assumed that when the values of u are small ($u \leq 0.01$), only first two terms of Eqn. (11.9) are sufficient to compute W(u). With this assumption, Eqn. (11.6) or Eqn. (11.8) can be written as:

$$s(r,t) = \frac{Q}{4\pi T}\left[-0.577216 - \ln(u)\right] \qquad (11.10)$$

$$\Rightarrow \quad s(r,t) = \frac{Q}{4\pi T}\left[-0.577216 - \ln\left(\frac{r^2 S}{4Tt}\right)\right]$$

$$\Rightarrow \quad s(r,t) = \frac{Q}{4\pi T}\left[-\ln(1.78) - \ln\left(\frac{r^2 S}{4Tt}\right)\right]$$

$$\Rightarrow \quad s(r,t) = \frac{Q}{4\pi T}\ln\left(\frac{4Tt}{1.78\times r^2 S}\right)$$

$$\therefore \quad s(r,t) = \frac{Q}{4\pi T}\ln\left(\frac{2.25Tt}{r^2 S}\right) \qquad (11.11a)$$

Or,

$$s(r,t) = \frac{2.3Q}{4\pi T}\log\left(\frac{2.25Tt}{r^2 S}\right) \qquad (11.11b)$$

Eqn. (11.11a) or (11.11b) is well known as Cooper-Jacob equation. Note that the Cooper-Jacob equation does not require the table of Theis well function, and hence it can be evaluated directly by using a calculator. However, the limitation of this equation is that it is valid only when $u \leq 0.01$. For $u \leq 0.01$, the error is less than 1% which can be considered negligible for practical purposes. Therefore, while using the Cooper-Jacob equation, it is necessary to check whether its use is justified for solving a given problem.

The Theis equation and the Cooper-Jacob equation can be used for forward calculation (i.e., to calculate transient drawdown in a pumping well and/or observation wells or discharge of the pumping well or distance of the drawdown observation point or radius of influence) if the aquifer parameters and the remaining variables are known. In addition, they can also be used for calculating aquifer parameters (T and S), which involves the analysis of time-drawdown data or time-recovery data (observed during pumping tests) following standard procedures as described in Lesson 14.

11.3 Solution of Unsteady Flow to Wells in Leaky Confined Aquifers

As we know that in leaky confined aquifers, leakage takes place through the confining layers from an adjacent aquifer (Fig. 11.2). Therefore, when leaky confined aquifers are pumped, the water withdrawn from leaky confined aquifers comes from aquifer storage as well as from leakage. Pumping gradually increase the leakage rate due to increase in head difference between the leaky confined aquifer and the adjacent aquifer. The net effect of this leakage is to reduce the drawdown in the confined aquifer compared to that expected from an ideal confined aquifer (Theis-type response). The solutions of unsteady flow to wells in

leaky confined aquifers have been obtained for two situations. In the first situation, leakage is considered as the flow across a confining layer (aquitard) without storage in the confining layer. In the second situation, however, it is considered that the storage in the aquitard is also a source of leakage. These two types of solutions are discussed below.

11.3.1 Hantush-Jacob Equation (Without Storage in the Aquitard)

Hantush and Jacob (1955) derived an analytical solution for the transient flow of groundwater to pumping wells tapping a leaky confined aquifer system (Fig. 11.2). In this case, they considered that all the water flowing to the pumping well comes from either elastic storage in the confined aquifer and/or leakage across the confining layer (aquitard). No water is derived from elastic storage in the aquitard. For solving Eqn. (11.3), Hantush and Jacob (1955) made the following assumptions besides the basic assumptions mentioned in Lesson 10:

(1) Groundwater flow to the pumping well is under unsteady-state condition.

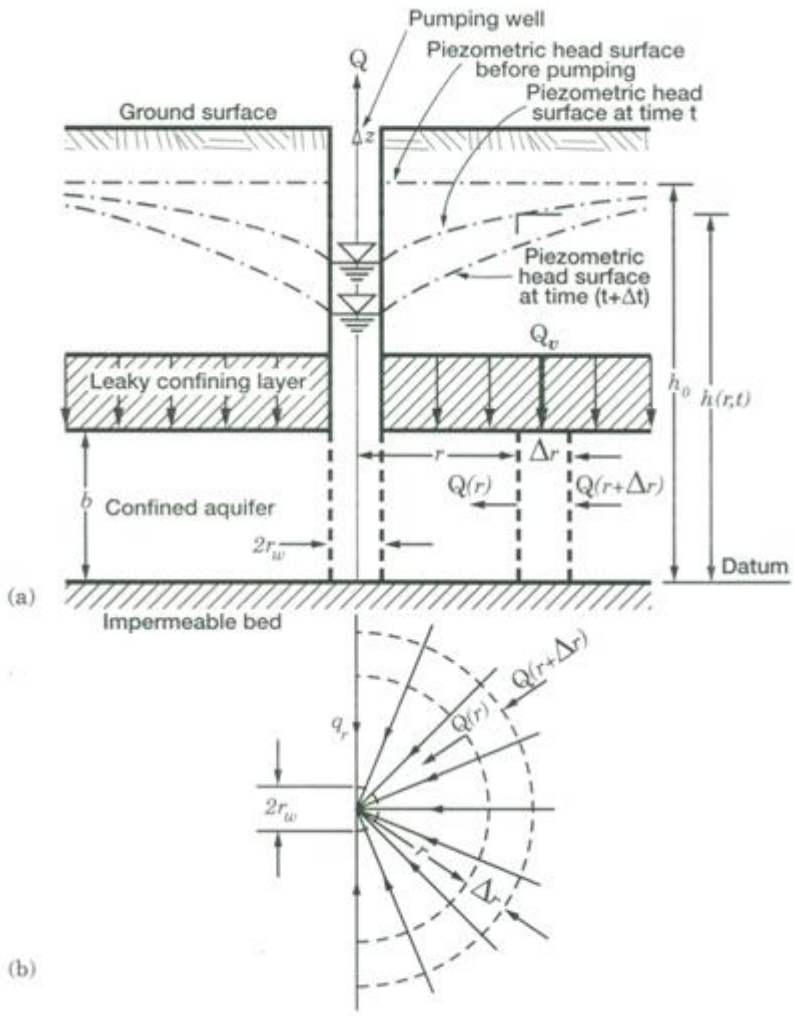


Fig. 11.2. Unsteady flow to a fully penetrating well in a leaky confined aquifer: (a) Vertical cross section; (b) Plan. (Source: Batu, 1998)

(2) The confined aquifer is leaky and the leakage occurs into the aquifer through the confining layer (aquitard) from an adjacent aquifer wherein hydraulic head remains constant during pumping.

(3) Storage in the confining layer (aquitard) is negligible. That is, no water is released from storage in the aquitard when the aquifer is pumped.

(4) The confining layer that overlies or underlies the leaky confined aquifer has a uniform hydraulic conductivity (K¢) and thickness (b¢).

(5) Groundwater flow is vertical in the confining layer and radial in the leaky confined aquifer.

(6) The leaky confined aquifer is compressible and the water is released instantaneously from the aquifer storage with the decline in head due to pumping.

(7) The well is pumped at a constant rate.

Given the above assumptions and considering appropriate initial and boundary conditions, Hantush and Jacob (1955) found an analytical solution to Eqn. (11.3) which is known as Hantush-Jacob equation and it is given as:

$$s(r,t) = \frac{Q}{4\pi T} W\left(u, \frac{r}{B}\right) \qquad (11.12)$$

Where, is called well function for leaky confined aquifers or Hantush well function, and is expressed as:

$$W\left(u, \frac{r}{B}\right) = \int_u^{\infty} \frac{1}{y} \exp\left(-y - \frac{r^2}{4B^2 y}\right) dy \qquad (11.13)$$

$u = \frac{r^2 S}{4Tt}$, and B = leakage factor which is expressed as:

$$B = \sqrt{T \times \frac{b'}{K'}} \qquad (11.14)$$

Since W(u,r/B) is a much more complicated function than the Theis well function mentioned above, its calculation is complex. Therefore, the values of W(u,r/B) have been presented in a tabular form for different values of u and r/B by Hantush (1956), which can also be found in standard books on groundwater hydrology or hydrogeology.

Like the Theis equation, the Hantush-Jacob equation can also be used for forward calculation as well as for the calculation of aquifer parameters (T, S, and B) from pumping-test data of leaky confined aquifers without storage in the aquitard. Moreover, the rate that water is being withdrawn from elastic storage in the confined aquifer (q_s) at a given time (t) since pumping started can be estimated from the following equation (Fetter, 1994):

$$q_s = Q\exp\left(-\frac{Tt}{SB^2}\right) \qquad (11.15)$$

If the total discharge (rate of groundwater withdrawal) at time t is Q and the water withdrawn from aquifer storage at that time is q_s, then the rate of leakage through the aquitard at that time (q_L) can be estimated as:

$$q_L = Q - q_S \qquad (11.16)$$

If the well is pumped long enough, all the water will be coming from leakage across the confining layer and no water will be coming from elastic storage in the confined aquifer. This situation creates steady-state flow condition in the leaky confined aquifer and it occurs when $t > \frac{8b'S}{K'}$. In this case, the steady drawdown in the leaky confined aquifer is given as follows (Hantush and Jacob, 1955):

$$s(r) = \frac{Q}{2\pi T}K_0\left(\frac{r}{B}\right) \qquad (11.17)$$

Where, K_0 = Zero-order modified Bessel function of the second kind. The values of $K_0(x)$ are available in Hantush (1956), and are also tabulated in standard books on groundwater hydrology or hydrogeology. Like the Thiem equation, Eqn. (11.17) can be used for forward calculation as well as for the calculation of aquifer parameters.

11.3.2 Hantush Equation (With Storage in the Aquitard)

If the assumption 'no water is released from storage in the aquitard when the aquifer is pumped' made in the previous case is not valid, then the solution for leaky confined aquifers with a contribution of water from storage in the aquitard must be used (Hantush, 1960). This solution is based on the basic assumptions mentioned in Lesson 10 as well as all of the assumptions used in the previous section (Section 11.3.1), except the assumption that storage in the aquitard is negligible. Thus, in this case, it was considered that some water comes from elastic storage in the aquitard. This case has two solutions as described below.

Solution 1: During the early part of pumping when $t < \frac{b'S'}{10K'}$ (where b', S' and K' are the thickness, storage coefficient and hydraulic conductivity of the aquitard, respectively), all the water will come from elastic storage in the aquifer and the aquitard. In this case, the analytical solution to the governing equation [Eqn. (11.3)] was derived by Hantush (1960) as follows:

$$s(r,t) = \frac{Q}{4\pi T}H(u,\beta) \qquad (11.18)$$

Where, H(u,β) is called modified well function for leaky confined aquifers or modified Hantush well function, and β is expressed as:

$$\beta = \frac{r}{4B}\left(\frac{S''}{S}\right)^{1/2} \qquad (11.19)$$

The remaining symbols in Eqn. (11.18) have the same meaning as defined earlier. Eqn. (11.18) is known as Hantush equation. The values of H(u,β) have been presented in a tabular form for different values of u and β by Hantush (1961), which can also be found in standard books on groundwater hydrology or hydrogeology. The Hantush equation can also be used for forward calculation as well as for the calculation of aquifer parameters from pumping-test data of leaky confined aquifers with storage in the aquitard.

Solution 2: If sufficient time elapses, the aquifer will reach equilibrium and all of the water will be coming from drainage from the overlying aquifer (source bed). The time to reach this equilibrium is given as:

$$t > \frac{8\left[S + (S''/3) + S''\right]}{\left[(K''/b'') + (K''/b'')\right]^2} \qquad (11.20)$$

Where, K" and b" are the hydraulic conductivity and saturated thickness of the overlying aquifer (source bed), respectively.

If the value of r_w/B is less than 0.01, then the solution is:

$$s(r) = \frac{Q}{2\pi T} K_0\left(\frac{r}{B}\right) \qquad (11.21)$$

This is the same solution as that for steady-state (equilibrium) flow condition in the case where no water comes from elastic storage in the aquitard [i.e., Eqn. (11.17)]. This is so because all the water comes from the source bed (overlying aquifer).

11.4 Solution of Unsteady Flow to Wells in Unconfined Aquifers

11.4.1 Neuman Equation for Unconfined Aquifers with Delayed Yield

Unlike confined aquifer, unconfined aquifers essentially yield water by two mechanisms when they are pumped: (i) like the confined aquifers, the decline in water pressure (hydraulic head) in the unconfined aquifer yields water due to elastic storage of the aquifer which is characterized as S_s (specific storage of the aquifer); and (ii) the declining water table also yields water as it drains (dewaters) under gravity from the aquifer material, which is characterized as S_y (specific yield of the aquifer). The second mechanism of water yielding dominates in most unconfined aquifers. However, the gravity drainage (dewatering) of pores is a slow process, and hence it does not keep pace with the rapid decline of the water table during pumping. Large-size pores drain faster than the small-size pores. As a result, some water drains from above the cone of depression (which was saturated before pumping) much later than the fall of the water table and this vertical (or gravity) drainage continues for some time (Fig. 11.3b). This typical response of unconfined aquifers to pumping is termed 'delayed yield' or 'delayed gravity response' (Boulton, 1954; Neuman, 1972), which is briefly explained below.

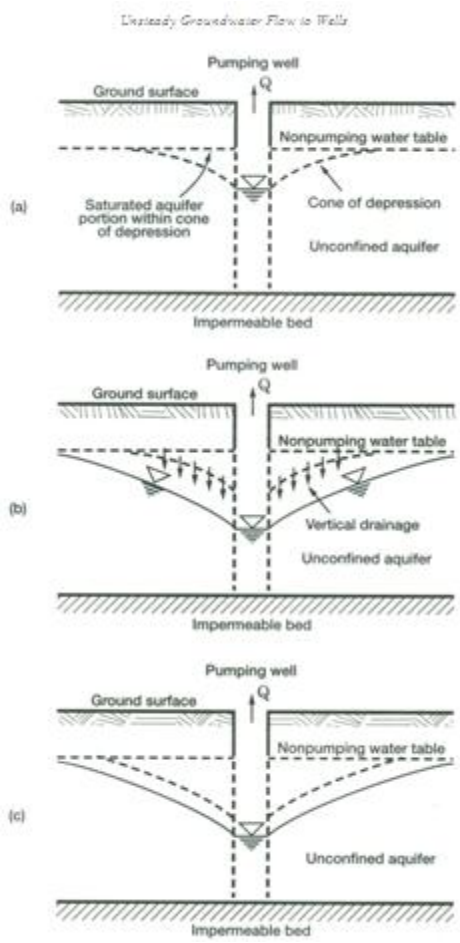
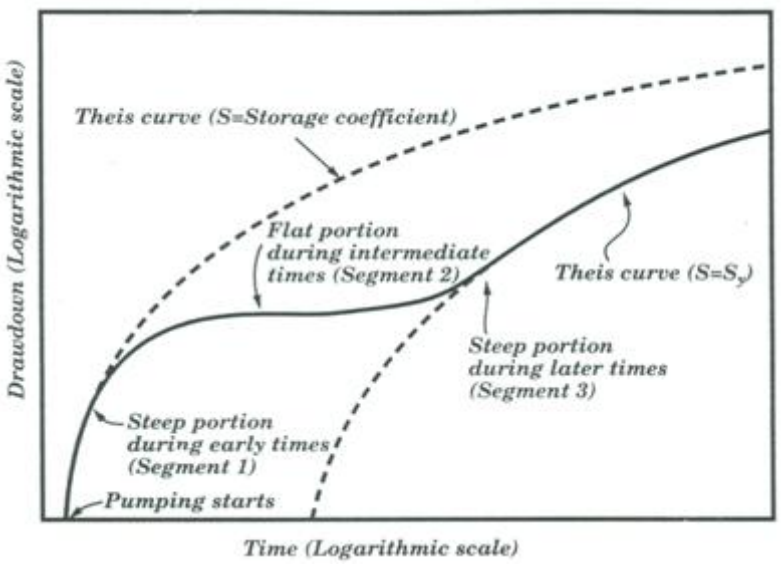


Fig. 11.3. Development of cone of depression in an unconfined aquifer: (a) Initial cone of depression; (b) Gravity drainage of the initial cone of depression; (c) Cone of depression under equilibrium condition.

(Source: Batu, 1998)

Fig. 11.4. Illustration of delayed gravity response and its effect on drawdown in unconfined aquifers. (Source: Batu, 1998)



There are three distinct phases (time segments) of time-drawdown curves in the unconfined aquifer showing delayed yield (Fig. 11.4):

1. First Segment (Early Segment): In this segment, which lasts for some seconds to a few minutes of pumping, water is released essentially instantaneously from aquifer storage by the aquifer compaction and by water expansion similar to confined conditions. Almost all water supplied to the well comes from the aquifer storage in the saturated zone. Gravity water above the hydraulic head within the cone of depression still has not reached the saturated zone (Fig. 11.3a). The storage coefficient during this phase is close to a confined aquifer of the same porous material as that of the unconfined aquifer and is denoted as S (Fig. 11.4).

(2) Second Segment (Intermediate Segment): This segment displays a gradual flattening in the time-drawdown slope caused by gravity-drainage replenishment from the pore spaces above the cone of depression, which were saturated before pumping. It indicates that the gravity water is reaching the saturated zone (Fig. 11.3b), but is still not in equilibrium with the saturated flow. The flat (straight-line) portion of the time-drawdown curve (Fig. 11.4) indicates that the rate of gravity drainage is equal to the rate of pumping from the aquifer.

(3) Third Segment (Late Segment): This segment represents equilibrium between the gravity drainage and the saturated flow when the delayed gravity response ceases (Fig. 11.3c). Such an equilibrium condition is achieved when the rate of gravity drainage is equal to the rate of decline in the water table. This hydraulic condition occurs in an unconfined aquifer after several minutes to several days of pumping. The storage properties during this phase are those of a truly unconfined aquifer and are termed 'specific yield' (S_y) (Fig. 11.4).

The flow of groundwater towards a pumping well in unconfined aquifers showing delayed yield is described by the following partial differential equation (Neuman and Witherspoon, 1969):

$$K_r \frac{\partial^2 h}{\partial r^2} + \frac{K_r}{r}\frac{\partial h}{\partial r} + K_v \frac{\partial^2 h}{\partial z^2} = S_s \frac{\partial h}{\partial t} \qquad (11.22)$$

Where, K_r = radial hydraulic conductivity of the unconfined aquifer, K_v = vertical hydraulic conductivity of the unconfined aquifer, S_s = specific storage of the unconfined aquifer, z = elevation above the base of the aquifer, and the remaining symbols have the same meaning as defined earlier.

Neuman (1975) solved Eqn. (11.22) after making some specific assumptions in addition to the basic assumptions mentioned in Lesson 10. The solution is known as Neuman equation, which is given as follows:

$$s(r,t) = \frac{Q}{4\pi T} W(u_a, u_y, \eta) \qquad (11.23)$$

Where, $W(u_a, u_y, h)$ is known as well function for unconfined aquifers or Neuman well function, and the dimensionless parameters u_a, u_y and h are defined as:

$$u_a = \frac{r^2 S}{4Tt}$$ (applicable for early drawdown data) (11.24)

$$u_y = \frac{r^2 S_y}{4Tt}$$ (applicable for later drawdown data) (11.25)

$$\eta = \frac{r^2 K_v}{h_0^2 K_h}$$ (11.26)

Where, S = storativity or storage coefficient of the unconfined aquifer, T = transmissivity of the unconfined aquifer, h_0 = initial saturated thickness of the unconfined aquifer, K_h = horizontal hydraulic conductivity of the unconfined aquifer, and the remaining symbols have the same meaning as defined earlier. The values of $W(u_a, u_y, h)$ have been presented in a tabular form by Neuman (1975), which can also be found in standard books on groundwater hydrology or hydrogeology. Like the Theis equation, the Neuman equation can also be used for forward calculation as well as for the calculation of aquifer parameters (K_h, K_v, S and S_y) from pumping-test data obtained from the unconfined aquifer exhibiting delayed yield.

11.4.2 Theis Equation for Unconfined Aquifers without Delayed Yield

The Theis equation for confined aquifers can also be applied to unconfined aquifers provided that the basic assumptions are satisfied. In general, if the drawdown is small in relation to the saturated thickness of the unconfined aquifer, reasonably good approximations are possible (Stallman, 1965). Furthermore, it has been found that many unconfined aquifers do not exhibit delayed yield, i.e., the time-drawdown curve does not show three distinct phases as discussed above. In this case also, the Theis equation can be used for unconfined aquifers without much error. Alternatively, a refined approach as suggested in Section 10.2.5 of Lesson 10 for the application of the Thiem equation to unconfined aquifers can be followed while using the Theis equation for unconfined aquifers. This approach has proved to be very effective for solving the flow problems related to unconfined aquifers.

11.5 Solution of Unsteady Flow to Cavity Wells

The solutions for unsteady drawdown behavior around a pumped cavity well (Fig. 11.5) have been obtained by Kanwar and Chauhan (1974) and Chauhan et al. (1975). These solutions are based on the following assumptions:

(1) The confined aquifer is homogeneous, isotropic and has an infinite areal extent and an extensive thickness.

(2) A spherical sink of infinitesimal radius r in the form of a non-penetrating well with sides impermeable and hemispherical bottom is situated at the boundary of the impermeable layer and the confined aquifer.

(3) The water removed from aquifer storage is discharged instantaneously with decline in head.

(4) The water is pumped at a constant rate and the specific storage coefficient is constant.

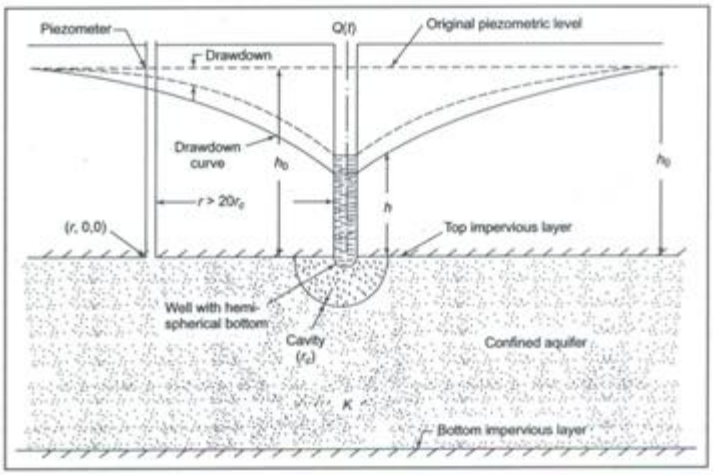


Fig. 11.5. Illustration of unsteady spherical flow into a cavity well in a confined aquifer. (Source: Michael et al., 2008)

The following partial differential equation (Michael et al. (2008) governs the unsteady (transient) spherical flow in a homogeneous and isotropic non-leaky confined aquifer (Fig. 11.5):

$$\frac{\partial^2 h}{\partial r^2} + \frac{2\partial h}{r\partial r^2} = \frac{S_s}{K}\frac{\partial h}{\partial t} \qquad (11.27)$$

Where, h = hydraulic head in the confined aquifer, r = radial distance from the centre of the cavity well, S_s = specific storage of the confined aquifer, K = hydraulic conductivity of the confined aquifer, and t = time since pumping started.

Chauhan et al. (1975) obtained the solution of Eqn. (11.27) for drawdown at a distance r from the pumping well (cavity well) at any time t as follows:

$$s(r,t) = \frac{Q}{2\pi r K} erfc(\sqrt{u}) \qquad (11.28)$$

Where, s(r,t) = drawdown at a distance r and time t, Q = constant discharge from the cavity well, K = hydraulic conductivity of the confined aquifer, and u is defined as:

$$u = \frac{r^2 S_s}{4Kt} \qquad (11.29)$$

Like the Theis equation, Eqn. (11.28) can also be used for forward calculation as well as for the calculation of aquifer parameters (K, S_s and S) from pumping-test data obtained using a cavity well. The procedure for determining hydraulic properties of confined aquifers using cavity wells can be found in Michael et al. (2008).

Lesson 12 Determination of Aquifer Parameters

12.1 Introduction

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

12.2 Laboratory Methods

12.2.1 Grain-Size Analysis (GSA) Method

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

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

Several formulae, varying from very simple to complex, based on analytic or experimental work have been developed for the estimation of K from the grain-size distribution data; for

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

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

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

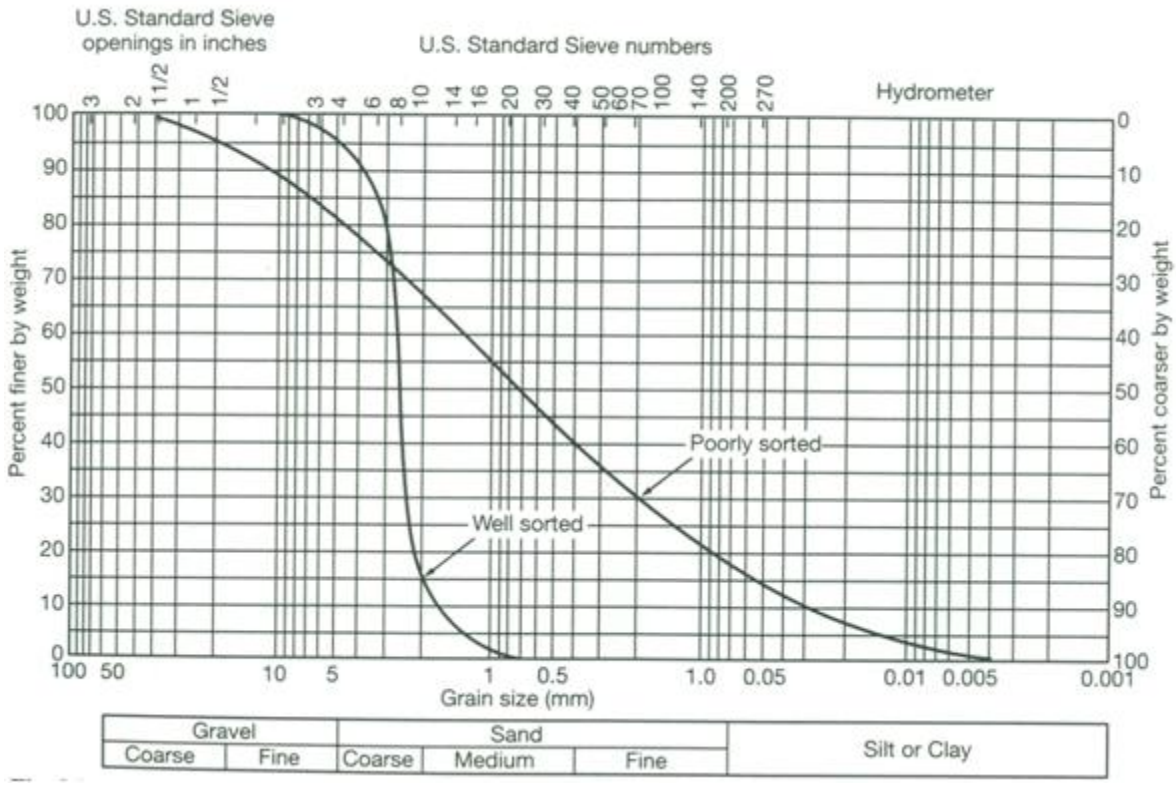


Fig. 12.1. Grain-size distribution curves for well sorted and poorly sorted samples. (Source: Brassington, 1998)

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

12.2.2 Permeameter Methods

In the laboratory, hydraulic conductivity of undisturbed geologic samples or soil samples can be determined in the laboratory by a permeameter. The permeameter methods essentially provide saturated hydraulic conductivity. If undisturbed geologic samples can be collected from shallow aquifers or confining layers using a core sampler, these samples can

be used to determine the saturated hydraulic conductivity of aquifer or non-aquifer materials in the laboratory in the same way as undisturbed soil samples. In permeameters, flow is maintained through a small sample of material while the measurements of flow rate and head loss are made. The constant-head and falling-head types of permeameters (Fig. 12.2) are simple to operate and widely used.

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

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

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

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

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

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

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

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

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

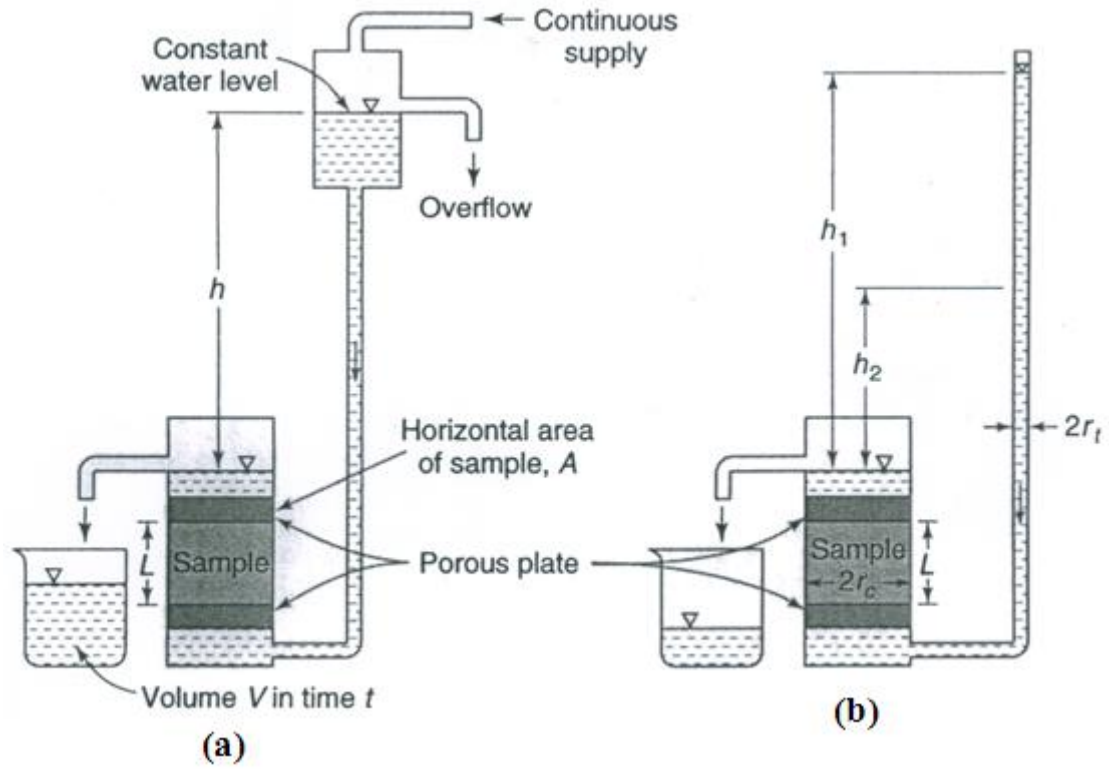


Fig. 12.2. Permeameters for measuring saturated hydraulic conductivity of geologic or soil samples: (a) Constant-head permeameter; (b) Falling-head permeameter. (Source: Mays, 2012)

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

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

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

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

12.3 Field Methods

12.3.1 Tracer Test

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

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

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

However, V_s can also be calculated as:

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

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

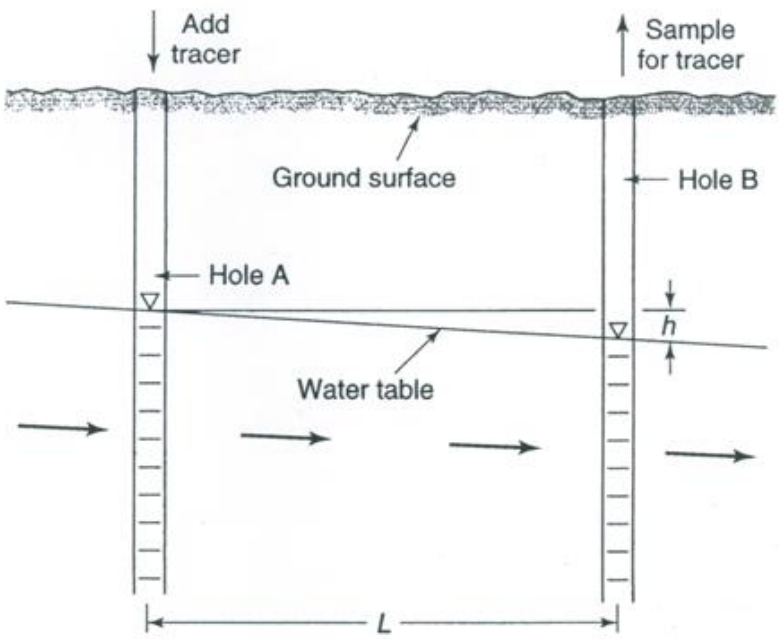


Fig. 12.3. Illustration of a tracer test in an unconfined aquifer for determining hydraulic conductivity. (Source: Mays, 2012)

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

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

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

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

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

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

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

Example Problem:

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

Solution:

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

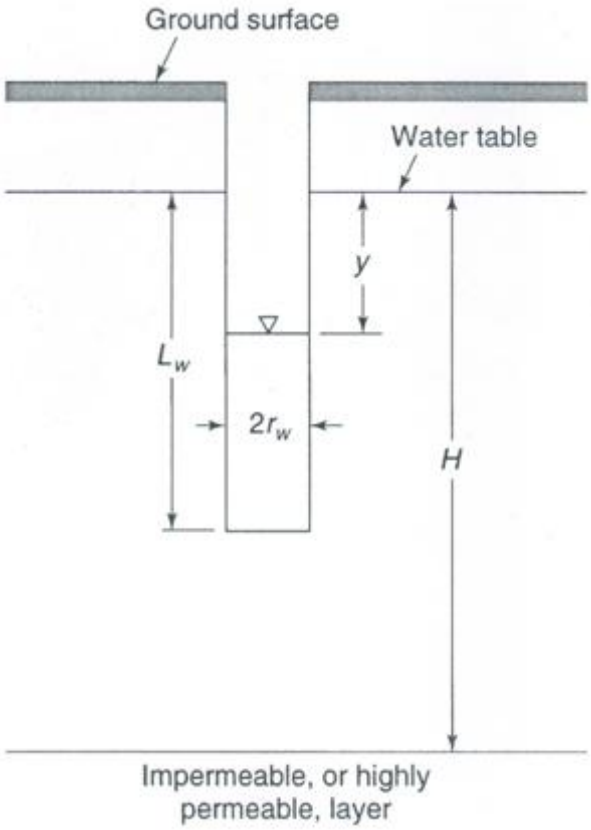


Fig. 12.4. Schematic of an auger hole and its dimensions for determining aquifer hydraulic conductivity. (Source: Mays, 2012)

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.

Lesson 13 Overview of Field Pumping Test

13.1 Introduction

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

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

13.2 Purpose of Pumping Test

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

13.3 Types of Pumping Tests

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

(1) Time-Drawdown Test:

(a) Interference Test

(b) Distance-Drawdown Test

(c) Single Well Test

(2) Recovery Test

(3) Step-Drawdown Test or Variable Rate Test

(4) Injection Tests:

(a) Time-Groundwater Level Rise Test

(b) Step-Injection Test

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

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

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

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

13.4 Tests for Determining Hydraulic Parameters of Aquifers

13.4.1 Steady and Unsteady Time-Drawdown Tests

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

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

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

13.4.1.1 Interference Test

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

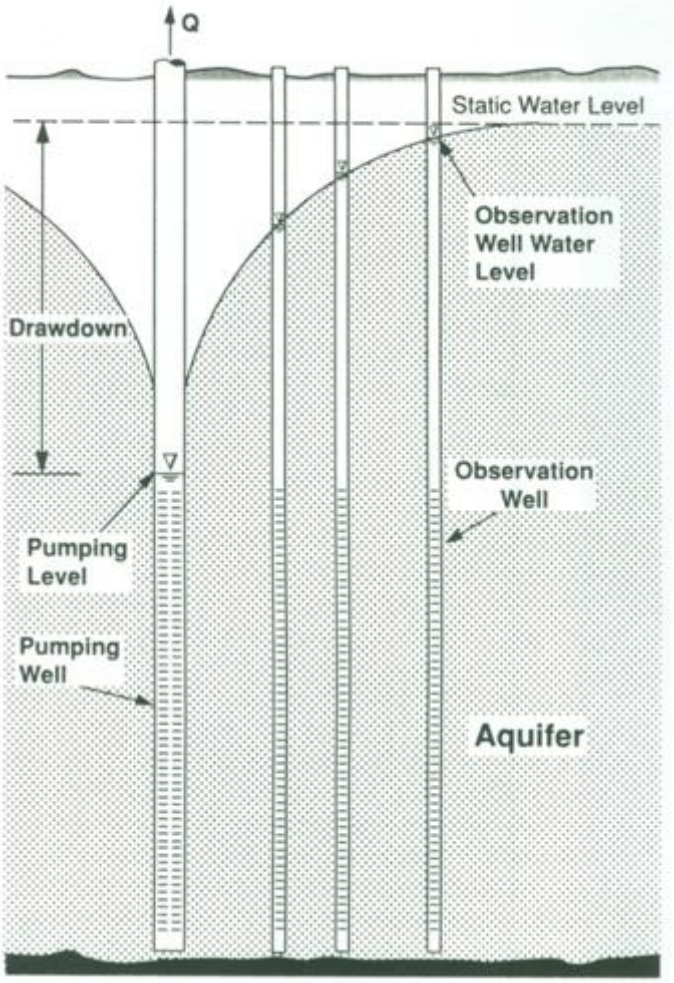


Fig. 13.1. Illustration of an interference pumping test.

(Source: Roscoe Moss Company, 1990)

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

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

13.4.1.2 Distance-Drawdown Test

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

13.4.1.3 Single Well Test

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

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

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

Or,

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

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

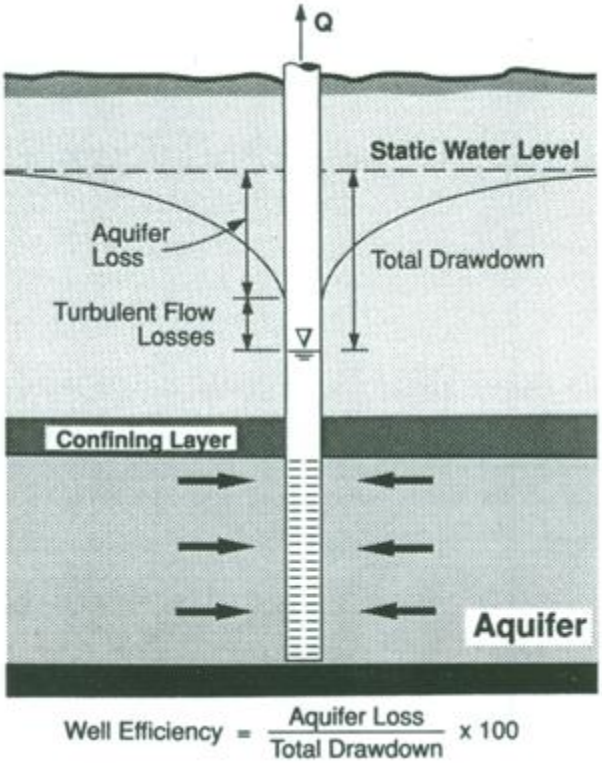


Fig. 13.2. Components of drawdown in a pumping well.

(Source: Roscoe Moss Company, 1990)

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

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

13.4.2 Recovery Test

Recovery test is conducted at the end of a time-drawdown aquifer test. It is an unnsteady-state aquifer test in which groundwater rise is measured with time in a pumping/production well or in an observation well after pumping has been stopped (Fig. 13.3). It is apparent from Fig. 13.3 that if the pumping of a well is stopped, the groundwater level in the aquifer gradually starts increasing and it should theoretically return to its pre-

pumping groundwater level; this time period is known as recovery period or recovery phase. From the water-level data measured during recovery period, 'residual drawdown' (difference between the pre-pumping groundwater level and the depth to groundwater after the pump is stopped) or 'recovery' (difference between the extrapolated time-drawdown curve and the residual drawdown) is computed at different times during recovery period (Fig. 13.3). The 'time-residual drawdown' data or the 'time-recovery' data thus obtained are used for determining aquifer parameters.

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

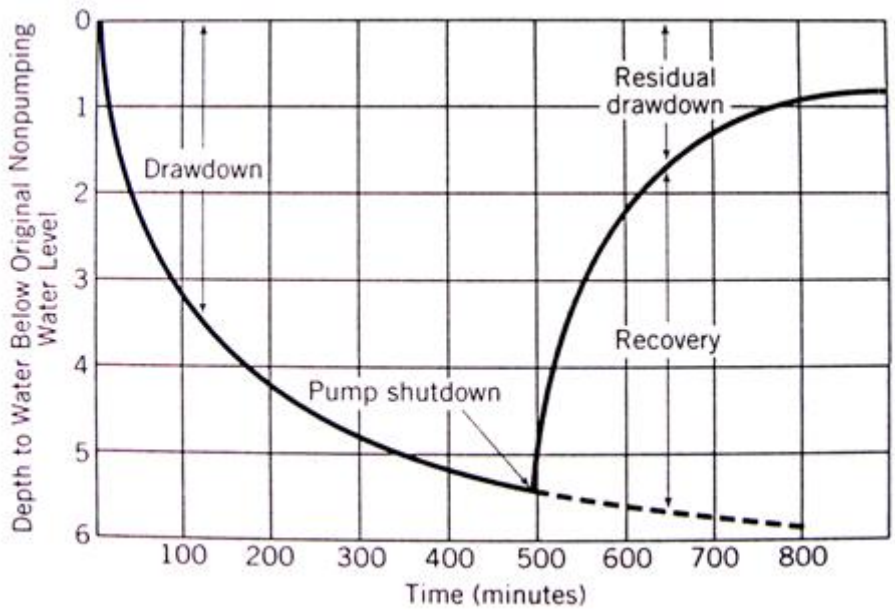


Fig. 13.3. Illustration of time-drawdown and recovery tests.

(Source: Schwartz and Zhang, 2003)

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

13.4.3 Injection Tests

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

13.5 Test for Determining Well Hydraulic Characteristics

13.5.1 Step-Drawdown Test

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

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

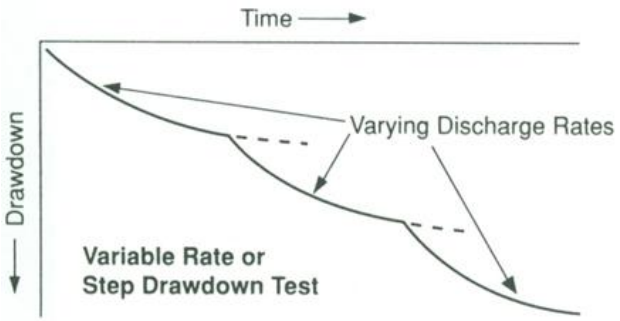


Fig. 13.4. Illustration of a step-drawdown test.

(Source: Roscoe Moss Company, 1990)

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

13.5.2 Critical Pumpage and Safe Well Yield

Based on the step-drawdown test data, a safe pumping rate or safe well yield for a production well can be determined. The safe well yield is generally defined as the highest pumping rate at which equilibrium conditions can be maintained in the aquifer system (Roscoe Moss Company, 1990). When the equilibrium rate is exceeded, the pumping level continues to decline at a rate proportional to the well and aquifer parameters. When this decline is quite rapid (e.g., several centimeters per minute), the well may dewater in a

relatively short time period, with the pump 'breaking suction'; the latter condition is very serious and could result in the collapse of well casing.

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

13.6 Design of Pumping Test

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

13.6.1 Placement of Observation Wells and Number of Tests

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

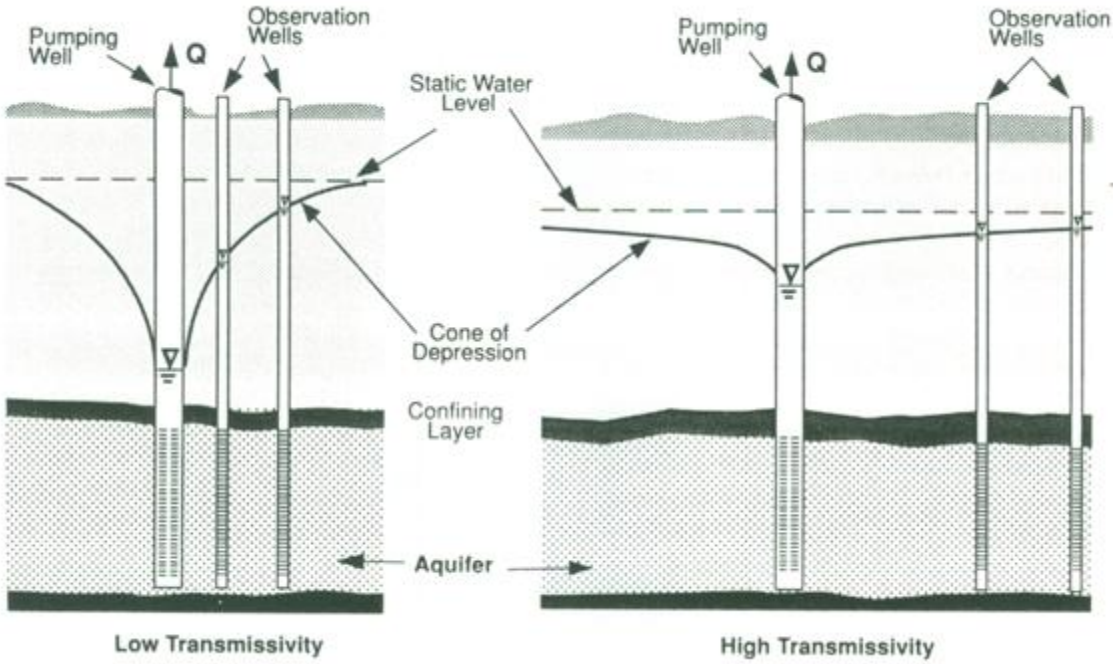


Fig. 13.5. Placement of observation wells for a pumping test.

(Source: Roscoe Moss Company, 1990)

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

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

13.6.2 Pumping Rate and Test Duration

The rate of pumping during aquifer tests should be such that an appreciable drawdown occurs in most of the observation wells placed at different radial distances. It should also be ensured that the drawdowns measured during an aquifer test are not affected by any other

factors. If any external factor affects the drawdown measured in a given observation well, the data become erroneous (noisy) and such data are usually not considered for analysis.

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

Table 13.1. Guidelines for deciding the duration of aquifer tests

(Source: Todd, 1980)

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

13.6.3 Measurement of Pumping Rate and Groundwater Level

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

for disposal) can be used. The depth to groundwater level can be measured either manually using a water-level indicator and/or automatically by a water-level recorder ('clock and chart type water-level recorder' or 'water-level sensor').

13.6.4 Schedule of Data Collection

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

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

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

13.7 Response of Bounded Aquifers to Pumping

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

depression by providing a source of recharge to the aquifer. They also influence the pattern of drawdown in the vicinity of pumping wells.

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

13.7.1 Image Well Theory

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

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

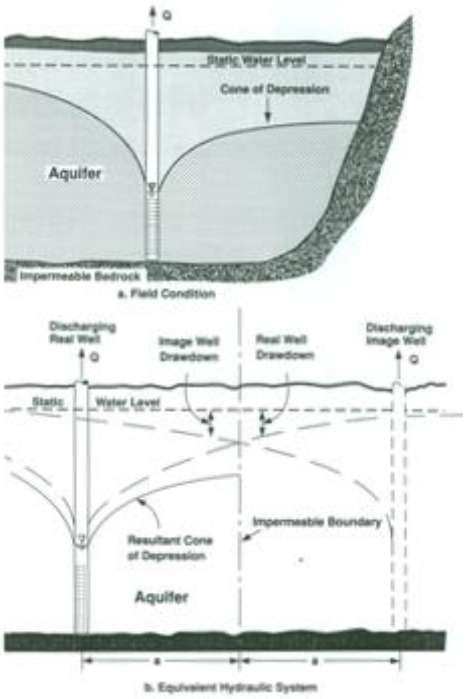


Fig. 13.6. Pumping well near a recharge boundary.

(Source: Roscoe Moss Company, 1990)

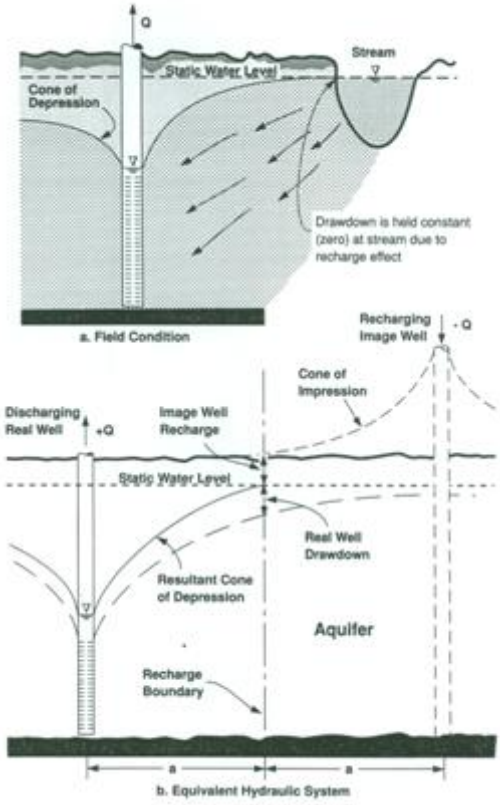


Fig. 13.7. Pumping well near an impermeable boundary.

(Source: Roscoe Moss Company, 1990)

13.7.2 Pumping Near Recharge Boundary

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

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

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

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

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

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

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

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

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

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

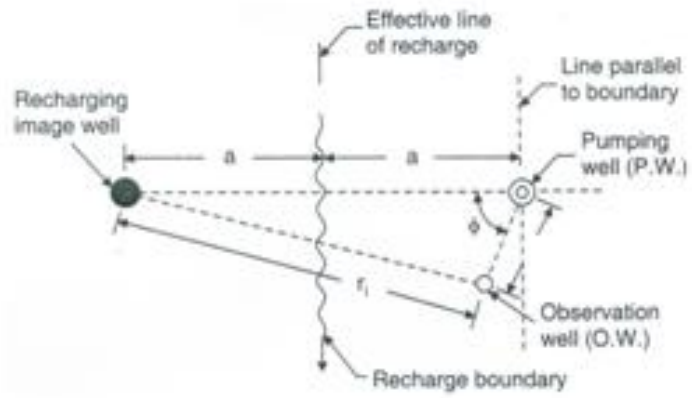


Fig. 13.8. Typical configuration of pumping well, observation well and image well in an aquifer bounded by a recharge boundary. (Source: Raghunath, 2007)

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

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

13.7.3 Pumping Near Impermeable Boundary

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

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

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

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

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

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

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

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

13.7.4 Impact of Aquifer Boundaries on Time-Drawdown Curve

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

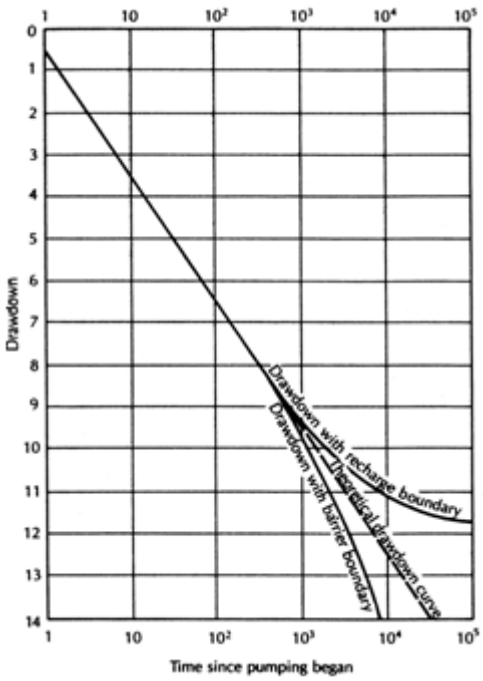


Fig. 13.9. Impact of recharge and impermeable boundaries on the time-drawdown curve. (Source: Redrawn from Fetter, 1994)

13.8 Advantages and Disadvantages of Pumping Tests

The advantages of the pumping test method are self-evident. A pumping test provides in situ parameter values and these values are, in effect, averaged over a large and

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

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

Lesson 14 Analysis of Pumping-Test Data

14.1 Introduction

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

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

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

Prof. Madan Kumar Jha, has copyrighted a user-friendly software package named GA-AquiAnalyzer which facilitates the analysis of aquifer-test data by the genetic algorithm (GA) technique (Samuel and Jha, 2003)

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

Sl. No.	Type of Aquifer	Type of Pumping-Test Data	Name of Methods
1	Confined Aquifer	(i) Time-Drawdown data	• Theis Type Curve Method • Cooper-Jacob Straight-Line Method
		(ii) Unsteady Distance-Drawdown data	• Cooper-Jacob Straight-Line Method
		(iii) Quasi-Steady/Steady Distance-Drawdown data	• Thiem Method • Graphical Method
		(iv) Recovery data: - Time-Residual Drawdown data - Time-Recovery data	Residual Drawdown-Time Ratio Method Cooper-Jacob Straight-Line Method
2	Unconfined Aquifer without Delayed Yield	(i) Time-Drawdown data	• Theis Type Curve Method • Cooper-Jacob Straight-Line Method
		(ii) Unsteady Distance-Drawdown data	• Cooper-Jacob Straight-Line Method
		(iii) Quasi-Steady/Steady Distance-Drawdown data	• Thiem Method • Graphical Method
		(iv) Recovery data: - Time-Residual Drawdown data - Time-Recovery data	Residual Drawdown-Time Ratio Method Cooper-Jacob Straight-Line Method

3	Unconfined Aquifer with Delayed Yield	(i) Time-Drawdown data	• Type-Curve Method • Neuman Straight-Line Method
		(ii) Quasi-Steady/Steady Distance-Drawdown data	• Thiem Method • Graphical Method
4	Leaky Confined Aquifer without Storage in Aquitards	(i) Time-Drawdown data	• Walton Type-Curve Method • Hantush Inflection-Point Method
		(ii) Quasi-Steady/Steady Distance-Drawdown data	• Type-Curve Method
5	Leaky Confined Aquifer with Storage in Aquitards	(i) Time-Drawdown data	• Hantush Type-Curve Method
		(ii) Quasi-Steady/Steady Distance-Drawdown data	• Type-Curve Method

14.2 Determination of Confined Aquifer Parameters

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

14.2.1 Theis Type-Curve Method

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

Theis equation can be rearranged as:

$$T = \frac{Q}{4\pi s} W(u) \qquad (14.1)$$

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

$$S = \frac{4Ttu}{r^2} \qquad (14.2)$$

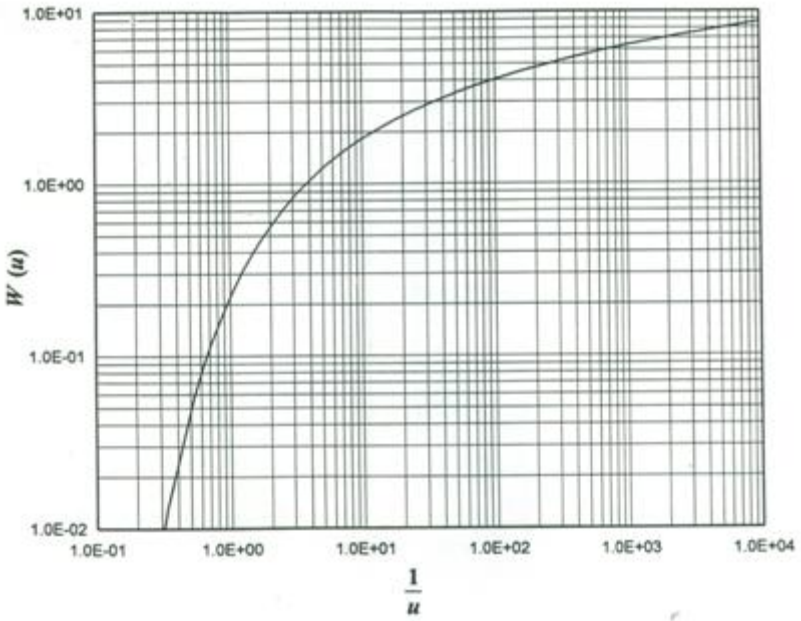


Fig. 14.1. Theis Type Curve for confined aquifers. (Source: Roscoe Moss Company, 1990)

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

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

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

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

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

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

14.2.2 Illustrative Example 1

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

Table 14.2. Time-Drawdown data of an observation well

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

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

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

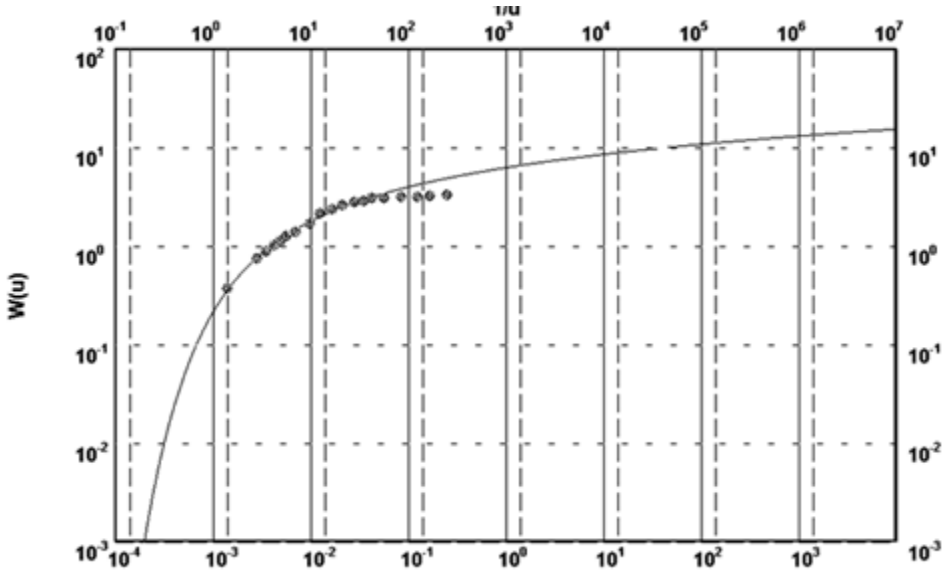


Fig. 14.2. Matching the field data with the Theis Type Curve.

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

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

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

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

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

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

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

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

$$T = \frac{2.3Q}{4\pi\Delta s} \qquad (14.3)$$

and $$S = \frac{2.25Tt_o}{r^2} \quad (14.4)$$

14.2.4 Illustrative Example 2

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

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

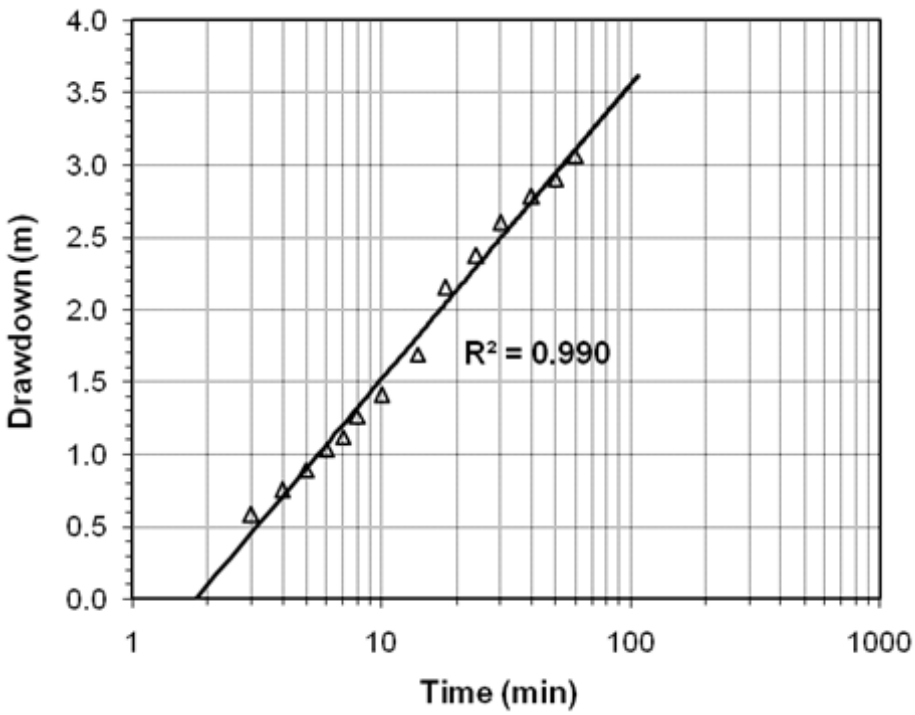


Fig. 14.3. Illustration of the Cooper-Jacob straight-line method.

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

$$T = \frac{2.3Q}{4\pi\Delta s} = \frac{2.3\times(200\times60\times60\times24/1000)}{4\pi\times2.05} = \frac{39744}{25.761} = 1542.80\ \text{m}^2/\text{day, Ans.}$$

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

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

Check:

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

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

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

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

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

and

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

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

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

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

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

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

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

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

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

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

and

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

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

14.2.6 Illustrative Example 3

Problem: During a pumping test conducted in a confined aquifer, the aquifer was pumped at a constant rate of 280 m^3/h. After 180 minutes of pumping, drawdowns were

simultaneously measured in nine observation wells located at different radial distances from the pumping well as shown in Table 14.3. Using the observed distance-drawdown data, determine transmissivity (T) and storage coefficient (S) of the confined aquifer.

Table 14.3. Unsteady Distance-Drawdown data

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

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

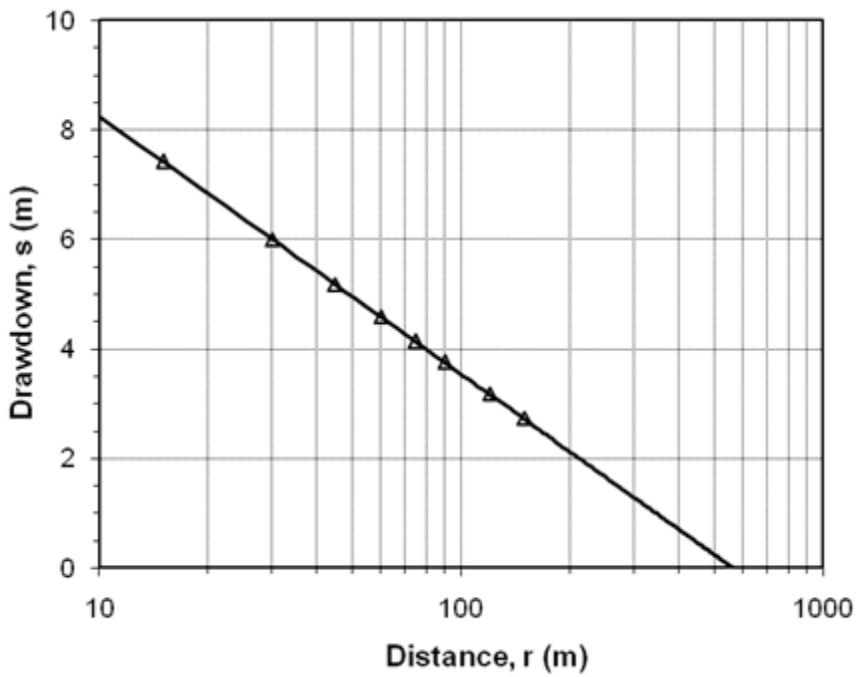


Fig. 14.4. Straight-line fitting to the distance-drawdown data.

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

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

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

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

14.2.7 Residual Drawdown-Time Ratio Method for Recovery Data

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

If t is the time since pumping starts and t¢ is the time since pumping stops, the residual drawdown (s¢) (Fig. 14.5) in a confined aquifer at any time (t′) after the end of pumping (i.e., during recovery period) can be obtained from the principle of superposition as follows:

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

Where,

$$u = \frac{r^2 S}{4Tt} \qquad (14.11)$$

and

$$u' = \frac{r^2 S}{4Tt'} \qquad (14.12)$$

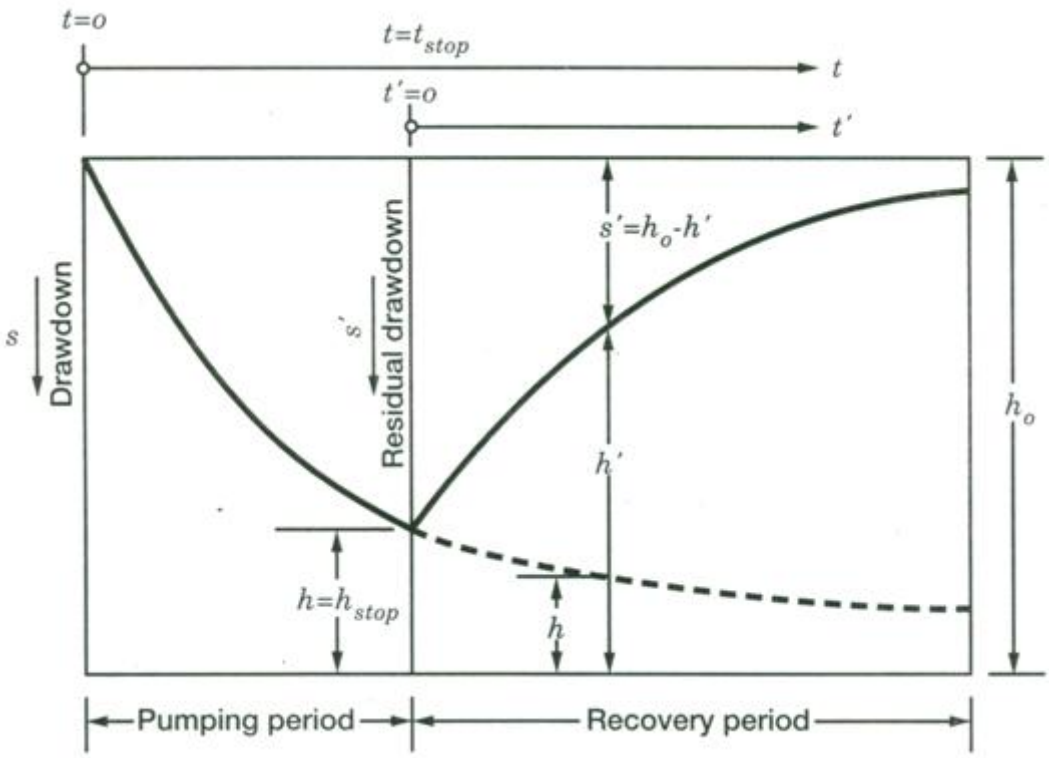


Fig. 14.5. Schematic diagram of the aquifer recovery after the pump is turned off. (Source: Batu, 1998)

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

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

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

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

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

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

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

Step 3: Calculate the value of the residual drawdown per log cycle (Δs′) from the slope of the straight line.

Step 4: Calculate the transmissivity (T) as:

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

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

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

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

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

14.2.8 Illustrative Example 4

Problem (Modified from Schwartz and Zhang, 2003): A pumping test was conducted in a confined aquifer and drawdowns were measured during both pumping period and recovery period in the pumping well (Table 14.4) as well as in an observation well (Table 14.5). In Tables 14.4 and 14.5, the first column contains the time since the pumping started (t), the second column contains the drawdown (s) during the pumping period, the third column contains the time since pumping stopped (t′), the fourth column contains the time ratio (t/t′), and the last column contains the residual drawdown (s′) during the recovery period.

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

t (min)	s (m)	t¢(min)	t/t¢	s¢(m)
20	3.44	20	41.00	0.46
80	3.54	80	11.00	0.30
140	3.60	140	6.71	0.24
195	3.60	195	5.10	0.21
255	3.60	255	4.14	0.18
315	3.66	315	3.54	0.16
375	3.72	375	3.13	0.15
435	3.72	435	2.84	0.14
495	3.72	495	2.62	0.12
560	3.72	560	2.43	0.10
616	3.75	616	2.30	0.10
668	3.78	668	2.12	0.10
737	3.81	737	2.10	0.07
800	3.81	800	2.00	0.07

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

t (min)	s (m)	t¢(min)	t/t¢	s¢(m)
5	0.02	5	161.00	0.54
10	0.07	10	81.00	0.50
15	0.10	15	54.33	0.47
20	0.12	20	41.00	0.44
25	0.15	25	33.00	0.42
30	0.17	30	27.67	0.40
40	0.20	40	21.00	0.37
50	0.22	50	17.00	0.35
60	0.24	60	14.33	0.33
70	0.26	70	12.43	0.31
80	0.28	80	11.00	0.30
90	0.29	90	9.89	0.29
100	0.30	100	9.00	0.27
110	0.32	110	8.27	0.27
120	0.33	120	7.67	0.26
180	0.38	180	5.44	0.21
240	0.41	240	4.33	0.19

300	0.44	300	3.67	0.16
360	0.46	360	3.22	0.15
420	0.48	420	2.90	0.14
480	0.50	480	2.67	0.12
540	0.52	540	2.48	0.11
600	0.53	600	2.33	0.11
660	0.54	660	2.21	0.10
720	0.55	720	2.11	0.09
800	0.57	800	2.00	0.09

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

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

$$T = \frac{2.3Q}{4\pi \Delta s'} = \frac{2.3 \times 270 \times 24}{4\pi \times 0.30}$$

= 3953.41 m^2/day, Ans.

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

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

$$T = \frac{2.3Q}{4\pi\,\Delta s'} = \frac{2.3 \times 270 \times 24}{4\pi \times 0.25}$$ = 4744.09 m²/day, Ans.

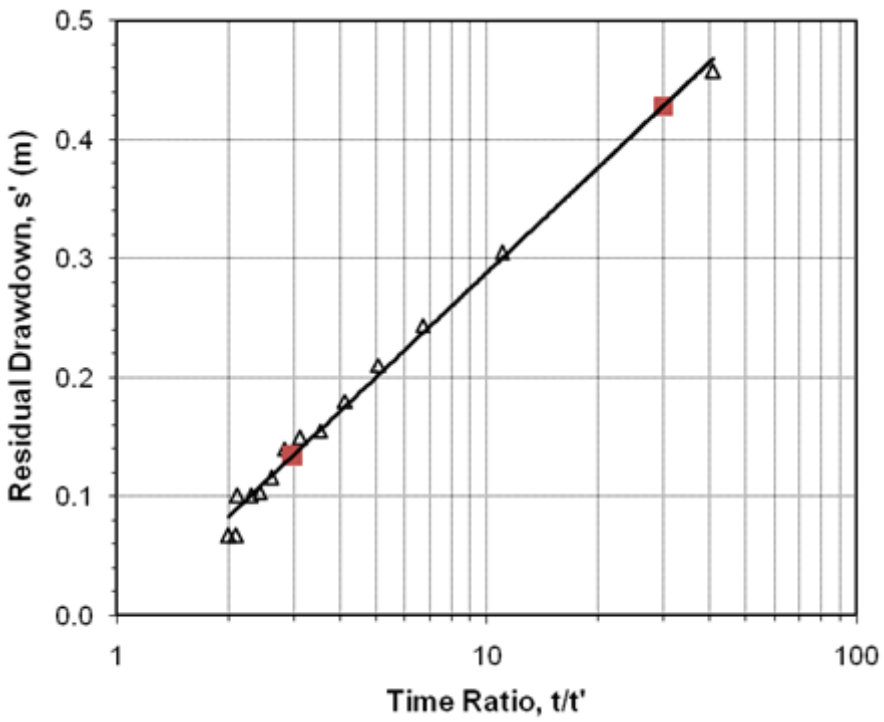


Fig. 14.6. Straight-line fitting to the residual drawdown-time ratio data obtained from the pumping well.

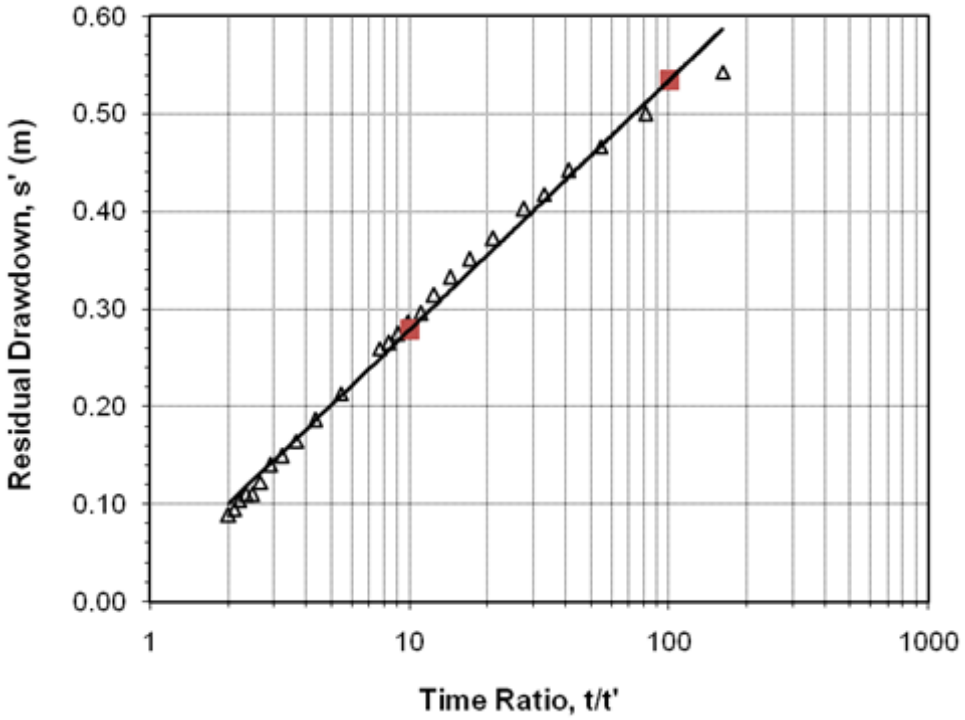


Fig. 14.7. Straight-line fitting to the residual drawdown-time ratio data obtained from the observation well.

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

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

$$= \frac{2.25\times3.29\times800}{22^2}\times10^{-\left(\frac{4\pi\times3.29\times0.57}{2.3\times4.5}\right)}$$

$$= 12.235\times10^{-2.277} = 0.0647, \textbf{ Ans.}$$

14.2.9 Cooper-Jacob Straight-Line Method for Recovery Data

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

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

14.3 Determination of Unconfined Aquifer Parameters

14.3.1 Unconfined Aquifer without Delayed Yield

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

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

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

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

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

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

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

14.3.2 Unconfined Aquifer with Delayed Yield

If the time-drawdown data obtained from unconfined aquifers exhibit significant delayed yield, the above-mentioned methods cannot be used for determining aquifer parameters. In this case, different methods have been suggested for reliable results which are: (i) Type-

Curve Method, and (ii) Neuman Straight-Line Method. Excellent descriptions of these methods can be found in Fetter (1994), Schwartz and Zhang (2003) and Batu (1998).

14.4 Identification of Aquifer Boundaries

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

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

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

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

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

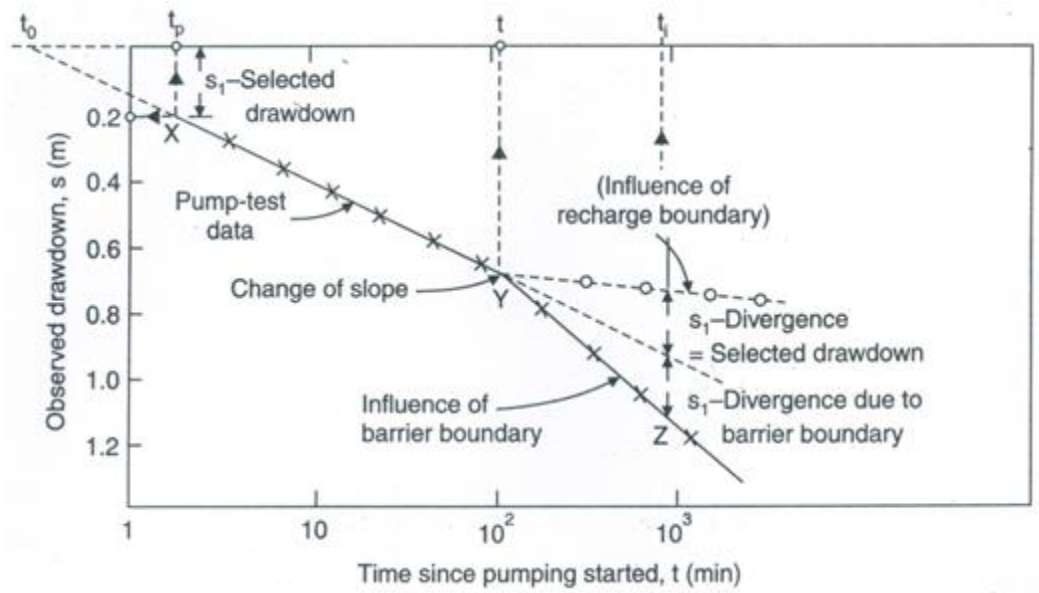


Fig. 14.8. Identification of aquifer boundaries using semi-logarithmic plot of time-drawdown data. (Source: Raghunath, 2007)

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

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

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

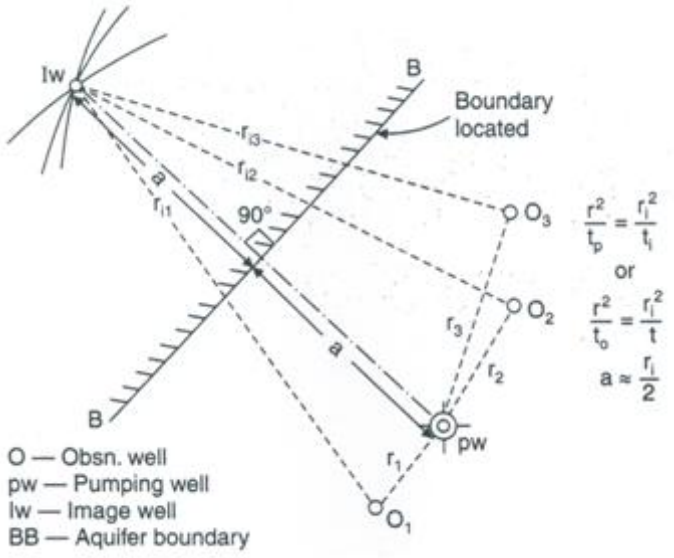


Fig. 14.9. Finding the location of an aquifer boundary.

(Source: Raghunath, 2007)

Lesson 15 Well Interference and Multiple Well Systems

15.1 What is Well Interference?

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

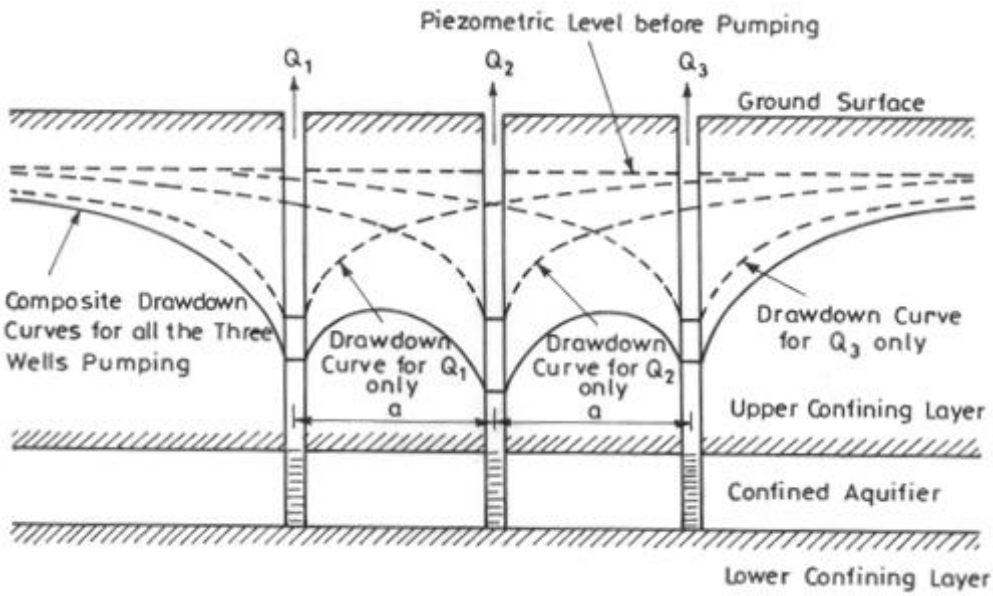


Fig. 15.1. Well interference in a well field having three pumping wells. (Modified from Todd, 1980)

15.1.1 Well Interference in Confined Aquifer Systems

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

$$s = s_a + s_b + s_c \ldots\ldots\ldots\ldots\ldots\ldots + s_n \qquad (15.1)$$

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

15.1.2 Unconfined Aquifer and Well Interference

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

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

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

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

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

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

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

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

15.2 Salient Applications of Well Interference

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

pumping level; considerably high energy is required for withdrawing groundwater from deeper depths.

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

15.3 Analysis of Multiple Well Systems

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

15.3.1 Well Systems Parallel to Line Source

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

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

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

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

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

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

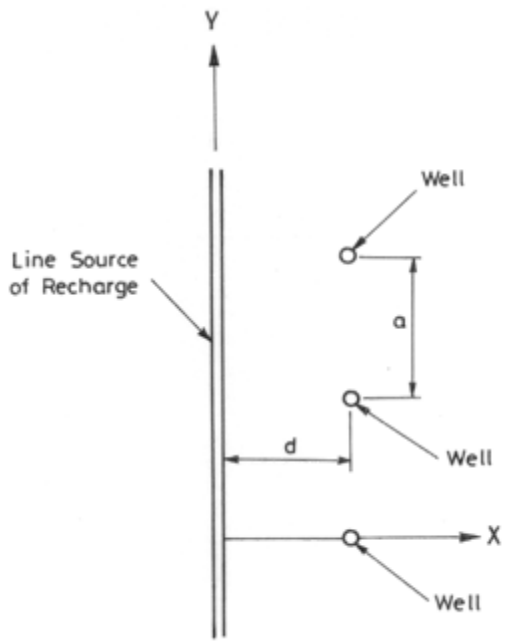


Fig. 15.2. Well parallel to a line source of recharge.

15.3.2 Well Discharge for Different Well Configurations

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

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

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

For Confined Aquifers: (15.5a)

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

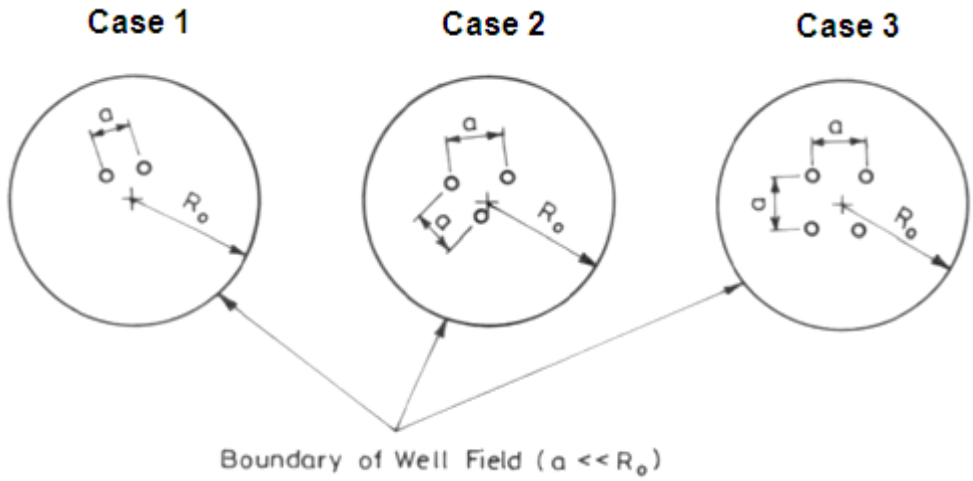


Fig. 15.3. Three configurations of wells closely spaced in a well field.

For Unconfined Aquifers:

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

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

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

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

For Confined Aquifers: (15.6a)

For Unconfined Aquifers:

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

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

For Confined Aquifers:

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

For Unconfined Aquifers:

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

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

15.3.3 Multiple Well Systems for Dewatering

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

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

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

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

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

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

(1) Circular Well System

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

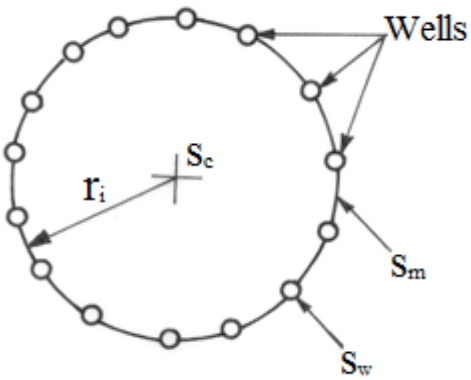


Fig. 15.4. Geometry for a circular dewatering system.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

(2) Linear Well System

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

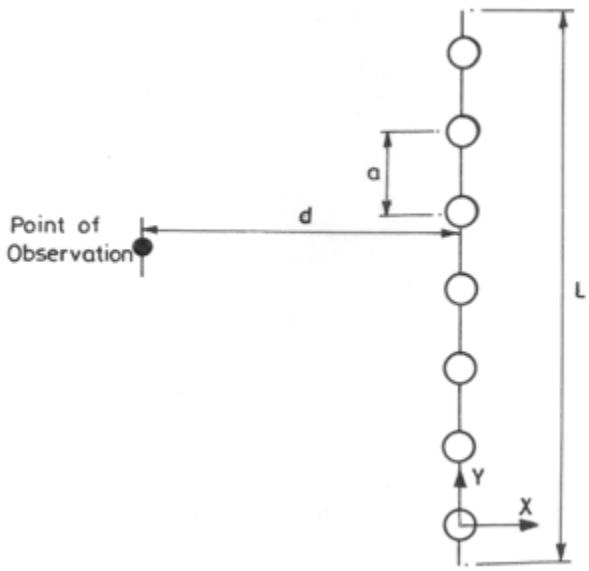


Fig. 15.5. Geometry for a linear dewatering system.

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

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

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

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

As L/a = N and NQ = Q_T, Eqn. (15.16) can be written as:

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

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

Similarly, for unconfined aquifer systems, we have:

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

Module 3_Design, Installation and Maintenance of Water Wells

Lesson 16 Design of Tubewells and Gravel Pack

16.1 Introduction

Proper design of water wells (tubewells and open wells) is essential in order to obtain optimum quantity of groundwater economically from a given aquifer system. The choice of open wells or borewells (tubewells) mainly depends on the economic condition of users, depth to groundwater availability and the quantity of water required. However, the availability of groundwater in a region depends on several factors such as geologic setting, geomorphology, rainfall, climate, topography, soil, drainage density, land use/land cover and vadose zone condition. Therefore, proper groundwater exploration and hydrogeologic investigation are pre-requisite to the design of water wells.

Major steps involved in the design of tubewells are: (i) selection of suitable size of the well and casing; (ii) length and location of the screen, including slot size and shape, and percentage of opening; (iii) selection of casing and screen material, and (iv) design of gravel pack (if gravel pack is necessary). These design steps are discussed in subsequent sections. Generally, the design of tubewells requires more design details than that of open wells. Also, tubewells in unconsolidated geologic formations involve consideration of more design details as compared to the wells in consolidated geologic formations (i.e., fractured/fissured rocks). Well designed water wells (tubewells or open wells) intend to ensure proper performance of the wells, reduced pumping and maintenance costs, and long service life of the well. This lesson focuses on the design of tubewells and gravel pack only. For the design of open wells in unconsolidated and hard rock formations, interested readers are referred to Michael et al. (2008).

16.2 Well Diameter

The size of a well needs to be carefully selected because it considerably affects the cost of well construction. It should be large enough to accommodate the pump used for groundwater withdrawal with a proper clearance (at least 5 cm) for installation and efficient operation. In deep wells which have both large static and pumping water levels, the well diameter can be reduced below the level of the lowest pump setting during dry periods, especially in the confined aquifers having relatively high piezometric level.

From the Thiem equation, it is apparent that for the same drawdown in the pumping well, the well yield is inversely proportional to, where R_0 is the radius of influence and r_w is the radius of the pumping well. Considering R_0 = 300 m, a pumping well of 60 cm diameter will yield 12% more than the well of 30 cm diameter. This demonstrates that drilling a large diameter well will not necessarily mean proportionately large yields. Table 16.1 summarizes the recommended diameters of pumping wells for various expected well yields.

Table 16.1. Recommended well diameters (Source: Raghunath, 2007)

Sl. No.	Expected Well Yield (L/min)	Internal Diameter of Well Casing (cm)		Nominal Size of Pump Bowl (cm)
		Minimum	Optimum	
1	400	12.5	15	10
2	400 - 600	15	20	12.5
3	600 - 1400	20	25	15
4	1400 - 2200	25	30	20
5	2200 - 3000	30	35	25
6	3000 - 4500	35	40	30
7	4500 - 6000	40	50	35
8	6000 - 10000	50	60	40

16.3 Well Depth

The depth of a pumping well depends on the depth at which aquifer layers exist and the number of aquifers to be tapped; this information could be obtained from the well logs (also called 'lithological logs') of an area. Usually, a pumping well is drilled up to the bottom of the aquifer so as to obtain greater well yield. If multiple aquifer layers exist in an area and money is not a constraint, pumping wells are drilled to penetrate two or more aquifer layers so that large well yield can be obtained for a longer time period. The poor-quality aquifer, if available, is backfilled or sealed in order to avoid the upward migration of the poor-quality groundwater when the well is pumped.

16.4 Design of Well Screen

The design of a well screen (also known as 'strainer') involves the determination of screen length, location of the screen, percentage open area, size and shape of openings (slots), screen diameter, and the selection of screen material. These design points are discussed in subsequent sub-sections.

16.4.1 Location and Length of the Screen

The length of a well screen is selected in relation to the aquifer thickness, available drawdown and stratification of the aquifer. In homogeneous confined aquifers, about 70 to 80% of the aquifer thickness is screened (Raghunath, 2007). The screen should best be positioned centrally at an equal distance between the top and bottom of the aquifer. In the case of heterogeneous confined and unconfined aquifers, it is better to screen the most permeable strata. On the other hand, in the case of homogeneous unconfined aquifers, the well screen is best positioned in the bottom portion of the aquifer. Selection of screen length is actually a compromise between two factors viz., specific capacity of the well and drawdown in the well. A higher specific capacity can be obtained by using as long a screen as possible, while more drawdown results by using short screens. The theory and experience have shown that screening the bottom one third of the aquifer provides the optimum design of a screen.

16.4.2 Size and Shape of Slots

The size and shape of the openings (slots) in the screen depend on the gradation, and the size and shape of the aquifer material so as to avoid entering of fine particles into the screen openings and to ensure that all the fine particles around the screen are washed out to improve the permeability of the aquifer material. For naturally developed wells, the size of the opening is selected as 40 to 70% of the size of the aquifer material (Raghunath, 2007). If the opening size selected on this criterion is smaller than 0.75 mm, then the use of an artificial gravel pack becomes essential.

16.4.3 Screen Diameter

After the length of the screen and the slot size has been selected, the screen diameter should be selected. Well screens (strainers) are available in a range of diameters. Suitable screen diameter is selected based on the desired yield of the well and the thickness of the aquifer. Recommended minimum diameters for well screens and casings are summarized in Table 16.2. Also, the entrance velocity near the well screen should not exceed 3 to 6 cm/s in order to avoid incrustation and corrosion and minimize friction losses (Raghunath, 2007). The entrance velocity is calculated by dividing the expected well yield with the total area of openings in the screen length.

Since the aquifers consisting of fine-grained materials tend to clog more easily than the aquifers consisting of coarser materials, it has been found that there exists a relationship between aquifer hydraulic conductivity and screen entrance velocity as shown in Table 16.3. To express the screen entrance velocities given in Table 16.3 in terms of screen size, the following equation can be used (Todd, 1980):

$$V_e = \frac{Q}{C\pi d_s L_s P} \qquad (16.1)$$

Where, V_e = optimum screen entrance velocity, Q = discharge of the pumping well, C = clogging coefficient (usually estimated at 0.5 on the basis that approximately 50% of the open area of a screen will be clogged by the aquifer material), d_s = diameter of the screen, L_s = length of the screen, and P = percentage of open area in the screen (available from the manufacturer's specifications).

Thus, for a given aquifer material, aquifer thickness, well yield and type of well screen, an appropriate diameter and length of well screen could be selected.

Table 16.2. Recommended minimum diameters for well casings and screens (Source: U.S. Bureau of Reclamation, 1977)

Sl. No.	Well Yield (m^3/day)	Nominal Pump Chamber Casing Diameter (cm)	Surface Casing Diameter (cm)		Nominal Screen Diameter (cm)
			Naturally Developed Wells	Gravel-Packed Wells	
1	<270	15	25	45	5
2	270 - 680	20	30	50	10
3	680 - 1900	25	35	55	15
4	1900 - 4400	30	40	60	20
5	4400 - 7600	95	45	65	25
6	7600 - 14000	40	50	70	30
7	14000 - 19000	50	60	80	35
8	19000 - 27000	60	70	90	40

Table 16.3. Optimum entrance velocity of water through well screens (Source: Walton, 1962)

Sl. No.	Aquifer Hydraulic Conductivity (m/day)	Optimum Screen Entrance Velocity (m/min)
1	>250	3.7
2	250	3.4
3	200	3.0
4	160	2.7
5	120	2.4
6	100	2.1
7	80	1.8
8	60	1.5
9	40	1.2
10	20	0.9
11	<20	0.6

16.4.4 Screen Material

The selection of screen material depends on the diameter and depth of the well, the type of aquifer layer, and the chemical composition of aquifer materials which dictates the quality of groundwater. The mineral content of the water, presence of bacterial slimes and strength requirements are important factors, which influence the selection of screen material. The screen material should be resistant to incrustation and corrosion, and should have enough strength to withstand the column load and collapse pressure. The principal indicators of corrosive groundwater are: low pH, presence of dissolved oxygen, CO_2 > 50 ppm, and Cl > 500 ppm (Raghunath, 2007). The principal indicators of incrusting groundwater are: total hardness > 330 ppm, total alkalinity > 300 ppm, iron content > 2 ppm, and pH > 8. Slime producing bacteria are often removed by chlorine treatment. This is followed by acid treatment to redissolve the precipitated iron and manganese.

As far as the choice of the screen material is concerned, steel has good strength, but it is not corrosion resistant. In contrast, brass has fair to good resistance to corrosion, but it has less strength than the steel. However, the strength of a well-made brass screen is adequate in most situations. Stainless steel has excellent strength and is highly resistant to most corrosive conditions. Well screens of corrosion resistant alloys such as Everdur metal, type 304 stainless steel and silicon red brass could be used for permanent installations (Raghunath, 2007). Different metals used for fabricating screens and their suitability in terms of resistance to corrosion are presented in Table 16.4.

Table 16.4. Corrosion resisting metals (Source: Raghunath, 2007)

Sl. No.	Name of Metal	Composition	Cost Factor	Colour of Finish	Suitability
1	Monel Metal	70% nickel and 30% copper	1.5	Bluish Silver	High NaCl and DO as in seawater, normally not used for drinking water.
2	Super Nickel	70% copper and 30% nickel	1.2	Bright nickel	High NaCl and DO as in seawater, normally not used for drinking water.
3	Everdur Metal (Silicon-bronze)	96% copper, 3% silicon and 1% manganese	1.0	Rich copper red	High TH, NaCl (without DO), and Fe. Usually used for municipal and industrial production wells. Highly resistant to acid treatment.
4	Stainless Steel	74% l.c. steel, 18% chromium and 8% nickel	1.0	Dark silvery steel	Water containing H_2S, DO, CO_2, Fe and bacteria. Usually used in municipal and industrial production wells.
5	Cupro Nickel	70% copper, 29% nickel and 1% arsenic	-	Bright nickel	-
6	Silicon Red Brass	83% copper, 16% zinc and 1% silicon	-	-	Resistant to acid and corrosion.

16.4.5 Type of Screens

The types of well screens are mainly decided based on the shape of screen openings (slots). Some of the commonly used screen types are shown in Figs. 16.1(a, b, c, d). The V-shape continuous-slot type of well screen is fabricated by winding cold-drawn wire, approximately triangular in cross-section, spirally around a circular array of longitudinal rods [Fig. 16.1(a)].

The V-shaped openings facilitate the fine particles to move into the well during development without clogging them. This type has the maximum percentage of open area per unit length of the screen, and the area of openings can be varied by adjusting the spacing of the wires wrapped (Raghunath, 2007). These screens are usually made of galvanished iron (GI), steel, stainless steel and various types of brass.

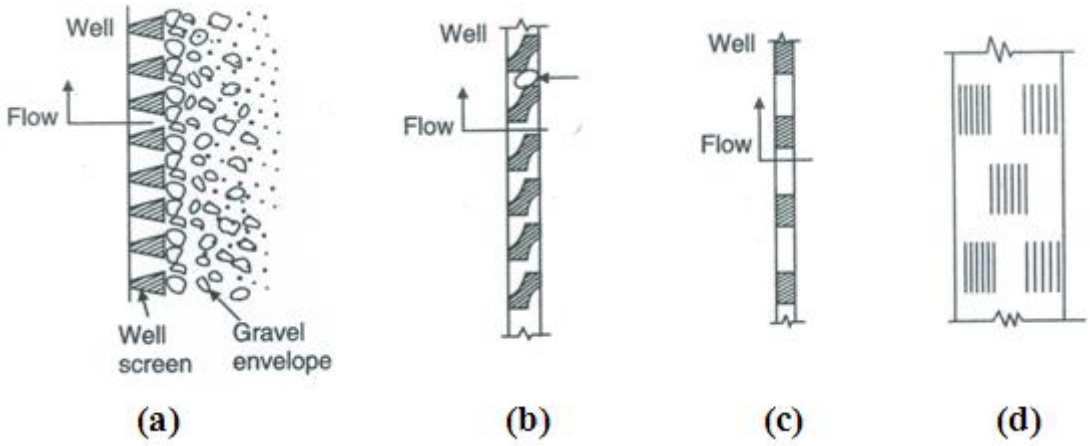


Fig. 16.1. Types of commonly used well screens: (a) V-shape continuous slot screen, (b) Louver-type screen, (c) Rectangular slot screen, and (d) Pipe-base well screen or Metallic filter point. (Source: Raghunath, 2007)

The Louver-type of screen has openings in the form of shutters [Fig. 16.1(b)], which are susceptible to be blocked by the fine particles during well development. Hence, this type of screen is most suitable for gravel-packed wells. The rectangular slot screen or the slotted pipe screen [Fig. 16.1(c)] is fabricated by cutting slots, vertical or horizontal, with a sharp saw, oxyacetylene torch, or by punching with a chisel and die or casing perforator. Some of the limitations of the slotted type well screens are wide spacing from the strength viewpoint, resulting in a low percentage of open area, lack of continuity and uniform size of the openings; the perforations made in the steel pipe may be more readily subject to corrosion at the jagged edges and surfaces, and the chance of blockage of such openings is high. This type of screen is least expensive. These days, slotted PVC pipes are widely used as screens, because especially in developing nations because they are light, cost effective, easy to handle, and free from corrosion. The use of slotted PVC pipes is generally limited to small diameter wells because of their relatively low strength and difficulty in providing proper fittings.

Furthermore, the pipe-base well screen or metallic filter point is made by using a perforated steel pipe [Fig. 16.1(d)]. A wire mesh is wrapped around the perforated pipe and is in turn is covered by a brass perforated steel. The percentage of open area in this type is normally low and the perforations are blocked by incrustation. This type of screen is relatively inefficient. In developing countries (including India), the coir-rope screen is sometimes used as an economical substitute for other types of screens. Coir rope is wrapped tightly around a circular array of steel flat, rods or bamboo strips. The life of the coir ranges from 7 to 8 years and can be increased by treating the coir with cashew shell oil (Raghunath, 2007). Hand boring sets are generally used for constructing the wells to be fitted with a coir-rope screen.

The coir-rope screen does not require gravel packing and development, but still it gives very good water supply. This type of screen is most suitable for shallow wells (depth not exceeding 12 to 15 m) in deltaic regions.

The best type of screen opening is the V-shaped slot that widens towards the inside of the screen, i.e., openings are bevelled inside. A major factor in controlling head loss through a perforated well section is the percentage of open area. For practical purposes, a minimum open area of 15% is desirable, which is easily obtained with many commercial (manufactured) screens but not with pre-perforated casings (Todd, 1980). Therefore, manufactured screens are preferred to pre-perforated casings because of larger open area and the ability to tailor opening sizes to aquifer conditions.

16.5 Design of Gravel Pack

16.5.1 Natural Gravel Pack

In many situations, the grain-size distribution of aquifer material is such that a properly selected well screen allows finer particles to enter the well, and to be removed during well development. Thus, after the development of the well, coarser particles are retained outside the well screen and form a permeable envelope around the well screen which is known as a 'natural gravel pack' and the well is called a naturally developed well. If the uniformity coefficient $\left(C_u = \frac{D_{60}}{D_{10}}\right)$ of an aquifer material for a naturally developed well (without an artificial gravel pack) is £5, the selected slot size should retain 40 to 50% of the aquifer material. However, if the uniformity coefficient (C_u) is greater than 5, the slot size should be selected such that it should retain 30 to 50% of the aquifer material (Todd, 1980).

16.5.2 Artificial Gravel Pack

A gravel-packed well is the well having an artificially placed gravel envelope around the well screen (Fig. 16.2). Salient advantages of the artificial gravel pack are (Todd 1980): (i) it stabilizes the aquifer tapped by the well, (ii) it avoids/minimizes sand pumping, (iii) it allows to use a large screen slot with a maximum open area, and (iv) it provides a zone of high permeability surrounding the well screen, which increases the well radius (known as 'effective radius' of the well) and well yield. When a well screen of a pumping well is to be surrounded by an artificial gravel pack, the size of the screen openings is decided based on the size of the gravel used for gravel packing.

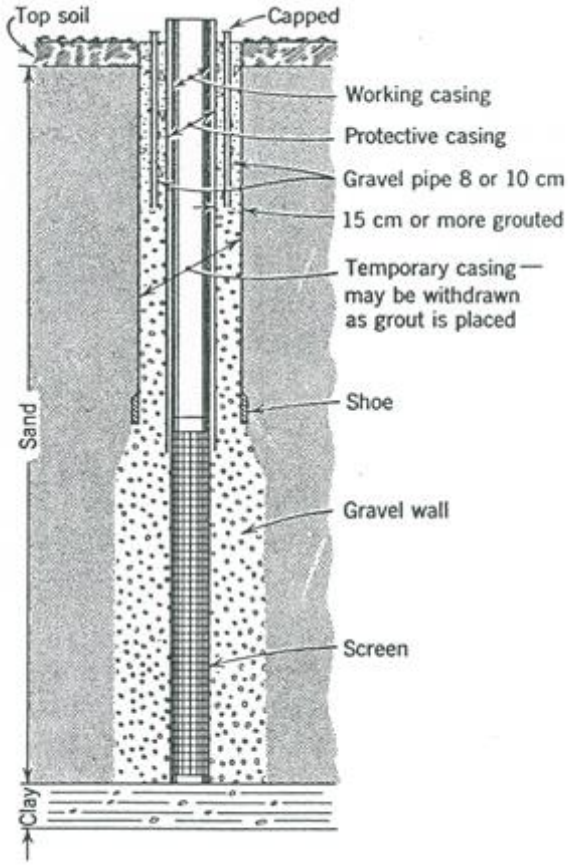


Fig. 16.2. Vertical cross-section of a gravel-packed well.

(Source: Todd, 1980)

Artificial gravel pack is required when the aquifer material is homogeneous with a uniformity coefficient (C_u) of less than 3 and an effective grain size (D_{10}) of less than 0.25 mm (Raghunath, 2007). The pack-aquifer ratio (ratio of the 30 or 50% size of the gravel-pack material to 30 or 50% size of the aquifer material) should be 4:1 if the aquifer material is fine and uniform. However, if aquifer material is coarse and non-uniform, the pack-aquifer ratio should be 6:1. The gravel-pack material should have a uniformity coefficient (C_u) of less than 2.5. The design procedure for selecting the gravel material is to determine the point D_{30} of the gravel pack which is equal to 4 to 6 times the D_{30} of the aquifer material obtained from the sieve analysis of the aquifer material samples and then drawing a smooth curve through this point (corresponding to D_{30} of the gravel pack) representing a material with a uniformity coefficient of 2.5 or less. This is the gradation of the gravel pack to be used (Raghunath, 2007). The slot size of the well screen is selected as D_{10} of the gravel-pack material to avoid segregation of fine particles near the screen openings. The width of screen slots ranges from 1.5 to 4 mm and the length ranges from 5 to 12.5 cm. A pack-aquifer ratio of 5 (i.e., ratio of 50% size of the gravel-pack material and 50% size of the aquifer material) has been successfully used in water wells (Raghunath, 2007).

The maximum grain size of a gravel pack should be less than 1 cm and the thickness of the gravel pack should be between 10 and 20 cm (Raghunath, 2007). Generally, the size of the pea gravel varies from 4 to 8 mm. Although a variety of formulas have been developed,

which relate gravel-pack grain-size gradations to aquifer grain-size gradations, the criteria for selecting gravel-pack material developed by the U.S. Bureau of Reclamation are shown in Table 16.5.

Table 16.5. Criteria for the selection of gravel pack material

(Source: U.S. Bureau of Reclamation, 1977)

Sl. No.	Uniformity Coefficient of Aquifer (C_u)	Gravel Pack Criteria	Screen Slot Size
1	<2.5	(a) C_u between 1 and 2.5 with the 50% size not greater than 6 times the 50% size of the aquifer. (b) If (a) is not available, C_u between 2.5 and 5 with 50% size not greater than 9 times the 50% size of the aquifer.	£ 10% passing size of the gravel pack
2	2.5-5	(a) C_u between 1 and 2.5 with the 50% size not greater than 9 times the 50% size of the formation. (b) If (a) is not available, C_u between 2.5 and 5 with 50% size not greater than 12 times the 50% size of the aquifer.	£ 10% passing size of the gravel pack
3	>5	(a) Multiply the 30% passing size of the aquifer by 6 and 9 and locate the points on the grain-size distribution graph on the same horizontal line. (b) Through these points draw two parallel lines representing materials with C_u £ 2.5. (c) Select gravel pack material that falls between the two lines.	£ 10% passing size of the gravel pack

The gravel selected for a gravel pack should be clean, dense, rounded, smooth and uniform, and should mainly consist of siliceous material (the allowable limit for calcareous material is up to 5%). Particles of shale, anhydrite and gypsum are also undesirable in the gravel-pack material. The detailed procedure for creating an artificial gravel pack is given in Todd (1980).

16.6 Design of Tubewells: An Example

Problem (Raghunath, 2007): A well log indicates that a clay layer of thickness 0-30 m from the ground surface is underlain by a layer of fine sand from 30 to 36 m and a layer of coarse

sand from 36 to 45 m followed by a silt layer at the bottom. If the expected well yield is 900 L/min, design all the components of tubewells for both naturally developed and artificially gravel packed cases, assuming that: (a) groundwater occurs under confined conditions with a piezometric level of 6 m below the ground surface, and (b) groundwater occurs under unconfined (water table) conditions with a water table level of 30.6 m below the ground surface. The results of the sieve analysis of the geologic sample obtained from depths 36 to 45 m are summarized in Table 16.6 below.

Table 16.6. Results of the sieve analysis

IS Sieve Size	Weight Retained on the Sieve (gm)	Cumulative Weight Retained (gm)	Cumulative Percentage Retained	Cumulative Percentage Passing
2.80 mm				
	57.4	57.4	14.4	85.6
2.00 mm				
	112.2	169.6	42.4	57.6
1.40 mm				
	84.8	254.4	63.6	36.4
1.00 mm				
	59.6	314.0	78.5	21.5
0.710 mm				
	43.2	357.2	89.3	10.7
Bottom Pan				
	41.3	398.5	100.0	0
Total	398.5	-	-	-

Solution:

Case 1: Tubewell in the Confined Aquifer

From Table 16.1, for an anticipated well yield (Q) of 900 L/min, a well casing diameter of 20 cm is recommended. The well screen should be located in the aquifer layer which lies between 36-45 m depths. The thickness of this aquifer is 9 m and it has a grain size mostly in the range of 0.6-2 mm and is classified as coarse sand according to the IS scale.

Length of the well screen, $l = \frac{3}{4} \times 9 = 6.75 \text{ m}$ (considering the position of the screen at the three-fourth of the aquifer thickness).

Considering 15% open area in the screen, the entrance velocity (V_e) for the well yield of 900 L/min can be calculated as:

$$\frac{900 \times 10^3}{60} = 0.15 \times (\pi \times 20 \times 6.75 \times 100) \times V_e$$

V_e = 2.358 cm/s, which is permissible.

Hence, a screen of 6.75 m length can be centrally located in the coarse sand aquifer. From the sieve analysis data of the aquifer sample, a grain-size distribution curve (or 'grading curve') is plotted on a semi-log paper (Fig. 16.3). From the grading curve, the effective size D_{10} = 0.69 mm and the uniformity coefficient U_c = 2.94. In this case, artificial gravel pack is not required since d_{10} > 0.25 mm and U_c > 2.5, and hence the well may be naturally developed when the screen slot size would be kept at D_{50} (i.e., 1.75 mm) to D_{60} (i.e., 2.03 mm), say 2 mm.

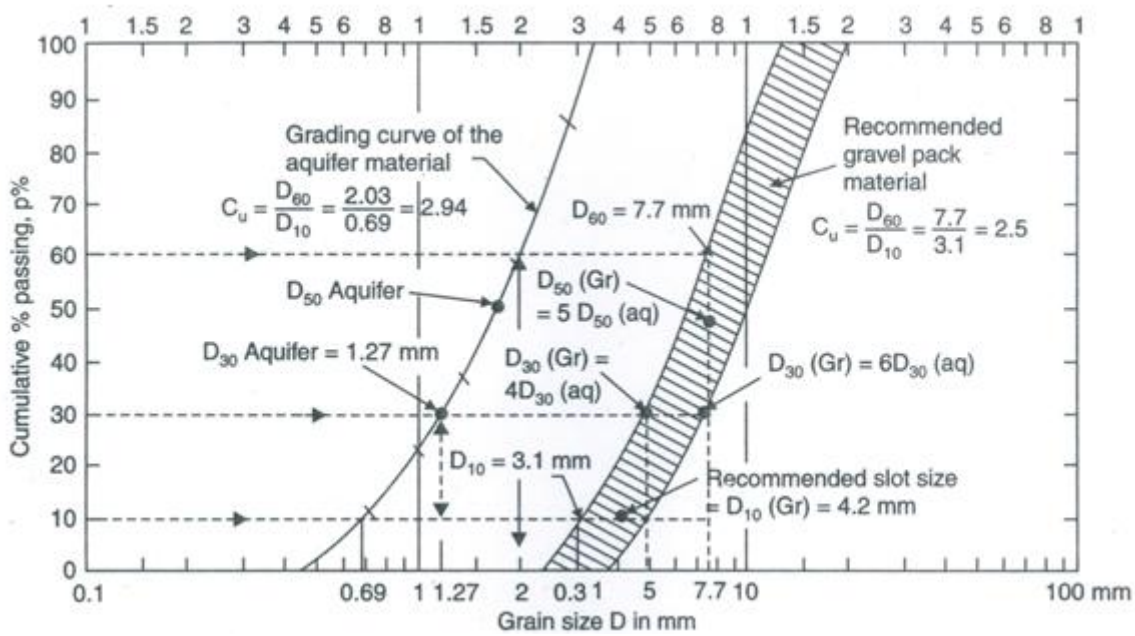


Fig. 16.3. Grain-size distribution curve illustrating the design of gravel pack. (Source: Raghunath, 2007)

Case 2: Tubewell in the Unconfined Aquifer

In this case also, the recommended well casing diameter will be 20 cm. Length of the well screen, $l = \frac{1}{3} \times 9 = 3$ m located at the bottom one-third of the coarse sand aquifer. Considering 15% open area in the screen, the entrance velocity (V_e) for the well yield of 900 L/min is calculated as:

$$\frac{900 \times 10^3}{60} = 0.15 \times (\pi \times 20 \times 3 \times 100) \times V_e$$

V_e = 5.305 cm/s, which is on the higher side.

Therefore, for a safer side, we should keep V_e within 3 cm/s. In order to bring the value of V_e below 3 cm/s, we can adopt a screen length of 4.5 m (i.e., half of the aquifer thickness) with a maximum open area of 18%. In this case, V_e is given as:

$$\frac{900 \times 10^3}{60} = 0.18 \times (\pi \times 20 \times 4.5 \times 100) \times V_e$$

V_e = 2.947 cm/s, which is permissible.

Artificial gravel pack is not required and the slot size may be taken as 2 mm as in the case of confined aquifer (Case 1).

Design of Artificial Gravel Pack: If an artificial gravel pack is desired, as C_u for the coarse sand aquifer is < 3.0, D_{30} of gravel-pack material = 4 to 6 times D_{30} of the aquifer material which is 4 to 6 times 1.27 mm from the grain-size distribution curve (Fig. 16.3), i.e., 5.08 mm to 7.62 mm. With these points for D_{30} for the gravel-pack material, smooth curves are drawn such that U_c for the gravel-pack material is 2.5. The hatched area in Fig. 16.3 shows the recommended gravel-pack material; clean pea gravel of size 10 mm can be used. The slot size should be kept at D_{10} of the gravel-pack material, which is 4.2 mm. The thickness of the artificial gravel pack could be 15-20 cm.

Check for Drawdown: In the absence of field or laboratory value of aquifer hydraulic conductivity (K), it can be estimated by the Hazen's formula as:

$$K = C \times d_{10}^{\,2} = 100 \times (0.069)^2 = 0.476 \text{ cm/s.}$$

Since the Hazen's formula overestimates the value of K, let's take two third of the above K value, i.e., $K = \frac{2}{3} \times 0.476 = 0.317$ cm/s.

Therefore, $\text{Transmissivity (T)} = K \times b = 0.00317 \times 9 = 0.029$ cm²/s.

$$\text{Specific Capacity, } \frac{Q}{s_w} = \frac{T}{1.4} \times \text{Efficiency of the well}$$

$$= \frac{0.029}{1.4} \times 0.60 \quad (\text{assuming well efficiency} = 60\%)$$

$$= 0.012 \text{ m}^3/\text{s/m.}$$

Note that the equations used for computing specific capacity of the well and aquifer hydraulic conductivity are approximate only; these estimates could be improved if more and better field data are available at the time of well design. Thus, we can estimate only probable drawdown in the well as follows:

$$\text{Probable drawdown in the well } (s_w) = \frac{\text{Well Discharge}}{\text{Specific Capacity}}$$

$$\Rightarrow s_w = \frac{900}{1000 \times 60 \times 0.012}$$

$$\therefore \quad s_w = 1.25 \text{ m.}$$

Thus, the probable drawdown in the well (s_w) is reasonably low, and hence it is permissible.

Lesson 17 Methods for Constructing Shallow Wells

17.1 Introduction

Different types of water wells are discussed in Lesson 8. For the construction point of view, water wells can be grouped under two categories: (a) Shallow wells, and (b) Deep wells. Shallow water wells can be either open wells or tubewells and are generally less than 15 m in depth. Deep water wells are usually tubewells and they are greater than 15 m in depth. However, in practice the terms 'shallow' and 'deep' are used in a relative sense and their depth limits vary considerably from one region/country to another. In this lesson, different methods of construction for shallow wells are described, while the methods of construction for deep wells are discussed in Lesson 18.

There are a variety of techniques for constructing water wells. Selection of a suitable method for well construction depends on the factors such as geologic conditions, purpose of the well, diameter and depth of the well, production capacity of the well, volume of work, maintenance, and availability of funds (Todd, 1980; Michael and Khepar, 1999; Raghunath, 2007). Shallow groundwater wells (<15 m deep) are constructed by digging, boring, driving, or jetting. Table 17.1 presents an extensive summary of the applications of these methods, together with the drilling methods used for constructing deep wells (>15 m deep). A brief description of digging, boring, driving and jetting methods is provided in the subsequent sections. Drilling methods/techniques are described in Lesson 18.

Table 17.1. Methods of well construction and their suitability

(Source: Todd, 1980)

Well Construction Method	Most Suitable Materials	Most Suitable Water Table Depth (m)	Usual Maximum Depth (m)	Normal Range of Diameter (cm)	Usual Casing Material	Customary Use	Well Yield (m^3/day)	Remarks
1.Augering (a) Hand	Clay, silt, sand, gravel (<2 cm)	2-9	10	5-20	Sheet metal	Domestic, drainage	15-250	Most effective for penetrating and removing clay. Limited by gravel over 2 cm. Casing required if

Auger								material is loose.
(b) Power Auger	Clay, silt, sand, gravel less than 5 cm	2-15	25	15-90	Concrete, steel or wrought-iron pipe	Domestic, irrigation, drainage	15-500	Limited by gravel over 2 cm, otherwise same as for the hand augers.
2. Driving (Hand, Air Hammer)	Silt, sand, gravel less than 5 cm	2-5	15	3-10	Standard weight pipe	Domestic, drainage	15-200	Limited to shallow water table, no large gravel.
3. Jetting (Light, Portable Rig)	Silt, sand, gravel less than 2 cm	2-5	15	4-8	Standard weight pipe	Domestic, drainage	15-150	Limited to shallow water table, no large gravel.
4. Drilling (a) Cable Tool	Unconsoli-dated and consolidated medium hard and hard rocks	Any Depth	450[b]	8-60	Steel or wrought-iron pipe	All uses	15-15,000	Effective for water exploration. Requires casing in loose materials. Mud-scow and hollow rod bits developed for drilling unconsolidated fine to medium sediments
(b) Rotary	Silt, sand, gravel less than 2 cm; soft to hard consolidated	Any Depth	45[b]	8-45	Steel or wrought-iron pipe	All uses	15-15,000	Fastest method for all except hardest rock. Casing usually not required during drilling. Effective for gravel envelope wells.

	rocks							
(c) Reverse Circulation Rotary	Silt, sand, gravel, cobble	2-30	60	40-120	Steel or wrought-iron pipe	Irrigation, industrial, municipal	2500-20,000	Effective for large- diameter holes in unconsolidated and partially consolidated deposits. Requires large volume of water for drilling. Effective for gravel envelope wells.
(d) Rotary Percussion	Silt, sand, gravel less than 5 cm; soft to hard consolidated rock	Any depth	600[b]	30-50	Steel or wrought-iron pipe	Irrigation, industrial, municipal	2500-15,000	Now used in oil exploration. Very fast drilling. Combines rotary and percussion methods (air drilling); cuttings removed by air. Would be economical for deep water wells.

Note: [a]Yield influenced mainly by the geology and availability of groundwater; [b]Greater depths reached with heavier equipment.

17.2 Digging

A pick and shovel are the basic implements for digging open wells in shallow aquifers. Loose material is brought to the surface in a container by means of rope and pulleys. Large dug wells can be constructed rapidly with portable excavating equipment such as clamshell and orange-peel buckets. For safety and to prevent caving, lining of wood or sheet piling should be placed in the hole to brace the walls (Todd, 1980; Michael and Khepar, 1999).

The depth of a dug well may be up to 20 m or more depending on the position of the water table, with the well diameter usually ranging from 1 to 10 m. Fig. 17.1 shows a typical dug well which is permanently lined with a casing/curb of wood staves, brick, rock, concrete or metal. The curb is perforated for entry of water and is firmly seated at the bottom. Gravel is backfilled around the curb and at the bottom of the well to control sand entry and possible caving. A properly constructed dug well penetrating a permeable aquifer can yield 2500 to 7500 m^3/day, although most domestic dug wells yield less than 500 m^3/day (Todd, 1980). Dug wells are generally used for individual water supplies in areas containing unconsolidated glacial and alluvial deposits. Further details of open-well (dug well)

construction in alluvial and hard rock formations can be found in Michael and Khepar (1999).

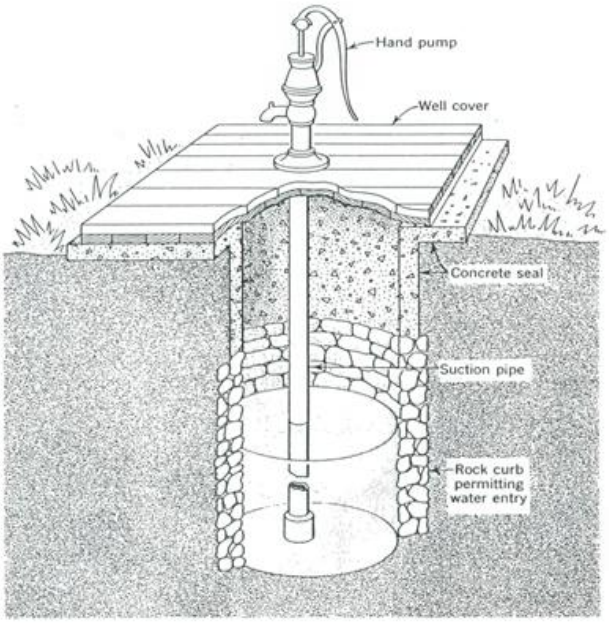


Fig.17.1. Typical domestic dug well with a rock curb, concrete seal and hand pump. (Source: Todd, 1980)

17.3 Boring

Augers are generally used for boring a well in shallow and unconsolidated aquifers. They are most suitable for the formations which don't cave. Augers are of two types: (a) Hand-operated augers, and (b) Power-driven augers.

17.3.1 Hand-Operated Augers

As shown in Fig. 17.2(a), hand-operated augers have cutting blades at the bottom that bore into the ground with a rotary motion. When the blades are full of loose earth, the auger is removed from the hole and emptied.

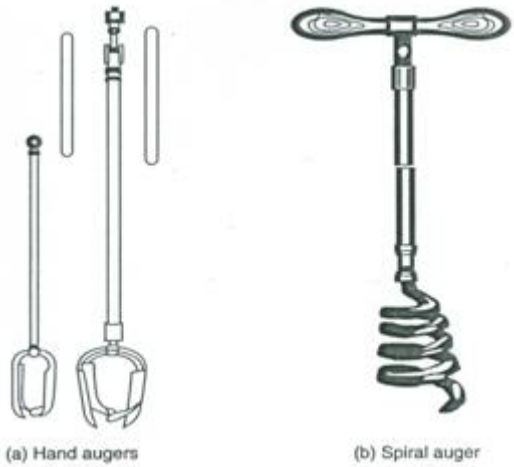


Fig. 17.2. Augers for boring wells: (a) Hand augers; (b) Spiral auger. (Source: Raghunath, 2007)

This procedure is repeated until the desired hole depth is reached. On the other hand, a spiral auger [Fig. 17.2(b)] is used to remove stones or boulders encountered during boring. Hand-bored wells can be up to 20 cm in diameter and 15 m in depth (Todd, 1980).

17.3.2 Power-Driven Augers

Power-driven auger consists of a cylindrical steel bucket with a cutting edge projecting from a slot in the bottom (Fig. 17.3). The bucket is filled up by rotating it in the hole by a drive shaft of adjustable length. When the container is full of excavated material, the auger is raised and emptied with the help of hinged openings located on the side or bottom of the bucket. Reamers, attached to the top of the bucket, help in enlarging holes to diameters exceeding the auger size. Power-driven augers can bore holes up to 1 m in diameter and to depths greater than 30 m (Todd, 1980).

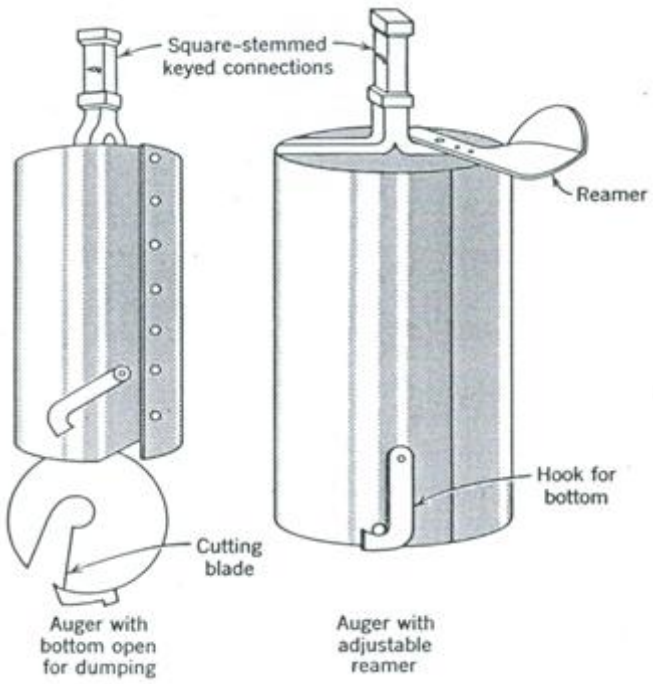


Fig. 17.3. Power-driven augers (Source: Todd, 1980).

There is another kind of power-driven auger called Continuous-flight power auger which has a spiral extending from the bottom of the hole to the surface. A screw conveyer is provided to carry the cuttings to the surface and the sections of the auger can be added as the depth increases. It is usually truck-mounted and can be operated by one person and can bore up to depths more than 50 m in unconsolidated formations devoid of large boulders. Note that where sticky clay formations are encountered, augers supplement other well-drilling methods because augers are more effective than any other penetrating device under such conditions.

17.4 Driving

In this method of well construction, a series of connected lengths of pipe is driven by repeated impacts into the ground to depths below the water table. Water enters the well through a drive point at the lower end of the well. A driven well with its driving mechanism is shown in Fig. 17.4. This consists of a screened cylindrical section protected during driving by a steel cone at the bottom. Driving can be done with a sledge, drop hammer or air hammer. The diameters of driven wells are in the range of 3 to 10 cm, and their depths are

usually less than 15 m although a few wells exceed 20 m depth (Todd, 1980). Suction-type pumps are used to extract water from driven wells.

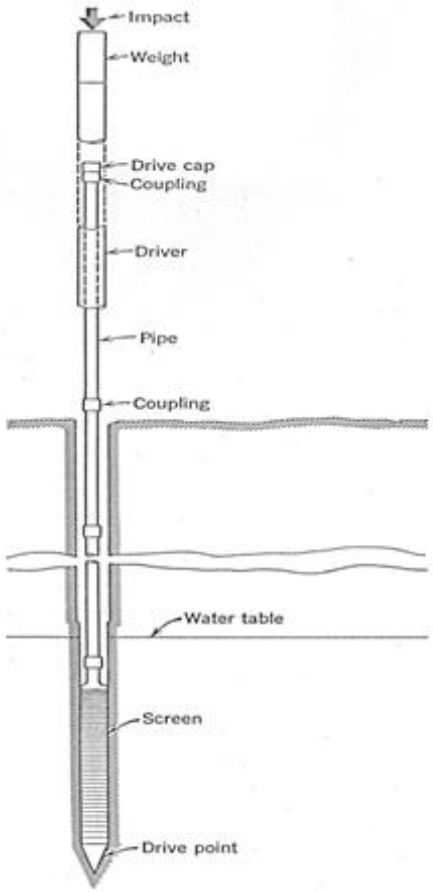


Fig. 17.4. A driven well with driving mechanism. (Source: Todd, 1980)

Driven wells can be installed only in unconsolidated formations which are relatively free of large gravels or rocks. The yield of driven wells is generally about 100-250 m^3/day (Todd, 1980). Driven wells are mostly used for domestic water supplies, temporary water supplies, and for exploration and observation. A series of driven wells connected by a suction header to a single pump is known as a well-point system which is used for dewatering excavations for foundations and other subsurface construction works. The main advantages of driven wells are that they can be constructed in a short time, at minimum cost, and even by one person.

17.5 Jetting

Jetted wells are constructed by the cutting action of a downward directed stream of water. The force of high velocity stream or jet of fluid loosens the subsurface materials and transports them upward and out of the hole. Jetting (Jet drilling) is achieved by a chisel-shaped bit attached to the lower end of a pipe string. Holes on each side of the bit serve as nozzles and water jets through these nozzles keep the bit clean and help loosen the material being drilled. Various types of jetting drill bits are shown in Fig. 17.5.

Fig. 17.5. Types of jetting drill bits. (Source: Todd, 1980)

A tripod and pulley, winch and a small pump of approximately 680 L/min at a pressure of 3.5 to 5 kg/cm^2 is used to force the drilling fluid (often normal water and in special cases, soft mud) through a hose on to the drill pipe and bit as shown in Fig. 17.6. During the jetting operation, the drill pipe is turned slowly to ensure a straight hole. When the casing extends to below the water table, the well pipe with screen attached is lowered to the bottom of the hole inside the casing. The outer casing is then removed, gravel is inserted in the outer space, and the shallow jetted well is completed.

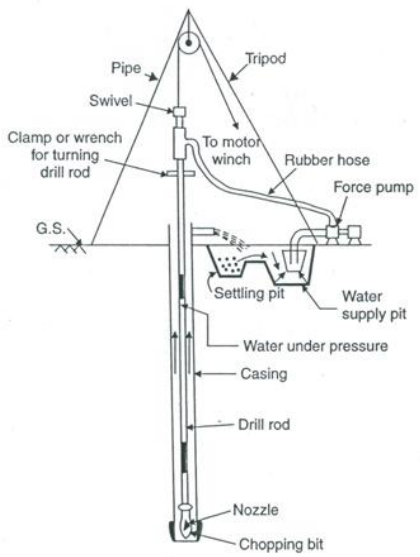


Fig. 17.6. Water jet method. (Source: Raghunath, 2007)

Jetting method is suitable for unconsolidated formations and can produce small- diameter holes of 3 to 10 cm to depths greater than 15 m (Todd, 1980). Jetted wells have only small yields, and are useful for exploratory test holes, observation wells and well-point systems.

Lesson 18 Methods for Constructing Deep Wells

18.1 Introduction

Construction of 'deep wells' (wells with depths more than 15 m) having high capacity as well as large diameter and depth is generally accomplished by using drilling methods. Various drilling methods used for constructing deep wells can be classified as: (i) Percussion drilling (also known as 'Cable tool drilling'), (ii) Rotary drilling, and (iii) Rotary-Percussion drilling (or 'Rotary-cum-Hammer drilling'). Each method has particular advantages under favorable hydrogeologic conditions; experienced drillers can easily select a suitable method for a given hydrogeologic setting. Brief descriptions of these drilling techniques are given below, whereas Table 18.1 summarizes the performance of these drilling techniques in diverse geologic formations. The construction procedure of a successful well is dependent on local conditions encountered during drilling, and hence the construction of each well should be treated as an individual project (Todd, 1980).

Table 18.1. Performance of drilling methods in different types of geologic formations (Source: Todd, 1980)

Sl. No.	Type of Geologic Formation	Performance of Drilling Methods		
		Percussion	Rotary	Rotary-Percussion
1	Dune Sand	Difficult	Rapid	Not Recommended
2	Loose Sand and Gravel	Difficult	Rapid	Not Recommended
3	Quicksand	Difficult, except in thin streaks. Requires a string of drive pipe.	Rapid	Not Recommended
4	Loose boulders in Alluvial Fans or Glacial Drift	Difficult; slow but usually can be handled by driving pipe.	Difficult, Frequently Impossible	Not Recommended
5	Clay and Silt	Slow	Rapid	Not Recommended

6	Firm Shale	Rapid	Rapid	Not Recommended
7	Sticky Shale	Slow	Rapid	Not Recommended
8	Brittle Shale	Rapid	Rapid	Not Recommended
9	Sandstone (poorly cemented)	Slow	Slow	Not Recommended
10	Sandstone (well cemented)	Slow	Slow	Not Recommended
11	Chert Nodules	Rapid	Slow	Not Recommended
12	Limestone	Rapid	Rapid	Very Rapid
13	Limestone with chert nodules	Rapid	Slow	Very Rapid
14	Limestone with small fractures	-	-	Very Rapid
15	Limestone (cavernous)	Rapid	Slow to Impossible	Difficult
16	Dolomite	Rapid	Rapid	Very Rapid
17	Basalts, thin layers in sedimentary rocks	Rapid	Slow	Very Rapid
18	Basalts, thick layers	Slow	Slow	Rapid
19	Metamorphic Rocks	Slow	Slow	Rapid
20	Granite	Slow	Slow	Rapid

18.2 Percussion Drilling

Percussion drilling is also known as 'cable tool drilling' or 'standard drilling', and it is accomplished with the help of a standard well-drilling rig, percussion tools and a bailer. Basically, the drilling procedure involves a regular lifting and dropping of a string of tools. On the lower end, a drill bit breaks/cuts the rock or other earth materials by impact. Thus, by repeated pounding and breaking/cutting operations, a borehole is formed. In particular, the percussion drilling equipment consists of a tool string (comprising a rope/swivel socket, a set of drilling jars, a drill stem, and a drill bit) suspended by a cable from a walking beam (truck mounted or tripod) or operated from a diesel engine, which lifts and drops the tool string (Fig. 18.1). Thus, the percussion/cable tool drilling rig consists of a mast, a multiline hoist, a walking beam and an engine. In modern designs, the entire assembly is truck mounted for easy portability.

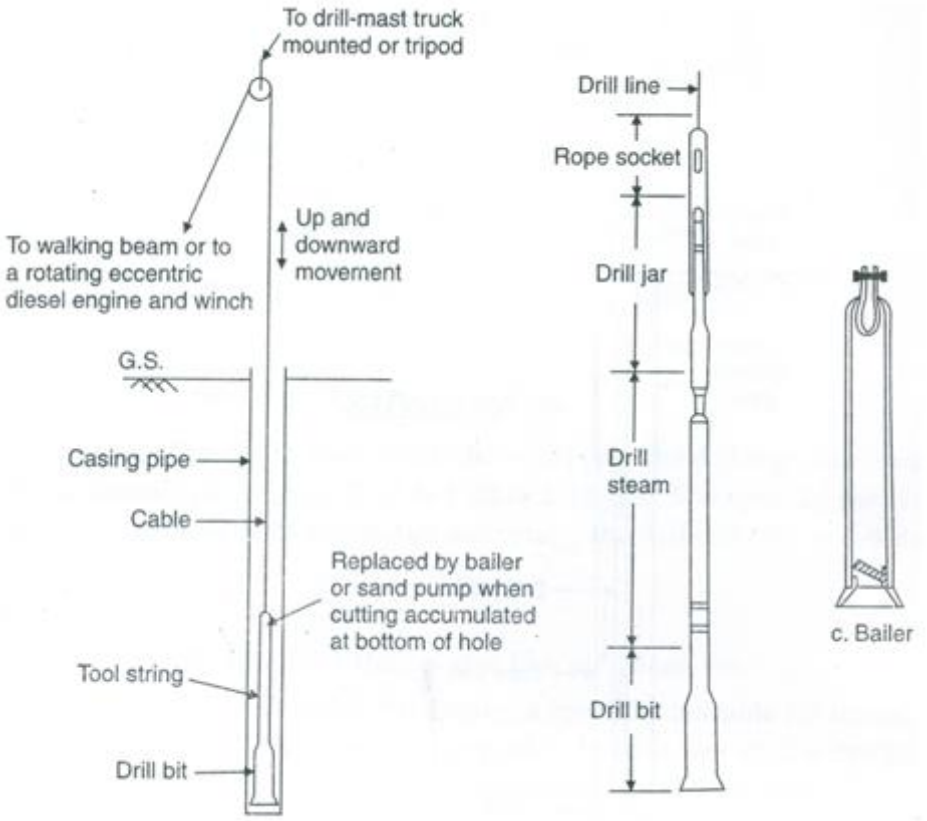


Fig. 18.1. Percussion drilling setup. (Source: Raghunath, 2007)

The most important part of the tool string is the drill bit (having a sharp chisel edge) which crushes/breaks almost all types of earth materials. The length of drill bits varies from 1 to 3 m and they weigh up to1500 kg (Todd, 1980). Drill bits of various shapes are manufactured for drilling in different subsurface formations. The drill stem is a long steel bar that provides additional weight to the bit and its lengths helps in maintaining a straight vertical hole while drilling in hard rock. Drilling jars consist of a pair of narrow linked steel bars and help in loosening the tools when they stick in the hole. Under normal tension on the drilling line, the jars remain fully extended. When tools get stuck, the drilling line is slackened and then lifted upward. This causes an upward blow to the tools because of which tools are released.

Swivel/rope socket connects the tool string to the cable. The wire cable, which carries and rotates the drilling tool on each upstroke, is known as drill line (Fig. 18.1).

Drill cuttings are removed from the well by a bailer or sand bucket (Fig. 18.1). A bailer consists of a section of pipe with a valve at the bottom and a ring at the top for attachment to the bailer line. The valve allows the cuttings to enter the bailer but prevents them from escaping. After filling, the bailer is hoisted to the surface and emptied. Drilling is accomplished by regular lifting and dropping of the tool string. As a result, the drilling line is rotated, the drill bit forms a round hole through the formation, the tool string is lifted and the hole is bailed. The cable tool method is capable of drilling holes of 8 to 60 cm in diameter through consolidated rock materials to depths of 600 m (Todd, 1980). It is least effective in unconsolidated sand and gravel formations, especially quicksand, because the loose material slumps and caves around the drill bit (Table 18.1).

Some of the advantages of the percussion drilling method are: (a) It is highly versatile in its ability to drill satisfactorily over a wide range of geologic conditions; (b) minimum water is required for drilling, a matter of concern in arid and semi-arid regions; (c) reasonably accurate sampling and logging of the formation material can be readily achieved; (d) the simplicity of design, ruggedness, and easy maintenance and repair of the rigs and tools are important advantages in isolated areas; and (e) rough checks on the water quality and yield from each water-bearing stratum can be made as drilling progresses. On the other hand, major drawbacks of the percussion drilling method are: (a) Slower drilling rate, (b) limitation of the drilling depth, (c) necessity of driving casing coincidentally with drilling in unconsolidated geologic formations, and (d) difficulty in pulling casing from deep boreholes.

18.3 Rotary Drilling

Rotary drilling method is a rapid method for drilling in unconsolidated formations. It consists of a rotating drill bit for cutting the borehole with a continuously circulated drilling fluid (usually a mixture of water and bentonite). The drilling fluid is forced through the hollow drill pipe on to the drill bit by a mud pump for removing the materials loosened by the drill bit (i.e., cuttings). The cuttings are carried upward in the hole by the rising mud, which flow to a settling pit where the cuttings settle out and the mud fluid overflows to a storage pit from where it is recirculated again (Fig. 18.2). The mud forms a clay lining on the wall of the borehole, which provides an adequate support for the wall of the hole, and hence casing is not normally required during drilling. The rotary drilling rig consists of a mast (derrick), a hoist, a power-operated revolving table that rotates the drill stem and bit, a pump for drilling mud, and an engine.

Deep wells up to 45 cm in diameter, and even larger with a reamer, can be constructed by the rotary drilling method (Todd, 1980). Drill bits for rotary drilling are available in different forms and commonly used designs are: (a) fishtail drill bit, (b) cone-type rock drill bit, and (c) carbide button drill bit as shown in Fig. 18.3.

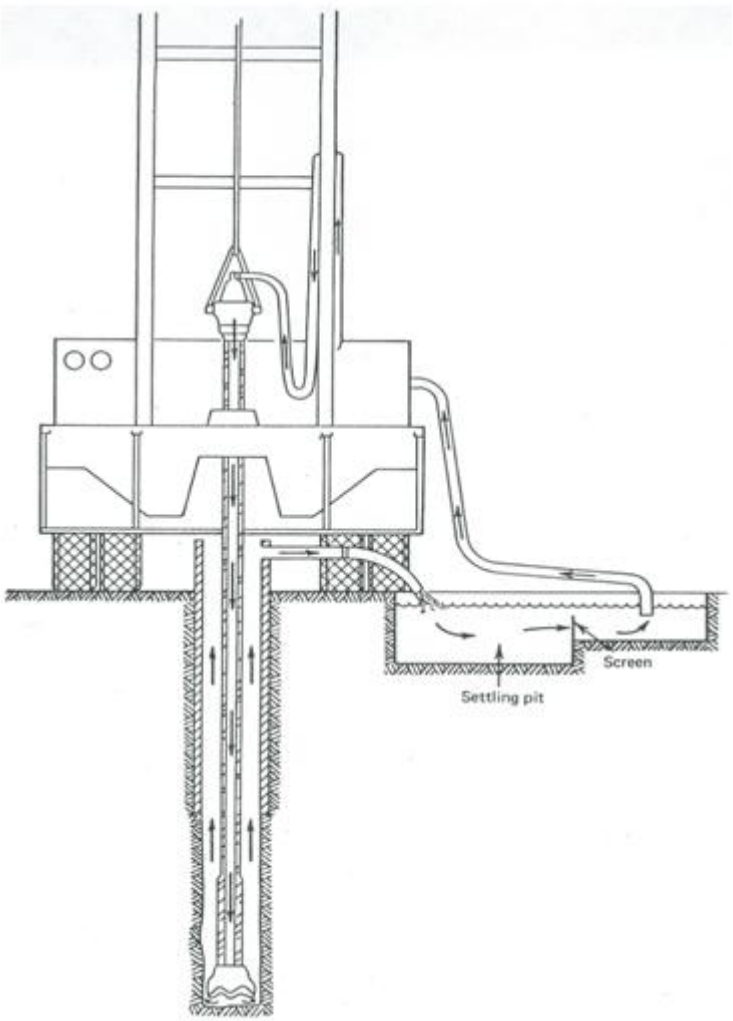


Fig. 18.2. Drilling mud circulation system for the rotary method. (Source: Todd, 1980)

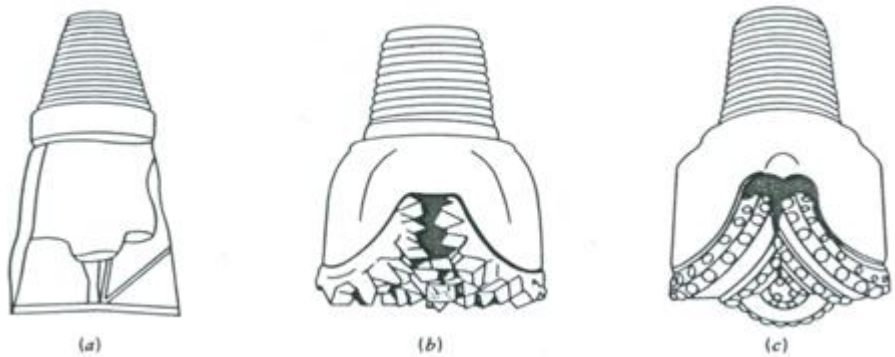


Fig. 18.3. Common types of rotary drill bits: (a) Fishtail drill bit, (b) Cone-type rock drill bit, (c) Carbide button drill bit. (Source: Todd, 1980)

The speed of rotation of drill bit in the borehole is 30 to 60 rpm. Drilling mud is essentially bentonite clay and the density of the mud fluid varies from 1.02 to 1.14 g/cm^3. The upward velocity of flow in the borehole is 0.7 to 1 m/s (Raghunath, 2007).

Initially, rotary drilling was employed for drilling oil wells and its application to water-well drilling has gradually increased over the years. The main advantages of the rotary drilling method are: (a) fast drilling rate, (b) no requirement of casing during drilling, and (c) the convenience for electric logging. There are also some disadvantages of this drilling method, which are: (a) high equipment cost, (b) more complex operation, (c) necessity to remove mud

cake (clay lining) during well development, and (d) the problem of lost circulation in highly permeable or cavernous geologic formations.

18.3.1 Air Rotary Method

Rotary drilling can also be done using compressed air instead of drilling mud. This technique is rapid and convenient for small-diameter holes in consolidated geologic formations (e.g., fractured rocks) where a clay lining is not required to support the walls against caving. Large-diameter holes can be drilled by employing foams and other air additives (Todd, 1980). It can be used for drilling wells to the depth of more than 150 m under favorable conditions.

Air rotary drilling is used in fractured/fissured rocks and is especially suitable for limestones. A striking feature of the air rotary drilling is its ability to drill consolidated geologic formations with little or no water.

18.3.2 Reverse-Circulation Rotary Method

The reverse-circulation rotary method is a modified form of the standard rotary method of drilling. In this method, the direction of water flow is reversed, i.e., from the annular space between the drill pipe and the wall of the hole through the drill bit into the hollow drill pipe upwards and discharged by a large-capacity pump into a large settling pit where cuttings settle out. The clear water returns to the borehole by gravity flow (Fig. 18.4). Relatively high velocity of water in the drill pipe (usually >2 m/s) enables the cuttings to be carried to the ground surface without the use of clay or other additives; the use of additives will increase viscosity which is not desirable for this method.

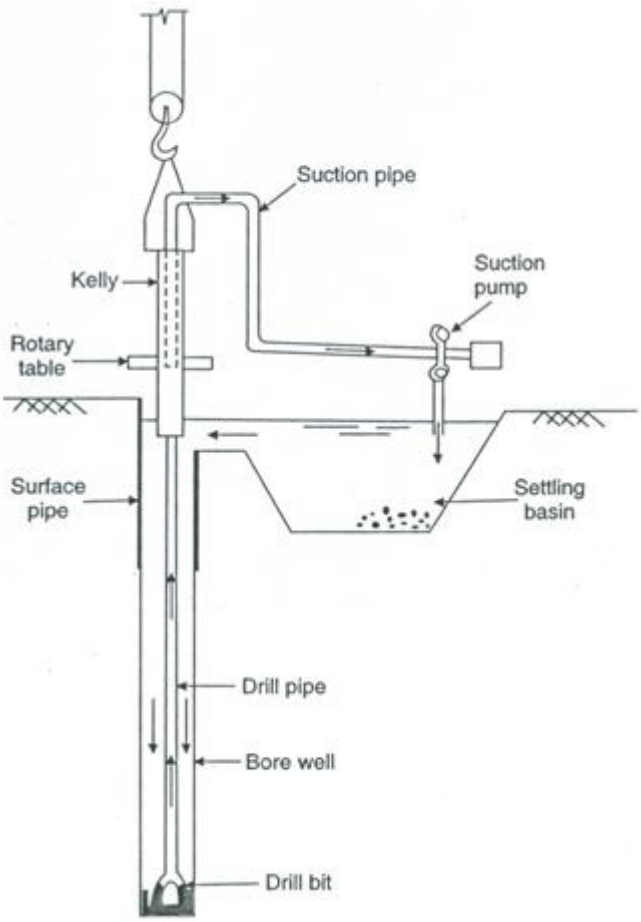


Fig. 18.4. Hydraulic rotary drilling with reverse circulation.

(Source: Raghunath, 2007)

In this system, drilling is done without a casing and hydrostatic pressure is used to support the walls of a borehole during drilling. Water level in the borehole is maintained at about 2 m above natural level or at ground level. The settling pit is about three times the volume of the material to be removed from the borehole. The diameter of the borehole is large compared to the drill pipe so that the velocity of the descending water in the annular space is low (≤ 30 cm/s), and the drill bit and drill pipe are rotated at speeds ranging from 10 to 40 rpm (Raghunath, 2007). The diameter of drill bits varies from 0.4 to 1.8 m. The reverse-circulation rotary rigs generally can drill to depths of 125 m; though suitable modifications with air-lift pumping can substantially increase this depth limit (Todd, 1980).

The reverse-circulation rotary method has become increasingly popular for drilling large-diameter boreholes in unconsolidated geologic formations. In fact, it is the most rapid drilling technique available for unconsolidated formations. The large diameters facilitate completion of the wells by artificial gravel packing. The minimum borehole diameter should be about 40 cm in order to avoid the erosion of the sides of the borehole (Todd, 1980), thereby restricting the downward velocity of the water in the borehole. The main disadvantage of this method is that it requires a large quantity of water to be readily available.

18.4 Rotary-Percussion Drilling

This method of drilling combines the percussion effect of cast tool drilling and the rotary action of rotary drilling. It is also known as 'Rotary-cum-Hammer drilling'. It uses compressed air as the drilling fluid which provides the fastest method for drilling in hard-rock formations (Todd, 1980); it can drill 15-20 cm holes to a depth of 120 m in 10-15 hours (Raghunath, 2007). A rotating drill bit, with the action of a pneumatic hammer delivers 10 to 20 impacts (blows) per second to the bottom of the hole. The diameter and depth of the hole is limited by the volume of air that can be exhausted through the hammer to remove the cuttings. A flush pump is used for flushing the hole and bringing the cuttings to the ground surface. Air compressor, pump and prime mover are usually mounted on a truck.

Compressed air must be supplied at a pressure of 750 to1350 kN/m^2 (to effectively remove the cuttings) and free air supply of at least 9 to 10 m^3/min for drilling 15-cm holes. The upward velocity in the space outside the drill pipe should be about 900 m/min. The rotation speed of the drill bit should be from 15 to 50 rpm (Raghunath, 2007). Reduced speed is required for drilling in harder and more abrasive rocks. In case of caving formations or incidence of large quantities of water, this method is not suitable. In this situation, the conventional rotary drilling with mud as a drilling fluid works satisfactorily (Todd, 1980).

Lesson 19 Well Completion, Development and Maintenance

19.1 Completion of Well Installation

After the construction of a pumping well, proper sanitary completion is necessary to produce safe water required for drinking and other purposes. Different well completion operations generally required for the wells constructed in unconsolidated formations are as follows (Todd, 1980):

1. Placement of casing and well screens,
2. Cementing/Grouting of casing, and
3. Gravel packing.

However, the wells constructed in consolidated formations where the material surrounding the well is stable, can be left as open holes (i.e., uncased wells) into which groundwater can enter directly. Hence, the above well completion operations may not be required for the wells constructed in consolidated formations. The details of well construction in consolidated formations can be found in Michael and Khepar (1999) and Sarma (2009).

19.1.1 Placement of Well Casing and Well Screen

(1) Types of Well Casing

Well casing is a lining to maintain an open vertical hole from ground surface to the aquifer. It seals out surface water and any undesirable quality groundwater and also provides structural stability against caving materials outside the well. Materials used for construction of well casings are wrought iron, alloyed or unalloyed steel and ingot iron (Todd, 1980). Polyvinyl chloride pipe is widely used as casing for shallow or deep, small-diameter observation wells. In cable tool drilling, the casing is driven into place, whereas in rotary drilling, the casing is smaller than the drilled hole. Well casing generally involves: (i) surface casing, and (ii) pump-chamber casing.

(i) Surface Casing

It is installed from ground surface through upper strata of unstable or fractured materials into a stable or relatively impermeable material. Surface casing has several functions: (a) it supports unstable materials during drilling, (b) it reduces loss of drilling fluids, (c) it facilitates installation or removal of other casing, and (d) it helps in placing a sanitary seal and serves as a reservoir for a gravel pack.

(ii) Pump-Chamber Casing

It comprises all the casing above the screen in wells of uniform diameter. The pump-chamber casing should have a nominal diameter at least 5 cm larger than the nominal diameter of the pump bowls (Todd, 1980). Non-metallic pipes such as ceramic clay, concrete, asbestos-cement, plastic, or fiberglass-reinforced plastic pipes are used where corrosion or incrustation is a problem.

(2) Placement of Well Screen

The method of installing well screens is influenced by the design of the well, drilling method and the problems encountered during drilling. The commonly used methods for screen installation are (Todd, 1980; Raghunath, 2007): (i) pull-back method, (ii) open-hole method, (iii) bail-down method, and (iv) wash-down method. A brief description of these methods is given below.

(i) Pull-Back Method

In this method, the casing is driven to the full depth of the well. Thereafter, the screen is lowered inside the casing and allowed to rest on the bottom of the hole. The casing pipe is then pulled upward enough to expose the full length of the screen in the water bearing formation. The lead packer provided at the top of the well screen is expanded by the swedge block in order to form a seal between the inside of the casing and the screen (Fig. 19.1). This method is commonly used in cable-tool drilled wells as well as in rotary drilled wells.

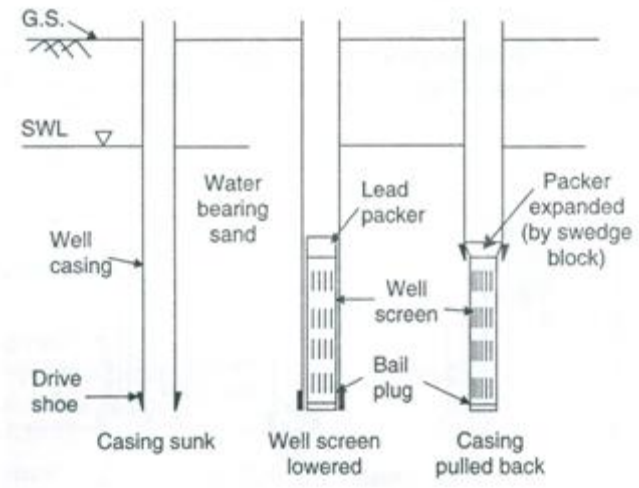


Fig. 19.1. Setting well screen using pull-back method.

(Source: Raghunath, 2007)

(ii) Open-Hole Method

In this method, the casing is first driven to a depth a little below the desired position for the top of the well screen. An open hole is then drilled in the sand below the casing and the casing is filled with the mud fluid (Fig. 19.2). The well screen is then lowered and the lead packer is swedged to the casing. This method is applicable to rotary-drilled wells.

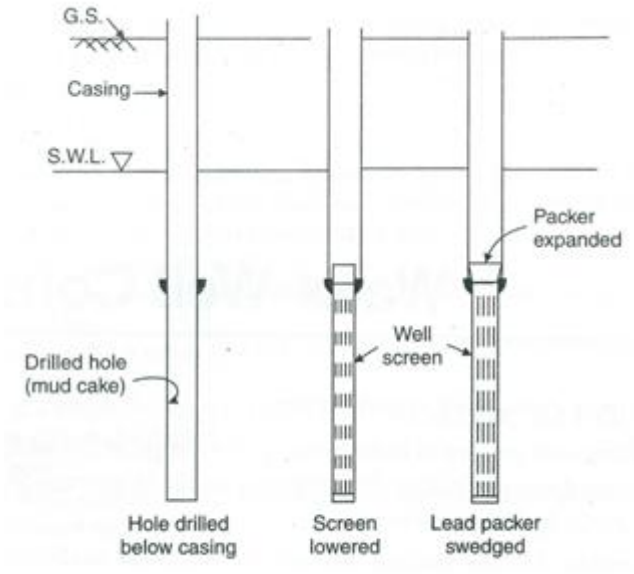


Fig. 19.2. Setting well screen using open-hole method.

(Source: Raghunath, 2007)

(iii) Bail-Down Method

In this method, the casing is driven to the intended position of the top of the screen. A bail-down shoe with connection fittings is fitted to the bottom of the screen. A string of bailing pipe is screwed on to the coupling of the bail-down shoe and the screen is suspended on this string. The screen is then lowered inside the casing till it bailed out from below the screen (Fig. 19.3). The lead packer is provided and expanded with the swedge block to seal the casing and screen. This method is suitable for rotary drilled wells as well as for percussion drilled wells.

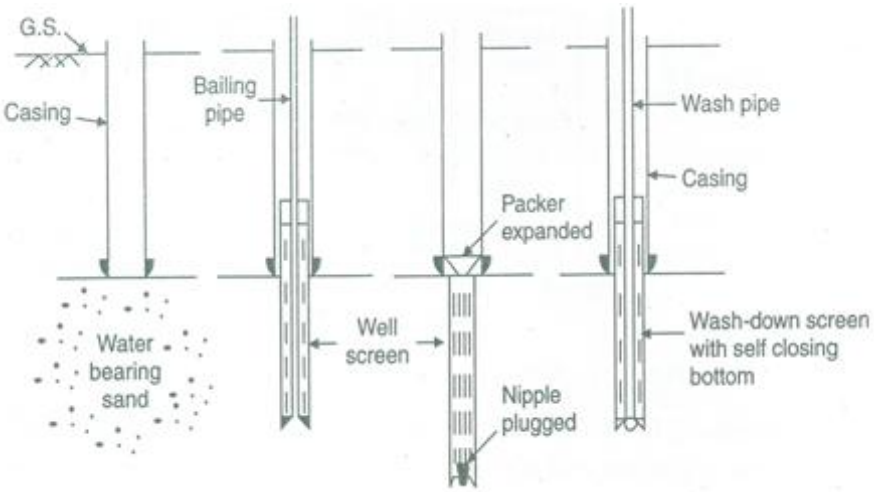


Fig. 19.3. Setting well screen using bail-down or wash-down method. (Source: Raghunath, 2007)

(iv) Wash-Down Method

In this method, the well casing is first set to the desired depth. A high velocity jet of drilling fluid is applied from a wash-down bottom, fitted to the end of the screen (Fig. 19.3). As a result, the sand is loosened and the screen is driven to the desired depth. Thereafter, water is circulated through the wash pipe to remove the drilling mud and the lead packer is expanded.

19.1.2 Grouting of Casing

Wells are grouted/cemented in the annular space surrounding the casing to prevent entry of polluted water, to protect the casing against the external corrosion and to stabilize caving rock formations. The grouting is achieved by using a cement grout between the outside of the casing and the inside of the drilled hole (Fig. 19.4). The grout is a mixture of cement and water of such a consistency that can be forced through the grout pipes and placed as needed.

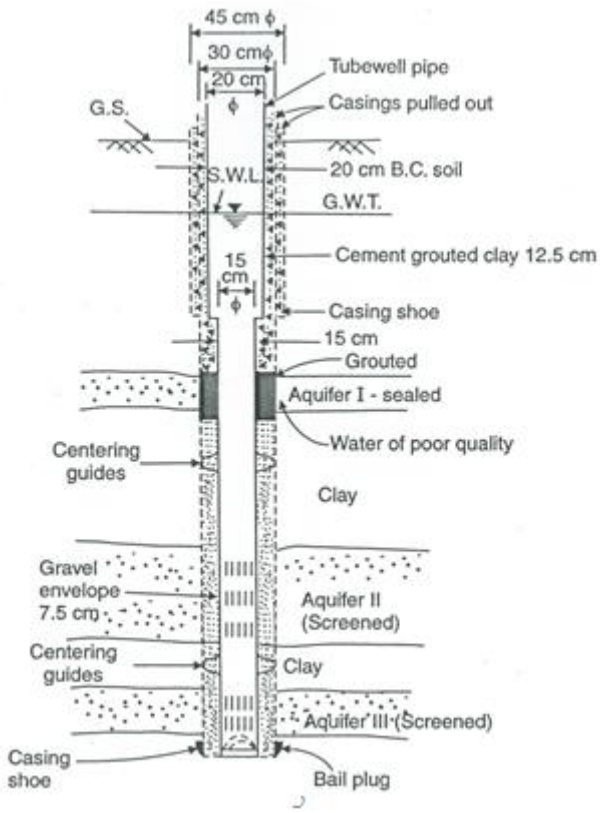


Fig. 19.4. Cementing/Grouting of the well casing (Shallow aquifer of inferior quality sealed and Aquifers II and III screened).

(Source: Raghunath, 2007)

The top of the casing should normally extend at least 50 cm above the level of the surrounding surface in such a way that it is isolated from direct contact with drainage wastes and sudden drainage discharges (Raghunath, 2007). The space around the casing should be grouted to a depth of about 6 m to seal the well from the entrance of surface drainage. A concrete platform should be constructed around the casing at the ground surface.

19.1.3 Gravel Packing

Gravel packing is done by placing an artificially packed gravel screen or envelope around the well screen. As mentioned in Lesson 16, the gravel pack has several advantages including the increase in well yield. A cross-section of a gravel-packed well is shown in Fig. 19.5.

Maximum grain size of a gravel pack should be nearly 1 cm, whereas the thickness should be in the range of 8 to 15 cm (Todd, 1980). The selected gravel should be washed and screened siliceous material that is rounded, abrasive-resistant and dense. Gravel should be placed in such a manner that it completely fills the annular space and minimizes segregation.

Gravel packing is generally done by placing two tremie pipes to the bottom of the well on opposite sides of the screen. Then the gravel is poured, washed, or pumped into the tremie pipes. The pipes are then pulled out of the well in stages

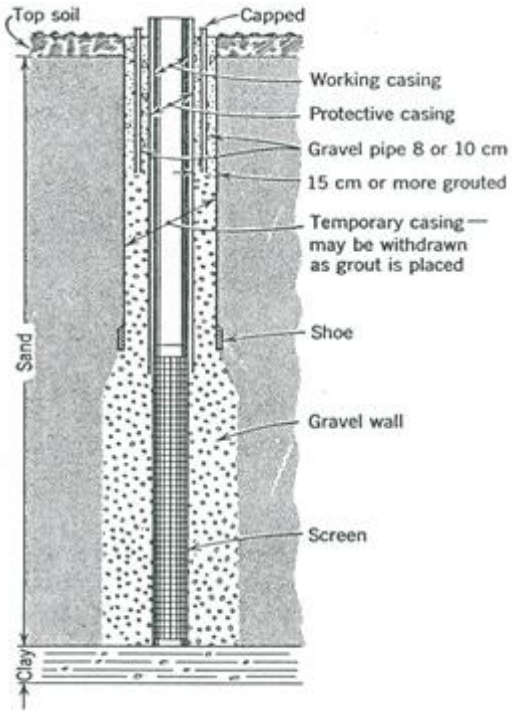


Fig. 19.5. Vertical cross-section of a gravel-packed well.

(Source: Todd, 1980)

as the pack is placed. In the cable-tool/percussion method of well drilling, the inner casing and screen are set inside the blank outer casing, the annular space is filled with gravel and then the outer casing is withdrawn out of the well. In sandy aquifers where a gravel pack is most essential, deep wells should be constructed by the rotary or reverse-circulation rotary method. The drilling fluid should be circulated and diluted with water before the gravel is introduced so as to avoid the clogging of the gravel pack.

19.2 Disinfection and Protection of Water Wells

19.2.1 Disinfection of Water Wells

After the completion of well installation, the pumping well and its appurtenances like the casing, pump, and pipe systems have to be disinfected or sterilized properly. Chlorinated water, prepared by dissolving dry calcium hypochlorite, liquid sodium hypochlorite or gaseous chlorine in water, is most effectively used for this purpose. The solution is poured into the well through the top of the casing, the water in the well is thoroughly agitated and allowed to stand for several hours. The well is then flushed to remove the entire disinfecting agent.

19.2.2 Protection of Water Wells

Proper sanitary precautions should be taken to protect the groundwater pumped from a well which is meant for human and animal consumption. Sources of pollution may exist either above or below the ground surface. Three commonly used protection measures for water wells are: (i) sanitary protection, (ii) frost protection, and (iii) abandonment of wells. These protection measures are succinctly described below.

(1) Sanitary Protection

The annular space between the outside of the casing and the inside of the drilled hole should be filled with cement grout in order to close avenues of access for undesirable water outside the casing. The top of the casing should be provided with a sanitary seal consisting of suitable packing glands that forms a water-tight seal between the pump column pipe and the well casing (Fig. 19.6). For pumps having an open-type base, a seal is required for the annular opening between the discharge pipe and the casing. The covers around the well should be made of concrete and should be elevated above the adjacent land level, and also should slope away from the well.

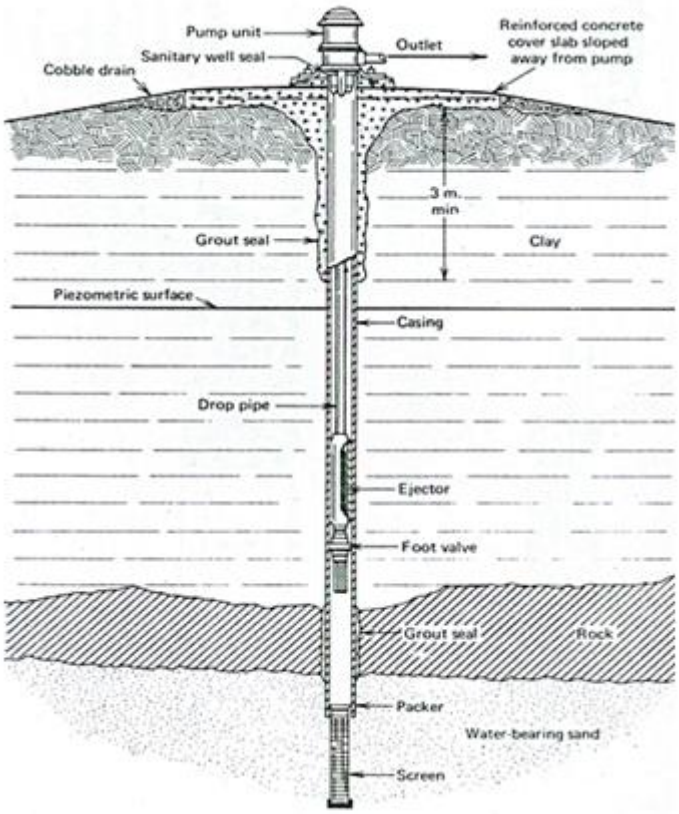


Fig. 19.6. A drilled well illustrating grout seal, concrete slab, and well seal for sanitary protection. (Source: Todd, 1980)

(2) Frost Protection

In the regions, affected by winter frost, the pumps and the water lines should be protected from freezing. The frost-proofing of a domestic well is performed by using a pitless adapter which is attached to the well casing and provides access to the well (Fig. 19.7). The discharge pipe runs about 2 m underground to the basement of the house (Todd, 1980).

(3) Abandonment of Wells

If the wells are of no use, i.e., they are abandoned, such wells must be sealed by filling them with clay, earth, or cement grout to prevent accidents, to avoid surface contaminants to enter the well and possible movement of polluted water from one aquifer to another as well as to conserve water in flowing wells.

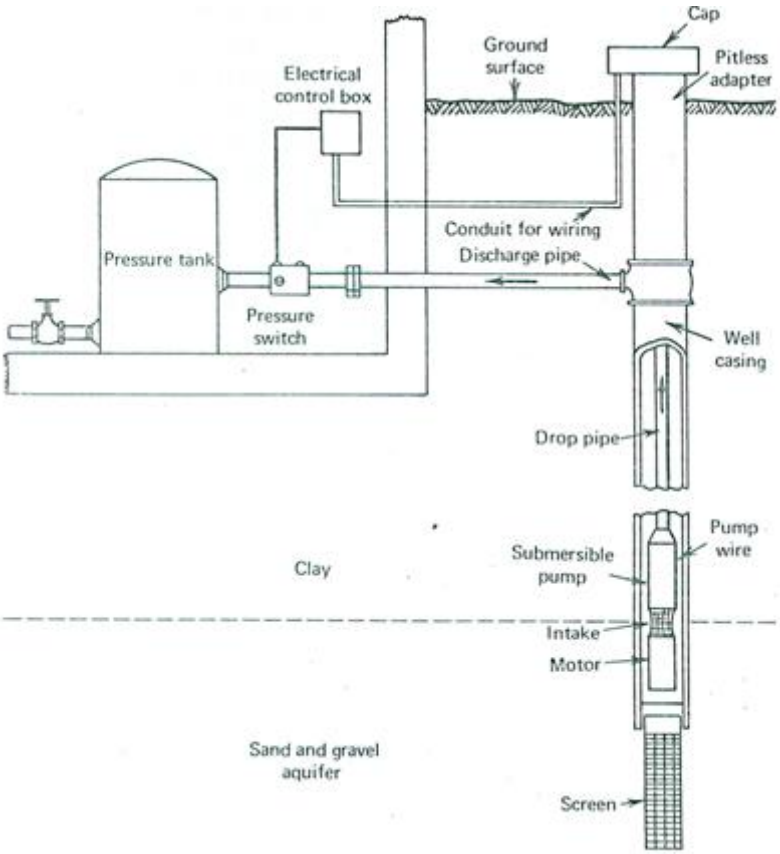


Fig. 19.7. Domestic well installation with a pitless adapter for protecting the well from frost. (Source: Todd, 1980)

19.3 Methods of Well Development

After the completion of a well, the new well is developed to increase its specific capacity (well discharge per unit drawdown), prevent sand pumping and obtain maximum economic well life. Well development is the process which causes reversals of flow through the screen openings so as to remove the finer material from the natural formations surrounding the perforated sections of the casing. As a result, the well provides clear (sand-free) water, thereby maximizing its specific capacity and well efficiency.

Various methods are available for developing a well, which include: (i) pumping, (ii) surging, (iii) use of compressed air, (iv) hydraulic jetting, (v) addition of chemicals/dispersing agents, (vi) hydraulic fracturing, (vii) backwashing, and (viii) use of explosives. These methods are briefly discussed below.

19.3.1 Pumping

This method of well development involves pumping a well in a series of steps from a low discharge to one exceeding the design capacity. At each step, the well is pumped until the water clears, after which the power is shut off and water in the pump column surges back into the well. The step is repeated until only clear water appears. The discharge rate is then increased and the procedure repeated until the final rate is the maximum capacity of the pump or well. This process agitates the fine material surrounding the well so that it can be carried into the well and pumped out. The courser fraction entering the well is removed by a bailer or sand pump from the well bottom.

19.3.2 Surging

In this method, a surge block attached to the bottom of a drill stem is repeatedly operated up and down in the well casing like a piston in a cylinder, thereby producing the required alternate reversals of flow. The procedure is completed when the loose materials accumulating in the bottom of the well become negligible. Further details of surging method can be found in Todd (1980) and Raghunath (2007).

19.3.3 Using Compressed Air

This method essentially involves both surging and pumping. During surging, a large volume of air is suddenly released and strong surge is produced by virtue of the resistance of water head, friction and inertia. Pumping is done using an ordinary air lift. The equipments required for well development by compressed air are: (a) air compressor capable of developing a maximum pressure of 700-1000 kN/m^2 and capacity of providing about 6 liters of free air for each liter of water, (b) pumping pipeline and air line with suitable means of raising and lowering each other independently, and (c) accessories such as flexible high pressure hose, relief value, quick opening value, pressure gauge, etc. The arrangement of pumping pipeline and air line is shown in Fig. 19.8.

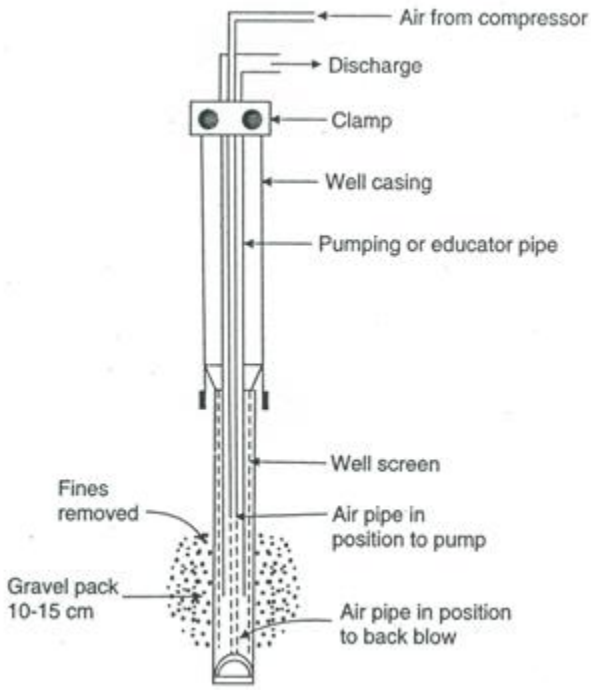


Fig. 19.8. Well development by compressed air.

(Source: Raghunath, 2007)

19.3.4 Hydraulic Jetting

In this method, a high velocity jet of water is applied horizontally through the screen openings with the tip of the nozzle at about 12-25 mm from the inner wall of the screen (Raghunath, 2007). A jetting tool coupled to the end of the pipe is lowered into the well. The top of the pipe is connected by a hose to a high pressure pump. As a pump is started, the jetting tool is slowly rotated and gradually raised or lowered so that the entire surface of the

screen receives jetting action. The well is pumped by another pump to maintain the hydraulic gradient so that water and the loosened fine particles will keep entering the well. Hydraulic jetting is particularly effective in developing gravel-packed wells (Todd, 1980). However, this method is not suitable when a perforated pipe is used as a screen. The disadvantage of the hydraulic jetting method is that it requires considerable amount of water for effective operation.

19.3.5 Addition of Chemicals/Dispersing Agents

Sometimes, chemicals/dispersing agents are added during well development to disperse the clay particles in the mud cake or in the formation to avoid their sticking to sand grains and to speed up the well development process. Several polyphosphates are used for this purpose such as tetrasodium pyrophosphate, sodium tripolyphosphate, and sodium hexametaphosphate and sodium septaphosphate (Raghunath, 2007). Sometimes, blocks of solid carbon dioxide (dry ice) are added to a well after aciditizing and surging with compressed air. The accumulation of gaseous carbon dioxide released by sublimation creates a pressure within the well and upon release this causes a burst of muddy water from the well, thereby helping in well development.

For developing open-hole wells in limestone or dolomite formations, hydrochloric acid is added to water, which removes fine particles and widens fractures/fissures leading into the well bore (Todd, 1980).

19.3.6 Backwashing

The backwashing method provides a surging effect for well development and is widely used by well drillers. In this method, the top of the well is fitted with an air-tight cover. The backwashing system consists of a discharge pipe, a long air pipe, a short air pipe, and a three-way valve as shown in Fig. 19.9.

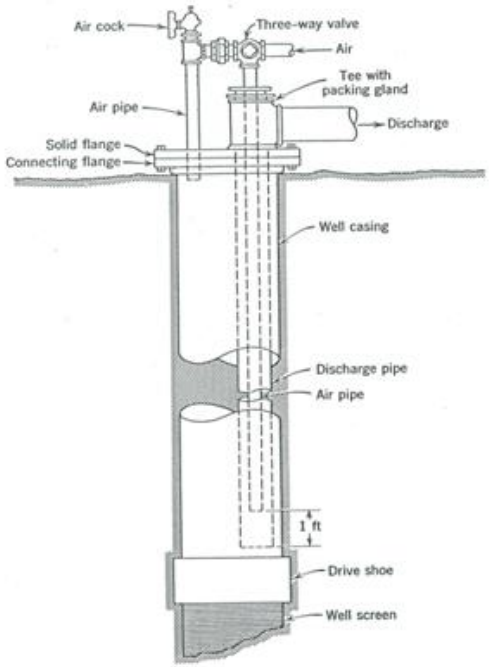


Fig. 19.9. Setup for well development by backwashing with air.

(Source: Todd, 1980)

Compressed air is released through the long air pipe, forcing air and water out of the well through the discharge pipe. After the water becomes clear, the air supply is stopped and the water is allowed to return to its static level. Thereafter, the three-way valve is turned to supply air into the top of the well through the short air pipe. This backwashes water from the well through the discharge pipe and at the same time agitates sand grains surrounding the well. Air is forced into the well until it starts escaping from the discharge pipe, after which the three-way valve is turned and the air supply is again directed down the long air pipe to pump the well. This procedure is repeated until the well is fully developed.

19.3.7 Hydraulic Fracturing

This technique is widely used in petroleum industry and it is occasionally employed for enhancing the yield of open-hole rock wells (Todd, 1980). In this method, a section of aquifer is isolated by inflatable packers on a pipe extending to the ground surface. After filling the pipe and isolated section with water, pump pressure is applied to fracture the rock. Sometimes, sand is pumped into the section to force the grains into the rock fractures in order to maintain the openings.

19.3.8 Using Explosives

Detonation of explosive in rock wells often increases yields by enlarging the borehole, increasing rock fractures, and removing fine-grained deposits on the face of the well bore. The method of well development using explosives is generally employed by experts, and hence its use is somewhat limited.

19.4 Well Maintenance and Rehabilitation

Although the expected service life of a well depends on the design, construction, development and operation of the well, proper maintenance of wells helps to improve their performance and increase their service life. Proper records of well discharge, drawdown, power consumption, operating hours, periodic checking of water quality and other such observations are very useful for formulating proper maintenance and rehabilitation plans.

19.4.1 Well Maintenance Criteria

In order to ensure that well performance does not deteriorate considerably, periodic test for well efficiency should be conducted and the results compared with the value obtained when the well was new. The well can be considered to be maintaining reasonable performance, if the well efficiency (ratio of theoretical drawdown to the measured drawdown in a pumping well) does not decline more than 15% from the original value (Roscoe Moss Company, 1990). When performance falls below an acceptable standard, redevelopment should be considered. As the monitoring of well efficiency is difficult and expensive, specific capacity (ratio of discharge to drawdown in a pumping well), which is a measure of the productivity of a pumping well, is often used to evaluate well performance with time. Seasonal and yearly influences on the pumping level must also be considered. The methodology for evaluating pumping wells can be found in Todd (1980), Raghunath (2007) and Roscoe Moss Company (1990).

19.4.2 Major Causes of Deteriorating Well Performance

The performance of wells can deteriorate because of one or combination of the following factors (Roscoe Moss Company, 1990):

(1) Change in hydrogeologic conditions, leading to declining groundwater levels.

(2) Excessive pumping of sand causing deterioration of the filter zone.

(3) Clogging of the filter zone by fine particles.

(4) Reduction in well discharge and well efficiency due to incrustation or bacterial growth in the well screen and/or filter zone.

(5) Degradation of water quality due to contamination.

(6) Structural damage of well casing and/or screen due to corrosion or other reasons.

19.4.3 Well Maintenance and Rehabilitation Techniques

Once the cause of deterioration in well performance is identified, one or more maintenance and rehabilitation techniques can be effective in restoring specific capacity and discharge of a pumping well or in protecting the well from further deterioration. Salient maintenance and rehabilitation techniques are as follows (Roscoe Moss Company, 1990):

(1) Redevelopment of the well using well development techniques.

(2) Chemical redevelopment of the well using acid or dispersing agents.

(3) Mechanical cleaning of the well screen by wire brushing or high-pressure jetting.

(4) Cleaning the screen with vibratory explosives.

(5) Structural repairs by setting liners, complete relining, or screen replacement.

(6) Reducing pumping rates, resetting the pump or deepening the well to offset decline in well production from lowering water tables.

Module 4_Groundwater Assessment and Management

Lesson 20 Evaluation of Groundwater Potential and Quality

20.1 Estimation of Static and Dynamic Groundwater Potential

20.1.1 Estimation of Static Groundwater Reserve

'Static groundwater reserve' refers to the groundwater which is available below the zone of natural groundwater-level fluctuation (GWREC, 1997). Static groundwater resources could be considered for withdrawal only during the period of severe and prolonged drought that also for drinking purposes. The computation of static groundwater resources in an area or basin can be done after delineating thickness of the aquifer and determining specific yield of the aquifer over the area or basin. The static groundwater reserve (SGWR) is estimated using the following equation (GWREC, 1997):

$$SGWR = (D_{AQ}-D_{WT}) \times A_Q \times S_Y \quad (20.1)$$

Where, D_{AQ} = depth to the aquifer base [L], D_{WT} = depth to water table in the pre-monsoon season [L], A_Q = areal extent of the aquifer [L^2], and S_Y = specific yield of the aquifer [fraction].

Depth to the aquifer base is estimated with the help of borewell (well log) data of multiple sites over the area under study (basin or sub-basin). Depth to water table can be measured by water-level indicator or water-level recorder. Areal extent of the aquifer can be estimated from the groundwater fluctuation data of multiple sites. The best method for determining specific yield is the field pumping test. It should be noted that Eqn. (20.1) can also be used to estimate static groundwater reserve at a site or in a zone.

20.1.2 Estimation of Dynamic Groundwater Reserve

'Dynamic groundwater reserve' refers to the long-term average annual recharge under conditions of maximum groundwater use (GWREC, 1997). Generally up to the end of October, the soil is saturated with moisture and no additional groundwater for irrigation is required. Groundwater irrigation actually starts from the beginning of November and continues until May of the next year. Therefore, the dynamic groundwater reserve (DGWR) can be estimated as follows (GWREC, 1997):

$$DGWR = (D_{WTE} - D_{WTO}) \times A_Q \times S_Y \quad (20.2)$$

Where, D_{WTE} = depth to water table in the pre-monsoon season of next year [L], D_{WTO} = depth to water table in the post-monsoon season of the current year [L], A_Q = areal extent of the aquifer [L^2], and S_Y = specific yield of the aquifer [fraction].

Eqn. (20.2) can be used to estimate dynamic groundwater reserve at a site or in a zone. It is worth mentioning that the dynamic groundwater reserve is also called exploitable groundwater reserve or utilizable groundwater reserve, which means that this amount of groundwater can be fully withdrawn to meet water demands in a year without causing any detrimental effect on the available groundwater reserve.

In general, the pre-monsoon season for a particular year refers to the period from October/November of previous year to the May/June of the particular year, whereas the post-monsoon season for a particular year refers to the period from October/November of that year until May/June of next year. However, for calculating dynamic groundwater reserve using Eqn. (20.2), the depth to water table in the month of May/June is taken as representative for the pre-monsoon season and the depth to water table in the month of October/November is taken as representative for the post-monsoon season. For the purpose of groundwater assessment in India, the monsoon season can be taken as May/June to September/October for all the areas of India except those areas where predominant rainfall occurs during the Northeast monsoon season. An additional period of one month after the cessation of monsoon is taken into account for the baseflow or groundwater recession which occurs immediately after the monsoon season (GWREC, 1997). In the areas (e.g., Tamil Nadu, parts of Kerala) where predominant rainfall is due to Northeast monsoon, the period of recharge assessment is based on pre-monsoon (October) to post-monsoon (February) water table fluctuations (GWREC, 1997).

Note that the static or dynamic groundwater reserve in an area/basin should be calculated zone-wise after dividing the area/basin into suitable zones and then computing static groundwater reserve or dynamic groundwater reserve in each zone on a yearly basis. The annual estimates of static/dynamic groundwater reserve, if necessary, could be used to calculate average values of static/dynamic reserve in each zone. A user-friendly computer software package named GWARA has been developed by Prof. Madan Kumar Jha, IIT Kharagpur (developer of this course), which facilitates the assessment of static and groundwater potential at a basin or sub-basin scale as well as the estimation and analysis of groundwater recharge.

20.1.3 Assessment of Status of Groundwater Development

Status/Level of Groundwater Development (GWD) is defined as (GWREC, 1997):

$$GWD = \frac{Existing\ Total\ Groundwater\ Draft}{Net\ Annual\ Groundwater\ Availability} \times 100 \qquad (20.3)$$

The term 'Net annual groundwater availability' refers to the available annual recharge after allowing for natural groundwater discharge in the monsoon season in terms of baseflow and/or groundwater outflow. 'Existing total groundwater draft' refers to the sum of groundwater withdrawals for different uses (i.e., irrigation, domestic, industry, etc.). Using Eqn. (20.3), a groundwater basin or sub-basin can be categorized based on the level/extent of groundwater extraction/exploitation (development) as shown in Table 20.1 (GWREC, 1997).

Table 20.1. Guidelines for assessing the extent of groundwater development

(Source: GWREC, 1997)

Sl. No.	Category	Level of Groundwater Development
1	White Zone/Region	< 65%
2	Grey Zone/Region	65 to 85%
3	Dark Zone/Region	85 to 100%
4	Overexploited Zone/Region	>100%

Besides the guidelines provided in Table 20.1 for evaluating the status of groundwater development, it has been recommended that long-term trend of groundwater levels in an area or basin should also be analyzed (GWREC, 1997) for an efficient evaluation of the status/level of groundwater extraction. A graph is prepared showing annual variation of pre-monsoon and post-monsoon groundwater levels for a minimum period of 10 years. The trends of both pre-monsoon and post-monsoon groundwater levels are depicted in the same graph. The trends of these two types of groundwater-level plots are used to interpret the scope of further groundwater development/exploitation with the help of standard guidelines suggested by GWREC (1997). This exercise can be performed for planning future groundwater resources development in a basin/sub-basin concerning additional withdrawal of existing groundwater resources.

20.2 Water Quality and Groundwater Contamination

20.2.1 Water Chemistry vis-à-vis Water Quality

The quality of water that we drink as well as the quality of water in our lakes, streams, rivers, and oceans is a critical parameter in determining the overall quality of our lives. Water quality is determined by the solutes and gases dissolved in the water, as well as the matter suspended in and floating on the water. Water quality is a consequence of the natural physical, chemical and biological state of water as well as of the changes occurred due to human activities. It determines the usefulness of water for a particular purpose. If human activities alter the natural water quality so that it is no longer suitable for a use for which it had been suited earlier, the water is said to be polluted or contaminated. It should be noted that in many regions of the world, water quality has been altered by human activities, but the water is still usable; though ever-growing pollution of both surface water and groundwater is posing a serious threat to our life and ecosystems throughout the world.

One basic measure of water quality is the total dissolved solids (TDS), which is the total amount of solids (in milligrams per liter) left when a water sample is evaporated to dryness.

Table 20.2 shows the classification of water based on TDS (Fetter, 1994). Water naturally contains a number of different dissolved inorganic constituents. The major cations are calcium, magnesium, sodium, and potassium; the major anions are chloride, sulfate, carbonate, and bicarbonate. Although not in ionic form, silica can also be a major constituent. These major constituents constitute the bulk of the mineral matter contributing to total dissolved solids (TDS). In addition, there may be minor constituents present in water such as iron, manganese, fluoride, nitrate, strontium, and boron. Trace elements such as arsenic, lead, cadmium, and chromium may be present in amounts of only a few micrograms per liter, but they are very important from a water-quality viewpoint because of their harmful effects on human health.

Table 20.2 Classification of water based on TDS

(Source: Fetter, 1994)

Sl. No.	Water Class	Value of TDS (mg/L)
1	Fresh	0-1,000
2	Brackish	1,000-10,000
3	Saline	10,000-100,000
4	Brine	>100,000

Furthermore, dissolved gases are present in both surface water and groundwater. The major gases of concern are oxygen and carbon dioxide. Nitrogen, which is more or less inert, is also present. Minor gases of concern are hydrogen sulfide and methane. Hydrogen sulfide is toxic and imparts a bad odor, but it is not present in the water that contains dissolved oxygen (DO).

Surface water may be adversely impacted by human activities. If organic matter, such as untreated human or animal waste, is placed into the surface water body, dissolved oxygen (DO) levels diminish as microorganisms grow using organic matter as an energy source and consuming oxygen in the process. The total dissolved solids (TDS) may increase due to the disposal of wastewater, urban runoff, and increased erosion due to land-use changes in a river basin. Generally groundwater has higher dissolved mineral concentrations than surface water because of the extended contact time between groundwater and rocks and soils.

The natural quality of groundwater varies substantially from place to place. It can range from total dissolved solids contents of 100 mg/L or less for some fresh groundwater to more than 100,000 mg/L for some brine found in deep aquifers (Fetter, 1994). Taking this variability into account, the U.S. Environmental Protection Agency (U.S. EPA) has

developed a three-part classification system for the groundwater of the United States (U.S. EPA, 1984).

Class I: Special Groundwaters are those that are highly vulnerable to contamination because of the hydrological characteristics of the areas under which they occur and that are also either an irreplaceable source of drinking water or ecologically vital because they provide baseflow for a particularly sensitive ecosystem.

Case II: Current and Potential Sources of Drinking Water and Waters Having Other Beneficial Uses are all other groundwaters except Class III.

Class III: Groundwaters Not Considered Potential Sources of Drinking Water and of Limited Beneficial Use because the salinity is greater than 10,000 mg/L or the groundwater is otherwise contaminated beyond levels that can be removed using methods reasonably employed in public water-supply treatment.

The U.S. EPA uses the above classification scheme in promulgating rules and regulations at the federal level (Fetter, 1994). The highest degree of protection is given to Class I groundwater.

Note that the pollution of surface water frequently results in a situation where the contamination can be seen or smelled. However, the contamination of groundwater most often results in a situation that cannot be detected by human senses. Groundwater contamination can be due to bacteriological or toxic agents or simply due to an increase in common chemical constituent to a concentration whereby the usefulness of the water is impaired.

20.2.2 Sources of Groundwater Contamination

Humans have been exposed to hazardous substances dating back to prehistoric times when they inhaled noxious gases from volcanoes and in cave dwellings. Pollution problems started in the industrial sector with the production of dyes and other organic chemicals developed from the coal tar industry in Germany during the 1800s (Bedient et al., 1999). In the 1900s, a variety of chemicals and chemical wastes increased drastically from the production of steel and iron, lead batteries, petroleum refining and other industrial practices. During that time, radium and chromic wastes also started creating serious quality problems. The World War II era ushered in a massive production of wartime products that required use of chlorinated solvents, polymers, plastics, paints, metal finishing, and wood preservatives (Bedient et al., 1999). Very little was known those days about the environmental impacts of many of these chemical wastes; only after several years, the impacts of various chemical wastes, municipal wastes and other chemicals could be known. Presently, there is a serious concern for the degradation of environment due to deteriorating water and air quality in both developed and developing nations.

Table 20.3 presents a list of major organic contaminants according to the Environmental Protection Agency (EPA). This is the target list of 126 priority pollutants defined by EPA for their contract laboratory program. The volatile compounds are determined by standard EPA method 624, the semivolatiles by method 625, and pesticides and PCBs by method 608 (Bedient et al., 1999).

Table 20.3. Sources of groundwater contamination

(Source: Bedient et al., 1999)

Category I	Category II	Category III
Sources designed to discharge substances	Sources designed to store, treat, and/or dispose of substances; discharge through unplanned release	Sources designed to retain substances during transport mission
- Subsurface percolation (e.g., septic tanks and cesspools) - Injection wells - Land application	- Landfills - Open dumps - Surface impoundments - Waste tailings - Waste piles - Materials stockpiles - Above ground storage tanks - Underground storage tanks - Radioactive disposal sites	- Pipelines - Materials transport and transfer
Category IV	Category V	Category VI
Sources discharging as consequence of other planned activities	Sources providing conduit or inducing discharge through altered flow patterns	Naturally occurring sources whose discharge is created and/or exacerbated by human activities
- Irrigation practices - Pesticide applications - Fertilizer applications - Animal feeding operations - De-icing salts applications	- Production wells - Other wells (non-waste) - Construction excavation	- Groundwater-surface water interactions - Natural leaching - Salt-water intrusion/brackish water upconing due to pumping

- Urban runoff - Percolation of atmospheric pollutants - Mining and mine drainage		

Fig. 20.1 shows the various mechanisms of groundwater contamination associated with some of the major sources. These sources are: chemical and fuel underground storage tanks, septic tanks, municipal landfills, and surface impoundments. A wide variety of organic and inorganic chemicals coming from both natural and man-made sources have been identified as potential contaminants in groundwater. They include inorganic compounds such as nitrates, brine, and various trace metals; synthetic organic chemicals such as fuels, chlorinated solvents, and pesticides; radioactive contaminants associated with defense sites; and pathogens.

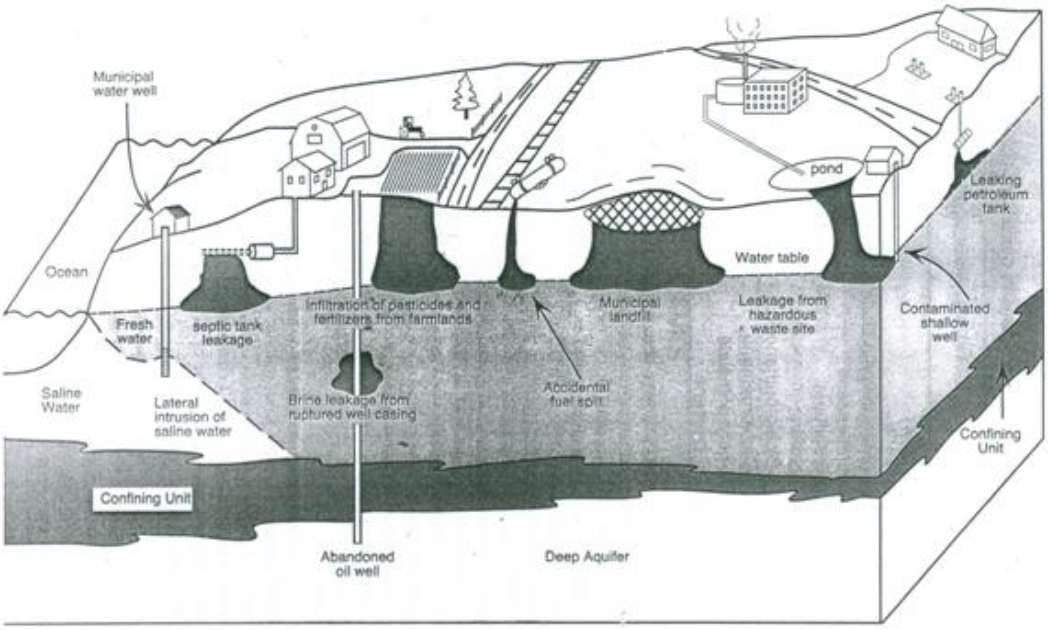


Fig. 20.1. Mechanisms of groundwater contamination.

(Source: Bedient et al., 1999)

The detailed discussion on the sources of groundwater contamination, the properties/characteristics of various organic and inorganic compounds identified as major threats to groundwater, and their impacts on groundwater can be found in Bedient et al. (1999), Todd (1980) and Fetter (1994).

20.2.3 Water Quality Standards

Chemical analyses are widely used to determine the suitability of water for different purposes such as drinking, irrigation, industrial and ecosystems. For this, we require sets of

standards based on which the quality of given water can be evaluated. These standards change from time to time and vary from country to country.

Water-quality standards are the regulations that set specific limitations on the quality of water which may be applied to a specific beneficial use. Water-quality criteria are the values of dissolved substances in water and their toxicological and ecological meaning. These data are often used to set water-quality standards. Two sets of water-quality standards gained international status in early seventies (Roscoe Moss Company, 1990): (i) World Health Organization (WHO) International Standards, and (ii) WHO European Standards. The WHO International Standards were intended as minimal standards and were considered to be achievable by all countries of the world. The WHO European Standards were more rigorous and reflected the water treatment technologies available to developed nations. Recognizing the many social, economic and cultural constraints of implementing uniform international standards, WHO developed a comprehensive set of guidelines for drinking water quality in 1984. These supersede the formal European and International Standards and instead provide a basis on which individual countries can develop standards and regulations of their own. Nevertheless, the WHO International Standards (WHO, 2006) are updated from time to time and are often used in many countries, especially in developing countries.

Besides the WHO standards, the rigorous U.S. Environmental Protection Agency Standards have also achieved almost an international recognition. Apart from the drinking-water standards, there exist separate water-quality standards for irrigation (Ayers and Westcot, 1985) as well as different industrial uses (Roscoe Moss Company, 1990), together with the water-quality standards for livestock consumption and that for maintaining the health of aquatic ecosystems.

Drinking water standards are especially important for evaluating groundwater quality because many consumers utilize untreated groundwater that is directly pumped from a well. Public water-supply systems that rely on groundwater are required to perform a complete analysis of the water for the drinking-water standards prior to the time a well is put into service and periodically thereafter. Private wells are often tested for bacteria and nitrate only when they are first drilled and then never tested again (Fetter, 1994). It is very important to maintain high quality in groundwater in order to protect private well owners. The situation of water-quality monitoring and its protection is much worse in many developing countries, including India. On the other hand, the important issues involved in using water for irrigation are requirements to match the salinity of irrigation water to the salt tolerance of selected crops, to avoid salt buildup in the soil, and to avoid a breakdown of the soil structure and a reduction in permeability by using water having high sodium concentration.

20.3 Collection of Water Samples

This section focuses on the methods of collecting representative water samples for chemical analyses in a specialized analytical laboratory. The program to sample both groundwater and surface water must be carefully planned considering the purpose of water sampling, number of sampling points, types of chemical constituents to be analyzed, frequency of sampling and the quality assurance/quality control (QA/QC) issues. In contamination studies or research investigations, wells are usually installed to provide samples from specified locations and to assure the quality of the sample. In this case, an investigator must make decisions concerning the location of the wells, their design, and the method of

installation (Schwartz and Zhang, 2003). Making appropriate decisions in this respect can minimize potential errors in water sampling.

20.3.1 Methods of Groundwater Sampling

Sampling is a critical step in obtaining valid water-quality data. A sample must be representative of the water residing in an aquifer (or produced from a well), and its integrity must be maintained until the laboratory tests are completed. Note that water standing in a well casing is probably not representative of the overall groundwater quality. This can be due to the presence of drilling contaminants, biological growths, and corrosion by-products or changes in environmental conditions such as redox potential. For these reasons it is necessary to pump or bail a well before collecting water samples. The recommended time of pumping depends on several factors, including the hydrogeology of the aquifer, the constituents or parameters to be tested, and the characteristics of the well. For small monitoring wells that are not easily bailed, a common practice is to pump or bail the well until a minimum of 4 to 10 bore volumes have been removed (Roscoe Moss Company, 1990). If possible, it is desirable to pump a production well for one to two hours before collecting water samples.

Newly completed wells sometimes require extended periods of pumping before a truly representative sample can be obtained. Samples collected during the first few hours of operation may be of a different quality than the samples collected after several days. This phenomenon has been observed in wells that penetrate more than one aquifer (Roscoe Moss Company, 1990). Sampling protocols or Standard operating procedures (SOPs) have been developed by the U.S. EPA and other government agencies. They specify the type of sample that is needed (grab or composite), the type of container that is to be used for the sample, the method by which the sample container is cleaned and prepared, whether or not the sample is filtered, the type of preservative that is to be added to the sample in the field, and the maximum time period for holding the sample prior to analysis in the laboratory. Based on these sampling protocols, Fetter (1994) and Schwartz and Zhang (2003) present useful guidance on groundwater sampling protocols, together with the design and installation of monitoring wells, sampling techniques, and the quality assurance/quality control (QA/QC) program.

It is a good field practice to clean thoroughly the sampling device prior to use. The method of cleaning should be such that no residue remains. The sampling devices and bottles should be rinsed with a sample of the water being sampled, if they are not thoroughly dry. This avoids the mixing of rinse water with the final water sample. Many of the chemical, physical and biological parameters in groundwater are unstable. Therefore, proper care should be taken in maintaining water sample integrity. Various sources of error involved in water sampling are described in the subsequent sub-section.

20.3.2 Sources of Error

Collecting water samples involves several steps, and problems can arise in each step. The groundwater sampling process begins by taking water from wells using a bailer or a pump. After bringing a groundwater sample to the surface, a few parameters like pH and specific conductance are measured immediately with portable equipment. Water is also stored in various containers, some of which could be special containers or contain preservatives to prevent deterioration. Many types of analyses require that the samples be chilled on ice and

then transported to an appropriate laboratory for analyses. Although these steps appear to be simple, there may be a number of pitfalls and problems which

STEP	SOURCES OF ERROR
***In Situ* Condition**	
Establishing a sampling Point	Improper well construction/placement; Inappropriate materials selection
Sample Collection	Sampling mechanisms bias; operator error
Sample Delivery/Transfer	Sampling mechanism bias; sample exposure, degassing, oxygenation, field conditions
Field Blanks, Standards	Operator error; matrix interference
Fields Determinations	Instrument malfunction; operator error; Field conditions
Preservation/Storage	Matrix interferences; handling/labeling errors
Transportation	Delay; sample loss

Fig. 20.2. Steps in groundwater sampling and sources of error.

(Source: Schwartz and Zhang, 2003)

can invalidate the sample results. Fig. 20.2 summarizes the sources of errors in sampling groundwater. They include improper procedures for installing the well, reactions between the groundwater and the well casing or sampler, and poor sample-handling protocols on the surface. Following the standard procedures of water sampling and their strict adherence during sampling can minimize these errors. Unfortunately, some problems can happen to a water sample even after reaching the analytical laboratory (Schwartz and Zhang, 2003). These problems encompass the errors due to deterioration of samples and laboratory standards, and the poor analytical methods. The laboratory problems also require that QC checks be designed to detect errors that originate in the laboratory.

20.4 Analysis of Water Quality

Describing the concentration of major and minor cations and anions, and the pattern of water quality variability is part of many groundwater investigations. A variety of graphical and statistical techniques are presently available for analyzing water quality data. Machiwal and Jha (2010) present a review of various tools and techniques for the analysis of water quality data as well as some case studies demonstrating the application of some of these techniques. Each technique has certain advantages and disadvantages in representing features of the data. Therefore, researchers/hydrogeologists should select appropriate tools and techniques for the effective analysis of water-quality data.

In short, the methods of water quality analysis can be divided into three major groups: (a) First group comprises a set of graphical methods for describing abundance or relative abundance, (b) Second group comprises methods that present patterns of variability in addition to abundance, and (c) Third group comprises derived maps that involve various types of calculations with the basic data. A brief discussion about these methods is provided below.

20.4.1 Graphical Methods

Several different graphical approaches can depict the abundance or relative abundance of ions in individual water samples. The most common approaches are: (i) Collins bar diagram, (ii) Stiff pattern diagram, (iii) pie diagram, and (iv) Piper diagram. In these plots, concentrations need to be expressed as meq/L or %meq/L. The Collins and Stiff diagrams require absolute concentrations (meq/L), whereas the pie and Piper diagrams require relative concentrations (%meq/L). Figs. 20.3(a, b, c, d) illustrate the sample data plotted in four different ways. The Collins, Stiff, and pie diagrams are relatively simple to construct. They require only that concentrations be plotted as a bar segment, a point on a line, or a percentage of the pie. The appropriate fields are shaded and possibly labeled in the case of the Collins and Piper diagrams (Fig. 20.3). The Stiff diagram can be plotted with or without the labeled axes.

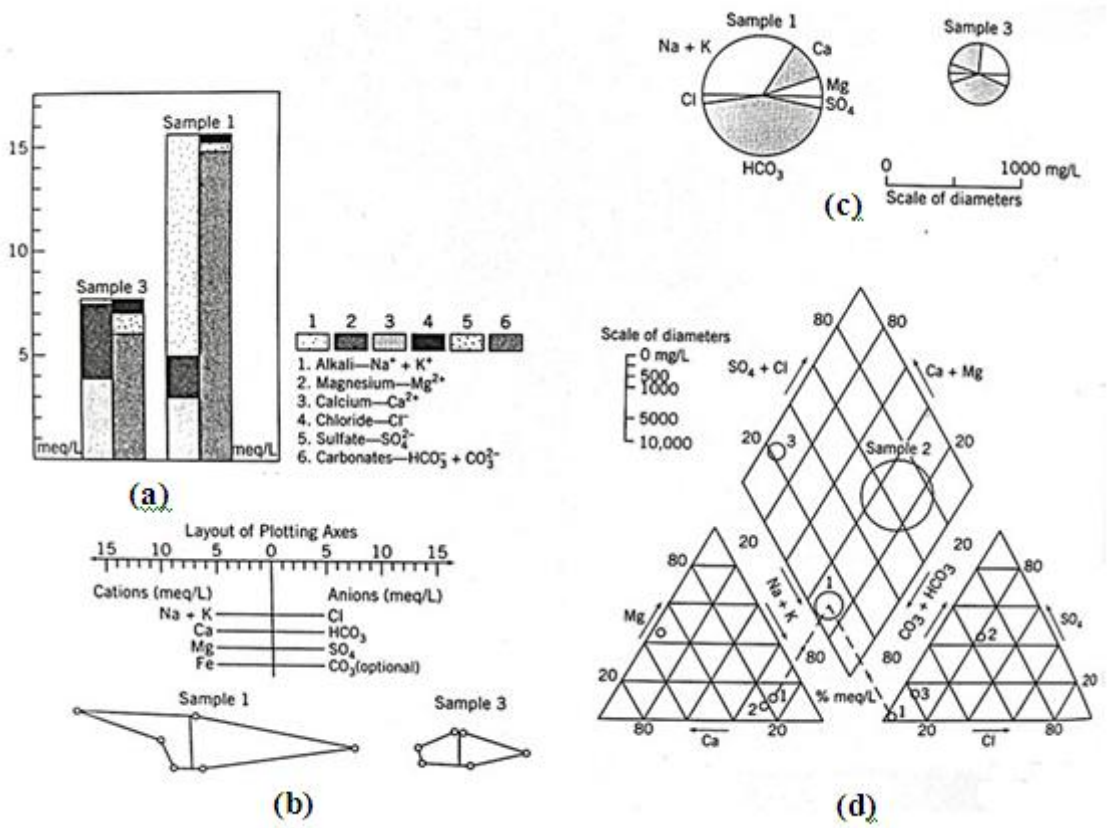


Fig. 20.3. Four different ways of plotting major ion data: (a) Collins diagram; (b) Stiff diagram; (c) Pie diagram; (d) Piper diagram.

(Source: Schwartz and Zhang, 2003)

Plotting data for a Piper diagram is complicated because there are three separate diagrams (Fig. 20.3d). The relative abundance of cations with the %meq/L of

$Na^+ + K^+$, Ca^{2+}, and Mg^{2+} assumed to equal 100% is first plotted on the cation triangle. Similarly, the anion triangle displays the relative abundance of Cl^-, and. Straight lines projected from the two triangles into the quadrilateral field define a point on the third field (Fig. 20.3d). To provide some indication of the absolute quantity of dissolved mass in the sample, the size of the data point is sometimes related to the salinity (TDS).

One advantage of all four techniques is that they present the major ion data for a sample on one figure. However, with the exception of the Piper diagram, these approaches are useful only in displaying the results for a few analyses. Presenting a large number of these diagrams together is confusing and is not much more helpful than showing concentration values in a tabular form (Schwartz and Zhang, 2003).

20.4.2 Other Methods

Graphical/illustrative type diagrams or statistics can define the pattern of spatial change among different geologic units, along a line of section, or along a pathline (Schwartz and Zhang, 2003). The simplest way for representing spatial variation of water quality over a particular geological layer is to take the single sample diagrams (e.g., pie or Stiff) and place them on a map. Such maps can show how the pattern of ion abundances changes within a given geological unit. Showing all the chemical constituents on a single map is often helpful.

When the water-quality data (measured or computed) vary systematically in space, it is often best to plot contour concentrations (or other data) on maps or cross sections (Fig. 20.4). This type of presentation clearly depicts the variation of individual parameters. However, this type of presentation involves a large number of figures to completely describe the chemistry of an area. Furthermore, GIS techniques can be used to effectively perform spatio-temporal analysis of water-quality data.

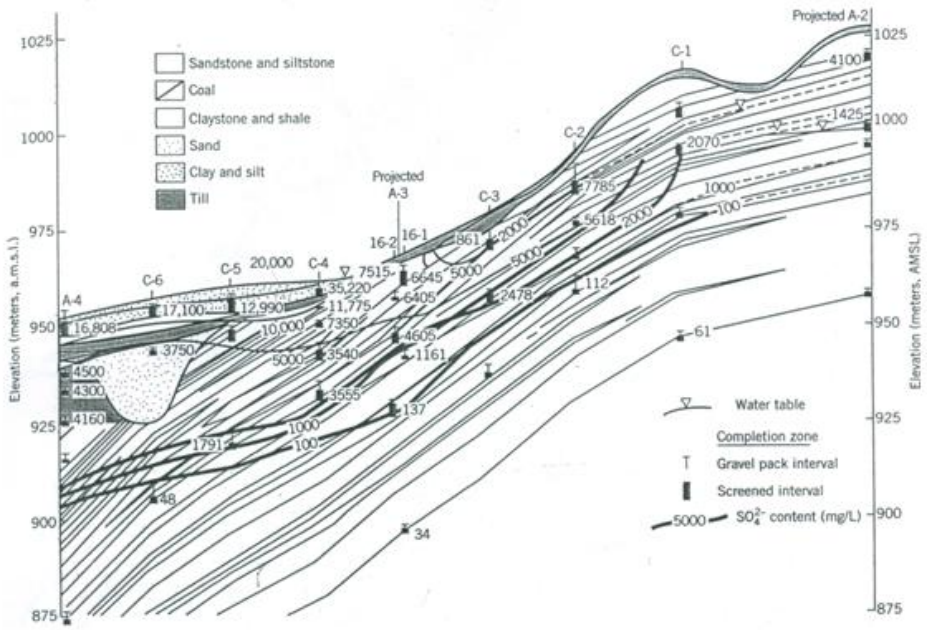


Fig. 20.4. Contours of ion concentration plotted on a cross section.

(Source: Schwartz and Zhang, 2003)

If the water-quality data are 'noisy', a Piper diagram is preferable to concentration maps. By classifying samples on the Piper diagram, one can identify geologic units with chemically similar water and define the evolution in water chemistry along a flow system (Schwartz and Zhang, 2003). Also, noisy water-quality data can be smoothed before plotting on a map or geological cross-section. The facies mapping approach provides one way of smoothing chemical data (Schwartz and Zhang, 2003).

Apart from the above methods of water-quality analysis, the statistical techniques such as time series analysis, t-test, regression/correlation analysis, multivariate analysis, etc. are also very useful and are being used by several researchers.

Lesson 21 Management of Groundwater

21.1 Introduction

The assessment and development of groundwater resources are essential for ensuring sustainable water supplies to domestic, agricultural and industrial sectors along with the ecosystems. There are overriding advantages of storing surplus water in subsurface reservoirs compared to surface reservoirs, which are as follows:

1. Large storage volume which can be developed in stages according to the water demand;
2. Little land-area requirement;
3. Resilience to droughts;
4. Relatively low environmental impact of well-field developments;
5. More or less uniform water temperature;
6. Relatively high purity and less vulnerability to contamination;
7. Slight to no evaporation loss;
8. No requirement of conveyance systems; and
9. Slight to no danger of catastrophic structural failure.

With the ever increasing use of groundwater throughout the world along with the mismanagement or lack of management of groundwater basins, there is a continuing need to manage this vital resource effectively so as to ensure sustainable groundwater development. The modern concept of water management states that surface water and groundwater are integrated resources, and hence they should be managed together. This concept must be followed in practice for efficient water resources management at a local, regional or national scale.

Groundwater management involves planning, implementation, and operation necessary to provide safe and reliable groundwater supplies. This necessitates groundwater management at a basin scale. Groundwater management objectives typically focus on aquifer yield, recharge, and water quality (i.e., groundwater quantity and quality) as well as on socio-economic, legal, and political factors. After the proper evaluation of available water resources in a basin and the preparation of alternative management plans, action decisions can be made by suitable government or public agencies. The formal groundwater management approach, though generally more important for large-scale development, can also be applied to smaller-scale projects or even individual well projects (Roscoe Moss Company, 1990).

In this lesson, basic concepts of groundwater management are briefly discussed, together with salient methods of groundwater management. An overview of groundwater modeling, which plays a significant role in managing groundwater resources of large and complex basins, is presented in next lesson (Lesson 22).

21.2 Basic Concepts of Groundwater Management

To manage a groundwater basin, a proper knowledge of the quantity of water that can be developed is a prerequisite. Determination of available water within a basin requires the evaluation of the elements constituting the water cycle. Therefore, the most fundamental approach to groundwater management is based on water balances within a groundwater basin. The water balance equation (or hydrologic budget) for a groundwater basin can be written as:

$$R + \frac{Q_i}{A} - ET - \frac{Q_o}{A} - \frac{Q_p}{A} = \pm \Delta S \qquad (21.1)$$

Where, R = recharge to groundwater [L/T], Q_i = surface-water inflow into groundwater storage in the basin [L^3/T], A = area of the basin [L^2], ET = loss of groundwater due to evapotranspiration [L/T], Q_o = groundwater outflow from the basin (groundwater outflow into surface water) [L^3/T], Q_p = total groundwater pumping from the basin [L^3/T], and ΔS = change of groundwater storage in the basin [L/T]. The values of these parameters are considered over a specific period of time for which the groundwater balance is sought.

Eqn. (21.1) indicates that for a given amount of recharge ($R+Q_i$) in a groundwater basin, the increase of pumping rate (Q_p) will eventually decrease the groundwater outflow into surface water (Q_o), evapotranspiration (ET), and groundwater storage (S). The decrease in the groundwater outflow into surface water may reduce flow in streams, creeks, lakes and springs, whereas the decrease in groundwater storage will lower the groundwater level in aquifers. Exactly how, when, and where these changes will be manifested depends on several factors such as the basin size, hydrogeologic setting, and the times involved. Complexity often arises in real-world basins because an increase in pumping also increases recharge to some extent; it is known as induced recharge.

Proper management of groundwater basin is concerned with renewability of the groundwater resource and its practical exploitation. Historically, one of the earliest approaches to analyzing groundwater yields was built on the concept of safe yield, which is associated with the amount of groundwater supply that a water user can depend upon (Todd, 1980; Fetter, 1994; Schwartz and Zhang, 2003). Safe yield is defined as the ratio of groundwater extraction from a basin for consumptive use over an indefinite period of time that can be maintained without producing negative effects on groundwater quantity, quality or environment. The goal of the safe yield is to achieve a 'long-term balance' (e.g., annual) between groundwater use and groundwater recharge in a basin so as to avoid groundwater depletion. Note that the purpose of the safe-yield goal is not to prevent pumping and use of groundwater, rather to limit pumping to the amount of groundwater that can be safely withdrawn each year. A few rules of thumb concerning safe yield are (Schwartz and Zhang,

2003): (i) the annual withdrawal of groundwater should not exceed the average annual recharge, (ii) the withdrawal of groundwater should not lower the groundwater level so that the permissible cost of pumping is exceeded (i.e., pumping becomes uneconomical), (iii) groundwater pumping should not lead to an undesirable deterioration in the quality of groundwater due to influx of contaminants, and (iv) groundwater pumping should not lead to land subsidence.

Although the concept of 'safe yield' is widely used as a groundwater management tool, it has been criticized by some groundwater experts for not taking surface water into consideration. As indicated by Eqn. (21.1), excessive pumping not only lowers the groundwater level but also decreases the groundwater outflow into surface water bodies. Many perennial streams across the world dried up as groundwater levels significantly declined due to excessive pumping. Also, plants and animals thrive in fragile ecosystems developed along the perennial streams. These ecosystems are particularly at risk when the overdevelopment of groundwater resources lowers water tables in the riparian zones or results in significant water-table fluctuations. Thus, groundwater plays an important role in sustaining life as well as in sustaining some aquatic and terrestrial ecosystems (Humphreys, 2009; Steube et al., 2009). Further discussion on the concepts of 'safe yield' and 'sustainable yield' can be found in Alley et al. (1999), Alley and Leak (2004), and Jha (2013). A thorough discussion on the constraints and challenges of sustainable development and management of groundwater resources in developing nations can be found in Jha (2013).

21.3 Salient Techniques for Groundwater Management

A well-organized plan is essential to any groundwater management program, because it relates all necessary tasks, resources and time. During the preparation of a groundwater management plan, the knowledge of possible management techniques plays an important role, among other information. In this section, some useful groundwater management techniques such as 'conjunctive use of surface water and groundwater', 'artificial recharge of groundwater and seawater barriers', 'interbasin transfer of water', 'intrabasin transfer of water', 'indirect recharge through avoidance of pumping', and 'control well fields' are briefly discussed, while Fig. 21.1 illustrates these management techniques. Further details of groundwater management, with salient case studies can be found in Todd (1980), Fetter (2000), Schwartz and Zhang (2003) and Sarma (2009).

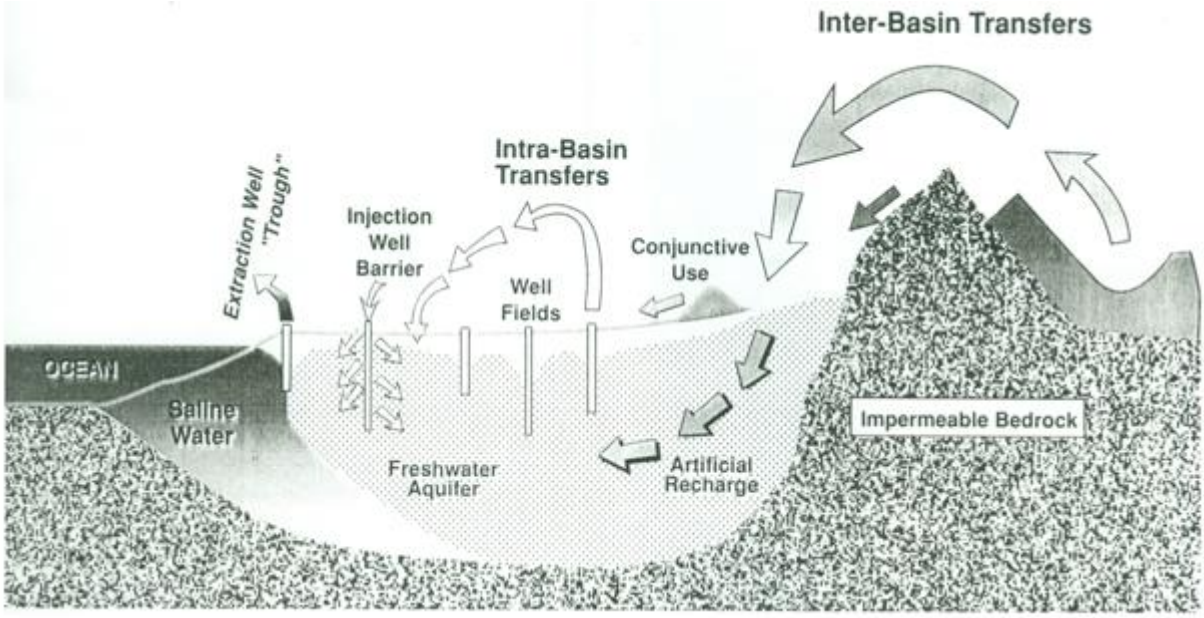


Fig. 21.1. Illustration of salient groundwater management techniques.

(Source: Roscoe Moss Company, 1990)

21.3.1 Conjunctive Use of Surface Water and Groundwater

Conjunctive use of surface water and groundwater is a management technique designed to maximize the use of available water resources. The major objectives of conjunctive use technique are: (i) to maximize net benefits, (ii) to increase reliability of supply, (iii) to enhance overall efficiency of a water system, and (iv) to minimize the degradation of ecosystems/environment. It requires a coordinated operation plan for both surface water and groundwater designed to meet demands while ensuring maximum water conservation (Roscoe Moss Company, 1990). Conjunctive use plans vary from percolation of natural streamflows to complex programs involving inter and intrabasin water transfers, with facilities for recharge, extraction, and distribution. Some important benefits of conjunctive use are (Roscoe Moss Company, 1990): (i) reduced surface-water storage facilities, (ii) water conservation, (iii) smaller surface-water networks, and (iv) less evaporation loss.

An excellent discussion on the concept, advantages and constraints of conjunctive use, together with some case studies can be found in Coe (1990), Todd (1980), and Sarma (2009).

21.3.2 Artificial Recharge of Groundwater and Seawater Barriers

Storing surface water into underground formations as groundwater for future use is an established practice in a conjunctive-use program. Groundwater recharge is accomplished by inducing percolation of surface water, thereby replenishing underlying aquifers. Further details of artificial groundwater recharge are given in Section 21.4.

When pumping near coastal areas creates depressions in groundwater levels, seawater migrates into the inland and contaminates underlying freshwater aquifers. Protection of coastal aquifers against seawater intrusion requires some kind of seawater barriers such as a ridge of 'protective groundwater elevations' constructed through the use of a line of injection wells (recharge wells) along the seashore or a 'pumping trough' to intercept intruding seawater. These methods, together with other methods of controlling seawater intrusion into freshwater aquifers are discussed in ASCE (1987).

21.3.3 Interbasin Transfer of Water

In many areas of the world, low precipitation rates, coupled with limited natural surface-water supplies necessitate the import of water from far distances. For example, the California aqueducts bring water hundreds of miles from the areas where surface water is abundant to the southern semi-arid region (Roscoe Moss Company, 1990). This water is either consumed directly or stored in groundwater reservoirs for later recovery. This transfer of water can be done on a seasonal basis, and if sufficient surface-water supplies are available, it can change the dynamics of water utilization. Generally, the water management technique, 'interbasin transfer of water' involves huge expenses and raises serious environmental issues. Therefore, proper planning and analysis are essential prior to the adoption of this water management technique.

21.3.4 Intrabasin Transfer of Water

Complex geologic conditions exist in most groundwater development areas. For example, it may be possible to overdraft one area while excessively recharging another, and still not exceed the safe-yield values predicted by regional groundwater budget calculations (Roscoe Moss Company, 1990). Therefore, a detailed basin investigation and analysis is necessary to delineate the areas of excess or deficiency and effectively design optimum pumping, distribution, and recharge programs. This management technique is usually less expensive and more environment friendly (i.e., reduced environmental impact) than the 'interbasin transfer of water'.

21.3.5 Indirect Recharge through Avoidance of Pumping

This is one of the innovative groundwater management techniques, which makes use of an indirect method of recharge. This technique encourages or requires groundwater users to purchase imported water instead of pumping groundwater (Roscoe Moss Company, 1990). In fact, this is equivalent to recharging the basin by the quantity of water not pumped. Such water management programs are made effective by keeping the costs of imported water supplies equal to or less than the pumping costs. They are implemented periodically by groundwater basin managers to regulate groundwater levels (Roscoe Moss Company, 1990).

21.3.6 Control Well Fields

Another technique used to conserve groundwater is through the use of 'control well fields'. Control well fields are strategically placed to produce interference effects for the control of hydraulic gradients and induce desirable groundwater-flow directions (Roscoe Moss Company, 1990). Control well fields typically control outflow from basins or restrain contaminant plumes. Well head protection (WHP) strategy used in many developed countries is one example of groundwater management by using the technique of control well fields.

Besides the above-mentioned groundwater management techniques, the specialized techniques like Soil-Aquifer Treatment (SAT) and River Bank Filtration (RBF) are also promising techniques, among others, for managing water-quality problems at a basin or sub-basin scale.

21.4 Artificial Recharge of Groundwater

21.4.1 Concept and Significance

In order to augment the natural supply of groundwater, people artificially recharge groundwater basins. Artificial recharge can be defined as "augmenting the natural movement of surface water into underground formations by some method of construction, by spreading of water, or by artificially changing natural conditions" (Todd, 1980). Various methods have been developed for artificial recharge, including water spreading, recharging through pits and wells, and pumping to induce recharge from surface water bodies such as rivers and lakes (Asano, 1985; Huisman and Olsthoorn, 1983; Johnson and Finlayson, 1988). The choice of a particular recharge method depends on several factors such as local topography, geologic and soil conditions, amount of water to be recharged, and the ultimate use of water. Under special circumstances, the value of land, water quality, or climate can be important factors in the selection of recharge methods (Todd, 1980).

Artificial recharge projects are designed to serve one or more of the following purposes (Todd, 1980):

(1) Maintain or augment the natural groundwater as an economic resource.

(2) Coordinate operation of surface and groundwater reservoirs.

(3) Combat adverse conditions such as progressive lowering of groundwater levels, unfavourable salt balance, or saline water intrusion.

(4) Provide subsurface storage for locally available surplus surface water or imported surface water.

(5) Minimize or prevent land subsidence.

(6) Provide a localized subsurface distribution system for established wells.

(7) Provide on-site treatment and storage for the reclaimed wastewater for subsequent reuse.

(8) Conserve or extract energy in the form of hot or cold water.

Thus, in most situations, artificial recharge projects not only serve as water-conservation mechanisms but also help in overcoming problems associated with groundwater overdrafts (Brown and Signor, 1974). To place water underground for future use requires that adequate amounts of water should be present or obtained for this purpose. Sources of recharge water can be storm runoff collected in ditches, basins, or surface/subsurface reservoirs through rainwater harvesting (RWH). Also, in some places, water is imported into a region by a pipeline or aqueduct from a far-off surface water source, which can be used for recharge. A third possibility involves the utilization of treated wastewater, though it often raises environmental concerns.

As to the history of artificial recharge, the artificial recharging of groundwater began in Europe early in the nineteenth century and in the United States of America (USA) near the end of the century; since then artificial recharge schemes have gradually increased throughout the world (Todd, 1980). Recharge basins form integral parts of many Swedish municipal water supply systems. Artificial recharge is widely practiced in Germany to meet industrial and municipal water demands. In the Netherlands, water supply systems for Amsterdam, Leiden, and The Hague include basins for recharging surface water into coastal sand dunes (Todd, 1980). Today, the need for artificial recharge is being felt throughout the world, including developing nations due to already extensive exploitation of groundwater or gradually increasing groundwater withdrawal for different uses. In fact, artificial recharge and rainwater harvesting have emerged as promising and indispensable tools for the efficient management of vital water resources in the face of changing climate and socio-economic conditions.

21.4.2 Methods of Artificial Recharge

A variety of methods have been developed for recharging groundwater artificially (Huisman and Olsthoorn, 1983; Asano, 1985; ASCE, 2001). However, the most widely used methods of artificial recharge are different types of water spreading (Todd, 1980), which involve application of water to the soil/ground surface for enhanced infiltration and then its downward movement through the unsaturated/vadose zone to the aquifer (groundwater). Spreading methods include basin method, stream-channel method, ditch-and-furrow method, flooding method, and irrigation method. Field studies on water spreading methods have shown that many factors govern the rate at which water will enter the soil. However, from a quantitative viewpoint, area of recharge and length of time that water is in contact with soil are most important. The economy of water spreading methods hinges on the maintenance of a high infiltration rate.

Although artificial recharge methods are classified in different ways by different authors, the classification suggested by Bouwer (1999) is discussed in this lesson. Bouwer (1999) classified artificial recharge systems into five types according to permeable materials in

which they can be placed (Fig. 21.2): (i) surface basin, (ii) excavated basin, (iii) trench, (iv) shaft or vadose zone well, and (v) recharge well. The surface basin recharge system (Fig. 21.2a) is suitable for soils that are sufficiently permeable, vadose zones that have no clay or other restricting layers, and aquifers that are unconfined. Artificial recharge can be accomplished by excavated basins (Fig. 21.2b) where permeable soils are not available at relatively small depths (e.g., 1 m). Excavated basins are constructed sufficiently deep to reach permeable material available at the recharge site. Trenches (Fig. 21.2c) are used if the permeable material is too deep to remove overlying material, but is within trenchable depth (e.g., less than about 7 m). Trenches are also suitable in soils that are highly stratified with alternating layers of fine and coarse materials (Bouwer, 1999). Large-diameter wells, pits, or shafts (Fig. 21.2d) in the vadose zone can be used when permeable subsurface formation is too deep to use trenches. These shafts can be drilled by bucket augers to a depth of about 50 m with a diameter of about 1 m (Bouwer, 1999). Furthermore, recharge wells (gravity-flow recharge wells or injection wells) penetrating the aquifer (Fig. 21.2e) can be used in situations where permeable surface soils are not available, vadose zones are not sufficiently permeable to transmit water, or aquifers are confined.

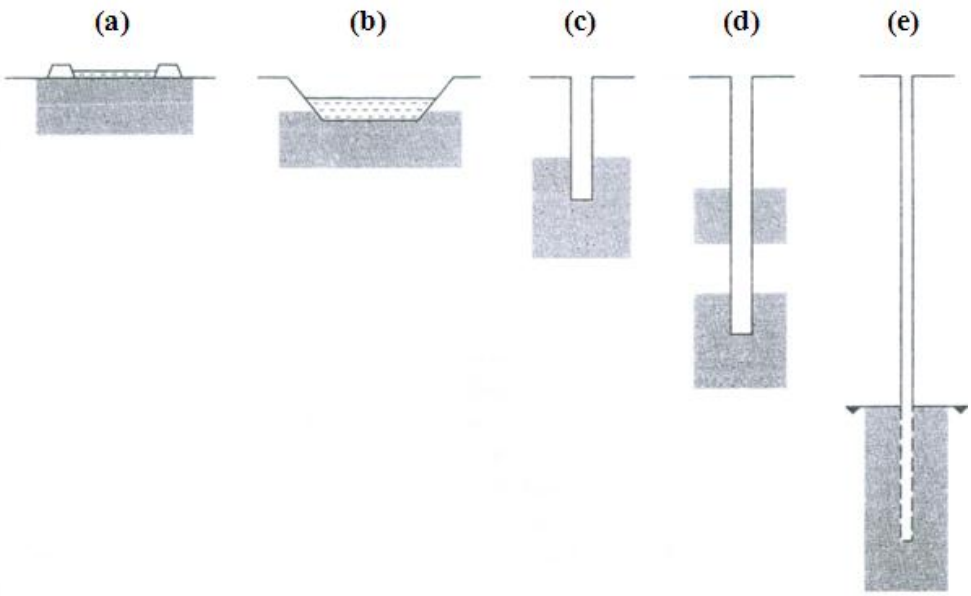


Fig. 21.2. Artificial recharge systems for increasingly deep permeable subsurface formations: (a) Surface basin; (b) Excavated basin; (c) Trench; (d) Shaft or Vadose zone well; (e) Recharge well. (Source: Bouwer, 1999)

For more detailed information on the theory and practice of artificial recharge, the readers are referred to Asano (1985), Pyne (1995), Huisman and Olsthoorn (1983), and ASCE (2001). Some cost-effective methods of artificial recharge are discussed in Jha et al. (2009), whereas the standard guidelines for artificial recharge of groundwater are given in ASCE (2001).

Lesson 22 Introduction to Groundwater Modeling

22.1 Introduction

Quantitative techniques are needed to best satisfy the ever-competing water demands for consumptive use, ecological protection, and maintenance of water quality. Management of complex water resources systems often benefits from mathematical simulation models. Simulation models along with optimization techniques are powerful tools for maximizing utilization of land and water resources, minimizing adverse impacts on the environment, and minimizing the costs of achieving management objectives. Particularly, groundwater being hidden and available in complex subsurface systems warrant for efficient tools and techniques in conjunction with a multi-disciplinary effort for its proper development and management. In this context, groundwater modeling has emerged as a powerful tool to help managers optimize groundwater use as well as to protect this vital resource (e.g., Anderson and Woessner, 1992; Spitz and Moreno, 1996; Pinder, 2002; Rushton, 2003; Bear and Cheng, 2010). Groundwater simulation models are currently in routine use for water supply management, pollution control, and environment protection. In most cases, these models are used to predict the response of complex groundwater systems to human-induced modifications/stresses (i.e., to determine cause and effect relationships and related model-based forecasts). Besides the advances in modeling techniques, the recent proliferation of GIS technology, digital terrain or digital elevation models (DTM/DEM), spatial data sets, and powerful desktop/laptop computers has enabled rapid progress in the development of quantitative research tools, especially simulation-optimization modeling tools with the capability to manage vast quantities of spatial and temporal data (e.g., Goodchild, 1993; Pinder, 2002; Jha, 2011).

In this lesson, fundamentals of modeling with a focus on groundwater modeling are presented. Emphasis is given on understanding the basic concepts of groundwater modeling and its proper applications. Interested readers are referred to Wang and Anderson (1982), Bear and Verruijt (1987), Istok (1989), Anderson and Woessner (1992), Spitz and Moreno (1996), Zheng and Bennett (2002), Rushton (2003), and Bear and Cheng (2010) for the details on the theory and practice of groundwater flow and contaminant transport modeling.

22.2 What is a Model?

The term 'model' can be defined as "a representation of reality that attempts to explain the behavior of some aspect of it and is always less complex than the real system it represents".

Simply put, "A model is a tool or device designed to represent a simplified version of reality (actual/natural systems)".

Similarly, we can define hydrologic models and groundwater models. "Hydrologic model is a tool designed to represent a simplified version of real hydrologic systems". On the same footing, "groundwater model is a tool designed to represent a simplified version of real groundwater systems".

Thus, a model is a representation of a portion of the natural or human-constructed world and can reproduce some but not all of its characteristics. It is always simpler than the prototype/natural system and can reproduce some but not all of its characteristics.

22.3 Why Modeling?

There are two major technical reasons why we need modeling: (a) natural systems (hydrologic, hydraulic, groundwater, etc.) are very complex and highly dynamic in nature, and hence our current knowledge about these systems is limited; and (b) there are limitations of current measurement techniques, i.e., limited availability of spatially and temporally distributed hydrologic, climatologic, geologic, pedologic, and land-use/land cover data.

Since we are not able to measure everything we would like to know about hydrologic or hydrogeologic systems, we need a means of extrapolating from available measurements in both space and time, particularly to un-gauged catchments (where data are not available) and into the future (where measurements are not possible) in order to assess the impacts of future hydrological changes. Models provide a means of quantitative extrapolation or prediction, which is finally helpful in decision-making. They can also help guide data-collection activities. Thus, models can be broadly used as one of the following tools (Anderson and Woessner, 1992):

(1) Predictive Tool: Models are used as predictive tools when the objective of hydrologic or groundwater modeling is to predict the impacts of a proposed action on existing hydrologic or hydrogeologic conditions.

(2) Interpretive Tool: Models are used as interpretive or research tools when the objective of modeling is to study system dynamics and understand flow and transport processes. Thus, interpretive modeling helps gain insight into the controlling parameters in a site-specific setting or helps assemble and organize field data and formulate ideas about system dynamics.

(3) Generic Tool: Models can also be used to study processes (e.g., flow or transport processes) in a hypothetical hydrologic or hydrogeologic system, which are called generic applications. Generic models are used to develop management standards and guidelines for a given region or as a screening tool to identify regions suitable or unsuitable for some proposed action. Note that 'predictive modeling' essentially requires calibration, whereas 'interpretive modeling' and 'generic modeling' do not necessarily require calibration.

There remains a continuing need for hydrologic, hydraulic, and hydrogeologic modeling to solve practical problems concerning water resources assessment, impacts of climate and anthropogenic activities on water quality and quantity, contamination incidents and mitigation, licensing of groundwater abstractions, flood forecasting and protection, design of water resources systems, and so on. In a nutshell, hydrologic/hydrogeologic models are developed either to guide the formulation of water resource management strategies (including the design of structures) or as tools for scientific research. Virtually, all applications of hydrology/hydrogeology to practical water-resource problems involve the use of models. The importance and the application domain of modeling have expanded to

such an extent that modeling is taught as a separate subject in major fields of water resources engineering (e.g., surface hydrology, subsurface hydrology, hydraulics, and environmental engineering), especially in developed nations.

22.4 Classification of Hydrologic/Hydrogeologic Models

A variety of models exist in the fields of hydrology and hydrogeology, and they can be classified based on certain criteria. The classification of various types of models used in hydrology and hydrogeology is shown in Fig. 22.1 based on two commonly used criteria. Hydrologic and hydrogeologic models in general can be classified into two broad groups: (a) material models, and (b) mathematical models (Fig. 22.1a). These two major groups of the hydrologic and hydrogeologic models are briefly described below.

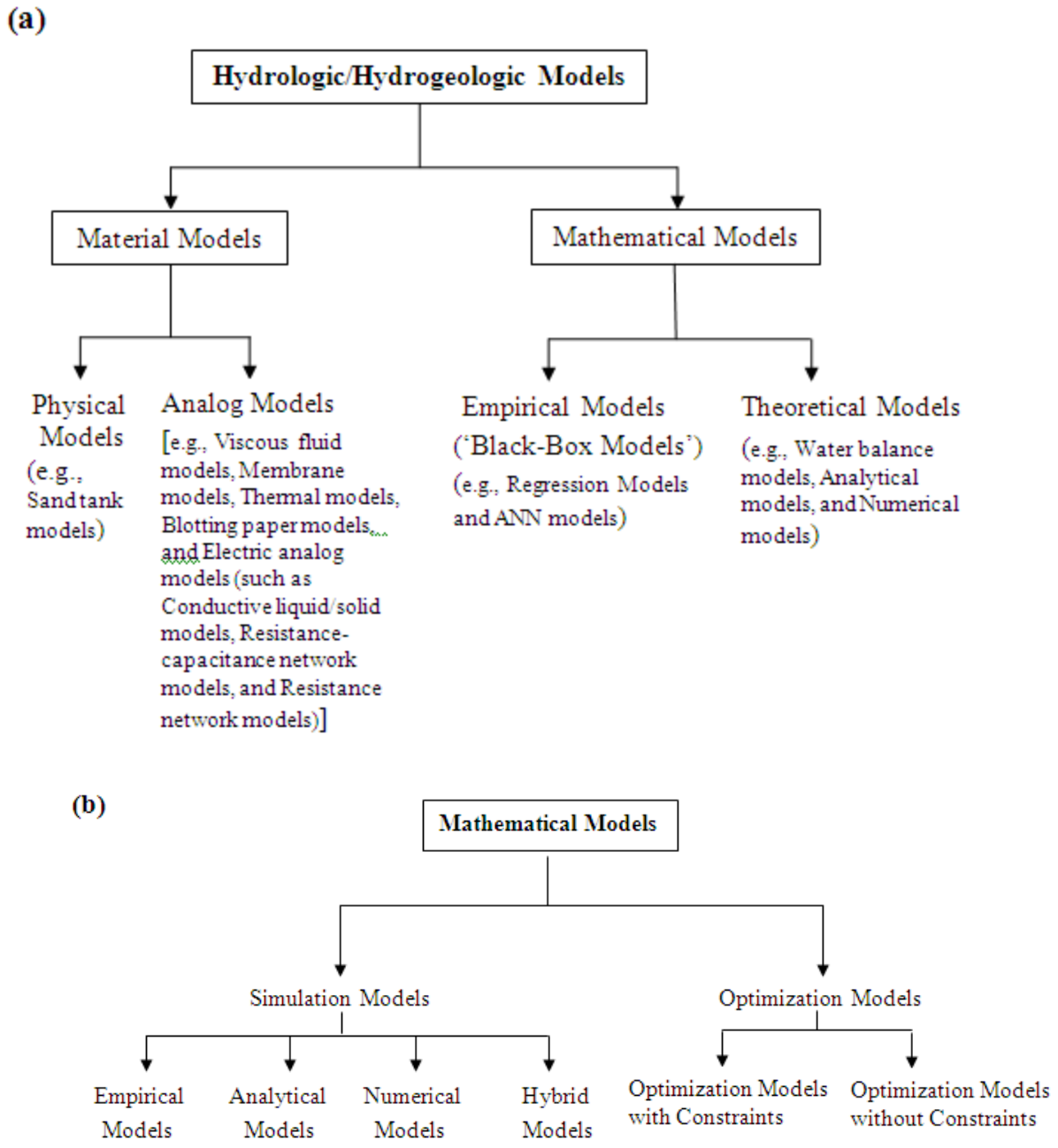


Fig. 22.1. Major classes of hydrologic/hydrogeologic models.

22.4.1 Material Models

Material models can be either 'physical models' or 'analog models'. A physical model is a tangible constructed representation of a portion of the natural world (i.e., scaled-down versions of a real system). A miniature, scaled version of a particular watershed and channel and a scaled sand tank model are the examples of physical models. Physical models have been important means to understanding the problems of hydraulics and fluid mechanics, and they are

On the other hand, analog models use substances other than (but analogous to) those in a real system. They use observations of one process to simulate a physically analogous natural process. An example of an analog model is an electric analog model in which the flow of electricity represents the flow of water. In the past, a variety of analog models have been used successfully in simulating groundwater flow under various boundary conditions (e.g., Todd, 1980). Other examples of analog models are given in Fig. 22.1(a).

22.4.2 Mathematical Models

A mathematical model is a set of mathematical expressions and logical statements combined in order to simulate a natural system. It uses a governing equation thought to represent the physical processes that occur in the system, together with the equations that describe heads or flows along the model boundaries (i.e., 'boundary conditions'). For time-dependent problems, an equation describing the initial distribution of heads in the system is also required (called 'initial conditions'). Mathematical models range from a simple linear regression equation to highly complex partial differential equations.

Mathematical models can be solved analytically using calculus (possible for a limited number of cases), which are known as 'analytical models', or they can be solved numerically using numerical techniques (possible for simple as well as complex cases), which are known as 'numerical models'. Analytical models may or may not involve a computer, but the use of computer is necessary for solving numerical models. The set of commands used to solve a mathematical model on a computer is called 'computer program', 'computer code', or simply 'code'.

As the availability of more powerful computers, modeling techniques and computer software packages has rapidly increased over the past few years, the use of both physical and analog models in hydrology and hydrogeology has been largely replaced by that of computer-implemented mathematical models, which are usually cheaper and much more flexible.

Mathematical models can also be classified as 'empirical models' or 'theoretical models' (Fig. 22.1). Empirical models are based on input-output relationships and do not necessarily simulate the actual processes involved. Such models rely on observed input and output data, and simply relate the output to a given set of inputs through a structure which may be fully statistical or partly mathematical. Therefore, empirical models are also known as 'black-box models' and they do not help in the physical understanding of processes involved. Examples of empirical models are 'regression models' and 'artificial neural network (ANN) models'.

In contrast, theoretical models rely on physical laws and theoretical principles. It is assumed that the hydrologic functions or relationships in a system are well understood and can be mathematically approximated directly from system characteristics. Thus, theoretical models use equations derived from basic physics (e.g., conservation of mass, conservation of energy, conservation of momentum, force balance, diffusion, etc.) to simulate flow, transport and storage. Examples are: 'water balance models', 'analytical models' and 'numerical models'. Theoretical models are called 'white-box' models or 'grey-box' models depending on whether model parameters and spatially-varying inputs are considered spatially distributed (white-box models) or lumped (grey-box models) for the model area (basin or sub-basin). Grey-box models are also known as 'lumped or conceptual models', whereas white-box models are also known as 'physically based or process-based models'.

22.5 Elements of Groundwater Modeling

22.5.1 Design and Development of Groundwater Models

22.5.1.1 Phases of Groundwater Model Development

Mathematical models incorporate the descriptions of key processes that determine a system's behavior with varying degrees of sophistication. Hillel (1987) suggests four principles which should guide model development: parsimony, modesty, accuracy, and testability. The main phases of numerical groundwater-model development are:

- Compiling and analyzing field data
- Understanding the natural groundwater system
- Conceptualizing the groundwater system
- Selecting a suitable numerical method or code
- Calibrating and verifying/validating the numerical model
- Predicting or simulating future scenarios
- Presenting model results and their analysis

In designing and developing a numerical groundwater flow or contaminant (solute) transport model, the first step is data collection and analysis followed by understanding the flow and/or transport processes in a natural groundwater system. The next step is to develop a conceptual model consisting of a description of the physical, chemical and biological processes which are thought to be governing the behavior of the system being modeled (Istok, 1989). A conceptual model is a pictorial representation of the groundwater flow and transport system, frequently in the form of a block diagram or a cross section (Anderson and Woessner, 1992). The nature of the conceptual model determines the dimensions (1-D, 2-D or 3-D) of the numerical model and the design of the grid. The subsequent step is to translate the conceptual model (i.e., our understanding of physical, chemical and biological processes occurring in a real groundwater system) into mathematical terms or a mathematical model (i.e., a set of partial differential equations and an associated set of auxiliary boundary conditions). The resulting groundwater model is only as good as the conceptual understanding of the processes being modeled. Finally, solutions of the governing equations subject to a set of boundary conditions can be obtained by using analytical or numerical methods. For most practical problems, it is not possible to solve the mathematical models analytically because of aquifer heterogeneity and irregular shapes of basin boundaries. Therefore, most groundwater flow and transport simulation models applied in practice are based on the numerical approach. The commonly used

numerical methods for solving the partial differential equations of flow and solute transport are: (i) finite difference method (FDM), (ii) finite element method (FEM), (iii) method of characteristics (MOC), and (iv) boundary element method (BEM), among others. However, the FDM and FEM are very popular in subsurface hydrology (e.g., Spitz and Moreno, 1996; Anderson and Woessner, 1992; Wang and Anderson, 1982; Bear and Verruijt, 1987; Istok, 1989).

It should be noted that numerical models are essential for analyzing subsurface flow and contamination problems in a groundwater basin because they are designed to incorporate the spatial variability within a groundwater system as well as spatial and temporal variations in hydrologic parameters that an analytical model cannot incorporate. Numerical solutions are also more flexible and useful than analytical solutions owing to the fact that the user can approximate complex geometries and combinations of recharge and pumping wells by judicious arrangement of grids as well as can easily handle complex boundary conditions. Theoretically, numerical models impose no restrictions on the boundary type, initial conditions, characteristics of the groundwater system, or the characteristics of the solute/contaminant to be investigated. At present, the use of numerical models is the state-of-the-art in practice for groundwater modeling. In numerical models, the physical layout of the area/basin to be modeled is replaced with a discretized model domain called 'grid' which consists of several cells/blocks (if the numerical method used is FDM) or elements (if the numerical method used is FEM). Thus, numerical models basically represent an assembly of many single-cell or single-element models. After developing a numerical model for a given study area (groundwater basin or river basin), it is calibrated and verified (validated) against the observed historical data of hydraulic head or concentration. Once the numerical model is calibrated and verified satisfactorily, various future management scenarios for the study area can be generated without much effort. Finally, model results are analyzed and conclusions/recommendations are made for the problem under investigation. Numerical models can solve both simple and complex problems.

22.5.1.2 Modeling Protocol

The above-mentioned phases of model development have been elaborated to formulate a step-by-step procedure for performing numerical modeling. This standard procedure of modeling is known as modeling protocol and is illustrated in Fig. 22.2. Some of the steps of the modeling protocol are described in the subsequent sub-sections, and the complete description of the modeling protocol could be found in Anderson and Woessner (1992).

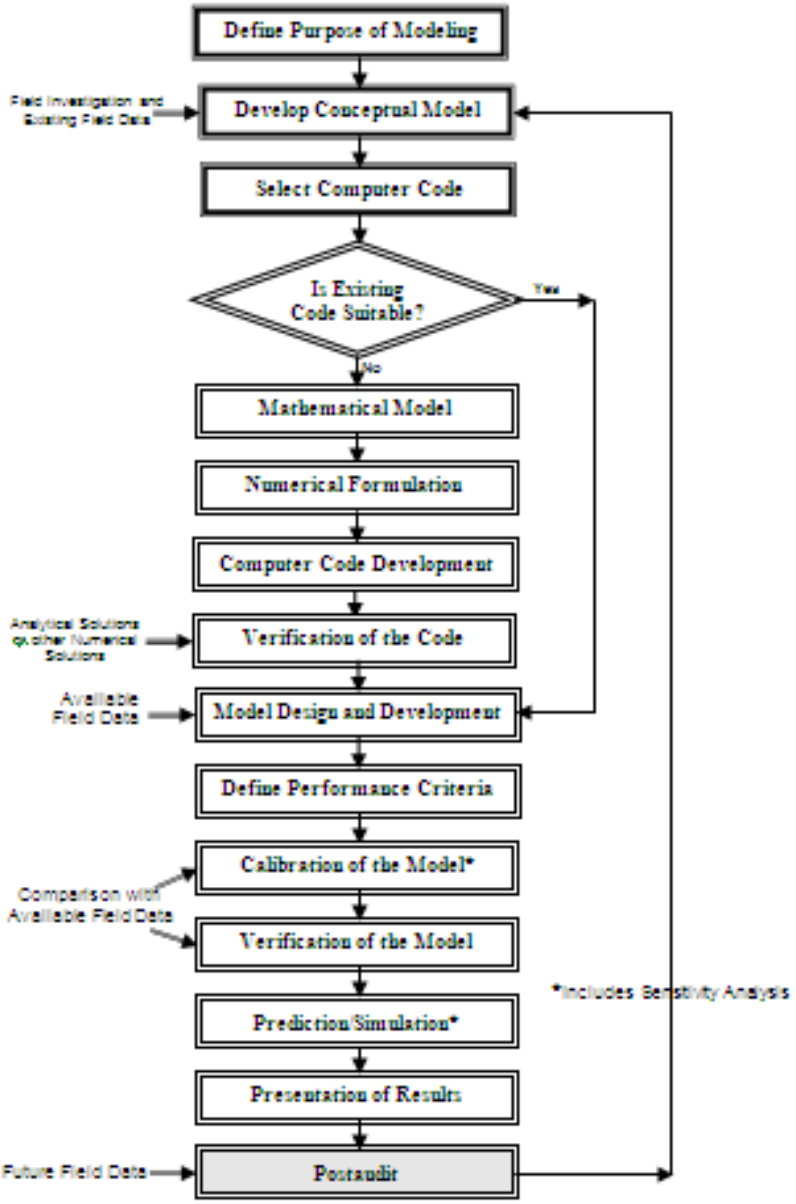


Fig. 22.2. Protocol for model development and application.

(Modified from Anderson and Woessner, 1992)

22.5.2 Numerical Solution of Governing Equation

In order to solve a governing groundwater flow or transport equation (i.e., to compute hydraulic head or solute concentration as a function of x, y, z and t), we need to know aquifer parameters, groundwater sources/sinks as well as the boundary and initial conditions. Thus, the following information is essential to fully define and solve a transient, subsurface flow or transport problem:

(i) The governing equation that applies within the domain (convert physical problem into a mathematical one).

(ii) Size and shape of the region of flow (flow domain).

(iii) Conditions at the boundaries of the flow domain (i.e., boundary conditions).

(iv) Initial conditions within the flow region (i.e., initial conditions).

(v) Spatial distribution of aquifer parameters.

(vi) A mathematical method of solution (e.g., FDM, FEM, or MOC).

Computer programs are generally used to solve the mathematical equation using initial and boundary conditions. However, initial conditions are not required for steady-state problems. The set of commands used to solve a mathematical model by a computer is known as computer program or computer code. The computer code is generic, and hence it can be considered as a generic model. When a generic model is applied to represent a particular geographic area by specifying a set of boundary and initial conditions, site-specific grid dimensions as well as site-specific parameter values and hydrologic/hydrogeologic stresses, the resulting computer program is known as a site-specific model. Thus, a computer code is written once but a new model (i.e., site-specific model) is designed and developed for each modeling application. These days, generic computer codes are available for the numerical modeling of groundwater flow and transport processes (e.g., MODFLOW, FEFLOW and FEMWATER) which can be used for different groundwater systems (study areas) without modifying the source codes.

It is worth mentioning that the computer code cannot set up the problem. If there is an error in setting up the problem (i.e., understanding the natural system or formulation of the conceptual model), the computer code may give the right answer to the wrong problem! Therefore, it is important to ensure that the problem being solved by the computer code is the same as the one that needs to be solved! Further, running the computer program is fairly straightforward provided that there is good documentation (though tedious sometimes). Setting up the problem, preparing the input data and interpreting the model results are more difficult!

22.5.3 Boundary Conditions

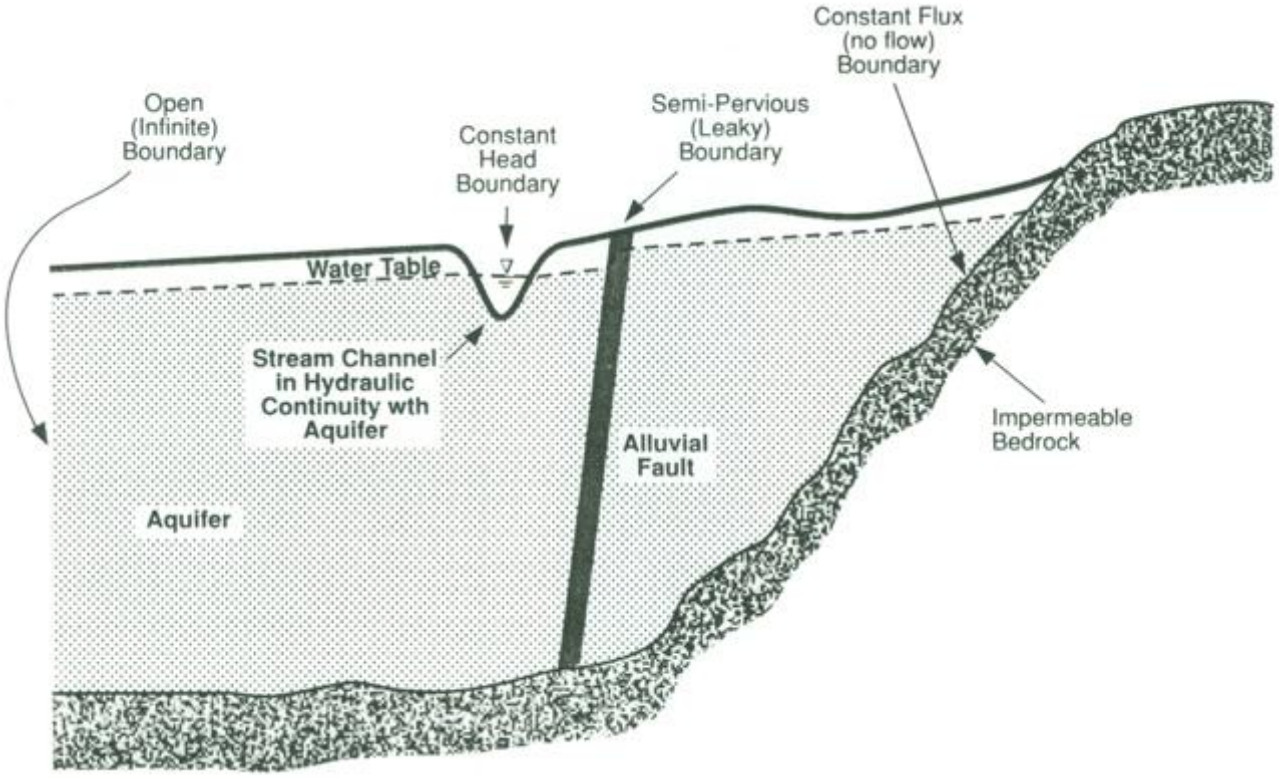
The term 'boundary' means a geometrical configuration of the surface enclosing the model domain. Boundary conditions must be specified for solving a mathematical model. The boundary conditions constrain the problem at hand and make solutions unique. Mathematically, the boundary conditions include the geometry of the boundary and the values of the dependent variable or its derivative normal to the boundary.

Generally, boundary conditions encountered in the field of water resources engineering can be of three types: (i) Dirichlet or Constant-Head Boundary Conditions, (ii) Neumann or Constant-Flux Boundary Conditions, and (iii) Cauchy or Mixed Boundary Conditions.

When head or concentration is known for surfaces bounding the model region, it is called Dirichlet boundary condition. Examples of the Dirichlet boundary from groundwater

hydrology are: ocean/sea and river/lake in direct hydraulic connection with the aquifer (Fig. 22.3). When flow rate (flux) is known across surfaces bounding the model region, it is called Neumann boundary condition. This type of boundary in groundwater hydrology is used to describe fluxes to surface water bodies, spring flow, and seepage to and from bedrock underlying the aquifer system. A special case of the Neumann boundary condition is when the flux is zero, and then it is also called no-flow boundary condition. Examples of the no-flow boundary from groundwater hydrology are: groundwater divides, streamlines, impermeable faults, and impermeable subsurface layers (Fig. 22.3).

If some combination of head and flux (i.e., head or water content dependent flux) or of concentration and mass flux (i.e., concentration dependent mass flux) is known for surfaces bounding the model region, it is called Cauchy or head-dependent flux boundary condition. Examples of the Cauchy boundary from groundwater hydrology are: semi-pervious subsurface layers (i.e., leaky confining layers) and semi-permeable or leaky faults (Fig. 22.3).



Note that the types of boundaries suitable for a particular field problem require careful consideration. If inconsistent or incomplete boundary conditions are specified, the problem itself will be ill defined!

22.5.4 Initial Conditions

Initial conditions are simply the values of the dependent variable specified everywhere inside the boundary (i.e., model domain) at the start of the simulation. Generally, the initial conditions are specified to be a steady-state solution. However, if initial conditions are specified so that transient flow is occurring in the system at the start of simulation, it should be recognized that heads (or concentrations) will change during simulation not only in response to the new pumping stress but also due to the initial conditions.

Note that for steady-state problems, only boundary conditions are required to solve a governing equation. However, both boundary and initial conditions are required for transient problems.

22.5.5 Calibration and Verification

Calibration of a simulation model is necessary because the real-world hydrogeologic and hydrologic systems are poorly known. Model calibration is defined as "the process in which model parameters are adjusted until the model output matches the field-observed conditions satisfactorily". For example, in groundwater-flow modeling, calibration is usually accomplished by finding out a set of system parameters (K and S or S_s) and other poorly-known inputs (e.g., recharge and leakance of streambed or aquitard) that produce simulated values of hydraulic heads and/or fluxes that match the measured/observed values within a specified range of error. Therefore, calibration is sometimes known as 'parameterization' or 'parameter identification'. Model calibration can be performed either by trial-and-error method or by automated technique using an inverse computer code.

There are no hard and fast rules for deciding a good calibration except that errors between observed and simulated values (hydraulic heads or solute concentrations) should be reasonably small. Several statistical goodness-of-fit criteria have been recommended for evaluating the performance of a model, of which mean error (ME) or bias, mean absolute error (MAE), root mean squared error (RMSE), coefficient of determination (r^2), and Nash-Sutcliffe efficiency (NSE) (also called 'model efficiency') are often used. They are mathematically expressed as follows:

$$ME \text{ or } Bias = \frac{1}{N}\sum_{i=1}^{N}(h_{si} - h_{oi}) \qquad (22.1)$$

$$MAE = \frac{1}{N}\sum_{i=1}^{N}\left|h_{oi} - h_{si}\right| \qquad (22.2)$$

$$RMSE = \sqrt{\frac{1}{N}\sum_{i=1}^{N}(h_{oi} - h_{si})^2} \qquad (22.3)$$

$$r^2 = \left[\frac{\sum_{i=1}^{N} \left[\left(h_{si} - \bar{h}_s \right) \left(h_{oi} - \bar{h}_o \right) \right]}{\sqrt{\left[\left(\sum_{i=1}^{N} \left(h_{si} - \bar{h}_s \right)^2 \right) \left(\sum_{i=1}^{N} \left(h_{oi} - \bar{h}_o \right)^2 \right) \right]}} \right]^2 \quad (22.4)$$

$$NSE = 1 - \frac{\sum_{i=1}^{N} (h_{oi} - h_{si})^2}{\sum_{i=1}^{N} (h_{oi} - \bar{h}_o)^2} \quad (22.5)$$

Where, h_{oi} = observed groundwater level at the ith time [L], h_{si} = simulated (predicted) groundwater level at the ith time [L], = mean of the observed groundwater levels [L], = mean of the simulated (predicted) groundwater levels [L], and N = total number of the observed data.

Besides the statistical goodness-of-fit criteria that facilitate quantitative evaluation, qualitative evaluation is also used for evaluating model performance. Qualitative evaluation involves visual inspection of observed and simulated values in certain graphical ways, for example, simultaneous plots of observed and simulated groundwater-level or concentration hydrographs, their scatter plots, plots of residuals, contour maps, etc.

Once the calibration of a model is complete, a verification (or validation) test is usually done so as to ascertain that the model is a valid representation of the hydrogeologic or hydrologic system under study. Model verification is defined as "the process in which the calibrated model is shown to be capable of reproducing a set of field observations independent of that used in model calibration". Verification results are also assessed using the above-mentioned quantitative and qualitative criteria. During verification, if the observed field conditions are not reproduced with a desired degree of accuracy, the model parameters should be changed to recalibrate the model with a new set of model parameters and the verification run should be repeated. This process eventually results in a calibrated and verified model for a particular study area.

22.5.6 Sensitivity Analysis

A well calibrated and verified site-specific model can be reliably used for simulating/predicting future management scenarios as well as for predicting impacts of hydrologic and socio-economic changes. Thereafter, a sensitivity analysis of the developed model is performed. The purpose of a sensitivity analysis is to quantify the effects of uncertainty in the estimates of model parameters on model results. It is usually done by perturbing the values of model parameters (one parameter at a time) by known amounts and measuring the effect of these variations on the model outputs. If a small change in the value of an input parameter produces a large change in the model's output, the model is said to be sensitive to that parameter and the parameter is said to have a high influence on the model. On the other hand, if a large change in the value of an input parameter produces a small change in the model's output, the model is said to be insensitive to that parameter and the parameter is said to have a low influence on the model.

Knowledge gained by performing the sensitivity analysis on a model can help to elucidate the way in which the modeled system functions and to identify those parameters of the model whose values need to be specified most accurately during field investigations.

22.5.7 Postaudit

As more field data and/or information are collected beyond the model development period (i.e., in the future), it is possible to compare the model predictions against the new set of field data. This process is known as 'postaudit' (Fig. 22.2). This may lead to further modifications and refinements (minor or major) of the previously developed 'site-specific model' due to significant changes in the system condition with time.

22.6 Concluding Remarks

Modeling is an excellent tool to study/understand complex hydrologic and hydrologic systems, and thereby help formulate efficient water management strategies. It facilitates efficient organization, analysis, and synthesis of field data. However, it is important to recognize that modeling is only one component in a comprehensive hydrological or hydrogeological study and not an end in itself. Modeling in conjunction with other tools and techniques can serve as a powerful tool for the planners and decision makers. The key point is that a good modeling methodology, proper knowledge of the system under study, as well as adequate and good-quality data are essential to enhance confidence in modeling analysis and results. Also, establishing the purpose of modeling effort at the outset and establishing realistic expectations can ensure much more effective utilization of modeling.

Finally, hydrologists and hydrogeologists (i.e., water resources scientists and engineers) must establish appropriate expectations for the role of modeling in supporting group dynamics and decision-making, and they must understand modeling costs and its actual benefits.

Module 5_Principle, Design and Operation of Pumps

Lesson 23 Introduction to Pumping System

23.1 Basic Mechanisms of Water Lifting

When the source of water is at a lower than the area to be irrigated and when free gravity flow is not available to drain surface or subsurface water, the water lifting devices are used. Water may be moved by the application of any one (or any combination) of the six following mechanical principles, which are mostly independent (FAO, 1986):

1. Direct lift: This involves physically lifting water in a container.
2. Displacement: This involves utilizing the fact that water is effectively incompressible and hence it can be 'pushed' or displaced.
3. Velocity head creation: When water is propelled to a high speed, the momentum can be used either to create a flow or to create a pressure.
4. Buoyancy of a gas: Air or other gases bubbled through water will cause movement of columns of water due to difference in specific gravity.
5. Impulse: Water hammer phenomenon creates impulse due to which a small portion of the water supply is lifted to a considerably high level.
6. Gravity: Water flows downward under the influence of gravity.

23.2 Types of Water Lifting Devices

Several types of indigenous water lifts are in use in small-scale irrigation. These families of lifting or propelling devices and pumps may be classified according to which of the above principles they depend on. Table 23.1 is an attempt to classify pumps under the categories given above. It is apparent from this table that most categories sub-divide into further classifications viz., 'reciprocating' or 'cyclic' and 'rotary'. The first category of these classifications relates to the devices that are cycled through a water-lifting operation (e.g., a bucket on a rope is lowered into the water, dipped to make it fill, lifted, emptied and then the cycle is repeated); in such cases the water output is usually intermittent, or at the best pulsating rather than continuous. Rotary devices were generally developed to allow a greater throughput of water, and they also are easier to couple to engines or other types of mechanical drive. Therefore, by definition, a rotary pump will generally operate without any reversal or cessation of flow, though its output may appear in spurts or pulsations in some cases (FAO, 1986).

Table 23.1. Taxonomy of water lifting devices (Source: FAO, 1986)

Category and Name	Type of Constr-uction	Head Range (m)	Power Range (W)	Output	Efficiency	Cost	Suction Lift?	Status for Irrigation
1. DIRECT LIFT DEVICES								
1. Reciprocating/Cyclic:								
- Watering can	1	>3	*	*	*	*	x	√
- Scoops and bailers	1	>1	*	**	*	*	x	√
- Swing basket	1	>1	*	**	*	*	x	√
- Pivoting gutters and "dhones"	2	1-1.5	*	**	**	**	x	√
- Counterpoise lift or "shadoof"	2	1-4	*	**	**	**	x	√
- Rope & bucket and windlass	1	5-50	*	*	*	*	x	√
- Self-emptying bucket or "mohte"	2	3-8	**	***	*	**	x	√
- Reciprocating bucket hoist	3	100-500	****	****	***	*****	x	X
2. Rotary/Continuous								
- Continuous bucket pump	2	5-50	**	**	***	**	x	√
- Persian wheel or "tablia"	2	3-10	**	***	***	**	x	√
- Improved Persian wheel "zawaffa"	2	3-15	***	****	****	***	x	√
- Scoop wheels or "sakia"	2	>2	**	****	****	****	x	√
- Waterwheels or		>5	*	**	**	**	x	√

"noria"								
II. DISPLACEMENT PUMP								
1. Reciprocating/Cyclic								
- Piston/bucket pumps	2&3	2-200	***	***	*****	****	√	√
- Plunger pumps	3	100-500	***	**	****	*****	√	?
- Diaphragm pumps	3	5-10	**	***	****	***	√	√
- "Petropump"	3	10-100	**	**	*****	****	√	?
- Semi-rotary pumps	3	5-10	*	**	**	**	√	x
- Gas or vapor displacement	3	5-50	****	****	***	***	√ or x	?
2. Rotary/Continuous								
- Gear and lobe pumps	3	10-20	*	*	**	***	√	x
- Flexible vane pumps	3	10-20	**	***	***	****	√	x
- Progressive cavity (Mono)	3	10-100	***	***	****	****	x	?
- Archimedean screw	3	>2	**	****	***	***	x	√
- Open screw pumps	2	>6	****	*****	****	*****	x	√
- Coil and spiral pumps	2 & 3	>6	**	**	***	***	x	√
- Flash-wheels & treadmills	2	>2	**	****	**	**	x	√
- Water-ladders "dragon spines"	2 & 3	>2	**	***	***	***	x	√
- Chain (or rope) and washer	3	3-20	***	***	****	****	x	√
	3	>3	*	*	***	***	√	x
	3	3-10	**	**	?	?	X	?

- Peristaltic pump - Porous rope								
III. VELOCITY PUMPS								
1. Reciprocating/Cyclic - Inertia and "joggle" pumps - Flap valve pump - Resonating joggle pump - Rebound inertia	2 & 3 1 & 2 2 3	2-4 2-4 2-10 2-60	* * ** **	** * **** *	**** ** **** ****	** * *** ***	x x x √	√ √ ? x
2. Rotary/Continuous - Propeller (axial flow) pumps - Mixed flow pumps - Centrifugal (volute) pumps - Centrifugal (turbine) pumps - Centrifugal (regenerative) pumps - Jet pumps (water, air or stream)	3 3 3 3 3 3	5-3 2-10 3-20+ 3-20+ 10-30 2-20	**** **** ***** ***** *** ***	***** ***** ***** ***** *** ***	**** **** **** **** *** **	**** **** *** **** **** ***	x x √ √ √ x	√ √ √ √ x x
IV. BUOYANCY PUMPS								
Air Lift	3	5-50	**	***	**	****	X	X
V. IMPULSE PUMPS								
Hydraulic Ram	3	10-100	**	**	***	***	X	√

(Hydram)								
VI. GRAVITY DEVICES								
Syphon	1, 2 & 3	1-(-10)	-	*****	-	**	-	√
Qantas or Foggara	2	-	-	**	*****	-	-	√

Note: Construction Type: 1: Basic; 2: Traditional; 3: Industrial.

*: Very Low; **: Low Medium; ***: Medium, ****: Medium-high; *****: High.

√: Yes; x: No; ?: Possible; X: Unlikely.

23.3 Hydraulic Ram

23.3.1 Introduction

Hydraulic ram (or hydram) is a special type of pump, which utilizes the energy of a large quantity of water falling through a small height to lift a small quantity of this water to a much greater height. Therefore, no external power is required to operate this pump. The first hydram was invented in 1775 by John Whitehurst of Derby, England (Lal, 1969). Thus, hydram can be employed when some natural source of water like a spring or a stream is available at some altitude (e.g., in hilly regions). It can be used wherever a stream of water flows with a minimum of about 1 m fall in altitude (Michael and Khepar, 1999). It can also be used for water supply to countryside and remote areas where a water source having a large quantity of water at some height is available, but the power is scarce or not available so that other types of pumps cannot be used. The simplicity of construction and the automatic operation of the hydram make it particularly suitable for remote rural areas in a hilly region which often suffer from non-availability of commercial power sources (e.g., electricity or diesel) and lack of skilled technicians for the repair and maintenance of pumps and prime movers.

The hydram can be used for various purposes such as irrigation in sloping lands, domestic water supply in villages, water supply to small industries and fish ponds in hilly areas, supplying water to a high-level field channel in undulating hills, and boosting the discharge of lift irrigation schemes in hilly areas by taking part of the pump discharge to higher elevations for irrigation.

23.3.2 Main Components of Hydram

A hydraulic ram consists of a valve chamber (also called 'hydram chamber' or 'hydram body') having a waste valve and a delivery valve (Fig. 23.1). The waste valve opens into a waste water channel and the delivery valve opens into an air vessel to which a delivery pipe is connected which carries water to a water storage tank located at a higher elevation. The valve chamber is connected to a water supply tank through a supply pipe. The supply pipe

is fitted with a gate valve to operate the hydraulic ram. Figures 23.2(a, b) illustrate two typical designs of a hydraulic ram.

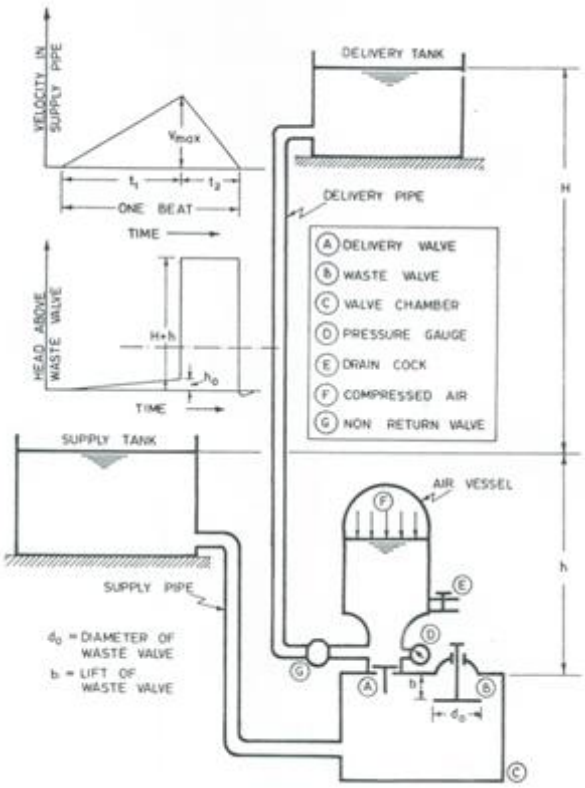


Fig. 23.1. Components and operating principle of a hydraulic ram.

(Source: Modi and Seth, 1998)

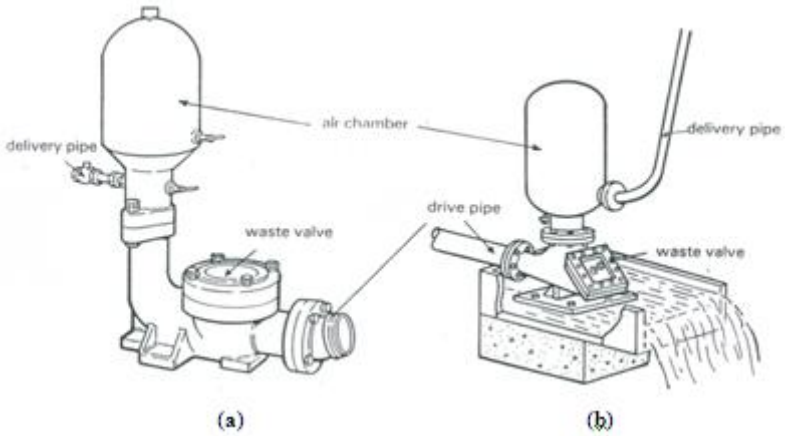


Fig. 23.2 (a,b). Typical designs of hydraulic rams: (a) Traditional European hydram design; (b) South-East Asia type of hydram. (Source: FAO, 1986)

23.3.3 Working Principle of Hydram

The hydram works on the principle of water hammer or inertia pressure developed in the supply pipe. Thus, in principle, it is an impulse pump. The momentum of a long column of water flowing through the supply pipe is made to force a part of the water to a height greater than that of the supply source itself. In order to develop maximum impulse, the supply pipe should be as long as possible. Installation of a hydram too close to the source of supply will reduce the impulse, and hence the delivery head.

23.3.4 Operation of Hydram

Initially, the water is at rest and the delivery valve and waste valve are closed (Fig. 23.1). The hydram is started by opening the gate valve of the supply pipe, thereby setting the water in motion. The column of water in the supply pipe rebounds a short distance creating partial vacuum in the valve chamber (hydram chamber). It causes the waste valve to open due to its own weight. Thus, water begins to escape through the waste valve into a waste water channel. As discharge through the waste valve increases, the flow of water in the supply pipe accelerates (i.e., velocity of flow increases). The acceleration of the water column in the supply pipe causes inertial pressure due to which pressure in the valve chamber increases. The pressure in the valve chamber rapidly increases to such an extent at which the dynamic thrust (due to the accelerating flow) acting on the lower face of the waste valve is greater than the weight of the waste valve. Consequently, the waste valve closes rapidly (i.e., instantaneously), which produces water hammer in the supply pipe. A very high pressure is momentarily produced in the valve chamber, by which the delivery valve is forced to open. The water then flows from the supply tank through the delivery valve into the air vessel and the delivery pipe (Fig. 23.1). Thus, some of the water flowing through the delivery valve is directly supplied to the water storage tank and some of it is stored in the air vessel. The water flowing into the air vessel compresses the air inside it, which pushes a part of the water in the delivery pipe even when the delivery valve is closed. Thus, an air vessel of a hydram helps provide a continuous delivery of water at a more or less uniform rate.

As the pressure gradually rises in the air chamber, inflowing water is brought to rest and the delivery valve then closes and the waste valve opens (due to the reduced pressure in the valve chamber), which again causes the water to flow from the supply tank to the waste water channel. This constitutes one cycle of operation or one beat of the hydram (Fig. 23.1). The same cycle is then repeated until the water supply is stopped. The operation of the hydraulic ram can be stopped by closing the gate valve fitted to the supply pipe.

It should be noted that the operation of a hydram depends on the successful creation and destruction of velocity of flow in the supply pipe. The waste valve must close suddenly to enable kinetic energy to be utilized to a maximum extent. Theoretically, the pumping head remains constant during the operation.

23.3.5 Analysis of Hydram Operation

With reference to Fig. 23.1, let:

d = diameter of the supply pipe,

d_o = diameter of the waste valve,

b = lift of the waste valve,

W = weight of the waste valve, and

V_o = velocity of flow of water through the waste valve just before its closure.

The dynamic pressure head acting on the waste valve is given as:

$$h_o = \frac{p}{\gamma} = \frac{V_o^2}{2g} \qquad (23.1)$$

Also, just before the closure of the waste valve, the force acting vertically upwards on the waste valve is equal to the weight of the waste valve. That is,

$$\frac{\pi}{4} d_o^2 \times p_i = W \qquad (23.2)$$

$$\Rightarrow p_i = \frac{W}{\pi/4\ d_o^2} \qquad (23.3)$$

$$\Rightarrow \gamma h_o = \frac{W}{\pi/4\ d_o^2} \qquad (23.4)$$

$$\therefore h_o = \frac{W}{\pi/4\ d_o^2 \gamma} \qquad (23.5)$$

If V_{max} = maximum velocity of flow of water in the supply pipe just before the closure of the waste valve, then by the continuity equation we have:

$$(\pi d_o b) \times V_o = \left(\frac{\pi}{4} d^2\right) \times V_{max} \qquad (23.6)$$

Also, if t_1 = time required to build up the velocity of water flow in the supply pipe from 0 (zero) to the maximum velocity (V_{max}) or, it is the time for which the waste valve remains open during each beat, then from the principle of water hammer we have:

$$h = \frac{l_s}{g} \times \frac{V_{max}}{t_1} \qquad (23.7)$$

Where, h = water-level in the supply tank above the waste valve and l_s = length of the supply pipe.

Similarly, if t_2 = time during each beat for which the waste valve remains closed, or the delivery valve remains open and H = water-level in the delivery tank above the water-level in the supply tank, then

$$H = \frac{l_s}{g} \times \frac{V_{max}}{t_2} \qquad (23.8)$$

Thus, the time taken to complete one cycle (or one beat) is t, i.e., $t = t_1 + t_2$ (Fig. 23.1) which can be calculated as:

$$t = t_1 + t_2 = \frac{l_s V_{max}}{g}\left(\frac{1}{h} + \frac{1}{H}\right) \qquad (23.9)$$

Furthermore, if q = discharge of water lifted by the hydram, Q = discharge of water flowing through the waste valve and the mean velocity of flow in the supply pipe is $V_{max}/2$, then q and Q can be computed as follows:

$$q = \left(\frac{\pi}{4} d^2\right) \times \frac{V_{max}}{2} \times \frac{t_2}{t} \qquad (23.10)$$

and

$$Q = \left(\frac{\pi}{4} d^2\right) \times \frac{V_{max}}{2} \times \frac{t_1}{t} \qquad (23.11)$$

It should be noted that the loss of head due to friction in the supply and delivery pipes has been neglected in Eqns. (23.7) and (23.8). However, considering this head loss, Eqns. (23.7) and (23.8) can be modified as follows:

$$\left(h - h_{fs}\right) = \frac{l_s}{g} \times \frac{V_{\max}}{t_1} \qquad (23.12)$$

and

$$\left(H + h_{fd}\right) = \frac{l_s}{g} \times \frac{V_{\max}}{t_2} \qquad (23.13)$$

Where, h_{fs} and h_{fd} are head losses due to friction in supply and delivery pipes, respectively.

23.3.6 Efficiency of a Hydram

The overall efficiency of a hydram is generally calculated in two ways: Rankine's approach and D'Aubuisson's approach. In the Rankine's approach, the water surface in the supply reservoir is considered as a datum for calculating delivery head and the base of the waste valve (i.e., top of the hydram chamber) is considered as a datum for calculating supply head. Hence, the input energy during one cycle is gQh, and the useful output energy during the same cycle is gqH, where g = unit weight of water. Thus, the Rankine's Efficiency is given as:

$$\eta_R = \frac{\gamma q H}{\gamma Q h} = \frac{qH}{Qh} \qquad (23.14)$$

On the other hand, in the D'Aubuisson's approach, the datum is taken as the base of the waste valve (i.e., top of the hydram chamber) and hence, the energy input during one cycle is g (Q+q) h and the useful output during the same cycle is gq (H+h). Thus, the D'Aubuisson's efficiency is given as:

$$\eta_D = \frac{\gamma q\left(H + h\right)}{\gamma\left(Q + q\right)h} = \frac{\left(H + h\right)q}{h\left(Q + q\right)} \qquad (23.15)$$

Note that the value of the D'Aubuisson's efficiency is always greater than the Rankine's efficiency. Because of significant energy losses in a hydram, its maximum efficiency is generally limited to about 70-75% (Lal, 1969). The main causes of energy losses are major and minor friction head losses in the supply and delivery pipes as well as minor friction head losses in the valves, and the kinetic energy wasted due to large flow through the waste valve.

It is worth mentioning that in Eqns. (23.14) and (23.15), friction head losses have been neglected. If these losses are taken into account, then the variables h and H appearing in the above equations should be replaced with $(h - h_{fs})$ and $(H + h_{fd})$, respectively.

23.3.7 Advantages of Hydraulic Ram

(i) No power source is needed to operate a hydram.

(ii) It requires no lubrication and no packing as there are no moving elements.

(iii) Maintenance cost is very low and almost no labor is required for supervision.

(iv) Hydrams can work continuously for 24 hours, and thus can provide a regular water supply.

(v) Hydrams can be adjusted to work with any quantity from their maximum capacity to less than one-half, and automatic adjustment is possible.

(vi) It has relatively high efficiency and reliability.

(vii) It has a long service life.

23.3.8 Criteria for Site Selection for Hydraulic Ram Installation

The selection of a suitable site, careful planning of various components of the system, and adherence to correct procedures in installation and maintenance contribute substantially to the economics and efficiency of a hydraulic ram. The following points should be taken into account while selecting a site for installing a hydraulic ram for irrigation or water supply (Michael and Khepar, 1999):

1. Quantity of water available in the stream during the cropping seasons and lean periods of flow.
2. Available fall in the stream (i.e., supply head).
3. Elevations of the water-supply points in the area proposed to be brought under irrigation and/or the elevation of the water storage tank.
4. Distance of the ultimate delivery point of water from the proposed site of the hydraulic ram.

5. Safe distance from the path of possible landslides and avalanches in hilly areas.
6. Possibility of stable foundation for the hydram, intake tank and a stable bed for the drive pipe.
7. Total area proposed to be brought under irrigation or proposed population to be served and the ancillary requirement of the community.
8. Existing and proposed cropping patterns of the area to be irrigated.
9. Estimated requirement of irrigation water.
10. Expected rainfall in terms of amount and periods of occurrence to estimate shut-off periods if the hydram is used exclusively for irrigation.

23.3.9 Example Problem on Hydram

Problem (Michael and Khepar, 1999): Assuming the following conditions, estimate the minimum expected flow rate of the source of water for installing a hydraulic ram in a rural water supply scheme:

Vertical fall = 9 m; Vertical lift = 60 m;

Population of village = 200; Growth rate = 25% in 15 years;

Water requirement = 45 L/day/person; and Efficiency of hydram = 60%.

Solution: Estimated population of the village after 15 years at the growth rate of 25% =, and Water demand = 250 × 45 = 11250 L/day.

Now, the minimum expected flow rate of the water source for a hydram can be calculated using the Rankine's formula [Eqn. (23.14)], which is given as:

$$Hydram\ Efficiency = \frac{q\left(h_d - H\right)}{QH}$$

$$\Rightarrow \quad Q = \frac{q\left(h_d - H\right)}{Hydram\ Efficiency \times H}$$

$$\therefore \quad Q = \frac{11250 \times \left(60 - 9\right)}{0.6 \times 9}$$

= 106250 L/day = 1.23L/s, Ans.

Note that in case the minimum rate of flow of the source of supply is lower than the computed value, it is still possible to get the desired output of water at the delivery. It can be possible by increasing the vertical fall or reducing the vertical lift, or by resorting to alternate locations and alignment.

23.4 Defining Pump

Pump can be broadly defined as 'a mechanical device to increase the pressure energy of a fluid'. Pumps are mostly used for lifting fluids (liquids or gases) from a lower level to a higher level. This is achieved by creating a low pressure at the inlet or suction end and a high pressure at the outlet or delivery end of the pump. Because of the low inlet pressure, the fluid rises from a depth where it is available and the high outlet pressure forces the fluid to a desired height. Here, work is done by a prime mover on the pump to enable it to impart energy to the fluid.

23.5 Classification of Pumps

Pumps are classified into two basic groups based on the method by which energy is imparted to the fluid. They are: positive displacement pumps and rotodynamic pumps. The positive displacement pumps are classified into two major groups: reciprocating pumps and rotary pumps, which are further classified into different groups as shown in Fig. 23.3. On the other hand, rotodynamic pumps are broadly classified into radial flow pumps, axial-flow pumps and mixed-flow pumps according to the direction of fluid flow inside the pump (Fig. 23.3). Radial flow pumps are further classified as volute pumps and diffusion (turbine) pumps based on the design of pump casing, both of which can be either of single stage (having one impeller) or multi stages (having more than one impellers), though most multi-stage pumps consist of diffusion casing.

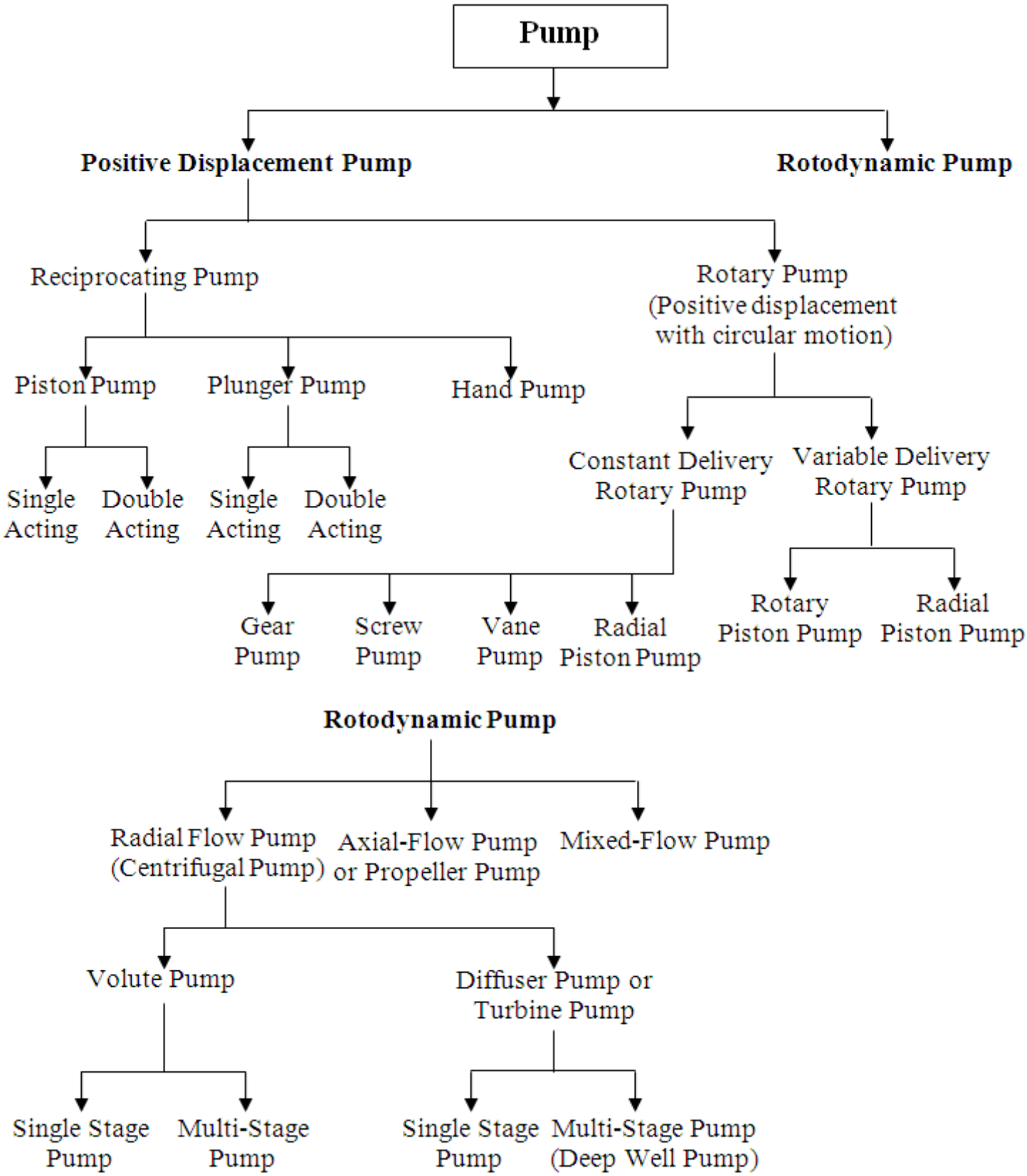


Fig. 23.3. Classification of different types of pumps.

23.5.1 Positive Displacement Pumps

These are the pumps in which the liquid is sucked and then it is actually pushed due to the thrust exerted on it by a moving element which results in lifting the liquid to a desired height. As such the discharge of liquid pumped by these pumps almost fully depends on the speed of the pump. The most common example of the positive displacement pump is reciprocating pumps. For the details of positive displacement pumps, the readers are referred to Michael and Khepar (1999) and Modi and Seth (1998).

23.5.2 Rotodynamic Pumps

They have a rotating element (called 'impeller') through which when the liquid passes, its angular momentum changes which results in an increase of the pressure energy of the liquid. Thus, a rotodynamic pump does not push the liquid as in the case of a positive displacement pump. The most common example of a rotodynamic pump is centrifugal pumps. The details about centrifugal pumps are given in Lessons 24 to 32, together with important references for further reading.

Note that the use of reciprocating pumps has become out of date for the water supply purpose, except for some popular indigenous water lifting devices used in rural areas. Rotodynamic pumps, especially of centrifugal type, have almost totally replaced the reciprocating pumps for lifting water.

Lesson 24 Centrifugal Pumps

24.1 Operating Principle of Centrifugal Pumps

Centrifugal pump is defined as 'a mechanical device to raise liquids from a lower elevation to a higher elevation by creating pressure with the help of centrifugal action'. If the liquid of a container is rotated with a sufficiently high velocity so as to enable it to rise beyond the walls of the container, and if more liquid is constantly supplied at the centre by a suitable means, the tendency of the liquid would be to flow out as shown in Fig. 24.1. Such a system in principle is a centrifugal pump.

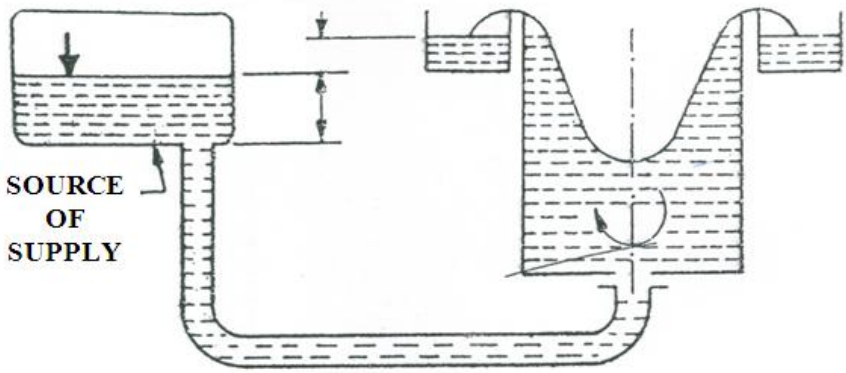


Fig. 24.1. Working principle of centrifugal pumps. (Source: Lal, 1969)

Whirling motion is imparted to the liquid by means of backward curved blades mounted on a wheel known as the impeller. The liquid to be pumped enters the impeller at its center, which is technically known as the eye of the pump, and discharges into the casing surrounding the impeller. The pressure head developed by centrifugal action is entirely due to the velocity imparted to the liquid by the rotating impeller, and not due to any displacement or impact.

On the other hand, if the liquid is flowed into the tank over the rim from an outside source at a higher elevation, and were drawn out at the centre, the flow being inward, the system will constitute a Francis Water Turbine in principle. The pump is usually named after the type of its casing (i.e., volute pump or diffuser pump). Diffuser casing is equipped with vanes and its design has been adopted from the Francis turbine. Therefore, the pump provided with a diffuser casing is also known as a turbine pump. Vertical turbine pumps (usually multi-stage), which are particularly suited for pumping water from deep wells, are often called deep well pumps.

24.1.1 Analysis of Flow through Impeller

In runners of turbines and in impellers of pumps, the flow of fluid is usually a combination of : (a) circulatory flow (i.e., flow in concentric circles), and (b) radial flow (i.e., flow involving a change of distance from the axis of rotation). The path resulting from the superimposition of these two motions is in the form of a spiral. In a centrifugal pump, water enters through an opening provided at the centre and leaves at the periphery. Guide vanes

are made of spiral shape to enable water to have both circulatory and radial flows inside the impeller.

As the radius increases from inlet to outlet, the area across the flow must also increase (Fig. 24.2), and consequently the relative velocity decreases so that $v_1 = \frac{q}{a_1}$ and $v_2 = \frac{q}{a_2}$ where q = quantity of water flowing per second, and a_1, a_2 are areas across flow at the inlet and outlet, respectively between two consecutive blades.

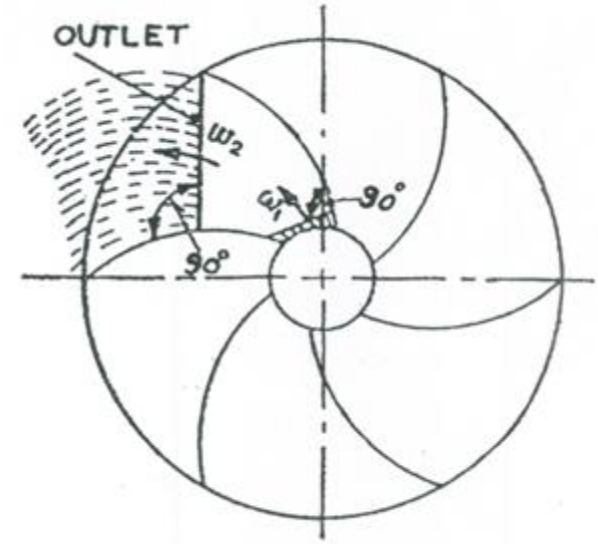


Fig. 24.2. Line diagram of impeller blades. (Source: Lal, 1969)

Therefore, pressure difference due to the change of kinetic energy is given as $\frac{v_1^2 - v_2^2}{2g}$ per unit weight of water. Also, pressure difference (ΔP) due to the centrifugal head can be expressed as: $\frac{\omega^2}{2g}\left(R_2^2 - R_1^2\right)$, where, R_1 and R_2 are inner radius and outer radius of the impeller, respectively. This pressure difference is due to cylindrical vortex only.

Therefore, the total pressure difference due to two flows (i.e., circulatory and radial) or due to spiral vortex will be:

$$\frac{p_2 - p_1}{\gamma} = \frac{\omega^2}{2g}\left(R_2^2 - R_1^2\right) + \frac{v_1^2 - v_2^2}{2g} \qquad (24.1)$$

$$\Rightarrow \quad H = \frac{u_2^2 - u_1^2}{2g} + \frac{v_1^2 - v_2^2}{2g} \qquad (24.2)$$

Where, H = head developed by the impeller, u_1 = peripheral velocity of water at the inlet of the impeller, and u_2 = peripheral velocity of water at the outlet of the impeller.

24.1.2 Example Problems

Problem 1: A certrifugal pump impeller has an inner diameter of 50 cm and its outer diameter is twice the inner diameter. Calculate the speed of the impeller (in rpm) at which the lifting of water will commence against a head of 15 m.

Solution: Given: R_1 = 50/2 cm = 25 cm = 0.25 m, and

R_2 = 100/2 cm = 50 cm = 0.50 m.

We know that Centrifugal Head = $\frac{\omega^2}{2g}\left(R_2^2 - R_1^2\right)$

$$\Rightarrow \quad 15.0 = \left(\frac{2\pi N}{60}\right)^2 \times \frac{1}{2 \times 9.81}\left(0.5^2 - 0.25^2\right)$$

$$\Rightarrow \quad 294.3 = \frac{4\pi^2 N^2}{3600} \times (0.1875)$$

$$\therefore \quad N = \sqrt{\frac{294.3 \times 3600}{0.1875 \times 4 \times \pi^2}}$$ =378.5 rpm, Ans.

Problem 2: Find the head developed by the impeller in the above problem, if the relative velocities of water at the inlet and outlet of the impeller are 7.5 and 5.4 m/s, respectively.

Solution: We have, v_1 = 7.5 m/s, v_2 = 5.4 m/s, and Centrifugal head (i.e., head developed by the centrifugal action) = 15.0 m.

From Eqn. (24.2), the head developed by the impeller (H) is given as:

$$H = \frac{u_2^2 - u_1^2}{2g} + \frac{v_1^2 - v_2^2}{2g}$$

$$= \left\{15.0 + \frac{(7.5)^2 - (5.4)^2}{2 \times 9.81}\right\} \text{ m}$$

=(15.0+1.38) m=16.38 m, Ans.

24.2 Components of a Centrifugal Pump

There are a variety of centrifugal pumps available in the market. However, commonly used centrifugal pumps have two basic components: a rotary element called impeller and a stationary element known as casing. Along with these two major components, it may also

have some other components such as pump inlet, pump outlet, suction valve, delivery valve, priming device, foot valve with a screen and/or pressure gauge (Fig. 24.3). A brief description about these components is given below.

(1) Impeller: It is a rotor, which is provided with a series of backward curved blades. It is mounted on a shaft, which is coupled to an external source of energy, which imparts the required energy to the impeller thereby making it to rotate.

(2) Casing: It is an air-tight chamber which surrounds the impeller and is usually in the form of a spiral or volute curve with a cross-sectional area increasing towards the discharge opening.

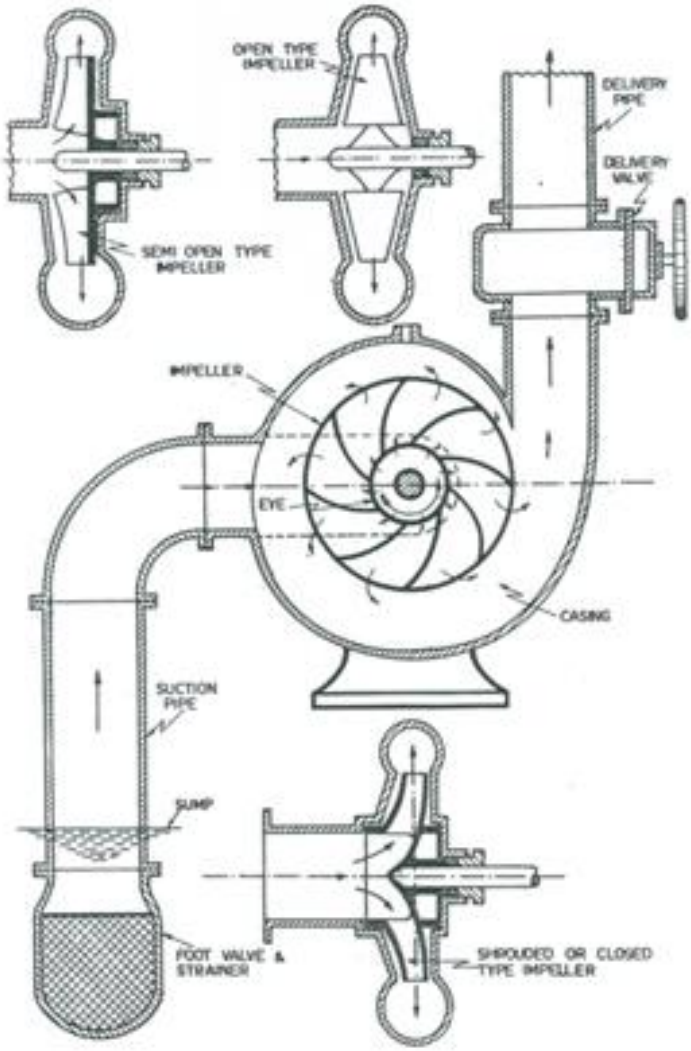


Fig. 24.3. Components of a centrifugal pump.

(Source: Modi and Seth, 1998)

(3) Pump Inlet (Suction Port): Liquid enters through the suction port or the pump inlet and flows into the pump casing. In centrifugal volute pumps the suction ports are located axially.

(4) Suction Valve (or Reflux Valve): Its main purpose is to retain water in the pump and the suction pipe section between the pump and the reflux valve, to help priming. The reflux

valve is used when pumps are directly connected to the suction pipe. In case of pumping to overhead tanks and long distance conveyance, the reflux valves are used in the delivery side also so as to allow flow in the forward direction only, thereby checking the return flow and preventing water hammering.

(5) Pump Outlet (Delivery Port): Liquid present inside the pump casing is thrown out of the casing and exits the pump through the delivery port or pump outlet. The delivery ports are placed centrally in radial direction in volute pumps.

(6) Delivery Valve: Sometimes centrifugal pumps are fitted with a one-way valve on the delivery pipe to regulate pump discharge and/or help in priming.

(7) Priming Device (optional): It is an optional component of a centrifugal pump. A priming device in a centrifugal pump eliminates the need for external priming mechanism and when it is provided in a centrifugal pump, it is known as a self-priming centrifugal pump. When the pump is shut down, the liquid in it drains out of the suction line. At the bottom of the casing in a priming device, a small quantity of liquid is retained. When the pump is started again, this water is pushed by the impeller out the discharge line, along with some air. This creates a vacuum at the impeller inlet, which draws liquid up the suction line. The priming cycle continues until all of the air is pushed out. Salient self-priming pumps and self-priming devices are discussed in Lesson 29.

(8) Foot Valve with a Screen (optional): The suction side of the pump consists of foot valve at the bottom of suction line. The foot valve is a one-way valve which helps to prevent the trash from entering the suction side and at the same time retains water in the suction line to help priming of the pump.

(9) Pressure Gauges (optional): A pressure gauge on the discharge side close to the outlet of the pump helps to diagnose pump system problems. It is also useful to have a pressure gauge on the suction side; the difference in pressure is proportional to the total head.

24.3 Types of Centrifugal Pumps

Centrifugal pumps possess some characteristic features based on which they are classified. These characteristic features are as follows:

(i) Working head,

(ii) Casing design (type of energy conversion),

(iii) Number of impellers (or, number of stages),

(iv) Relative direction of flow through the impeller,

(v) Number of entrances to the impeller (or, type of suction inlet),

(vi) Disposition of pump shaft,

(vii) Split of casing (Horizontally-split casing pumps and Vertically-split casing pumps),

(viii) Connection of the pump to the prime mover (Close-coupled/Monoblock centrifugal pumps, and Flexible-coupled centrifugal pumps),

(ix) Type of impeller (or, type of liquid handled), and

(x) Specific speed of the pump.

Note that the first characteristic feature is a commercial classification of centrifugal pumps from the viewpoint of their utility, but the characteristic features under (ii) to (ix) are all practical considerations each governing an important constructional feature of the pump. The last feature is a solely theoretical aspect and provides a technically sound basis for the absolute classification of centrifugal pumps.

24.3.1 Types of Centrifugal Pumps Based on Working Head

It is the head against which water is delivered by a pump. Based on the working head, centrifugal pumps could be classified as follows:

(1) Low-Head Centrifugal Pumps: They can produce heads up to 15 m. The impeller is usually surrounded by a volute and there are no guide vanes. The shaft is generally horizontal and water may enter the impeller from one or both sides depending on the quantity of water to be delivered.

(2) Medium-Head Centrifugal Pumps: They are capable of generating heads as high as 40 m. They are usually provided with guide vanes. Water may enter from one or both sides depending on the quantity of water to be pumped.

(3) High-Head Centrifugal Pumps: They can generate heads greater than 40 m. They are generally multi-stage pumps because an ordinary single impeller cannot build up such a high pressure. High-head centrifugal pumps may be horizontal or vertical; the latter is mostly used in deep wells.

24.3.2 Types of centrifugal Pumps Based on Casing Design

Pump casing should be so designed as to minimize the loss of kinetic head through eddy formation, etc. Efficiency of the pump largely depends on the type of casing. In general, the casings are of three types as shown in Figs. 24.4(a, b, c) and the pump is named after the type of casing it has. They are: (i) volute pump, (ii) volute pump with vortex chamber (or double volute pump), and (iii) diffuser pump (or turbine pump). A brief description about these pump types is given below.

(i) Volute Pump: It has a volute casing (spiral shaped casing, which is known as volute chamber) into which the impeller discharges water at a high velocity. The shape of the casing is such that the cross-sectional area of the flow around the impeller periphery gradually increases from the tongue towards the delivery pipe.

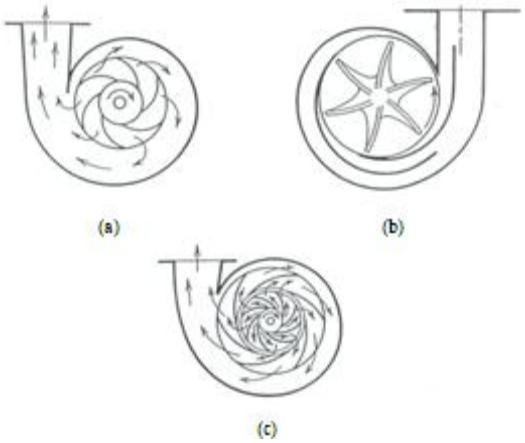


Fig. 24.4. Types of centrifugal pump casings: (a) Single volute; (b) Double volute casing; (c) Diffuser casing. (Source: James, 1988)

(ii) Volute Pump with Vortex Chamber (or Double Volute Pump): A subsequent improvement in the simple volute casing was made by providing a circular chamber between the impeller and the volute chamber. The circular chamber that acts as a diffuser is known as a vortex or whirlpool chamber, and such a pump is called volute pump with vortex chamber. Almost all the volute pumps are single-stage type, with horizontal shafts irrespective of the shape of impeller.

(iii) Diffuser Pump: In this pump, the impeller is surrounded by a series of 'guide vanes' (stationary vanes) or 'diffusers' mounted on a ring called 'diffuser ring'. Diffuser pump is also known as a turbine pump. Diffuser pumps may be either horizontal or vertical shaft type.

24.3.3 Types of Centrifugal Pumps Based on Number of Impellers

Based on the number of impellers per shaft, centrifugal pumps are classified as: (i) single-stage centrifugal pump, and (ii) multi-stage centrifugal pump. They are succinctly discussed below.

(i) Single-Stage Centrifugal Pump

Single-stage centrifugal pumps have only one impeller fitted to the pump shaft. Such a pump is generally horizontal, but can be vertical also. It is usually a low-lift (low-head) pump. Vertical turbine pumps can have 1 to about 25 stages.

(ii) Multi-Stage Centrifugal Pump

Multi-stage centrifugal pumps have two or more identical impellers fitted to a single shaft (Fig. 24.5) and enclosed in the same casing. Thus, pressure is built up in steps. The impellers are surrounded by guide vanes and the water is led through a by-pass channel from the outlet of one stage to the entrance of the next until it is finally discharged into a wide chamber from where it is pushed on to the delivery pipe.

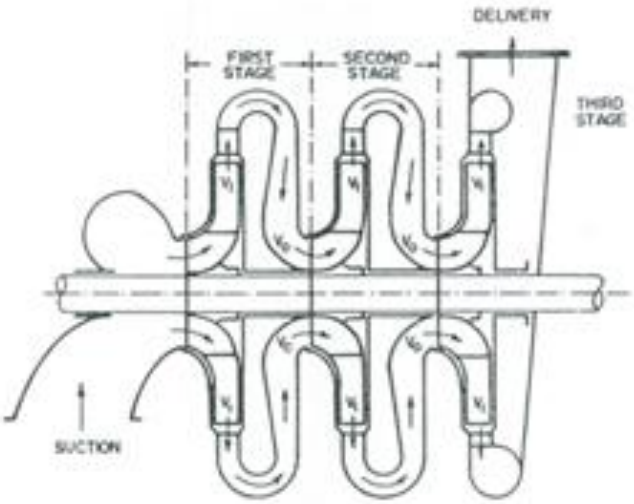


Fig. 24.5. Configuration of a three-stage centrifugal pump.

(Source: Modi and Seth, 1998)

Multi-stage pumps are used essentially for high working heads and the number of stages depends on the head required. Usually, not more than 15 stages are employed for ordinary multi-stage centrifugal pumps. According to the number of impellers fitted to a single shaft, a multi-stage pump is designated as two-stage, three-stage, four-stage, and so on. Multi-stage vertical turbine pumps can develop heads up to about 1500 m. However, some specially designed multi-stage pumps can have a discharge up to 946 L/s and can develop heads up to about 2100 m. Remember that for a given type of impeller, the head and power requirements of a multi-stage pump increase in direct proportion to the number of stages. However, the discharge and efficiency of a multi-stage pump are almost the same as for the single-stage pump operating alone.

Discharge and Head of Multi-Stage Pumps: As the liquid passes through each impeller, the absolute velocity of the liquid increases to V_1, and in each connecting passage, the absolute velocity decreases to V_o, but the pressure head continuously increases. If H_1 and H_2 are the pressure heads gained by the liquid in each impeller and the surrounding guide vanes, respectively, then the pressure head imparted on the liquid at each stage is $H_m = (H_1 + H_2)$. Now, if there are n impellers, then the total head (H) developed by the multi-stage pump will be:

$$H = n(H_m) = n(H_1 + H_2) \qquad (24.3)$$

This is because at each stage, the pressure head will be raised by the same amount. Moreover, since the same liquid flows through each impeller, the discharge of a multi-stage pump is the same as the discharge passing through each impeller of series.

24.3.4 Types of Centrifugal Pumps Based on Relative Direction of Flow

On the basis of the direction of flow of liquid through the impeller, the centrifugal pumps are classified as: (i) radial flow pump, (ii) axial flow pump, and (iii) mixed flow pump as shown in Figs. 24.6(a, b, c).

(i) Radial Flow Pump: In this pump, liquid flows through the impeller in the radial direction only. Generally, all centrifugal pumps are manufactured with radial flow impellers.

(ii) Axial Flow Pump: In this pump, the flow through the impeller is in the axial direction only. Axial flow pumps are designed to deliver very large quantities of water at relatively low heads. Thus, they are ideally suited for irrigation purposes. Although the axial pump is a rotodynamic pump, it is hardly justifiable to call it a centrifugal pump, because the centrifugal force is not used for the generation of pressure. Pressure is developed by flow of liquid over blades of aerofoil section just as the wings of an aeroplane produce the lift. The action is just the opposite of a propeller turbine.

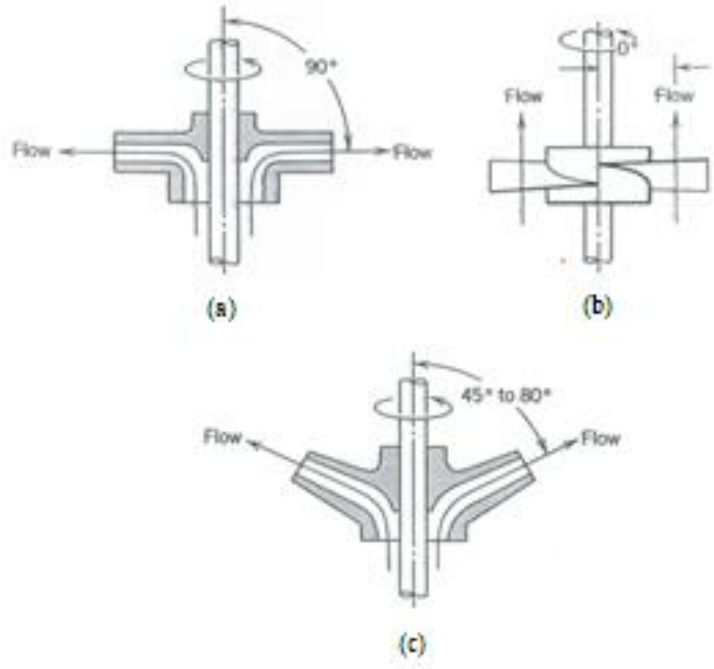


Fig. 24.6. Types of centrifugal pumps based on realtive direction of flow through the impeller: (a) Radial flow pump; (b) Axial flow pump; (c) Mixed flow pump. (Source: James, 1988)

(iii) Mixed Flow Pump: In this pump, the liquid flows through the impeller axially as well as radially; that is there is a combination of radial flow and axial flow. Some mixed flow impellers look like a screw and are known as screw impellers. A mixed flow impeller is just a modification of radial flow type enabling it to pump a large quantity of water. As such, mixed flow pumps are generally used where a large quantity of liquid is to be discharged at a low head. In older designs, a large quantity of water was delivered by running several pumps in parallel, but this arrangement is now obsolete.

24.3.5 Types of Centrifugal Pumps Based on Number of Entrances to the Impeller

Based on the number of entrances to the impeller, the centrifugal pumps are classified as: (i) single suction (entry) pump, and (ii) double suction (entry) pump. In the single suction pump, water from a suction pipe enters into the impeller from one side of the impeller only. However, in a double suction pump, water enters into the impeller from both sides of the impeller. The double suction centrifugal pump is suitable for pumping large quantities of liquid, because it provides a larger inlet area. The provision of double suction/entry has an added advantage that the axial thrust on the impeller is neutralized.

24.3.6 Types of Centrifugal Pumps Based on Disposition of Shaft

The shaft of a centrifugal pump can be disposed horizontally or vertically, and accordingly the centrifugal pump is called a horizontal centrifugal pump or vertical centrifugal pump. Normally, centrifugal pumps are designed with horizontal shafts and impellers are mounted vertically on the shafts. They are most commonly used for irrigation. Vertical centrifugal pumps have vertical shafts and impellers are mounted horizontally on the shafts. Vertical disposition of the shaft provides an economy in space occupied, and hence vertical pumps are suitable for deep wells, mines, etc. They can also be used for irrigation purposes. Note that the volute-type vertical centrifugal pumps may be either submerged or exposed. The body of an exposed vertical centrifugal pump is usually set in a sump at a level that can accommodate the suction lift.

24.3.7 Types of Centrifugal Pumps Based on Based on Type of Impeller

Depending on the type and viscosity of liquid to be pumped, the pump may have a closed, semi-open, or open impeller [Figs. 24.7(a, b, c)]; accordingly the centrifugal pumps are classified as: (i) closed impeller pump, (ii) semi-open impeller pump, and (iii) open impeller pump. Each of these impeller types may have ferrous, non-ferrous, or stone-coated impeller to resist the chemical effect of the liquid being pumped.

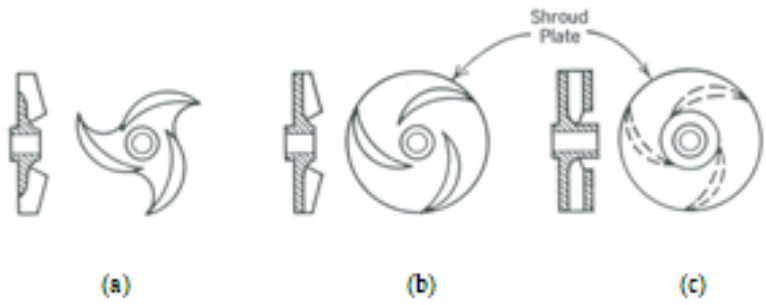


Fig. 24.7. Types of centrifugal pumps based on type of impeller: (a) Closed impeller pump; (b) Semi-open impeller pump; (c) Open impeller pump. (Source: James, 1988)

(i) Closed Impeller Pump

Closed impeller pumps have shrouded impellers. Shrouded or closed impeller is the one in which the vanes are covered with metal plates (also called 'shrouds') on both sides. These plates (shrouds) are known as crown plate (i.e., top plate) and base plate (i.e., bottom plate). The closed impeller provides better guidance for the liquid and is the most efficient. However, this type of impeller is most suited when the liquid to be pumped is free from debris and other such impurities. That is, the closed impeller is meant for handling non-viscous liquids such as ordinary water, hot water, hot oil and chemicals like acids, etc.

Material of the impeller should be selected according to the chemical properties of the liquid to be pumped. For example, for hot water at a temperature greater than 150 ^{o}C, cast steel impeller is recommended. Non-ferrous (e.g., gun metal) impellers are used for chemicals or brackish water, which are liable to corrode a ferrous surface. For pumping acids, the impeller and all inside surfaces in contact with liquid should be coated with stone. On the other hand, for pumping ordinary water, impellers may be made of bronze or cast iron. Impellers for light duty could be made of rigid plastic materials.

(ii) Semi-Open Impeller Pump

This type of pump is equipped with an impeller having a shroud on one side only, i.e., the vanes have only the base plate and there is no crown plate. It is used for viscous liquids such as sewage water, paper pulp, sugar molasses, etc. In order to minimize the chances of impeller clogging, the number of vanes is reduced and their height is increased while designing a semi-open impeller. Sewage pumps (also called 'non-clog pumps') are meant for handling sewage water containing solid particles, rags and other impurities. Hence, they are made without any protuberances around which rags could wrap and catch. They have impeller blades which are well rounded at their entrance ends and have large passage-ways between the vanes. The number of impeller blades is small, usually not more than two. They are always of the single suction type because the shaft extends through the eyes of the impeller in double-suction pumps, thereby creating an easy place for rags to catch and wrap, and thus clogging the pump.

The non-clog pumps for paper stock and other similar materials have open-type impellers, with entrance blades especially designed to prevent separation of stock and water. The choice of impeller material is dependent on the chemical nature of the liquid to be handled. Non-clog pumps must be heavily constructed mechanically in order to provide satisfactory service.

(iii) Open Impeller Pump

Open impeller pumps have the impellers which are not provided with any shroud, i.e., the vanes have neither the crown plate nor the base plate; the vanes are attached to a central hub. These are used in dredgers and elsewhere for handling a mixture of water, sand, clay and pebbles, wherein the solid contents may be as high as 25%. The open impeller is meant for performing very rough duty. It is generally made of forged steel. Its service life depends on the type of material handled, and it may be as small as 40-50 hours, or in some cases it may vary from 500 to 1000 hours.

24.3.8 Types of Centrifugal Pumps Based on Specific Speed

The specific speed is a sound basis for a technical classification of centrifugal pumps. It is the only characteristic index of a pump when several impellers can be used for the same head and capacity. It should be noted that the performance and dimensional proportions of pumps having same specific speed will be the same even though their outside diameters and actual operating speeds may vary.

Specific speed of a centrifugal pump is defined as the speed of a geometrically similar pump when delivering 1 L/s against a head of 1 meter. The most commonly adopted expression for the specific speed of pumps is:

$$N_s = \frac{N\sqrt{Q}}{H^{3/4}} \qquad (24.4)$$

Where, N = speed of the pump (rpm), Q = discharge of the pump (L/s), and H = total dynamic head (m).

Here, note that the values of Q and H to be used in Eqn. (24.4) for the purpose of calculating specific speed (N_s) are those corresponding to the maximum efficiency of the pump at its normal working speed. Furthermore, for a multi-stage pump, the value of H to be used for the calculation of N_s is obtained by dividing the actual head developed with the number of stages. Similarly, the value of Q for a double-suction pump is taken as half the actual discharge delivered by the pump.

Based on the specific speed, centrifugal pumps are classified into five classes as shown in Table 24.1.

Table 24.1. Types of centrifugal pumps based on the specific speed (Modi and Seth, 1998)

Sl. No.	Type of Pump	Specific Speed (N_s)
1	Slow-Speed Radial Flow	300−900
2	Medium-Speed Radial Flow	900−1500
3	High-Speed Radial Flow	1500−2400
4	Mixed Flow or Screw Type	2400−5000
5	Axial Flow or Propeller Type	3400−15000

Lesson 25 Pump Installation and Head Calculation

25.1 Installation of Centrifugal Pumps

Efficient operation of a centrifugal pump is based on its proper installation with suitable foundation in correct location and with appropriate alignment of coupling. The type of installation depends on the nature of water source (surface water bodies or groundwater), type of well (open well or tubewell), extent of lining in case of tubewells, seasonal variation in the static water level, and kind of prime mover (electric motor or diesel engine) used in operating the pump. Typical low-lift centrifugal pump installations are of two types (FAO, 1986): (a) suction installation, and (b) sump installation or below ground installation.

25.1.1 Suction Installation

The simplest installation is the suction installation (Fig. 25.1). The suction lift of an ordinary centrifugal pump is limited to 5-6 m in practice, and it is further reduced if a larger length of the suction pipe is used. Hence, a suitable length and larger size of suction pipe is desirable for better performance of the pump. Also, the suction lift reduces to about 2 m at an altitude of 2000 m (FAO, 1986). In case of higher suction lift, it is certain that the problem will be experienced in priming the pump, retaining its prime, etc. A foot valve (Fig. 25.1) is an important part of any such installation, without which the moment the pump stops or slows down, all the water in the pipeline will run back through the pump making it impossible to restart the pump unless the pipeline is first refilled with water. In addition, if the pump is driven by an electric motor and water flows back through the pump; it can run backwards and possibly damage the electrical system.

If the delivery pipeline is long, it is also important to have another check valve (non-return valve) at the pump outlet (Fig. 25.1). This is necessary because if suddenly pump stops, the flow will continue until the pressure drops enough to cause cavitation in the pipeline; when the upward momentum is exhausted, the flow reverses and cavitation bubbles implode creating severe water hammer. In addition, severe water hammer can occur when the flow reverses which causes the instantaneous closure of the foot valve. The impacts of such events are very dangerous; the pump casing and/or pipeline may burst. Thus, a check valve on the discharge line (at the pump outlet) protects the pump from any such back surge down the pipeline.

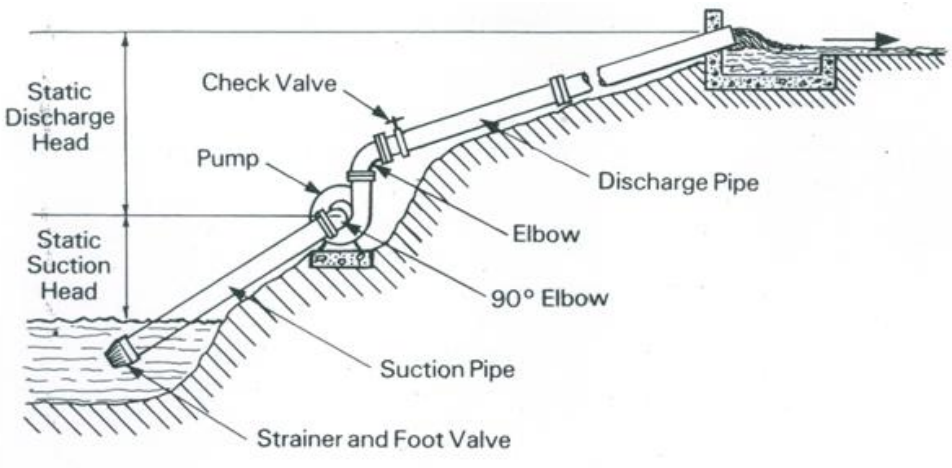


Fig. 25.1. Suction installation of a centrifugal pump. (Source: FAO, 1986)

25.1.2 Sump Installation

The other installation of centrifugal pumps could be below the ground surface as shown in Fig. 25.2; it is also known as 'sump installation'. If there is no surface mounting position low enough to permit suction pumping, centrifugal pumps are often placed in a sump or a pit where the suction head will be small or where the pump is located below the water level (Fig. 25.2). In the situation shown in Fig. 25.2, a long shaft is used to drive the pump from a surface mounted electric motor. Such a positioning of prime mover protects the motor and electric equipment from any possible flood, i.e., maximum water level in surface water bodies.

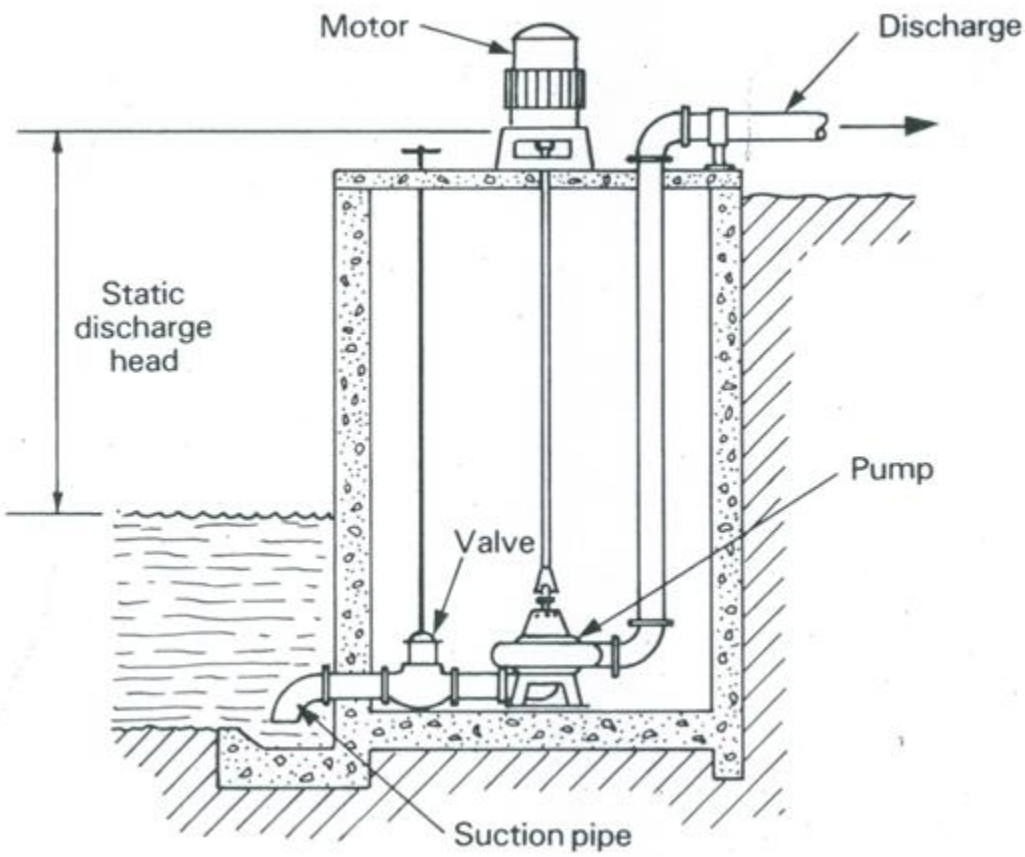


Fig. 25.2. Sump installation of a centrifugal pump. (Source: FAO, 1986)

25.2 Operation of Centrifugal Pumps

The basic principle on which a centrifugal pump functions has been discussed in Lesson 24. The first step in the operation of a pump is priming, i.e., the suction pipe and the pump-casing are filled with water so that no air/air-pocket is left. Now, when the pump is started, the revolution of the impeller inside a casing full of water produces a forced vortex, which is responsible for imparting a centrifugal head to the water. The rotation of the impeller causes a reduction in pressure at the center due to which the water in the suction pipe rushes into the eye of the pump. The speed of the pump should be high enough to produce centrifugal head sufficient to initiate discharge against the delivery head.

Mechanical action of the pump is to impart a velocity to the water. A water particle with a given velocity will rise to the same vertical height through which any particle should fall freely under gravity in order to attain the same velocity starting from rest. Therefore,

$$V = \sqrt{2gH} \quad \Rightarrow \quad H = \frac{V^2}{2g} \qquad (25.1)$$

Thus, if the outlet velocity of water in a pump is V, the pump can theoretically deliver water against a head of $\frac{V^2}{2g}$.

25.3 Limitation of Suction Lift

The absolute pressure head at the inlet of the pump is expressed as follows:

$$\frac{P_a}{\gamma} + \frac{P_s}{\gamma} = \frac{P_a}{\gamma} - \left[\frac{V_s^2}{2g} + h_s + h_{fs}\right] \qquad (25.2)$$

Where, P_a = atmospheric pressure acting on the free liquid surface in the sump, P_s = gage pressure at the inlet of the pump, γ= unit weight of water, V_s = velocity of flow in the suction pipe, $\frac{V_s^2}{2g}$ = suction velocity head, h_s = suction lift (static suction head), and h_{fs} = friction head losses in the suction pipe and in the foot valve and strainer.

It is not possible to create an absolute pressure at the pump inlet lower than the vapor pressure of the liquid. Thus, if P_v = vapor pressure of the liquid in the absolute unit, then in the limiting case, $P_a + P_s = P_v$, and hence from Eqn. (25.2), the limiting value of the suction lift (h_{sl}) can be given as follows:

$$h_{sl} = \left(\frac{P_a - P_v}{\gamma}\right) - \frac{V_s^2}{2g} - h_{fs} \qquad (25.3)$$

Note that the value of the suction lift (h_s) should not be more than that given by Eqn. (25.3). This is because greater h_s may result in a rapid vaporization of the liquid due to the reduction in pressure, which may ultimately lead to the incidence of cavitation that must be avoided for proper operation of the pump.

25.4 Calculation of Total Head

The energy imparted by a pump to the liquid increases pressure which is reflected as increased head. Total head generated by a pump is called 'pumping head' which is also known as 'manometric head' or 'total dynamic head'. Fig. 25.3 illustrates how the manometric head of a centrifugal pump can be calculated using the basic law of energy

conservation. For this, let's consider five points 0, 1, 2, 3 and 4 in a pumping system as shown in Fig. 25.3. The pumping head mainly comprise suction head and delivery head. Hence, the expressions for the suction head (H_s), delivery head (H_d), and manometric head (H_m) of a centrifugal pump are derived in the subsequent sub-sections.

25.4.1 Suction Head

Applying the Bernoulli's equation between Points 0 (at the water surface in the sump) and 1 (at the centerline of the pump or at the pump inlet) and considering water surface in the sump (i.e., Point 0) as a datum, we have:

$$0 = \frac{P_s}{\gamma} + \frac{V_s^2}{2g} + h_s + h_{fs} \qquad (25.4)$$

$$\Rightarrow \quad \frac{P_s}{\gamma} = -(h_s + h_{fs} + \frac{V_s^2}{2g}) \qquad (25.5a)$$

$$\therefore \quad H_s = -(h_s + h_{fs} + \frac{V_s^2}{2g}) \quad = \text{Total suction head} \qquad (25.5b)$$

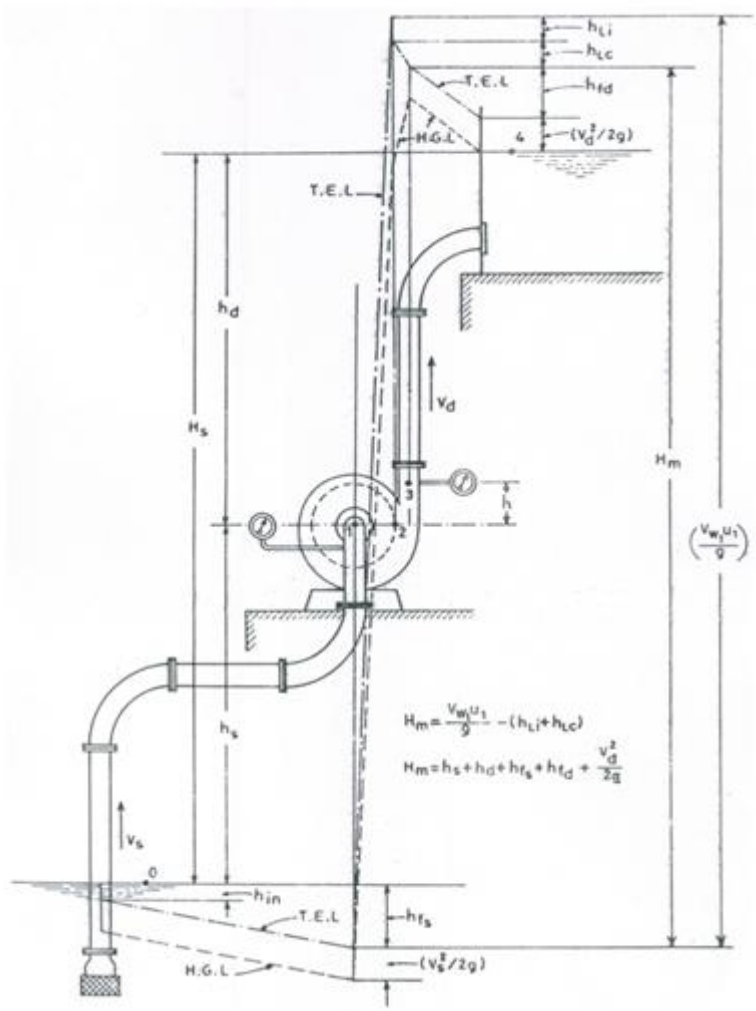


Fig. 25.3. Heads of a centrifugal pump. (Source: Modi and Seth, 1998)

25.4.2 Delivery Head

Similarly, applying the Bernoulli's equation between Points 3 (at the pump outlet) and 4 (at the water surface in delivery tank) and considering Point 3 as a datum, we have:

$$\frac{P_d}{\gamma} + \frac{V_d^2}{2g} = h_d + h_{fd} + \frac{V_d^2}{2g} \qquad (25.6)$$

$$\Rightarrow \quad \frac{P_d}{\gamma} = h_d + h_{fd} \qquad (25.7a)$$

$$\therefore \quad H_d = h_d + h_{fd} \qquad (25.7b)$$

Where, P_d = gage pressure at the outlet of the pump, V_d = velocity of flow in the delivery pipe, $\frac{V_d^2}{2g}$ = delivery velocity head, h_d = static delivery head, h_{fd} = friction head losses in the delivery pipe, and the remaining symbols have the same meaning as defined earlier. Note that the 'total delivery head' (H_d) is given as:

$$H_d = h_d + h_{fd} + \frac{V_d^2}{2g} \qquad (25.8)$$

25.4.3 Manometric Head

'Manometric head' of a pump is the total head produced by the pump to satisfy specific external requirements. If there are no energy losses in the impeller and casing of the pump (valid in an ideal or theoretical situation only), the manometric head will be equal to the energy given to the liquid by the impeller. As we know from the velocity triangles of an impeller, the energy (in terms of head) given to the liquid by the impeller is $\frac{V_{w_1} u_1}{g}$. However, in reality, head losses occur inside the pump (i.e., in the impeller and casing), and therefore the manometric head (H_m) for practical purposes is given as:

$$H_m = \frac{V_{w_1} u_1}{g} - Head\ losses\ in\ the\ pump \qquad (25.9a)$$

Or,
$$H_m = \frac{V_{w_1} u_1}{g} - (h_{Li} + h_{Lc}) \qquad (25.9b)$$

Where, V_{w1} = velocity of whirl of the liquid at the impeller outlet, u_1 = peripheral velocity of the impeller at the outlet, h_{Li} = head loss in the impeller, and h_{Lc} = head loss in the casing.

Now, applying the Bernoulli's equation between Points 1 and 3, and considering Point 1 as a datum, we have:

Pressure head at Point 1 + Velocity head at Point 1 + Head generated by the impeller = Pressure head at Point 3 + Velocity head at Point 3 + Difference in

the levels of two gages + Head losses in the pump

That is,

$$\frac{P_s}{\gamma} + \frac{V_s^2}{2g} + \frac{V_{w_1} u_1}{g} = \frac{P_d}{\gamma} + \frac{V_d^2}{2g} + h + h_{Li} + h_{Lc} \qquad (25.10)$$

$$\Rightarrow \quad \frac{V_{w_1} u_1}{g} - (h_{Li} + h_{Lc}) = \left(\frac{P_d}{\gamma} + \frac{V_d^2}{2g} + h\right) - \left(\frac{P_s}{\gamma} + \frac{V_s^2}{2g}\right) \qquad (25.11)$$

$$\therefore \quad H_m = \left(\frac{P_d}{\gamma} + \frac{V_d^2}{2g} + h\right) - \left(\frac{P_s}{\gamma} + \frac{V_s^2}{2g}\right) \qquad (25.12)$$

Thus, the manometric head (H_m) is equal to the difference between total energy of the liquid at outlet of the pump and that at inlet of the pump. In the above equations, P_d = pressure at the outlet of the pump (i.e., pressure gage reading on the delivery pipe); P_s = pressure at the inlet of the pump (i.e., vacuum gage reading on the suction pipe); h = vertical difference in the levels of the vacuum and pressure gages; V_d = velocity of flow in the delivery pipe; and V_s = velocity of flow in the suction pipe.

If the two gages are fitted at the same level, then h = 0, and hence Eqn. (25.12) reduces to:

$$H_m = \left(\frac{P_d}{\gamma} + \frac{V_d^2}{2g}\right) - \left(\frac{P_s}{\gamma} + \frac{V_s^2}{2g}\right) \quad (25.13)$$

Now, substituting the expressions of $\frac{P_d}{\gamma}$ [from Eqn. (25.7a)] and $\frac{P_s}{\gamma}$ [from Eqn. (25.5a)] in Eqn. (25.13), we have another expression of the manometric head (H_m) as follows:

$$H_m = h_s + h_{fs} + h_d + h_{fd} + \frac{V_d^2}{2g} \quad (25.14)$$

Equation (25.14) is the commonly used expression to compute manometric head (or pumping head) required for a given pumping system. Note that one external parameter which does not appear in Eqn. (25.14) is the operating head that is needed for operating sprinklers, gated pipe or drip irrigation systems. Therefore, for computing the total head which a pump must provide to operate an irrigation device, it is necessary to add the required operating head.

In practice, the total dynamic head (TDH) for the suction installation of a centrifugal pump (Fig. 25.1) is calculated as follows:

TDH = Discharge Gage Reading + Vacuum Gage Reading + Distance between Point of Attachment of the Vacuum Gage and Centerline of the Discharge Gage + $\left(\frac{V_d^2}{2g} - \frac{V_s^2}{2g}\right)$ (25.15)

However, in case of sump installation, the centrifugal pump is installed in such a way that there is a positive pressure on the suction side. In this situation, a pressure gage is used on the pump suction instead of a vacuum gage. Thus, for the sump installation of a centrifugal pump (Fig. 25.2), TDH is calculated as follows:

TDH = Discharge Gage Reading - Suction Gage Reading + Distance between Centerlines of Discharge and Suction Gages + $\left(\frac{V_d^2}{2g} - \frac{V_s^2}{2g}\right)$ (25.16)

25.5 Calculation of Friction Head Losses

As we know that there is a frictional resistance offered to the liquid flowing through suction and delivery pipes, which results in the loss of head. Head losses due to friction are known as friction head losses, which constitute significant components of the total pumping head for a particular pumping system. Friction head losses are classified as: (a) major friction head losses, and (b) minor friction head losses. Major friction head losses are defined as the friction head losses due to the roughness of the inner surface of pipe-network, and the viscosity and density of the flowing fluid. Minor friction head losses are defined as the head

losses due to pipe fittings, bends, entry and exit, sudden expansion and contraction, valves, screen, and so on.

The major friction head losses (h_f) in the suction and delivery pipes of a pumping system can be calculated by the Darcy-Weisbach equation, which is given as:

$$h_f = \frac{fLv^2}{2gd} \qquad (25.17)$$

Where, f = friction factor (its value is usually obtained from the Moody diagram), L = length of the pipe, v = velocity of flow, d = inside diameter of the pipe, and g = acceleration due to gravity.

Moreover, minor friction head losses can be calculated with the help of standard tables. The sum of major and minor friction head losses constitutes the 'total friction head losses'. Generally, the value of major friction head losses is much higher than that of minor friction head losses, and hence the minor friction head losses are often neglected while computing the total head of a pumping system (i.e., TDH). However, if the value of minor friction head losses is significantly large, they must not be neglected during the calculation of pumping head (TDH).

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QUIZ

State whether the following statements are true (T) or false (F):

1. The suction lift of an ordinary centrifugal pump is practically limited to 10 m.
2. At the pump inlet, an absolute pressure lower than the vapor pressure of the liquid to be pumped can be created.
3. Larger suction head in centrifugal pumps may lead to cavitation.

4. The theoretical manometric head of a centrifugal pump is equal to the energy given to the liquid by the impeller.
5. Both major and minor friction head losses can be calculated by the Darcy-Weisbach formula.
6. The total head required for operating a sprinkler irrigation system is equal to the manometric head of the pump plus the operating head of the sprinkler.

Lesson 26 Power Requirement and Efficiency of Centrifugal Pumps

26.1 Calculation of Power Requirement for a Pumping System

Power requirements for a pumping system can be calculated for a given pump installation. Power of a pumping system can be classified into four classes: (i) Water Power (WP), (ii) Shaft Power (SP), (iii) Brake Power (BP), and (iv) Input Power (IP). They are described in the subsequent sub-sections.

26.1.1 Water Power

Water power is the theoretical power required for pumping. It is the power required by a pump in lifting a given quantity of water in a unit time assuming no losses of power in the pump. The water power (WP) of a pump can be calculated using the following equation:

$$WP = \gamma \times Q \times H \qquad (26.1)$$

Where, WP = water power (W), = specific weight of the water (N/m^3), Q = discharge of the pump (m^3/s), and H = total head or total dynamic head (m).

26.1.2 Shaft Power

Shaft power is the power required at the pump shaft. It takes into account the loss of power in the pump. Shaft power (SP) of a pump can be computed using the following equation:

$$SP = \frac{\gamma Q H}{\eta_{pump}} \qquad (26.2)$$

Where, η_{pump} is the overall efficiency of the pump (in fraction) and the remaining symbols have the same meaning as defined earlier. If the SI units of γ, Q and H are used, the unit of SP will be in watts (W).

26.1.3 Brake Power

It is the actual power to be supplied by the prime mover (engine or electric motor) for driving a pump. When there is direct drive from an engine or electric motor to the pump, the brake power is equal to the shaft power. In this case, the drive efficiency is assumed to be

100%. However, if belt or other indirect drives are used to run a pump, the brake power (BP) needs to be computed. Brake power (BP) can be calculated as:

$$BP = \frac{\gamma Q H}{\eta_{pump} \times \eta_{drive}} \qquad (26.3)$$

Where, η_{drive} is the efficiency of the drive (in fraction) and the remaining symbols have the same meaning as defined earlier. Like the WP and SP, if the SI units of, Q and H are used, the unit of BP will be in watts (W).

26.1.4 Input Power

Input power (IP) of a pump is defined as the ratio of brake power to the efficiency of the motor or engine used for driving the pump. Mathematically, it is expressed as:

$$IP = \frac{BP}{\eta_{motor/engine}} \qquad (26.4)$$

Where, $\eta_{motor/engine}$ is the efficiency of the motor or engine (in fraction). If the SI unit of BP is used for calculating IP, the unit of IP will be in watts (W).

As mentioned above, if the SI units are used to compute WP, SP, BP and IP, the units of power will be in watts (W), which can be converted into horsepower (a popular unit of power) by dividing the power in watts with 746.

26.1.5 Energy Consumption by a Pump

The energy consumption by a pump can be calculated as follows:

Energy Consumption = Input Power × Hours of Pump Operation (26.5)

26.1.6 Illustrative Example

Problem (Michael et al., 2008): A pump lifts 100,000 liters of water per hour against a total head of 20 m. Calculate water power of the pump. If the pump has an efficiency of 75%, what size of prime mover is required to operate the pump? If a direct drive electric motor with an efficiency of 80% is used to operate the pump, compute the cost of electric energy in

a month of 30 days. Assume that the pump is operated for 12 hours per day for 30 days and that the cost of electricity is Rs 5.00 per Unit.

Solution: From the question, we have: Discharge of the pump (Q) = 100,000 L = 0.028 m^3/s, Total head (H) = 20 m, Efficiency of the pump () = 75%, Efficiency of the motor (η_{motor}) = 80%, and Cost of electricity = Rs 5.00/Unit.

Water Power, WP = γ×Q×H = 9810 × 0.028 × 20 = 5493.6 W, Ans.

Shaft Power, $SP = \frac{WP}{\eta_{pump}} = \frac{5493.6}{0.75}$ = 7324.8 W, Ans.

Note that since direct drive is used to operate the pump, the shaft power determines the required size of the prime mover.

Now, Input Power, $IP = \frac{BP}{\eta_{motor}} = \frac{7324.8}{0.80}$ = 9156 W

Total energy consumption per month = Input Power × Hours of Pump Operation

= 9156 × 12 × 30

= 3296160 watt-hours (Wh)

= 3296.16 Kilowatt-hours (kWh)

As we know that 1 kWh is equal to 1 Unit, therefore, the cost of electric energy = Rs. (3296.16 × 5) = Rs. 16480.80, Ans.

26.2 Efficiency of Centrifugal Pumps

The efficiency (η_{mano}) of a centrifugal pump can be expressed in four ways (Modi and Seth, 1998) and accordingly it is classified as: (i) Manometric efficiency, (ii) Volumetric efficiency, (iii) Mechanical efficiency, and (iv) Overall efficiency.

26.2.1 Manometric Efficiency

Manometric efficiency (h_{mano}) is defined as the ratio of the manometric head developed by the pump to the head imparted by the impeller to the liquid. That is,

$$\eta_{mano} = \frac{H_m}{(V_{wo}u_o / g)} = \frac{gH_m}{V_{wo}u_o} \qquad (26.6)$$

Where, V_{wo} = velocity of whirl of the liquid at the exit point (outlet) of the impeller; and u_o = peripheral (or, tangential) velocity of the impeller at the vane outlet.

If Q = discharge of the pump (i.e., volume of liquid actually delivered per second by the pump), and = specific weight of the liquid, then the manometric efficiency (h_{mano}) can also be expressed as follows:

$$\eta_{mano} = \frac{\gamma Q H_m}{\gamma Q (V_{wo}u_o / g)} \qquad (26.7)$$

26.2.2 Volumetric Efficiency

Volumetric efficiency (h_{vol}) is defined as the ratio of the quantity of liquid discharged per second from the pump to the quantity of liquid passing per second through the impeller. Leakage loss occurs in the centrifugal pump and if this loss is represented by, then the volumetric efficiency is given as:

$$\eta_{vol} = \frac{Q}{(Q + \Delta Q)} \qquad (26.8)$$

Where, Q is the actual discharge of the pump.

26.2.3 Mechanical Efficiency

Mechanical efficiency (η_{mech}) is defined as the ratio of the power actually imparted by the impeller to the power supplied to the shaft by the prime mover. That is,

$$\eta_{mech} = \frac{\gamma(Q + \Delta Q) \times (V_{wo} u_o / g)}{Shaft\ Power} \qquad (26.9)$$

Eqn. (26.9) means h_{mech} = Actual Power imparted by the impeller ¸ Shaft Power, which can be mathematically expressed as:

$$\eta_{mech} = \frac{V_{wo} u_o / g}{Energy\ head\ given\ to\ the\ shaft} \qquad (26.10)$$

Or,

$$\eta_{mech} = \frac{V_{wo} u_o / g}{(V_{wo} u_o / g) + (Mechanical\ head\ losses\ in\ the\ bearings)} \qquad (26.11)$$

26.2.4 Overall Efficiency

Overall efficiency (η_o) of a pump is defined as the ratio of the power output from the pump to the power input to the pump from the prime mover. That is,

$$\eta_o = \frac{\gamma\ Q\ H_m}{Shaft\ Power} \qquad (26.12)$$

Note that the overall efficiency (h_o) can also be calculated as the product of η_{mano}, η_{vol} and η_{mech}. That is,

$$\eta_o = \eta_{mano} \times \eta_{vol} \times \eta_{mech} \qquad (26.13)$$

$$\Rightarrow \eta_o = \frac{H_m}{V_{wo} u_o / g} \times \frac{Q}{Q + \Delta Q} \times \frac{\gamma(Q + \Delta Q)(V_{wo} u_o / g)}{Shaft\ Power} \qquad (26.14)$$

$$\therefore \ \eta_o = \frac{\gamma Q H_m}{Shaft\ Power} \qquad (26.15)$$

Equation (26.15) is the same as the expression obtained from the definition of overall efficiency [i.e., Eqn. (26.12)].

QUIZ

State whether the following statements are true (T) or false (F):

1. 'Input power' is the theoretical power required for pumping.
2. 'Water power' is the power required by a pump in lifting a given quantity of water in a unit time assuming no losses of power.
3. For the pumps directly driven by the electric motor, the 'brake power' is equal to the 'shaft power'.
4. Power imparted by the impeller is equal to the power delivered by the motor to the shaft.
5. 'Mechanical efficiency' of a centrifugal pump is the ratio of the actual power imparted by the impeller to the 'shaft power'.

Lesson 27 Design of Centrifugal Pumps: An Overview

27.1 Design Criteria for Centrifugal Pumps

Centrifugal pumps have two basic groups of parts: rotating and stationary. Rotating parts are impeller, shaft, wearing rings, shaft sleeves, bearings and mechanical seals. Stationary parts include casing, bearing housing, foot valves or reflux valves, strainer, pipes and pipeline accessories. Each component should be designed to obtain adequate operational efficiency. This lesson succinctly explains design concepts and some formulae related to the design of centrifugal pumps. The detailed procedures for designing different types of centrifugal pumps can be found in Michael and Khepar (1999), Stepanoff (1994), and Lobanoff and Ross (1985).

The design of centrifugal pumps are based on various criteria, of which the size of the impeller (i.e., inside and outside diameters), size of suction and delivery pipes, flow at the impeller inlet and exit, shape and size of casing, specific speed as well as the knowledge of cavitation phenomena and the concepts of 'net positive suction head available' (NPSHA) and 'net positive suction head required' (NPSHR) play an important role. They are discussed in subsequent sub-sections.

27.2 Design of Pump Impeller

The design of the pump impeller involves the selection of speed of rotation of the impeller, to meet a given head and capacity. This establishes the specific speed. With the help of the specific speed and a given capacity, the attainable efficiency of the proposed impeller may be predicted. The impeller profile and the layout of the vanes are designed with the following elements:

- Outside diameter of impeller.
- Inside diameter of the impeller.
- Flow at the impeller inlet and exit.

27.2.1 Outside Diameter of Impeller

If the outside diameter of the impeller is D_2 and the speed of the pump shaft is N (in rpm), then the peripheral (tangential) velocity of the impeller at the outlet (u_2) can be given as:

$$u_2 = (\pi D_2 N / 60) \qquad (27.1)$$

Furthermore, if H_m = total head or manometric head, then u_2 can also be expressed as follows:

$$u_2 = K_u \sqrt{2gH_m} \qquad (27.2)$$

Where, K_u = speed ratio for the centrifugal pump. The value of K_u usually ranges from 0.95 (for low specific speed impellers) to 1.25 (for high specific speed impellers).

From the above two equations, D_2 can be calculated as follows:

$$D_2 = \frac{60 K_u \sqrt{2gH_m}}{\pi N} \qquad (27.3)$$

It should be noted that Eqn. (27.3) can also be used to determine the head which a pump can develop if D_2 and N are known. It can serve as a check for an existing pump.

27.2.2 Inside Diameter of Impeller

The inside diameter of an impeller (D_1) is 2/3 to 1/3 of D_2 (outside diameter) depending on the specific speed (N_s) or total head (H_m). However, in most cases, $D_1 = 0.5\ D_2$.

27.2.3 Flow at Impeller Inlet and Exit

The rotating impeller of a centrifugal pump imparts energy to the fluid. As mentioned in earlier lesson, the impeller contains radial flow passages formed by rotating blades (vanes) arranged in a circle. A disk in the back (base plate) connects the impeller assembly to the shaft and another disk (crown plate) covers the blades on the front. The flow enters axially near the center of rotation and turns in the radial direction inside the impeller as shown in Fig. 27.1. Thus, the liquid enters the impeller at its center and leaves at its outer periphery. The flow follows certain streamlines inside the rotating impeller, approximately parallel to the blade surfaces. The shape of the blades and the resulting flow pattern in the impeller determine how much energy is transferred by a given size of the impeller and how efficiently it operates. The theoretical energy increase [i.e., theoretical head rise (H_{mth}) through the impeller] can be found by applying the principle of conservation of angular momentum.

The components of flow through an impeller can be best studied by means of velocity vectors as illustrated in Fig. 27.1. In this figure, the inlet and outlet velocity diagrams of an impeller with backward curved vanes are shown. Note that this figure shows a portion of the impeller of a centrifugal pump with one blade only. The velocity vector diagram is triangular and hence, it is known as a 'velocity triangle'. It can be drawn for any point the

flow path through the impeller. However, velocity triangles are usually drawn at the impeller inlet and outlet (exit) and are called 'inlet or entrance velocity triangle' and 'outlet or discharge velocity triangle', respectively.

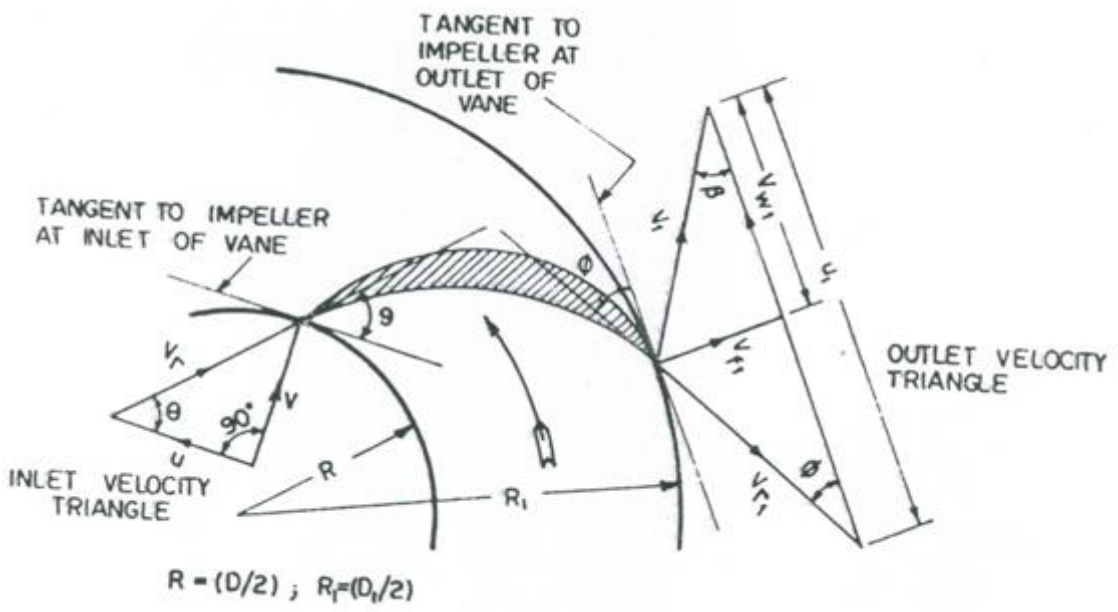

Fig. 27.1. Inlet and outlet velocity triangles for an impeller vane.

(Source: Modi and Seth, 1998)

Let V = absolute velocity of the liquid (measured in the stationary frame of reference),

u = peripheral (tangential) velocity of the impeller (i.e., rotational velocity),

V_r = relative velocity of the liquid (i.e., velocity of the liquid relative to the impeller),

V_f = velocity of flow of the liquid, and

V_w = velocity of whirl of the liquid.

Further, let Θ = impeller vane angle at the inlet, and Φ = impeller vane angle at the outlet. Similarly, α = angle between the absolute velocity of entering liquid and the peripheral velocity of the impeller at the inlet point, and β = angle between the absolute velocity of leaving liquid and the peripheral velocity of the impeller at the outlet (exit) point.

Since there are no guide vanes at the entrance to the impeller, the direction of absolute velocity of liquid at this point is not directly known. However, for the best efficiency of the centrifugal pump, it usually assumed that the liquid enters the impeller radially. Thus, a = 90° (see inlet velocity triangle in Fig. 27.1) and the velocity of whirl (V_w) at the inlet is zero, and hence V and V_f at the inlet are the same. Moreover, it is desired that the liquid enters and leaves the vane without shocks. This can be ensured if the inlet and outlet tips of the

vane are parallel to the direction of the relative velocities at the two tips. As such it is assumed that the peripheral velocities u and u_1 are parallel to the tangents to the impeller at the inlet and outlet vane tips, respectively (Fig. 27.1). The relative velocity of the liquid (V_r) is obtained by the vector sum of the absolute velocity (V) and the peripheral velocity of the impeller (u).

27.2.4 Minimum Impeller Diameter

An expression for the minimum diameter of an impeller can be derived on the basis of the fact that for the pump to start pumping, the centrifugal head must be equal to the total head or manometric head (H_m). That is,

$$\frac{u_2^2 - u_1^2}{2g} = H_m \qquad (27.4)$$

$$\Rightarrow \left(\pi D_2 N/60\right)^2 - \left(\pi D_1 N/60\right)^2 = 2gH_m$$

$$\therefore \left(D_2^2 - D_1^2\right) = \left(\frac{60}{\pi N}\right)^2 \times \left(2gH_m\right) \qquad (27.5)$$

Let's assume that $D_1 = 0.5\ D_2$ (as mentioned above). Now, Eqn. (27.5) can be expressed as follows:

$$D_2 = \frac{60 \times \sqrt{2gH_m}}{\pi\ N \times \sqrt{0.75}} \qquad (27.6)$$

Or,

$$D_2 = \frac{97.68 \times \sqrt{H_m}}{N} \qquad (27.7)$$

Equation (27.7) will yield the minimum outside diameter of an impeller for given manometric head (H_m) and speed of the pump (N).

27.2.5 Head Generated by the Impeller and Design Discharge

The work done per second by the impeller on the liquid is given as:

$$Work\ Done = \frac{W}{g}(V_{w1}u_1 - V_w u) \qquad (27.8)$$

Where, W = weight of the liquid per second passing through the impeller; V_w and V_{w1} = velocity of whirl of the liquid at the inlet and outlet of the impeller, respectively; and u and u_1 = peripheral velocity of the impeller at its inlet and outlet, respectively.

Since the liquid enters the impeller radially (Fig. 27.1), the value of V_w will be zero. Therefore, Eqn. (27.8) will reduce to:

$$Work\ Done = \frac{W}{g}(V_{w1}u_1) \qquad (27.9)$$

Thus, the work done per second per unit weight of the liquid is $\frac{V_{w1}u_1}{g}$, which represents the theoretical head imparted by the impeller to the liquid (i.e., theoretical manometric head). That is,

$$H_{mth} = \frac{V_{w1}u_1}{g} \qquad (27.10)$$

Now, from the inlet velocity triangle (Fig. 27.1), we have $\tan\theta = \frac{V}{u}$ and $\sin\theta = \frac{V}{V_r}$, and therefore,

$$u = \frac{\pi D N}{60} \qquad (27.11)$$

Similarly, from the outlet velocity triangle (Fig. 27.1), we have $\cot\phi = \frac{u_1 - V_{w1}}{V_{f1}}$ and hence

$$V_{w1} = (u_1 - V_{f1}\cot\phi) \qquad (27.12)$$

Where, $u_1 = \frac{\pi D_1 N}{60}$, and $V_{f1} = \frac{Q}{\pi D_1 \times B_1}$.

Now, the design discharge of the pump (Q) can be calculated as follows:

Case 1: If vane thickness is neglected, the design discharge (Q) is given as:

$$Q = (\pi D_1 \times B_1) \times V_{f1} \qquad (27.13)$$

Where, D_1 = outer diameter of the impeller, and B_1 = width of the impeller vane at the outlet.

Case 2: If the vane thickness of the impeller is considered, then Q is given as:

$$Q = (\pi D_1 B_1) \times V_{f1} \times A_1 \qquad (27.14)$$

Where, A_1 = percentage of the outlet area not occupied by impeller vanes.

27.3 Design of Suction and Delivery Pipes

27.3.1 Diameter of Suction Pipe

Let d_s be the diameter of the suction pipe and V_s be the velocity of flow in the suction pipe. Then, the amount of water to be pumped is given as:

$$Q = \frac{\pi d_s^2}{4} \times V_s \qquad (27.15)$$

Or,

$$d_s = \sqrt{\frac{4Q}{\pi V_s}} \qquad (27.16)$$

Here, the value of V_s is usually taken as 1.5 to 3 m/s (Modi and Seth, 1998).

27.3.2 Diameter of Delivery Pipe

Let d_d be the diameter of the delivery pipe and V_d be the velocity of flow in the delivery pipe. Then, the discharge of the pump can be given as:

$$Q = \frac{\pi\, d_d^2}{4} \times V_d \qquad (27.17)$$

Or,

$$d_d = \sqrt{\frac{4Q}{\pi V_d}} \qquad (27.18)$$

Here, the value of V_d is generally taken as 1.5 to 3.5 m/s. In fact, the value of V_d is normally equal to or slightly higher than that of V_s (Modi and Seth, 1998).

27.4 Design of Pump Casing

Pumps casing should be so designed as to minimize the loss of kinetic head through eddy formation, friction, etc. Efficiency of a pump largely depends on the type of casing. In general, centrifugal pump casings are of three types: (i) volute casing, (ii) volute casing with vortex chamber, and (iii) diffuser casing. Interested readers are referred to Michael and Khepar (1999) and Lobanoff and Ross (1985) for details about the design of pump casings and other elements.

27.5 Cavitation Problem

A problem commonly encountered in the operation of hydraulic machines is 'cavitation'. Cavitation occurs when the pressure drops below the vapor pressure of the liquid. It is caused as follows:

(a) Vaporization of the liquid and/or release of dissolved air at low pressure,

(b) Movement of the vapor/gas into a high-pressure region,

(c) Collapse of the vapor/gas bubbles when subjected to a high pressure, and

(d) Release of energy and pressure wave of high intensity (i.e., shock wave) resulting in (i) pitting and erosion of metal surfaces, and (ii) noise and vibration of the machine.

Thus, in brief, cavitation can be defined as the formation and subsequent collapse of vapor bubbles. When these collapses occur violently on interior surfaces of the hydraulic machines (e.g., pumps, turbines, etc.), they produce ring-shaped indentations on the surface called 'pits'. Continued cavitation and pitting not only deteriorate the efficiency of hydraulic machines but also severely damage the hydraulic machines and hence, it must be avoided.

Cavitation is more likely to take place in hydraulic machines in the places such as: (a) suction side of the pumps, (b) draft tube of the hydraulic turbines, (c) venturi or the minimum area section of passages, (d) boundaries in the vicinity of high velocity flow, and (e) siphon passages. It is necessary to avoid cavitation in hydraulic machines, which can be done by limiting the minimum suction pressure above a critical value which is denoted by the Thoma's cavitation parameter (s_c) and is expressed as follows:

$$\sigma_c = \frac{P}{\rho g H_m} \qquad (27.19)$$

Or,

$$\sigma_c = \frac{NPSH}{H_m} \qquad (27.20)$$

Where, P = local pressure in the hydraulic machine, ρ = density of the fluid, H_m = pumping head, and NPSH = net positive suction head.

Note that the efficiency of a hydraulic machine deteriorates rapidly if the Thoma's cavitation parameter (σ) is allowed to drop below the critical value (σ_c). The critical cavitation parameter (σ_c) is related to the specific speed of hydraulic machines.

It is sometimes preferred to express the cavitation parameter as the suction specific speed (S) instead of σ_c, which is given as:

$$S = \frac{N\sqrt{Q}}{(gH)^{3/4}} \qquad (27.21)$$

Where, N = speed of the pump, Q = discharge of the pump, and H = total suction head corrected for the local vapor pressure (i.e., NPSH).

The minimum safe value of S for pumps is 3 and that for turbines is 4. The usual design practice is to allow the formation of bubbles but not to allow the collapse of bubbles until downstream of the impeller passage. These are referred to as the supercavitating conditions in flow. Axial flow pumps are generally designed considering the supercavitating principle.

27.6 Net Positive Suction Head

The net positive suction head (NPSH) is defined as the absolute pressure head at the inlet of the pump minus the vapor pressure head (absolute) corresponding to the temperature of the liquid pumped plus the velocity head at this point (Modi and Seth, 1998). That is,

$$NPSH = \left(\frac{P_a}{\gamma} + \frac{P_s}{\gamma} \right) - \frac{P_v}{\gamma} + \frac{V_s^2}{2g} \qquad (27.22)$$

$$\Rightarrow \quad NPSH = \frac{P_a}{\gamma} - \frac{P_v}{\gamma} - h_s - h_{fs} \qquad (27.23)$$

Where, P_a = atmospheric pressure, P_v = vapor pressure of the liquid at the operating temperature, γ = unit weight of the liquid, h_s = static suction head, and h_{fs} = friction head losses on the suction side of the pump.

It is clear from Eqn. (27.23) that NPSH is equal to the total suction head $(H_s)_{abs}$ corrected for the local vapor pressure. Thus, the concept of NPSH is applicable to only suction installation of centrifugal pumps.

The concept of NPSH is very commonly used in the pump industry. For any pump installation, a distinction is usually made between the net positive suction head required (NPSHR) and the net positive suction head available (NPSHA). If sufficient energy is not present in the liquid on the intake side of the pump to move the liquid into the eye of the impeller, then the liquid will vaporize and cavitation will occur in the pump. As mentioned above, cavitation should be avoided.

In order to ensure that the required energy is available on the intake side of the pump, an analysis must be made to determine the net positive suction head available (NPSHA). The available head is a function of the system in which the pump operates. It can be calculated for all suction installations of pumps using Eqn. (27.23). Also, the relationship between drawdown and discharge from the water source must be known to compute the NPSHA-Q relationship.

On the other hand, NPSHR varies with the pump design, pump-speed, and its capacity. Its value is determined experimentally for each pump by the pump manufacturers. Manufacturers conduct laboratory tests to determine NPSHR values for each pump model they manufacture. Both the NPSHA and NPSHR vary with the discharge in such a manner that with an increase in the discharge NPSHA decreases but NPSHR increases. In order to have cavitation-free operation of centrifugal pumps, the value of NPSHA should be greater than that of NPSHR. In other words, if NPSHA is less than NPSHR, then the pumps will cavitate.

Module 6_Performance Characteristics, Selection and Maintenance of Pumps

Lesson 28 Characteristic Curves of Centrifugal Pumps

28.1 Introduction

A pump is usually designed for one speed, flow rate and head, but in actual practice the operation may be at some other condition of head or flow rate, and for the changed conditions the behavior of the pump may be quite different. For instance, if the flow through the pump is less than the designed quantity, the magnitude of flow velocity through the impeller will be changed, thereby changing the head developed by the pump, and at the same time the losses will increase so that the efficiency of the pump will be lowered. Therefore, in order to predict the behavior and performance of a pump under varying conditions, pump tests are performed, and the results of the tests are plotted. The curves thus obtained are known as the 'characteristics curves' of the pump.

Most pump manufacturers have their own pump testing laboratories. They normally publish a set of characteristic curves for each pump model manufactured by them. These curves are developed by testing several pumps of a specific model. Some manufacturers' curves represent the average performance of all pumps of a specific model tested, while other manufacturers prepare their curves for the pump having the poorest performance. In this lesson, firstly pump characteristics curves are defined and classified, and then different types of characteristics curves of centrifugal pumps are discussed.

28.2 Defining Pump Characteristic Curves

The pump characteristic curves can be defined as 'the graphical representation of a particular pump's behavior and performance under different operating conditions'.

The operating properties of a pump are established by the geometry and dimensions of the pump's impeller and casing. Curves relating total head, efficiency, power, and net positive suction head required (NPSHR) to discharge or pump capacity (Q) are utilized to describe the operating properties (characteristics) of a pump. This set of four curves is known as the pump characteristic curves or pump performance curves.

28.3 Classification of Pump Characteristic Curves

Pump characteristics curves can be classified into four groups: (i) Main characteristic curves, (ii) Operating characteristic curves, (iii) Constant efficiency curves, and (iv) Constant head and constant discharge curves. Each group characterizes one aspect of the pump's performance. They are described in subsequent sections.

28.3.1 Main Characteristic Curves

The pump is usually designed to run at the same speed as the driving unit (i.e., prime mover), which is generally an electric motor of the AC induction type. When the electric power is not available, the pump may be driven by a diesel engine, or may be coupled to the tractor engine. In such circumstances, it is necessary to know the performance of a pump at different speeds, which can be best seen from the main characteristic curves of a pump.

In order to obtain the main characteristic curves of a pump, it is operated at different speeds. For each speed, the pump discharge (Q) is varied by means of a delivery valve and for the different values of Q, the corresponding values of manometric head (H_m), shaft power (SP) and overall efficiency (h_o) are measured or calculated. Thereafter, H_m vs Q; SP vs Q, and h_o vs Q curves for different speeds are plotted as shown in Fig. 28.1, which represent the main characteristics of a pump. Clearly, these curves are useful in indicating the performance of a pump at different speeds.

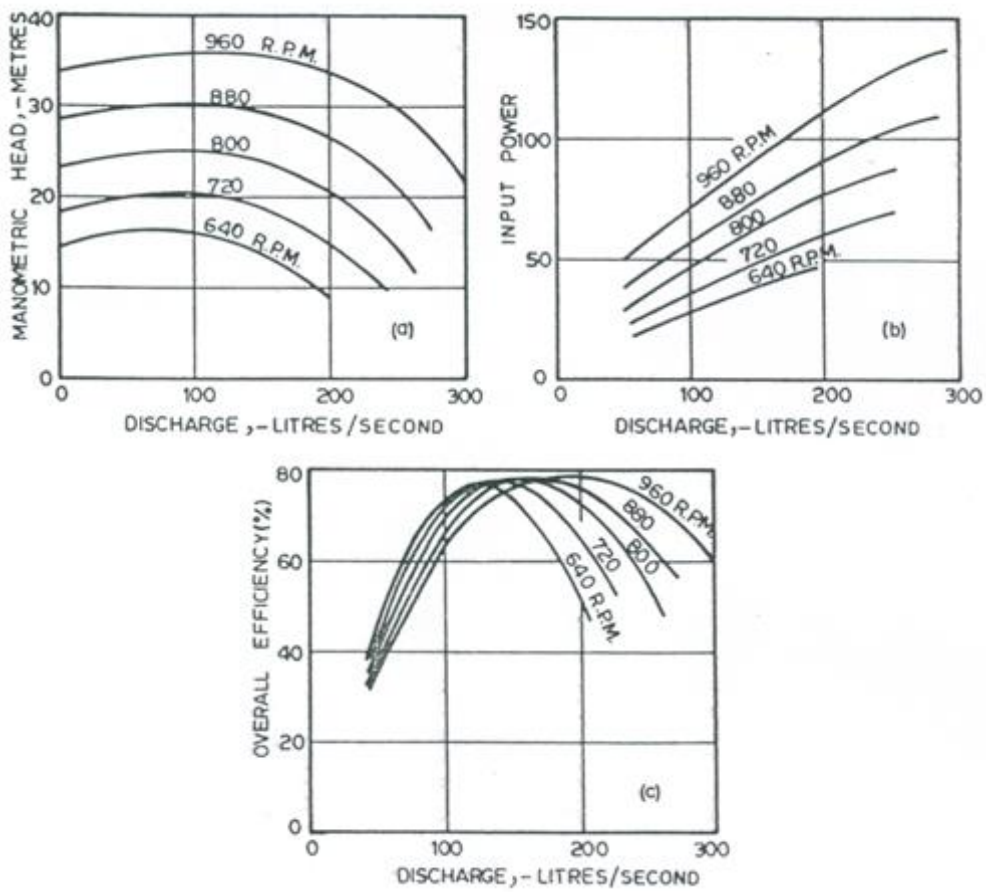

Fig. 28.1. Main characteristics curves of a centrifugal pump.

(Source: Modi and Seth, 1998)

28.3.2 Operating Characteristic Curves

During operation of a pump, the pump must run constantly with the speed of the prime mover; this constant speed is usually the design speed. The set of main characteristics curves which corresponds to the design speed is mostly used in pump operation, and hence such curves are known as the operating characteristics curves. A typical set of such characteristics of a centrifugal pump is shown in Fig. 28.2, which consists of four curves at a constant speed viz., head versus discharge (H_m vs Q) curve, efficiency versus discharge (h_o vs Q) curve, power versus discharge (BP or SP vs Q) curve, and net positive suction head required versus discharge (NPSHR vs Q) curve. From these characteristic curves, it is possible to determine whether the pump will handle the necessary quantity of liquid against the desired head and what will happen if the head is increased or decreased. In addition, these characteristic curves illustrate what size motor will be required to operate the pump at the required conditions and whether or not the motor will be overloaded under any other operating conditions. A brief description of these curves is provided below.

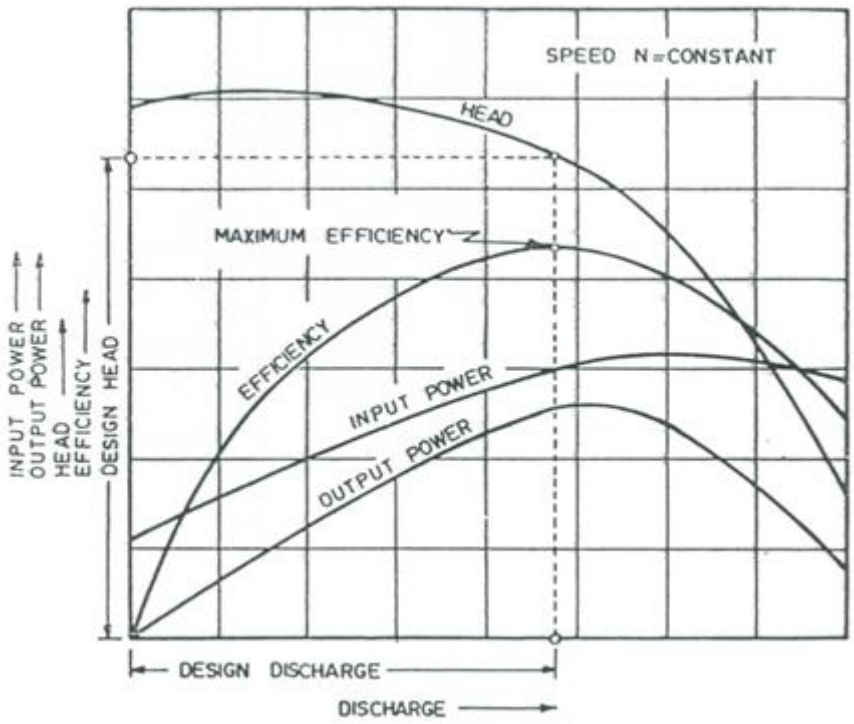

Fig. 28.2. Operating characteristic curves of a centrifugal pump.

(Source: Modi and Seth, 1998)

28.3.2.1 Head versus Discharge Curve

The head-discharge curve (H-Q curve) relates the head produced by a pump to the volume of water pumped per unit time (i.e., discharge of the pump). Generally, the head produced by a pump steadily decreases as the discharge of the pump increases (Fig. 28.2). The values of the head and the discharge corresponding to the maximum efficiency are known as the design head or normal head and the design discharge or normal discharge of a pump.

The shape of the H-Q curve varies with the specific speed. Typical H-Q curves for various specific speeds and impeller designs are shown in Fig. 28.3. For radial flow impellers, head decreases only slightly and then drops rapidly as discharge (Q) increases from zero. Slope changes along H-Q curves for the mixed and axial flow impellers are not as dramatic as those for radial flow impellers. Radial flow impellers operating on the flat portion of their H-Q curves work well in situations where head must remain essentially constant as Q fluctuates (e.g., as in set-move systems where the number of operating laterals varies during the irrigation season). In situations where a relatively constant Q is desired and H is expected to fluctuate (e.g., water sources like a well, small stream, or small reservoir), the impellers with higher specific speeds will probably perform the best.

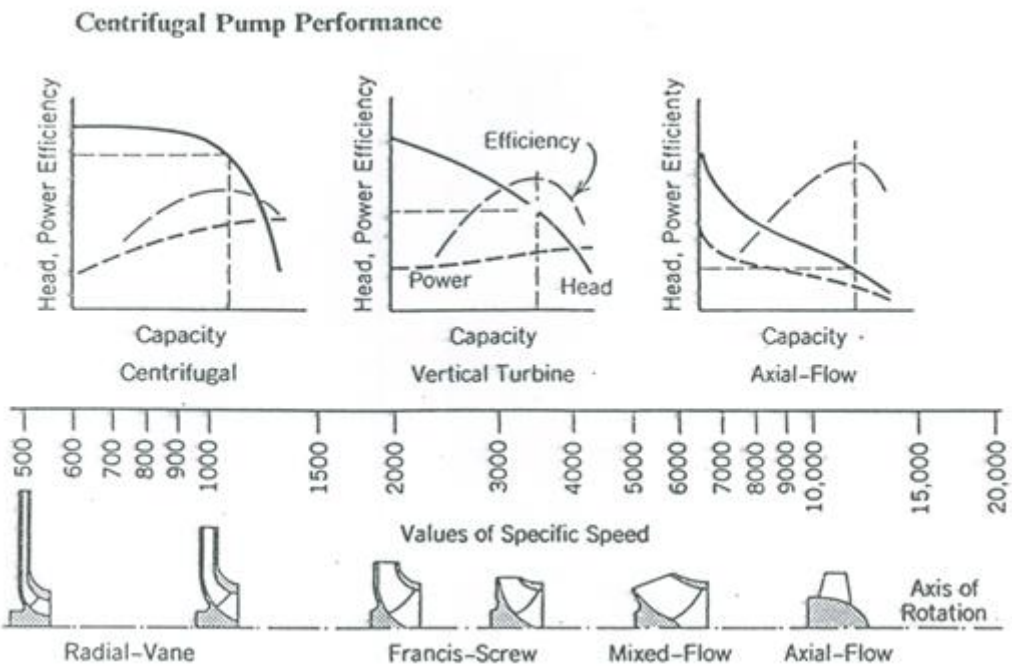

Fig. 28.3. Head-discharge curves as a function of specific speed and impeller design. (Source: James, 1988)

Moreover, the head generated by the pump when discharge is zero (i.e., when the pump is operating against a closed valve) is called shutoff head. For pumps with steadily declining H-Q curves, the shutoff head is the maximum head and must be known to design piping on the discharge-side of the pump. In such situations, discharge-side piping must be able to withstand the shutoff head when the discharge valve is closed. Note that the efficiency of a pump is zero at the shutoff head, since energy is used to turn the pump.

28.3.2.2 Efficiency versus Discharge Curve

The pump efficiency versus discharge (h_o-Q) curves for typical centrifugal pumps are illustrated in Figs. 28.2 and 28.3. The overall efficiency (h_o) for a pump gradually increases to a peak as Q increases from zero, and then it declines with further increase in Q. There is normally only one-peak efficiency for a given type of impeller.

The theoretical pump efficiency is a function of specific speed, impeller design, and pump discharge as shown in Fig. 28.4. This figure indicates that the larger capacity pumps can be expected to have the highest efficiency. The overall efficiency (h_o) is also related to the types of materials used in construction, the finish on castings, the quality of machining, and the type and quality of bearings used. For example, impellers with extremely smooth surfaces tend to be more efficient than rougher surfaced impellers.

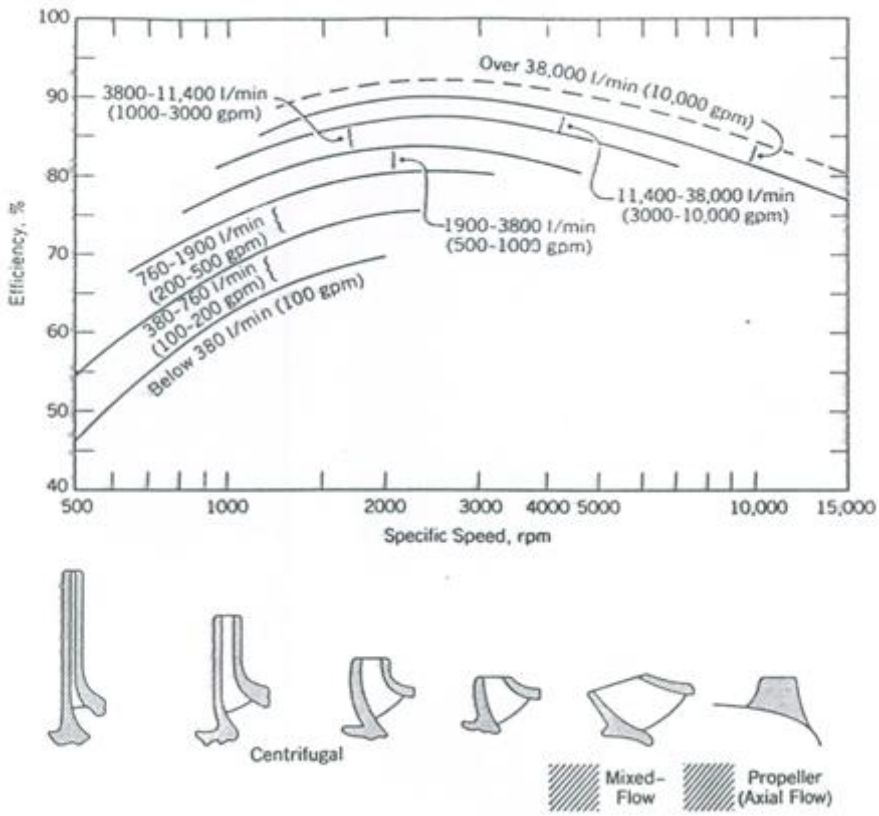

Fig. 28.4. Variation of theoretical pump efficiencies with specific speed, impeller design and discharge. (Source: James, 1988)

Furthermore, it should be noted that the efficiency versus discharge curve is usually for a specific number of stages. If a different number of stages are needed for a particular situation, efficiencies must be adjusted upward or downward depending on the number of stages. Manufacturers usually provide information for making these adjustments.

28.3.2.3 Power versus Discharge Curve

The brake power (BP) or shaft power (SP) versus discharge/capacity (Q) curve for a pump is derived from its H-Q and h_o-Q curves. The shape of the BP-Q curve depends on the pump's specific speed and impeller design. Figure 28.3 shows that for radial flow impellers, BP generally increases from a nonzero value to a peak and then decreases slightly as Q increases from zero. For mixed flow impellers, BP increases gradually from a nonzero value as Q increases. However, for axial flow impellers, BP is maximum when Q is zero and it steadily decreases as Q increases from zero. Thus, when axial flow pumps are started, the discharge-side valve (or discharge valve) should be open to the atmosphere in order to minimize the start-up load. In contrast, the discharge-side valve should be closed when radial flow and mixed flow pumps are started.

28.3.2.4 NPSHR versus Discharge Curve

The fourth characteristic curve typically published by pump manufactures is the net positive suction head required (NPSHR) versus pump discharge (Q) curve as shown in Fig. 28.5. It is evident from this figure that for a typical radial flow pump, NPSHR gradually increases as the pump discharge increases.

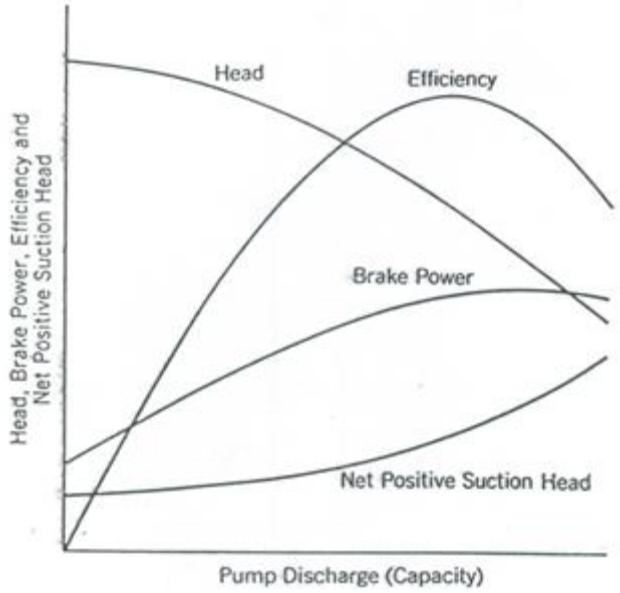

Fig. 28.5. NPSHR versus discharge curve. (Source: James, 1988)

28.3.3 Constant Efficiency Curves

The constant efficiency curves or Muschel curves (Fig. 28.6) help determine the range of pump operation for a particular efficiency. As shown in Fig. 28.6, the constant or iso-efficiency curves may be obtained from H_m vs Q and h_o vs Q curves of main characteristic curves. In order to plot the iso-efficiency curves, horizontal lines representing constant efficiencies are drawn on the h_o vs Q curves. The points at which these lines cut the efficiency curves at various speeds are transferred to the corresponding H_m vs Q curves. The points corresponding to the same efficiency are then joined by smooth curves, which represent the iso-efficiency curves or Muschel curves. From these curves, the line of maximum efficiency can be obtained (Fig. 28.6).

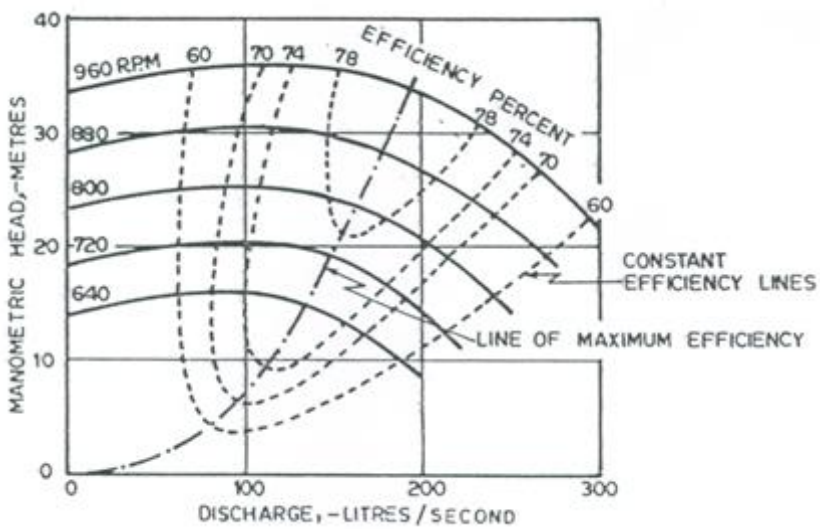

Fig. 28.6. Constant efficiency curves of a centrifugal pump.

(Source: Modi and Seth, 1998)

Thus, the Muschel curves facilitate the job of a salesman and enable the prospective customer to see directly the range of pump operation for a given efficiency. These curves further serve as a suitable basis for the comparison of pumps, especially from a commercial viewpoint.

28.3.4 Constant Head and Constant Discharge Curves

It is quite possible that a pump may be required to deliver water at a certain height, wherein head (H) is fixed. If for some reason, the pump speed (N) varies, the discharge of the pump will also be affected. In order to predetermine the performance of the pump under such conditions, it is necessary to draw a constant head curve by plotting Q versus N (Fig. 28.7). The constant head curve can be used to determine the speeds required to discharge varying amounts of water at a constant pressure head. Similarly, to determine the speeds required to discharge a certain quantity of water at different heads or to find variation of head with N, it is convenient to draw constant discharge curves by plotting H versus N (Fig. 28.7).

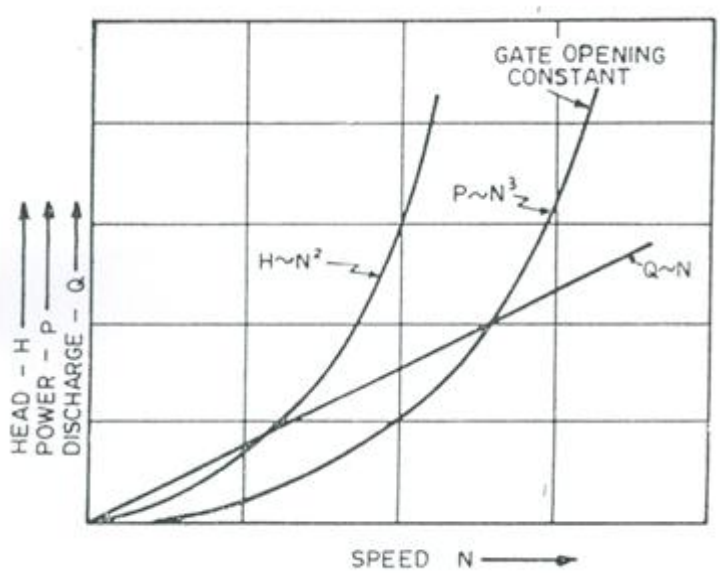

Fig. 28.7. Q versus N, H_m versus N, and P versus N curves of a centrifugal pump. (Source: Modi and Seth, 1998)

The constant head curves and the constant discharge curves are also useful for determining the performance of a variable speed pump having constantly varying speed.

Lesson 29 Rotodynamic Pumps for Special Purposes

In this lesson, rotodynamic pumps which are used for special purposes are discussed. Such pumps are: propeller pumps, mixed-flow pumps, submersible pumps, vertical turbine pumps, and jet pumps. In addition, self-priming devices for rotodynamic pumps are succinctly discussed.

29.1 Propeller Pump

An axial flow pump or propeller pump propels water by the reaction to lift forces produced by rotating its blades. This action both pushes the water past the rotor or impeller and also imparts a spin to the water which if left uncorrected would represent wasted energy, since it will increase the friction and turbulence without helping the flow of water down the pipe. Axial flow pumps therefore usually have fixed guide vanes, which are angled so as to straighten the flow and convert the spin component of velocity into extra pressure, in much the same way as with a diffuser in a centrifugal pump.

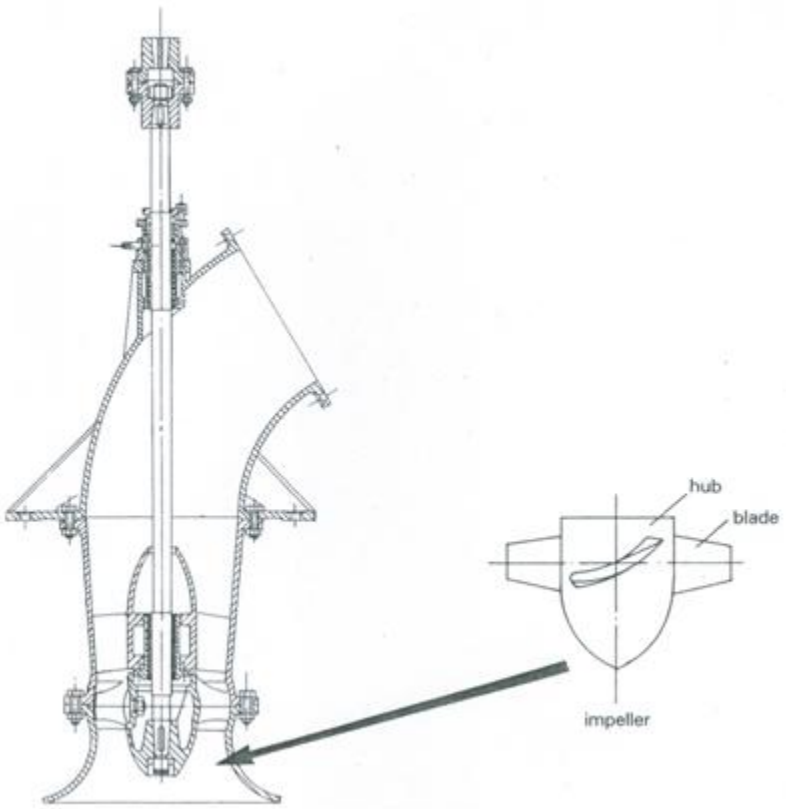

Fig. 29.1. Axial flow or propeller pump. (Source: FAO, 1986)

Figure 29.1 shows a typical axial flow pump of this kind, in which the guide vanes, just above the impeller, also serve a second structural purpose of housing a large plain bearing, which positions the shaft centrally (FAO, 1986). This bearing is usually water lubricated and has features in common with the stern gear of an inboard-engined motor boat.

Axial flow pumps are generally manufactured to handle flows ranging from 150 to 1500 m^3/h for vertically mounted applications, usually with heads in the range of 1.5 to 3.0 m. By adding additional stages (i.e., two or more impellers on the same shaft) extra lift up to 10 m or so can be obtained (FAO, 1986).

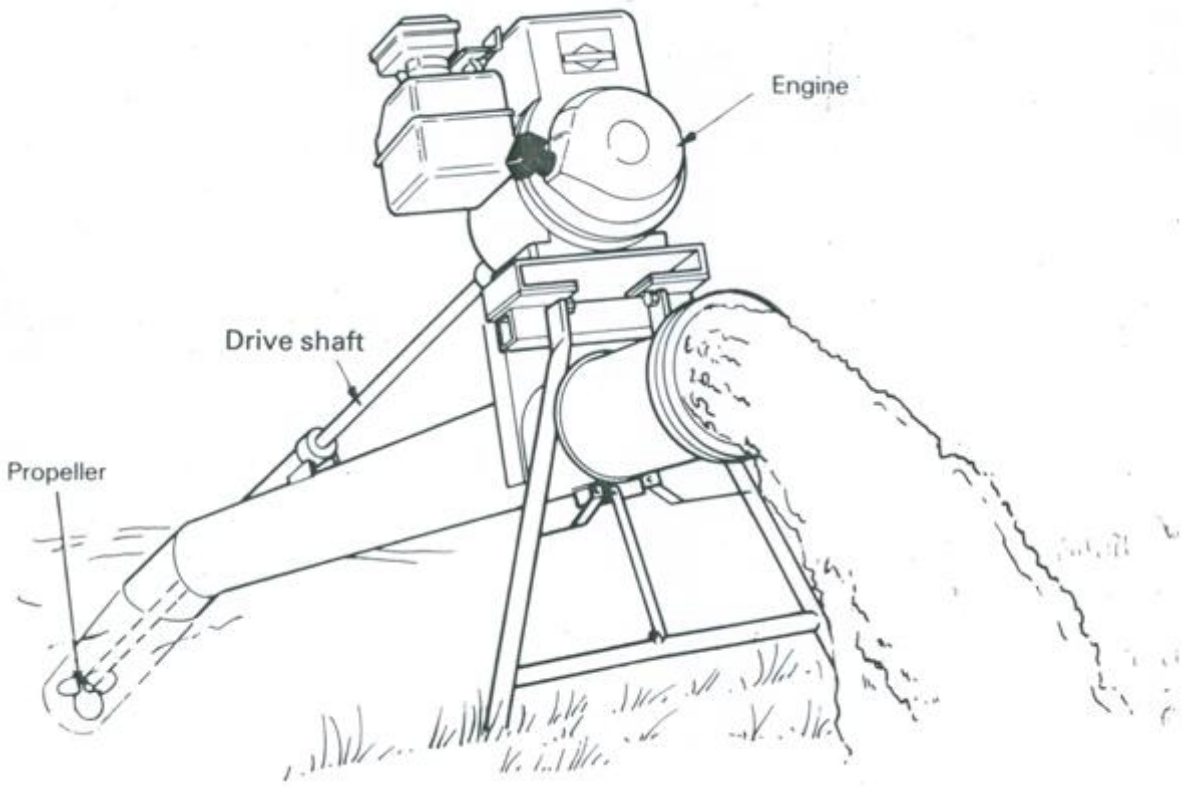

Fig. 29.2. Portable axial flow pump (IRRI model). (Source: FAO, 1986)

Since axial flow pumps are designed for very large flows at low heads, normally concrete pipes are used to avoid the high cost of large-diameter steel pipes. They are generally used in canal irrigation schemes where large volumes of water are to be lifted 2-3 m, typically from a main canal to a feeder canal.

Small-scale propeller pumps are usually locally manufactured such as the portable axial flow pump (Fig. 29.2) developed by the International Rice Research Institute (IRRI), Philippines. It is designed to deliver up to 180 m^3/h at heads in the range of 1 to 4 m. This pump requires a 5 hp diesel engine or electric motor capable of driving its shaft at 3000 rpm; its length is 3.7 m, the discharge tube is 150 mm in diameter and the overall weight without the prime mover is 45 kg (FAO, 1986).

29.2 Mixed-Flow Pump

The mixed-flow pump, as its name suggests, involves something of both axial and centrifugal pumps. In the irrigation context, it can often represent a useful compromise to

avoid the limited lift of an axial flow pump, but still achieve higher efficiency and larger flow rates than an ordinary centrifugal pump. Also, axial flow pumps generally cannot sustain any suction lift, but mixed-flow pumps can, although they are not self-priming. Figure 29.3 shows a surface mounted suction mixed-flow pump and its installation. Here, the swirl imparted by the rotation of the impeller is recovered by delivering the water into a snail-shell volute or diffuser (identical in principle to that of a centrifugal volute pump).

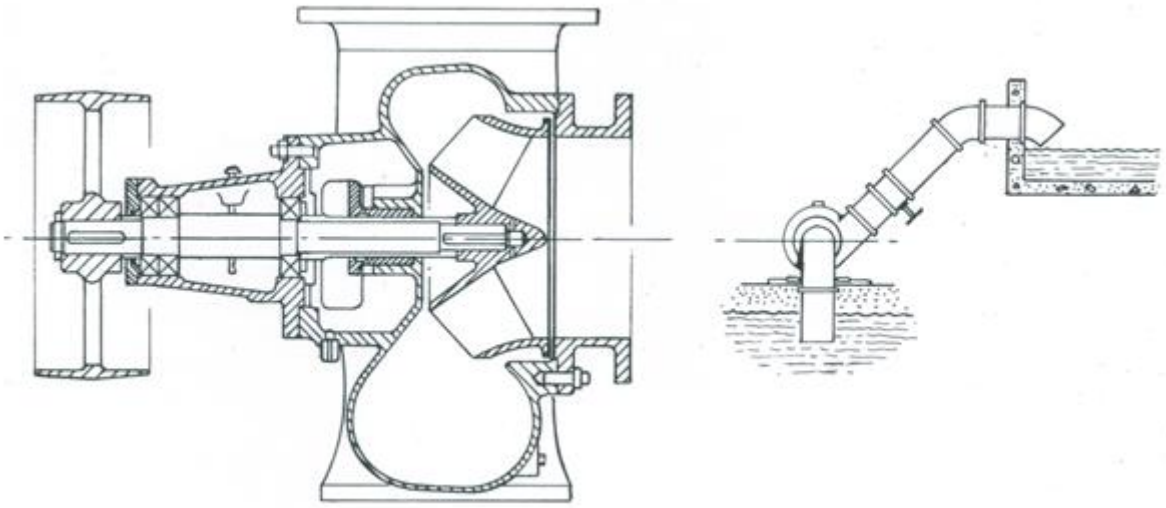

Fig. 29.3. Surface mounted mixed-flow pump. (Source: FAO, 1986)

An alternative arrangement somewhat similar to an axial flow pump is shown in Fig. 29.4, which is known as a submersible mixed-flow pump. In this case, a 'bowl' casing is used so that the flow spreads radially through the impeller and then converges axially through fixed guide vanes which remove the swirl and thereby add to the efficiency. Pumps of this kind are installed submerged, which avoids the priming problems that can afflict large surface suction rotodynamic pumps such as in Fig. 29.3. The 'bowl mixed-flow pump' is sometimes called a turbine pump, and it is actually equivalent to the centrifugal turbine pump described later in this lesson. In the 'bowl mixed-flow pump', the passage through the rotor reduces in cross-section and thereby accelerates water and imparts energy to it, while the fixed guide vanes are designed as a diffuser to convert kinetic energy into pressure energy and thereby increase both pumping head and efficiency. Two or more 'bowl mixed-flow pumps' can also be stacked on the same shaft to make a multi-stage turbine pump; such pumps are commonly used as borehole pumps because of their long narrow configuration. Bowl mixed-flow pumps typically operate with discharges from 200 to 12000 m^3/h over heads from 2 to 10 m. Their multiple stage versions are often used at heads of up to about 40 m (FAO, 1986).

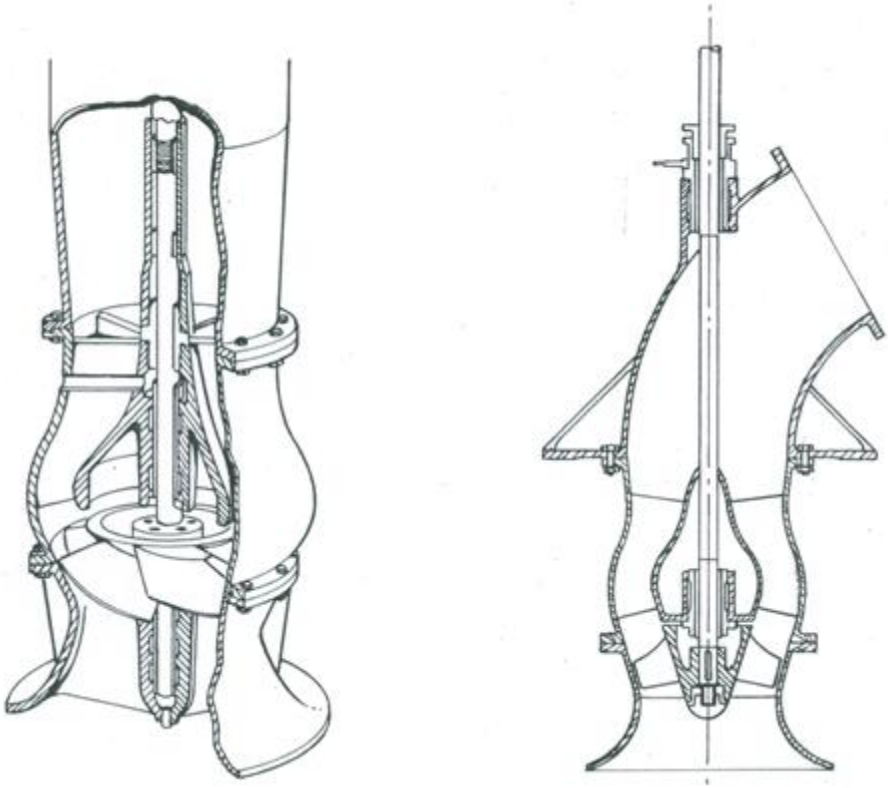

Fig. 29.4. Submerged mixed-flow pump. (Source: FAO, 1986)

29.3 Submersible Pump

Where high heads are needed, the primary means to achieve this with a single impeller centrifugal pump are either to increase the impeller speed or to increase impeller diameter. However, there are practical limits to this way of increasing head. Therefore, in practice either single impeller pumps are connected in series or a more practical solution is to use a specially designed pump (commonly known as submersible pump) wherein multiple (two or more) impellers mounted on the same shaft such that the output from one impeller feeds directly through suitable passages in the casing to the next impeller. Submersible pumps have the motor and the bowl assembly as a unit submerged below the lowest pumping water level (Fig. 29.5). A water-proof cable supplies power to the motor. Submersible pumps to fit inside 10, 15, 20 and 25 cm borewells are available in India. They can be used for discharges varying from 40 to 3000 L/min and heads varying from 15 to 150 m (Raghunath, 2007). They can be installed in crooked wells but the repair to motor or pump requires its removal from the well, and hence the motor or pump is subject to abrasion by sand.

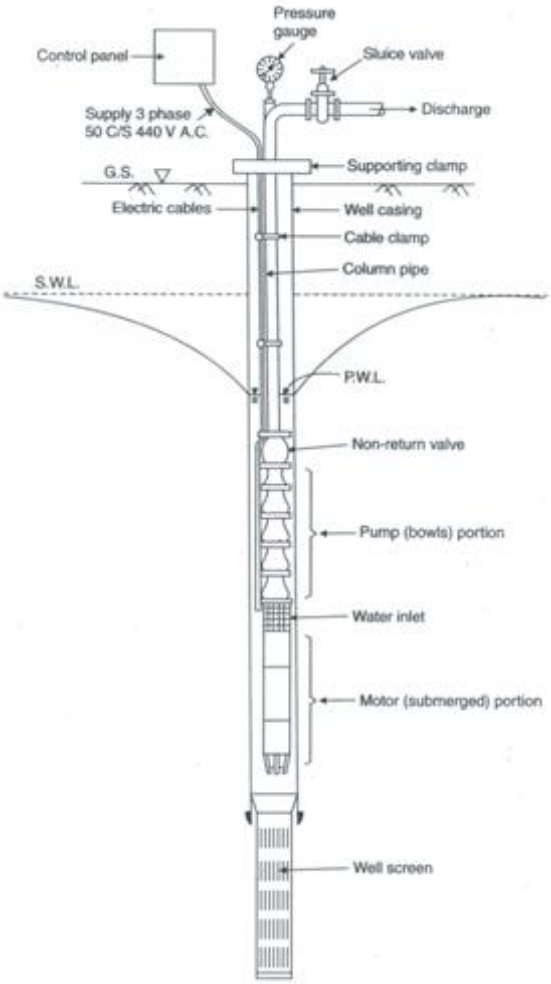

Fig. 29.5. Schematic diagram of a submersible pump.

(Source: Raghunath, 2007)

Submersible pumps have the advantage that they can be installed in the localities where there is little or no floor space to install a pump unit as well as in the localities where noise or theft is a serious issue. They can be either water or oil lubricated. The new types of voltage regulated starters have solved the problem of overloading. Although the initial costs of submersible pumps are lower than those of vertical turbine pumps, their repair and maintenance costs are higher.

29.4 Vertical Turbine Pump

Vertical turbine pumps are most widely used for large deep tubewells. The bowl-assembly (containing impellers) is placed below the lowest pumping water level, but the prime mover (electric motor or diesel engine) is placed on the ground surface and is connected by a long shaft (Fig. 29.6). Usually deep-well turbine pumps are used for fairly high flows under high heads. The overall efficiency of vertical turbine pumps ranges from 50 to 80% (Raghunath, 2007).

Vertical turbine pumps have the advantages of high efficiency, high head pumping capability and excellent serviceability. Their impellers can be of semi-open or fully enclosed types. However, they have some serious disadvantages. They require sufficiently straight and plumb well for installation and proper operation and are subject to abrasion by sand. The maintenance problem becomes severe when they are pumping corrosive water unless the pump, column pipe, line shaft and other components are made of non-corrosive materials. Also, the lubrication and vertical alignment of line shaft is critical.

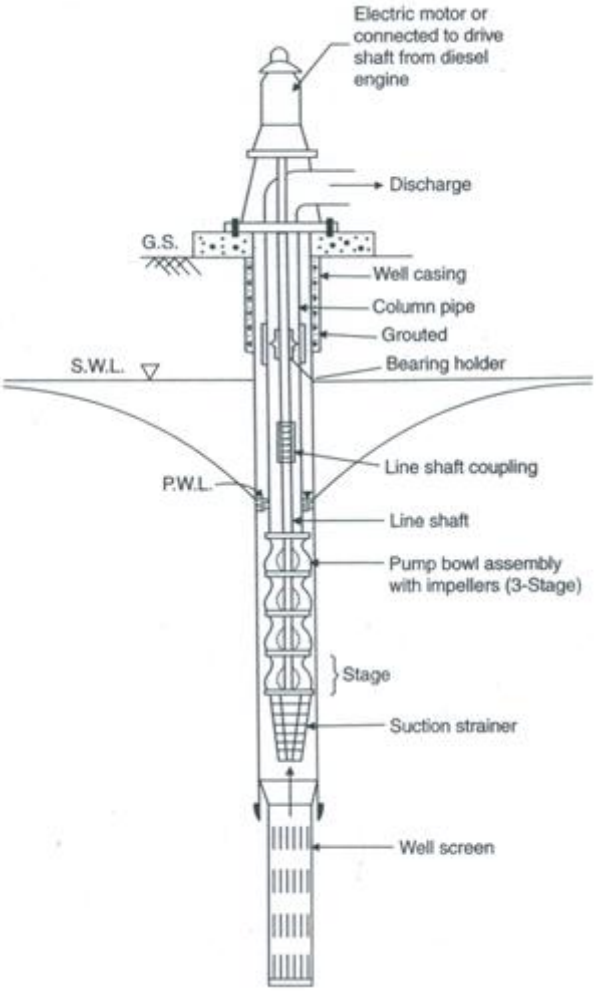

Fig. 29.6. Schematic diagram of a vertical turbine pump.

(Source: Raghunath, 2007)

29.5 Jet Pump

A jet pump consists of a centrifugal pump and a jet assembly (Fig. 29.7) and it is essentially a self-priming centrifugal pump which is based on the fact that if water is accelerated through a jet, it causes a drop in pressure. In the jet pump, the pump is fitted into a secondary casing which contains water at the discharge pressure. A proportion of the water from this chamber is bled back to a nozzle fitted into the suction end of the pump casing and directed into the eye of the impeller (Fig. 29.7). If the pump has been used once (having been manually

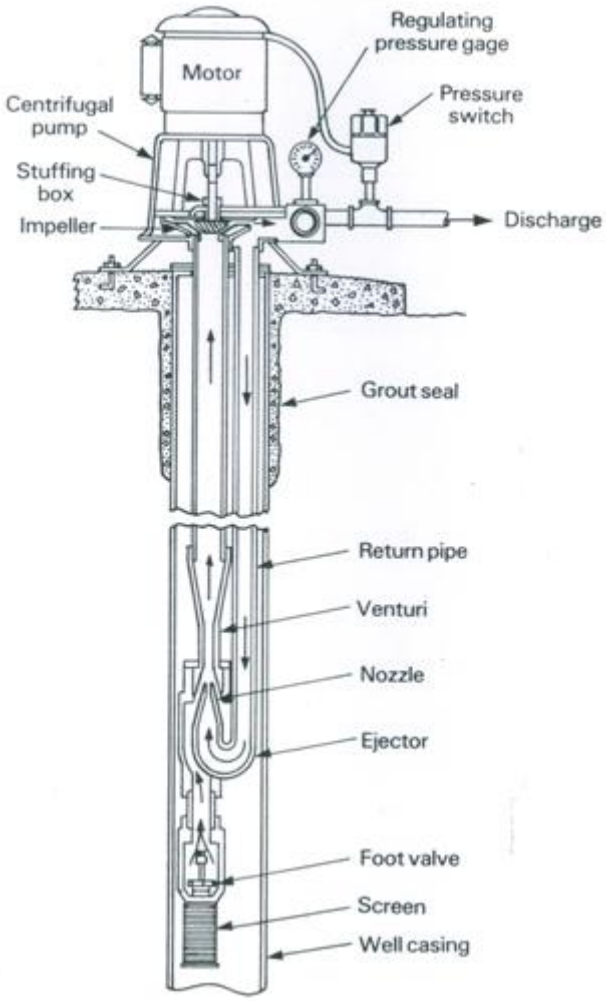

Fig. 29.7. Schematic diagram of a borehole jet pump. (Source: FAO, 1986)

primed initially), it remains full of water so that on start up the water is re-circulated from the delivery side of the pump to the bottom of the suction pipe and is injected through a nozzle to impart additional kinetic energy. Thus, the jet causes low pressure in the suction line and thereby increases suction considerably compared to the effect of impeller on its own, which in turn provides additional suction lift. Initially, the entrapped air is gradually drawn up the suction line. As soon as all the air is removed from the system, most of the discharge goes up the discharge line and a small proportion is fed back into the nozzle. In this way, the jet pump not only creates a higher suction lift than normal but also it can reliably run on 'snore' (i.e., sucking a mixture of air and water without losing its prime). This makes it useful in situations where shallow water is to be pumped under suction and it is difficult to obtain sufficient submergence of the foot valve, or where a water source may occasionally be pumped dry. The provision of a jet assembly allows a surface-mounted pump and motor to 'suck' water from depths of about 10 to 20 m; the diffuser after the jet raises the pressure in the rising main and avoids cavitation (FAO, 1986).

Jet pumps are often viable for pumping fairly small discharges (40-90 L/min) under low heads (15 to 45 m) when the water level is more than 7.6 m from the ground surface

(Raghunath, 2007). Their capacity reduces as the lift increases. They are generally used for residential buildings and hotels. Jet pumps are of two types: twin type for the borewells of 15 cm diameter and larger, and packer type (duplex) for the borewells of less than 15 cm diameter (Raghunath, 2007). Although the jet circuit usually needs 1.5 to 2 times the flow being delivered (discharge), and therefore is a source of significant power loss, the jet pumps are sometimes useful for lifting sandy or muddy water as they are not so easily clogged as an ordinary submerged pump. In such cases, however, a settling tank is provided on the ground surface between the pump suction and the jet pump discharge to allow the pump to draw clear water.

Moreover, jet pumps have two main disadvantages (FAO, 1986): (i) greater complexity and hence higher cost, and (ii) reduced efficiency because power is used in pumping water through the jet, though some of this power is recovered by the pumping effect of the jet. Considering these disadvantages of jet pumps, it is better to use a conventional centrifugal pump in the situations where there is little or no suction lift. However, in the situations where suction pumping is essential, a jet pump can offer a successful solution.

29.6 Self-priming Devices for Rotodynamic Pumps

Rotodynamic pumps will start pumping only if their impellers are flooded with water prior to start up. Obviously, the easiest way is to submerge the pump in the water source, but this is not always possible or convenient (e.g., portable pump sets). If sufficient water is present in the pump casing, then even if the suction pipe is empty, suction will be created and water can be lifted. A variety of methods are used to fill rotodynamic pumps with water when they are placed above the water level. It is important, however, to note that if the suction line is empty but the delivery line is full, it is necessary to drain the delivery line so as to remove the back pressure on the pump and enable it to be primed. Otherwise, it will be difficult, if not impossible, to flush out the air in the system.

The most basic method of priming is to use a foot valve to retain water in the system. In this case, the system has to be filled initially by pouring water into the pipes using a bucket and then it is hoped that the foot valve will retain water in the system even after the pump is not used for some time. In many cases, however, foot valves leak, especially if mud or grit is present in the water and settles between the valve and its seat when it attempts to close (FAO, 1986).

The two most common methods for priming surface-mounted and engine-driven suction centrifugal pumps are either by using an ordinary hand pump or a diaphragm pump on the delivery line. For example, Fig. 29.8 illustrates a hand-operated diaphragm pump fitted on the delivery side of a portable centrifugal pump for priming.

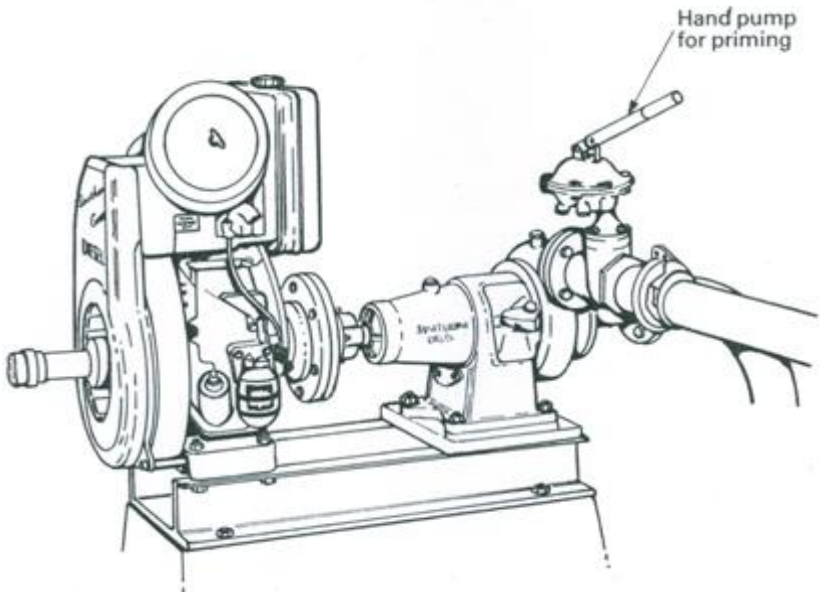

Fig. 29.8. Direct-coupled air-cooled diesel engine and centrifugal pump with a diaphragm pump for priming. (Source: FAO, 1986)

Several alternative methods of priming surface suction pumps can be designed using a large water container/tank with appropriate accessories to suitably fill the pump and suction line with water and maintain water in the tank for the next start. Yet another simple method to use, but only if the delivery line is long enough to carry a sufficient supply of water, is to fit a hand valve immediately after the pump discharge (instead of a non-return valve) so that when the pump is turned off, the valve can be manually closed. Then, the opening of this valve will refill the pump from the delivery line to ensure that it is flooded on restarting. Besides these simple and cost-effective self-priming devices, sometimes the most reliable arrangement is to use a special 'self-priming centrifugal pump' which has an enlarged upper casing with a baffle in it (FAO, 1986). Additionally, the jet pump (discussed in the previous section) is a special type of self-priming centrifugal pump.

Lesson 30 Affinity Laws

30.1 Introduction

In this lesson, affinity laws and their utility are discussed. As we know that the total head, discharge and power of a centrifugal pump are related to the size and speed of its impeller. Changing the size or speed of an impeller significantly affects the operational characteristics of a centrifugal pump. Therefore, such knowledge allows the pump manufacturers and/or users to modify the performance of a single pump so as to match the system need or understand the pump performance under different operating conditions.

The mathematical relationships of total head, discharge and power with the pump speed and with the impeller diameter are known as affinity laws. Thus, two types of affinity laws each consisting of a set of three equations exist, which are discussed in subsequent sections.

30.2 Effect of Changes in Pump Speed on the Pump Performance

Most pump manufacturers have standardized the speed for which they publish characteristic curves. Belt driven electric pumps or engine-driven pumps can operate at a range of speeds, which can be varied by suitable adjustments. Variation in the pump speed can be used advantageously to operate a single pump to match two different system head curves or to provide two different design discharges. Reduction in the pump speed is also one of the best ways to rectify surging pumps (Jensen, 1980). However, the selection of a proper pump which can operate efficiently at two different speeds enhances the complexity of pump selection several fold.

Total dynamic head (or total head), discharge and power of a pump are related to the speed of an impeller (i.e., pump speed). These relations can be developed by using fundamental equations of pumps. As we know, the peripheral velocity of the impeller (u) is given as:

$$u = \frac{\pi DN}{60} \qquad (30.1)$$

$$\therefore \quad u \propto N$$

Where, D = diameter of the impeller, and N = speed of the impeller or pump.

From the velocity triangles, it becomes evident that if u changes to u′ (vane angles remaining constant), then the flow velocity (V) is related as:

$$V \propto u$$

$$\therefore \quad V \propto N$$

Now, the pump discharge Q = Area across flow × w, which suggests that:

$$Q \propto V \text{ (because } V = \omega \times r)$$

$$\therefore \quad Q \propto N$$

That is, the pump discharge varies linearly with a change in the pump speed.

Moreover, from the fundamental equation of pumps, it is clear that the total head (H) depends on the squares of velocities, which are directly proportional to the pump speed (N). That is, $H \propto N^2$. In addition, the power required for a pump (P) is directly proportional to Q × H. $P \alpha N^3$ Therefore,.

Thus, if N_1 and N_2 are the two pump speeds, and Q_1, Q_2; H_1, H_2 and P_1, P_2 are respectively discharges, heads and powers corresponding to the two pump speeds N_1 and N_2, we obtain three equations relating the discharge, head and power with the pump speed as follows:

$$\frac{Q_1}{Q_2} = \frac{N_1}{N_2} \qquad (30.2)$$

$$\frac{H_1}{H_2} = \left(\frac{N_1}{N_2}\right)^2 \qquad (30.3)$$

and

$$\frac{P_1}{P_2} = \left(\frac{N_1}{N_2}\right)^3 \qquad (30.4)$$

The above three equations are known as Affinity Law I, which suggests that the discharge varies linearly with a change in the speed, the head varies as the square of the ratio of the two speeds, and the power varies as the cube of the ratio of the two speeds. Thus, a slight increase in the pump speed will deliver more water at a higher head, but will require considerably more power to drive the pump.

30.3 Effect of Changes in Impeller Diameter on the Pump Performance

As mentioned earlier, the discharge of a pump can be changed by modifying either its speed or the diameter of its impeller which changes the pump's performance. The former is normally not possible because the speed of the driving motor is fixed. Therefore, the outer diameter of the impeller (D) has to be enlarged or reduced according as H or Q is to be increased or decreased. We can reduce the outer diameter of the impeller by trimming. By fixing rings to the shroud of an impeller, we can increase the outer diameter and extend the impeller blades to the required size, though it is not a usual practice. The effect of the alteration of D is two fold. Firstly, the alteration of D changes peripheral velocity (u) without changing N. Therefore, from Eqn. (30.1) we have:

$$u \propto D$$

$$\therefore \quad V \propto D$$

Secondly, the alteration of D changes discharge (Q). Since Q = A α V, because the area across flow (A) will remain constant. Further, head (H) depends on u^2, V^2, etc. and hence, V^2 α D^2, thereby we have:

H^2 α D^2

In addition, the power required for a pump (P) is directly proportional to Q × H. Therefore,P α D^3 .

Thus, similar to the impact of change in pump speed on pump performance, we obtain three equations which relate the impact of change in impeller diameter to changes in pump performance. If D_1 and D_2 are the two impeller diameters, and Q_1, Q_2; H_1, H_2 and P_1, P_2 are respectively discharges, heads and powers corresponding to the impeller diameters D_1 and D_2, the three equations relating the discharge, head and power with the impeller diameter are as follows:

$$\frac{Q_1}{Q_2} = \frac{D_1}{D_2} \qquad (30.5)$$

$$\frac{H_1}{H_2} = \left(\frac{D_1}{D_2}\right)^2 \qquad (30.6)$$

and

$$\frac{P_1}{P_2} = \left(\frac{D_1}{D_2}\right)^3 \qquad (30.7)$$

The above three equations are known as Affinity Law II, which suggests that the discharge varies linearly with a change in the impeller diameter, the head varies as the square of the ratio of the two impeller diameters, and the power varies as the cube of the ratio of the two impeller diameters.

Note that the applicability of the Affinity Law II is limited to less than 20% changes in the original outer diameter of the impeller (James, 1988).

On the whole, it is worth mentioning that all of the above relationships [Eqns. (30.2) to (30.7)] are approximate because leakage, windage, and bearing losses have been neglected. Nevertheless, they are very useful for the pump manufacturers because the main characteristics of a new pump (i.e., modified pump) can be determined without testing it in the laboratory.

30.4 Illustrative Example

Problem: A centrifugal pump requires 5 kW power when it runs at 1450 rpm and delivers water against a head of 10 m. If the pump is operated at 1750 rpm, calculate the head developed and the power required by the pump.

Solution: From the question, N_1 = 1450 rpm, N_2 = 1750 rpm, H_1 = 10 m, and P_1 = 5 kW. The head developed by the pump (H_2) and the power required (P_2) at 1750 rpm can be calculated by using the Affinity Law I.

From the Affinity Law I, we have:

$$\frac{H_1}{H_2} = \left(\frac{N_1}{N_2}\right)^2$$

$$\therefore \quad H_2 = H_1 \times \left(\frac{N_2}{N_1}\right)^2$$

$$= 10 \times \left(\frac{1750}{1450}\right)^2$$

= 14.57 m, Ans.

Also, from the Affinity Law I we have:

$$\frac{P_1}{P_2} = \left(\frac{N_1}{N_2}\right)^3$$

$$\therefore \quad P_2 = P_1 \times \left(\frac{N_2}{N_1}\right)^3$$

$$= 5 \times \left(\frac{1750}{1450}\right)^3$$

= 8.79 kW, Ans.

Lesson 31 Selection of Suitable Pumps

31.1 Introduction

A variety of pumps (centrifugal and other types) and their different models are manufactured by a large number of manufacturers in almost every country. These pumps are available in a wide range of sizes and have widely varying characteristics. Therefore, a proper selection of pump for a given purpose is necessary. A properly selected pump not only effectively meet the desired water demand but also operates efficiently, thereby enhancing its service life and minimizing OMR (operation, maintenance and replacement) costs. The information required for selecting a suitable pump is: (i) total head of the pumping system (system head curve), (ii) design discharge, and (iii) pump characteristic curves. The system head curve and design discharge are discussed in the subsequent sections, and the pump characteristic curves are discussed in Lesson 28.

In this lesson, although general criteria for pump selection are described, the focus is on the selection of centrifugal pumps. The selection and operating characteristics of other types of pumps (e.g., vertical turbine pump, submersible pump, propeller pump, mixed-flow pump, and jet pump) are discussed in Michael and Khepar (1999).

31.2 System Head Curve

System head curve (Fig. 31.1) is a relationship between total head and discharge for a given pumping system and it illustrates how the total head varies with an increase in discharge. It essentially describes the head-discharge (H-Q) requirements of a pumping system. It indicates that the more head is required to increase flow (discharge) through the system.

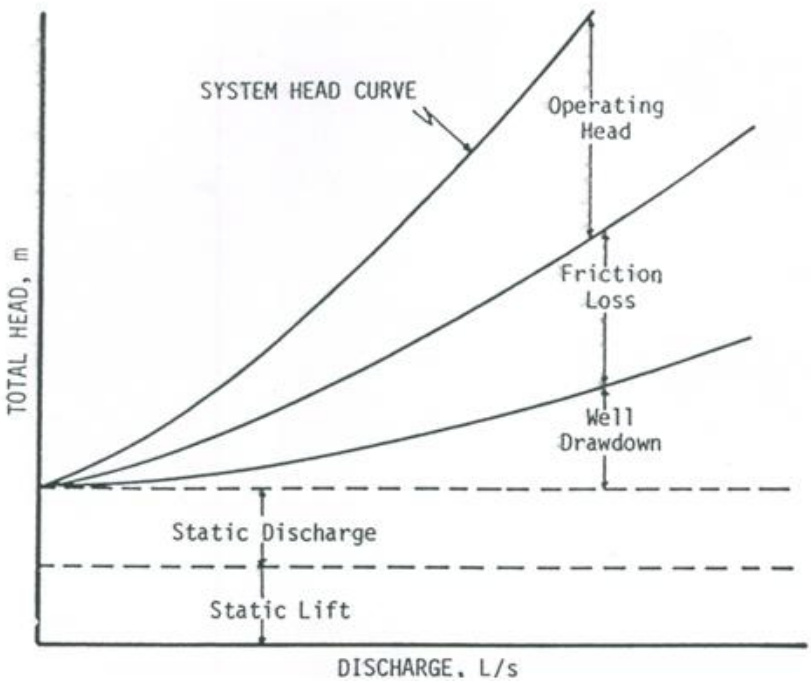

Fig. 31.1. A typical 'system head curve' with its different components. (Source: Jensen, 1980)

System head curves are developed by computing the total head required by a pumping system to deliver different discharges. The following equation, which shows different head components contributing to the system head curve, is used to calculate system head (SH) or total head for a pumping system:

$$SH = h_s + h_d + h_f + s_w + h_o + \frac{v^2}{2g} \qquad (31.1)$$

Where, h_s = static suction head, h_d = static delivery head, h_f = friction head losses (major and minor), s_w = drawdown in the sump (water source) during pumping, h_o = operating head, and $\frac{v^2}{2g}$ = velocity head.

For some systems, not all of the above-mentioned head components would be applicable. In the above equation, the static suction head or static lift (h_s) and static delivery head or static discharge head (h_d) are independent of the system flow (Q), whereas the remaining head components increase as the discharge increases (Fig. 31.1). The system head (SH) is independent of the pump, with the exception of friction losses occurring in the column pipe a vertical turbine pump (Jensen, 1980). Note that the system head curve is time dependent because of temporal variations in the well drawdown, friction, operating conditions, and the static water level of the water source. Also, the static delivery head (h_d) may be time dependent. The system head curve is useful for determining the operating point of a pump to be installed; it is discussed in Section 31.4.

31.3 Determination of Design Discharge

The discharge of a pump that meets the peak demand of water in a domestic, irrigation or industrial sector is known as design discharge of the pump. If the source of water is groundwater, the safe yield of a well becomes the dominating factor in deciding the design discharge. In this case, if the maximum water requirement per day during peak periods is less than the safe yield of the well, the peak water demand will be the design discharge. Conversely, if the peak water demand is more than the safe yield of the well, the water requirement during peak periods should be adjusted to be equal to the safe well yield in order to ensure efficient utilization of available groundwater. This adjustment could be done by supplying part of the peak demand from other wells or surface-water sources. On the other hand, if the source of water is surface water (e.g., river, lake, pond, or canal) and there is no constraint on the availability of water, the design discharge of the pump (i.e., pump capacity) can be determined based on the peak water requirement. An example of this situation is presented below considering the requirement of a pump for irrigating crops grown by a farmer or a group of farmers.

The discharge of the pump used for irrigation should meet the peak demand of water for a given cropping pattern. The rate of pumping (pump discharge) depends on the area under different crops, water requirements of the crops, irrigation interval and the duration of pump operation in a day. Hence, the design discharge or capacity of an irrigation pump can be calculated as (Michael and Khepar, 1999):

$$Q = \frac{27.78}{t_p} \times \sum_{i=1}^{n} \frac{A_i d_i}{t_i} \qquad (31.2)$$

Where, Q = design discharge of the pump (L/s); A_i = area under the ith crop (ha); d_i = depth of irrigation for the ith crop (cm); t_i = irrigation interval (day); and t_p= duration of pumping (h/day).

31.4 Criteria for Pump Selection

31.4.1 General Criteria

Proper selection of a pump requires the knowledge and use of the pump characteristic curves and the system head curve. The normal procedure for selecting a pump is to first determine the system head curve and desired discharge (design discharge). Thereafter, the pump manufacturer's catalogs having characteristics curves or tables are used to select a couple of pump models for consideration which can operate efficiently at or near the design discharge.

Engineers, pump installers and irrigation equipment salesmen have historically determined the design discharge and the system's total head for that discharge (Jensen, 1980). These values are used to select a pump. In practice, very few irrigation or drainage systems will have fixed operating conditions. Therefore, by determining the system head curve or curves for a range of discharges above and below the design discharge, sufficient information will be available to evaluate pump performance for all expected operating conditions. Thus, by matching the system head curve or curves for a range of discharges with the characteristic curves of various models of pumps, the pump that can provide maximum efficiency can be selected. Alternatively, if the pump characteristics are reported in a tabular form, the pump that requires minimum power to operate at the design head and design capacity can be selected after comparing the pump characteristics data from several reputed pump manufacturers (Michael and Khepar, 1999).

31.4.2 Criteria for Centrifugal Pump Selection

The selection criteria described in the pervious section in applicable for centrifugal pumps also. Some additional points are to be taken care of. Trimming of the impellers to match the pump to the specific use is quite common. The decisions that must be made for centrifugal pump selections are: which model of pump to use, how much to trim the impeller, and what

speed to turn the impeller. Manufacturer's characteristic curves contain information required to make these decisions (Jensen, 1980). It is also necessary to determine the size of prime mover (motor or engine) required to drive the pump.

Centrifugal pumps have a suction and discharge fitting or flange. The pump is connected with standard pipe fittings to both suction and discharge pipes. Care should be taken to minimize friction losses on the suction side. Large friction losses can cause cavitation which must be avoided. Most centrifugal pumps used for irrigation require that a foot valve be installed on the intake end of the suction pipe in order to ensure that the pump could remain primed at all times. There are some self-priming centrifugal pumps as discussed in Lesson 29, which may be selected according to the need and budget.

31.4.3 Operating Point of a Pump

A system characteristic between the head required (H) and the discharge (Q) to be maintained is generally expressed as a parabolic equation as follows (Jensen, 1980; FAO, 1986):

$$SH = K_1 + K_2 Q^2 \qquad (31.3)$$

Where, K_1 and K_2 are constants for the system. Equation (31.3) is graphically illustrated in Fig. 31.1.

A centrifugal pump usually operates at different combinations of head and discharge given by its head-discharge (H-Q) characteristic curve. The particular H-Q combination at which a pump operates is known as the 'pump's operating point'. A system head curve and the H-Q characteristic curve of the pump are used to determine the operating point. The point of intersection of the pump's H-Q characteristic curve and the system characteristic (H-Q) curve locates the actual operating point of a pump when it will be installed in a given system (Fig. 31.2). At the operating point, the head and discharge (H-Q) requirements of a system are equal to the head and discharge (H-Q) generated by the pump.

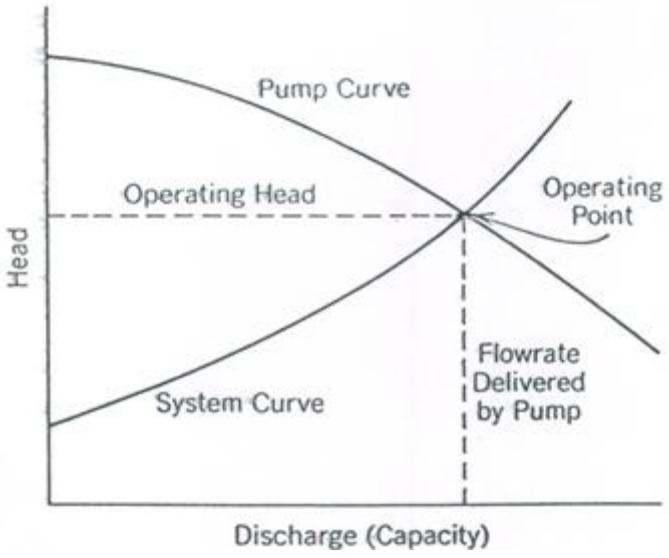

Fig. 31.2. Superposition of 'system head curve' and 'pump head-discharge curve' for determining operating point. (Source: James, 1988)

Once the operating point of a pump is determined, the power, overall efficiency, and NPSHR (net positive suction head required) for the pump can be obtained (Fig. 31.3).

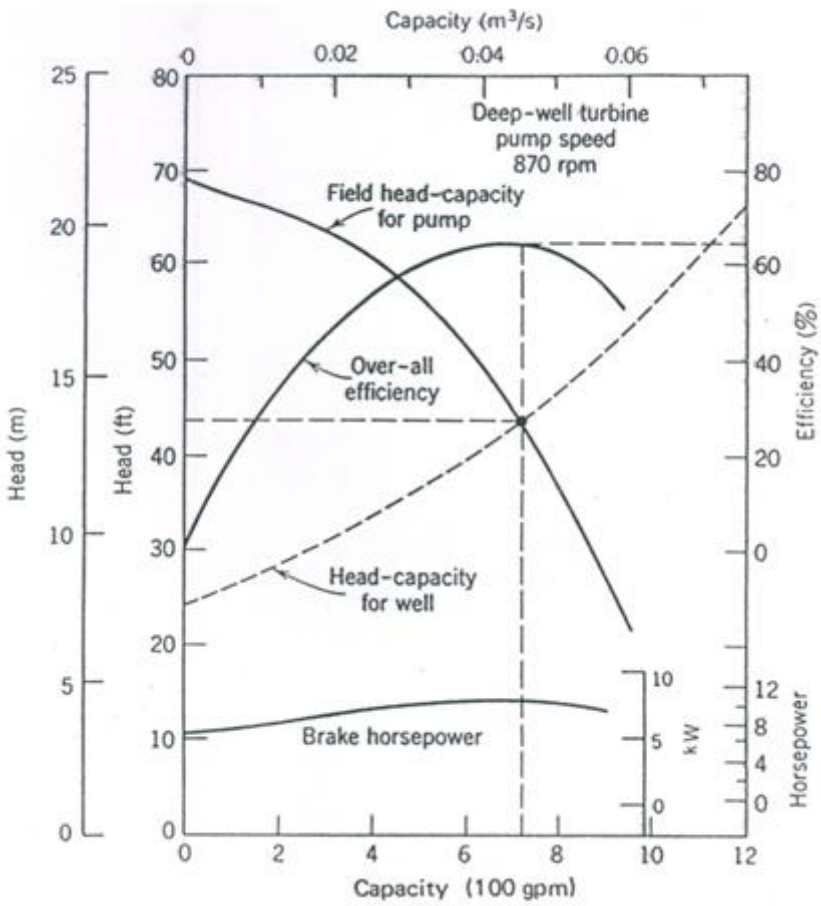

Fig. 31.3. Typical system and pump head-capacity curves.

(Source: Schwab et al., 2005)

31.5 Selection of Type of Pump

The relative merits and applicability of different types of pumps are discussed in earlier lessons. Volute centrifugal pumps are adapted to a wide range of head-discharge conditions compared to vertical turbine pumps and propeller pumps. The efficiency of vertical turbine pumps, submersible pumps and propeller pumps drops rapidly when applied under conditions different from those for which they have been designed (Michael and Khepar, 1999). The suitability of commonly used irrigation pumps to specific pumping situations is presented in Table 31.1, which can be used as guidelines for selecting a suitable pump type.

Table 31.1. Suitability of common types of irrigation pumps to specific pumping applications (Source: Michael and Khepar, 1999)

Sl. No.	Source of Water	Lift	Yield	Suitable Pump Type	Remarks
1	Stream/Canal	<2.5 m	High	Propeller pump	Volute centrifugal pumps inefficient at low heads
2	Stream/Canal	3-12 m	High	Mixed-flow pump	Volute centrifugal pumps also provide satisfactory efficiency at heads> 6 m. Tractor-operated pumps feasible upto about 6 m lift
3	Stream/Canal	>12 m	Medium to High	Volute Centrifugal Pump	Multi-stage pumps to be preferred for high beads
4	Shallow Open Well	3-6 m	Medium	Mixed-Flow Pump or Volute Centrifugal pump	Pump located at ground surface
5	Deep Open Well (Electricity available)	>6 m	Medium	Volute Centrifugal Pump directly coupled to Electric Motor	Pumping set located close to water table in well
6	Deep Open Well (Electricity	>6	Medium	Volute Centrifugal Pump driven by	Engine located at or near ground level,

	not available)	m		Diesel Engine	pump driven through belt/shaft
7	Deep Open Well (Electricity not available)	>6 m	Low	Jet Pump coupled to Diesel Engine	Pumping set located at ground surface
7	Sewage Pumping	<6 m	Medium	Non-clog Centrifugal Pump coupled to Electric Motor/Engine	Pumping set located at ground surface
8	Shallow Tube-well or Filter Point	<6 m	Medium	Volute Centrifugal Pump	Pump driven by electric motor or diesel engine located at the ground surface
9	Shallow Tube-well	5-15 m	Medium	Volute Centrifugal Pump installed in sump within suction and water table limits	In case of engine drive, the pump is located in the sump, but the engine is placed on the ground surface.
10	Deep Tubewell (Electricity available)	>15 m	Medium/High	Submersible Pump or Vertical Turbine Pump	Assured service facility essential, especially in case of submersible pumps.
11	Deep Tubewell (Electricity not available)	>15 m	Medium/High	Deep Well Turbine Pump	Engine located on the ground surface
12	Pumping from Sump or Drainage Pumping	<6 m	High	(i) Propeller Pump for lifts < 2.5 m (ii) Mixed-Flow Pump for lifts > 2.5 m	Motor-driven pumps to be provided with a float-controlled automatic switch

13	Canal Lifts	<15 m	High	Mixed-Flow Pump	To lift the flow of a canal from a lower elevation to a higher elevation

Lesson 32 Maintenance and Troubleshooting of Centrifugal Pumps

32.1 Introduction

Proper maintenance of a centrifugal pump is very important in order to ensure its trouble-free operation and long service life. The major causes of deteriorating pump performance can be summarized as follows (Roscoe Moss Company, 1990):

1. Improper pump installation. For example, leakage from the column pipe and power losses due to crooked shafts and improper tightening.
2. Changes in system conditions that force the pump to operate inefficiently.
3. Insufficient line-shaft lubrication that causes power loss and premature wear of line-shaft bearings.
4. Motor overloading and/or overheating that decreases efficiency and breakdown insulation.
5. Improper pump adjustment causing increased wear and power losses.
6. Cavitation either from entrapped air or from insufficient NPSH.
7. Abrasion from sand and/or silt produced from the well.
8. Wear from rubbing mechanical parts. This can be normal wear expected over time or abnormal wear caused by deformed or bent parts.
9. Corrosion and incrustation of pump components.
10. Mechanical plugging of the impellers or the pump suction.

Therefore, a good maintenance program should be implemented. A well-planned maintenance program maintains high pump efficiency, helps reduce power costs, improves dependability of the equipment, reduces operating costs and provides extended service life of the pump (Jensen, 1980). The maintenance operations of a centrifugal pump can be classified into two groups (Michael and Khepar, 1999): (a) preventive/routine maintenance, and (b) overhaul or repair operations. They are discussed in subsequent sections.

32.2 Preventive Maintenance of Centrifugal Pumps

32.2.1 Daily Maintenance

The operating conditions of pumps vary widely and so do the maintenance requirements. The performance of the pump should be observed daily. Any abnormality in operation should be taken care of promptly. This refers mainly to any change in the sound of running, undesired leakage in the stuffing box, abnormal change in voltage and current, and temperature. The alignment of the pump unit should be checked occasionally. The bearings that require lubrication should be given a regular attention.

32.2.2 Half-yearly Maintenance

At least once in six months, the shaft packing should be checked by observing leakage from it. Generally, a leak of 15-30 drops of liquid per minute from the stuffing box is desirable. If the leakage from the stuffing box is excessive or the packing is worn, the entire packing in the box will have to be replaced. Replacing just a ring or two will not result in an effective sealing. While overhauling the stuffing box, all the old packing rings are removed and the interior of the stuffing box cleaned thoroughly. The shaft and shaft sleeve surface are properly cleaned before inserting the packing rings. There should not be any burrs or scores on the working surface. If the shaft sleeve is badly worn or scored, it is replaced. Similarly, the straightness of the shaft is ensured.

The radial clearance between the shaft and the stuffing-box bore is measured for determining the size of a new gland packing to be provided. The packing should not be inserted in a spiral form; rather it is installed in individual rings. The rings are carefully cut to the exact size by wrapping the packing around the shaft. The packing rings are fitted carefully, opening them radially until the ends are as wide apart as half the shaft diameters, then the ends are turned apart in an axial direction, until the rings slide over the shaft. Then, each ring is pushed into the stuffing box, inserting the joint first. It should be ensured that the joints of succeeding packing rings are staggered. This is followed by assembling the gland ring and tightening the nuts by hand or a spanner. When the pump is in operation, the nuts are again adjusted to achieve the desired leakage from the stuffing box.

The pumps installed on cavity wells require half-yearly checking. Sometimes, pieces of stones or gravel get into the pump casing owing to overpumping which results in the decrease of discharge. In such situations, the pump is opened and cleaned.

32.3 Preventive Maintenance Techniques

A brief description of the salient techniques for preventive maintenance of pumps is provided below.

(1) Standby units: In most major industries failure of an essential item of equipment can result in a total emergency shut-down of the complete plant. It is for this reason that on the more vulnerable possible points of failure, such as rotating machinery, it has become normal practice to employ a working machine with a 100% standby machine standing alongside to take over the function in the event of failure occurring (Myles and Associates, 2003). If such an occurrence takes place, immediate action must be taken to restore the damaged unit to service with minimum downtime to ensure continuity of service in the event of a repeat occurrence.

(2) Regular routine checks: Despite the above precautions, regular checks on all the more vital items of equipment, including almost without exception any installed pump units should be made to identify in good time, possible developing problems which will have to be faced at the first opportunity.

(3) 'OEM' parts and service: To minimise failures in service, the repairs which are carried out on pumps and motors should be in strict accordance with the manufacturer's original

standards in all respects, and should incorporate only 'Original Equipment Manufacturers (OEM)' replacement components. The ideal would, of course, be to return the unit for periodic servicing to the maker's workshop.

(4) Cost of plant shutdown: It must be realized that the cost of possible plant shutdown can totally outweigh possible obvious saving in price by purchase of substandard parts, not of original design standards from pirate suppliers.

(5) Cost of power: If the original maker's efficiency levels are not maintained by the use of 'OEM' components, the costs of power consumption are also a most significant item in the total production costs, and again this emphasizes the need for use at all times of 'OEM' parts. Taking the equivalent capital cost of power from a calculation undertaken by a major South African Water Board for application in a contract, awarded in 1986, the equivalent capital cost of 1 kW consumption was calculated at Rs. 2300.00 (Myles and Associates, 2003).

(6) Danger to life and limb: The most important of all reasons for maintaining original standards by use of original manufactures' components and facilities is the possibilities of pump failure creating a hazardous situation, where the operating staff may, as a result, be subjected to unnecessary dangers.

(7) Quality assurance: The design, manufacturing, inspection and quality assurance standards and limitations set by the original manufacturer are a vital ingredient mix in ensuring that the above standards of performance, reliability in service, continued high efficiency and safety in operations are maintained. This is the foundation of preventive maintenance (Myles and Associates, 2003).

32.4 Overhauling of Centrifugal Pumps

Centrifugal pumps have two basic types of parts: rotating and stationary. Rotating parts include the impeller, shaft, wearing rings, shaft sleeves, and bearings. Stationary parts include the casing with the suction and discharge flanges, bearing housing and packing. Most overhaul work on centrifugal pumps is concerned with the rotating parts.

It is desirable that the pump is completely overhauled annually or once in two years. However, in many situations, the operating conditions do not permit annual shutdown periods for overhaul. In such cases, overhauling is done when it is absolutely essential, on the basis of pump performance and symptoms indicating major problems. The following situations call for a shutdown of the pumping plant for troubleshooting, repair and possible overhaul (Michael and Khepar, 1999):

1. Fall-off in pump performance,
2. Excessive noise during pump operation,
3. Excessive vibration of pump, and
4. Symptoms of corrosion or erosion trouble.

32.4.1 Dismantling of Centrifugal Pumps

The pump has to be dismantled for overhauling. As discussed in earlier lessons, there are different types of horizontal centrifugal pumps such as monoblock or close-coupled pumps, belt-driven pumps and directly-coupled pumps. The dismantling and reassembling procedure of a belt-driven pump is discussed in this section. However, the basic principles are the same, irrespective of the types of centrifugal pumps.

The pump is first disconnected from the piping system (if the pump is directly coupled, it is uncoupled by removing the coupling bolts and rubber bushes). The steps involved in dismantling a belt-driven centrifugal pump are as follows (Michael and Khepar, 1999):

1. Remove the inlet and outlet flanges.
2. Remove the bearing cap by removing the bolts holding it.
3. Remove the grease cup and bearing lock nut.
4. Remove the pedestal and take out the ball bearing, using a bearing puller.
5. Remove the belt shifter.
6. Remove the nuts and bolts joining the casings and remove the casing slowly, taking care not to damage the impeller and casing rings.
7. Remove the impeller nut.
8. Dismantle the rotating unit and remove the impeller slowly by gently hammering back the shaft using a wooden block.
9. Remove the impeller from the rotating unit.
10. Remove the pulleys using a pulley puller.
11. Finally, dismantle the stuffing box.

The detailed description about the above steps can be found in Michael and Khepar (1999).

32.4.2 Reassembling of Centrifugal Pumps

After overhauling, the pump is reassembled. The procedure of reassembling is more or less the reverse of dismantling. The following are the major steps involved in reassembling a centrifugal pump with pulley (Michael and Khepar, 1999):

1. Mount the pulleys on the shaft.
2. Mount the casing and stuffing box bushes on the shaft.
3. Gently mount the impeller on the shaft.
4. Insert the impeller key carefully.
5. Adjust the impeller at its correct position and tighten the impeller unit.

6. Insert the gasket and grease it properly.
7. Mount the casing by tightening its nuts and bolts.
8. Insert the belt shifter at its correct position.
9. Insert the shaft sleeve and tighten it properly.
10. Mount the pedestal and align it properly by inserting the desired packing.
11. Mount the ball bearing using hand press and tighten the bearing locking nut. Do not hammer the ball bearing!
12. Finally, mount the bearing cap.

32.5 Centrifugal Pump Troubles and Remedies

Troubles in centrifugal pumps can be grouped into two classes: mechanical troubles and hydraulic troubles. Mechanical troubles include breakage of the pump coupling or shaft. These troubles are easily traceable and can be attended to promptly. However, hydraulic troubles such as failure to deliver water, reduction in discharge and overloading of the prime mover are more difficult to rectify. The major troubles encountered in a centrifugal pump and their remedial measures are discussed below, which can serve as guidelines for the pump users.

Table 32.1. Summary of troubles encountered in centrifugal pumps and their remedies (Source: Myles and Associates, 2003)

SYMPTOM	PROBABLE FAULT	REMEDY
1. Pump does not deliver water	• Impeller rotating in wrong direction.	Reverse direction of rotation.
	• Pump not properly primed ¾ air or vapor lock in the suction line.	Stop pump and reprime.
	• Inlet of suction pipe insufficiently submerged.	Ensure adequate supply of liquid.
	• Air leaks in suction line or gland arrangement.	Make good any leaks or repack gland.
	• Pump not up to rated speed.	Increase speed.
2. Pump does not	• Air or vapor lock in the suction line.	Stop pump and reprime.

deliver rated quantity	• Inlet of suction pipe insufficiently submerged.	Ensure adequate supply of liquid.
	• Pump not up to rated speed.	Increase speed.
	• Air leaks in suction line or gland arrangement.	Make good any leaks or repack gland.
	• Foot valve or suction strainer choked.	Clean foot valve or strainer.
	• Restriction in delivery pipework or pipework incorrect.	Clear obstruction or rectify error in pipework.
	• Head underestimated.	Check head losses in delivery pipes, bends and valves, reduce losses as required.
	• Unobserved leak in delivery.	Examine pipework and repair leak.
	• Blockage in impeller or casing.	Remove half casing and clear obstruction.
	• Excessive wear at neck rings or wearing plates.	Dismantle pump and restore clearances to original dimensions.
	• Impeller damaged.	Dismantle pump and renew impeller.
	• Pump gaskets leaking.	Renew defective gaskets.
3. Pump does not generate its rated delivery pressure	• Impeller rotating in wrong direction.	Reverse direction of rotation.
	• Pump not up to rated speed.	Increase speed.
	• Impeller neck rings worn excessively.	Dismantle pump and restore clearances to original

		dimensions.
	• Impeller damaged or choked.	Dismantle pump and renew impeller or clear blockage.
	• Pump gaskets leaking.	Renew defective gaskets.

4. Pump loses liquid after starting	• Suction line not fully primed ¾ air or vapor lock in the suction line.	Stop pump and reprime.
	• Inlet of suction pipe insufficiently submerged.	Ensure adequate supply of liquid at suction pipe inlet.
	• Air leaks in suction line or gland arrangement.	Make good any leaks or renew gland packing.
	• Liquid seal to gland arrangement logging ring (if fitted) choked.	Clean out liquid seal supply.
	• Logging ring not properly located.	Unpack gland and relocate logging ring under supply orifice.
5. Pump overloads driving unit	• Pump gaskets leaking.	Renew defective gaskets.
	• Serious leak in delivery line, pump delivering more than its rated quantity.	Repair leakage.
	• Speed too high.	Reduce speed.
	• Impeller neck rings worn excessively.	Dismantle pump and restore clearance to original dimensions.
	• Gland packing too tight.	Stop pump, close delivery valve to relieve internal pressure on packing, slacken back the gland nuts and retighten to finger

		tightness.
	• Impeller damaged.	Dismantle pump and renew impeller.
	• Mechanical tightness at pump internal components.	Dismantle pump, check internal clearance and adjust as necessary.
	• Pipework exerting strain on pump.	Disconnect pipework and realign to pump.
6. Excessive vibration	• Air or vapor lock in suction.	Stop pump and reprime.
	• Inlet of suction pipe insufficiently submerged.	Ensure adequate supply of liquid at suction pipe inlet.
	• Pump and driving unit incorrectly aligned.	Disconnect coupling and realign pump and driving unit.
	• Worn or loose bearings.	Dismantle and renew bearings.
	• Impeller chocked or damaged.	Dismantle pump and clear or renew impeller.
	• Rotating element shaft bent.	Dismantle pump and straighten or renew shaft.
	• Foundation not rigid.	Remove pump, strengthen the foundation and reinstall pump.
	• Coupling damaged.	Renew coupling.
	• Pipework exerting strain on pump.	Disconnect pipework and realign to pump.
7. Bearing	• Pump and driving unit out of	Disconnect coupling and

overhauling	alignment.	realign pump and driving unit.
	• Oil level too low or too high.	Replenish with correct grade of oil or drain down to correct level.
	• Wrong grade of oil.	Drain out bearing, flush through bearings; refill with correct grade of oil.
	• Dirt in bearings.	Dismantle, clean out and flush through bearings; refill with correct grade of oil.
	• Moisture in oil.	Drain out bearing, flush through and refill with correct grade of oil. Determine cause of contamination and rectify.
	• Bearings too tight.	Ensure that bearings are correctly bedded to their journals with the correct amount of oil clearance. Renew bearings if necessary.
	• Too much grease in bearing.	Clean out old grease and repack with correct grade and amount of grease.
	• Pipework exerting strain on pump.	Disconnect pipework and realign to pump.
8. Bearing wear	• Pump and driving unit out of alignment.	Disconnect coupling and realign pump and driving unit. Renew bearings if necessary.
	• Rotating element shaft bent.	Dismantle pump, straighten or renew shaft. Renew bearings if necessary.

	• Dirt in bearing.	Ensure that only clean oil is used to lubricate bearings. Renew bearings if necessary. Refill with clean oil.
	• Lack of lubrication.	Ensure that oil is maintained at its correct level or that oil system is functioning correctly. Renew bearings if necessary.
	• Bearing badly installed.	Ensure that bearings are correctly bedded to their journals with the correct amount of oil clearance. Renew bearings if necessary.
	• Pipework exerting strain on pump.	Ensure that pipework is correctly aligned to pump. Renew bearings if necessary.
	• Excessive Vibration.	Refer to symptom 6.
9. Irregular delivery	• Air or vapor lock in the suction line.	Stop pump and reprime.
	• Fault in driving unit.	Examine driving unit and make good any defect.
	• Air leaks in suction line or gland arrangement.	Make good any leaks or repack gland.
	• Inlet of suction pipe insufficiently immersed in liquid.	Ensure adequate supply of liquid at suction pipe inlet.
10. Excessive noise level	• Air or vapor lock in suction line.	Stop pump and reprime.
	• Inlet of suction pipe	Ensure adequate supply of

	insufficiently submerged.	liquid at suction pipe inlet.
	• Air leaks in suction line or gland arrangement.	Make good any leaks or repack gland.
	• Pump and driving unit out of alignment.	Disconnect coupling and realign pump and driving unit.
	• Worn or loose bearings.	Dismantle and renew bearings.
	• Rotating element shaft bent.	Dismantle pump, straighten or renew shaft.
	• Foundation not rigid.	Remove pump and driving unit, strengthen foundation.