ASHOKA INSTITUTE OF TECHNOLOGY & MANAGEMENT, VARANASI

DIGITAL EDUCATION

FLOOD ESTIMATION

Branch : Civil Engineering

Subject: Engineering Hydrology

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Short Notes on Engineering Hydrology

Precipitation & General aspects of Hydrology

Index of Wetness

- index of wetness =

$$\frac{\text{rainfall in a given year at a given place}}{\text{average annual rainfall of that place}} \times 100$$

- % Rain deficiency = 100 - % index of wetness

Aridity index

$$A.I = \frac{PET - AET}{PET} \times 100$$

Where, A.I = Aridity index

PET = Potential Evapo- transpiration

AET = Actual Evapotranspiration

a. AI ≤ 0 → Nonarid
b. 1 ≤ A.I ≤ 25 → Mild Arid
c. 26 ≤ A.I ≤ 50 → Moderate arid
d. A.I > 50 → Severe Arid

Optimum Number of rain Gauge: (N)

$$N = \left(\frac{C_V}{\in}\right)^2$$

$$C_V = \frac{\sigma_{n-1}}{\bar{X}} \times 100$$

$$\sigma_{n-1} = \sqrt{\frac{\Sigma(X - \bar{X})}{(n-1)}}$$

$$\bar{x} = \frac{\Sigma x}{n}$$

where, C_v = Coefficient of variation,

ϵ = Allowable % Error,

σ Standard deviation of the data, n = Number of stations,

$\overline{x}$ mean of rainfall value

Estimation of missing rainfall data

$$(a)\ P_x \cdot = \frac{P_1 + P_2 + \ldots + P_n}{(n)}$$

$$If\ N_1, N_2 \ldots N_n < 10\%\ of\ N_x$$

where, $N_1, N_2, \ldots N_x..N_n$ are normal annual percipitation of 1,2,…x…n respectively.

$P_1, P_2 \ldots P_n$ are rainfall at station 1,2,…. N respectively.

And P_x is the rainfall of station x.

Case: A minimum number of three stations closed to station 'x'

$$P_x = \frac{P_1 + P_2 + P_3}{3}$$

$$(b)\quad P_x = \frac{N_x}{n}\left[\frac{P_1}{N_1} + \frac{P_2}{N_2} + \ldots + \frac{P_n}{N_n}\right]$$

If any of N_1, N_2, N_3…

N_n > 10% of N_x

Mean rainfall Data

To convert the point rainfall values at various into an average value over a catchment the following three methods ar in use

(i) Arithmetic Avg Method: when the rainfall measured at various stations in a catchment area is taken as the arithmetic mean of the station values.

$$P_{avg} = \frac{P_1 + P_2 + \ldots + P_n}{n}$$

Where, $P_1, P_2 \ldots P_n$ are rainfall values

Of stations 1,2…n respectively.

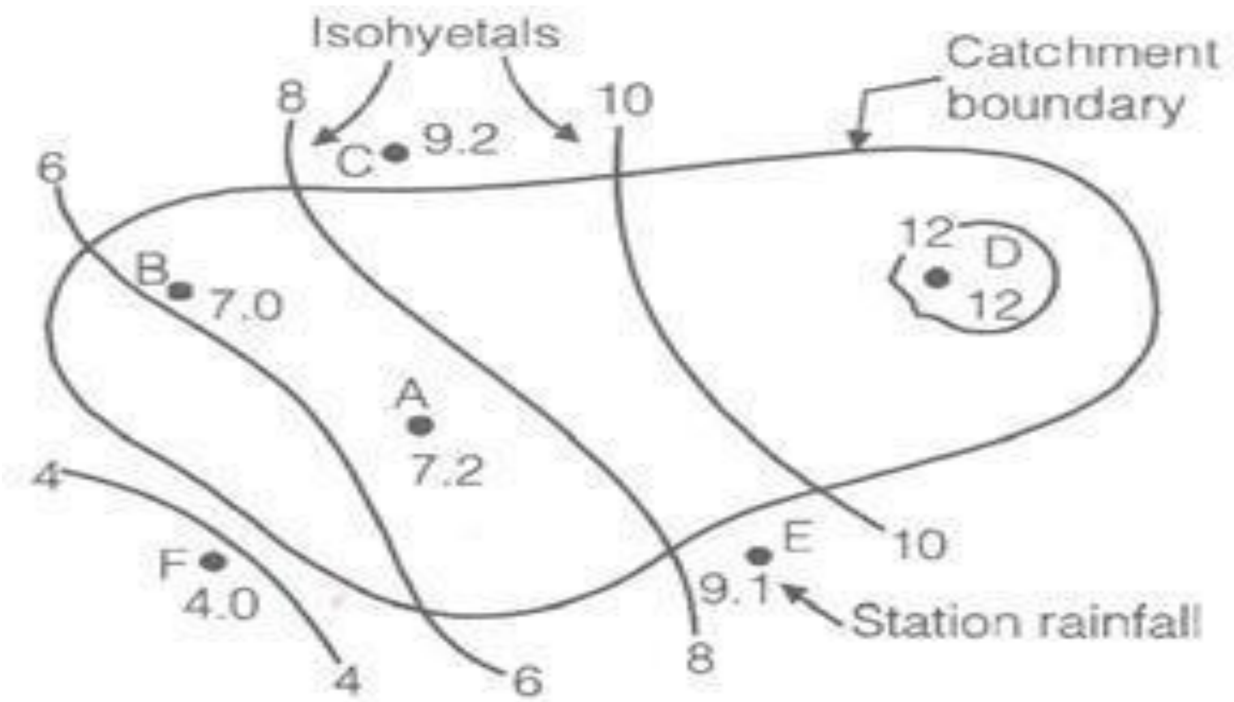
In practice this method is used very rerely.

(ii) Thiessen Polygon Method: In this method, the rainfall recorded at each station is given a weightage on the basis of an area closest to the station.

$$P_{avg} = \frac{P_1A_1 + P_2A_2 + \ldots + P_nA_n}{A_1 + A_2 + \ldots + A_n}$$

Where, $P_1, P_2 \ldots P_n$ are the rainfall data of areas $A_1, A_2 \ldots A_n$. The thiessen-polygon method of calculating the average percipitation over an area is superior to the arithmetic average method.

(iii) Isohyetal Method: An isohyet is a line joining points of equal rainfall magnitude. The recorded values for which areal average P is to be detarmined are then maked on the plot at appropirate stations.Neighbouring stations outside the catchment are also considered.



$$P_{avg} = \frac{A_1 \frac{(P_1 + P_2)}{2} + A_2 \frac{(P_2 + P_3)}{2} + A_{n\text{-}1} \frac{(P_{n\text{-}1} + P_n)}{2}}{A_1 + A_2 + \ldots + A_{n\text{-}1}}$$

Infiltration, Run off and Hydrographs

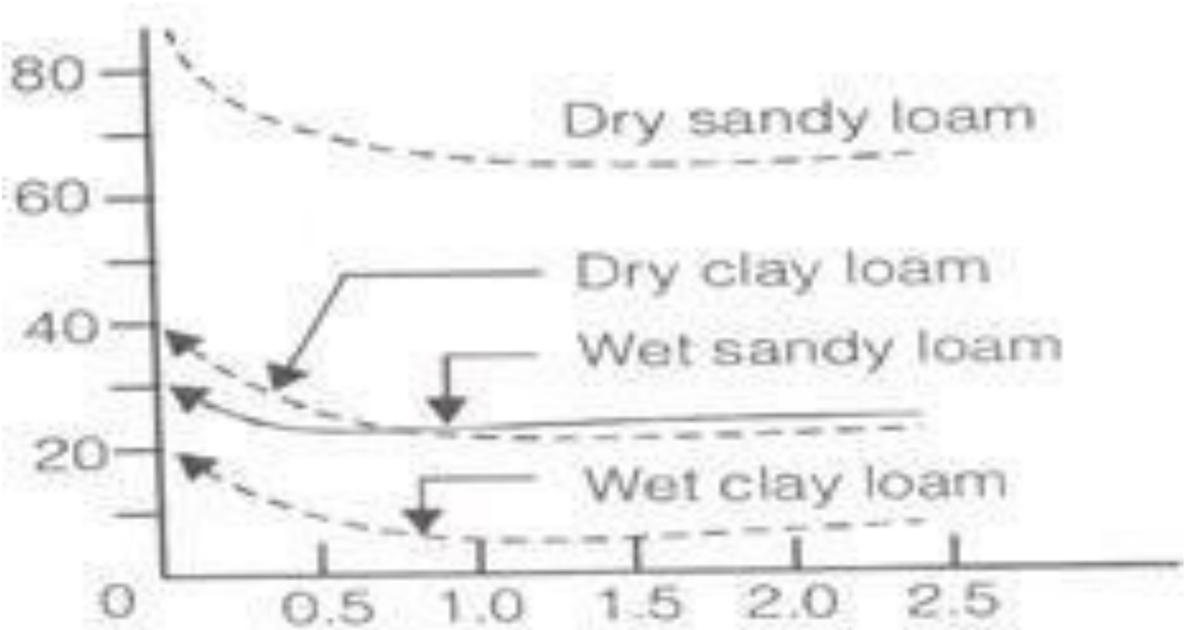
Infiltration

Infiltration is the flow of water into the ground through the soil surface.

- **Horton's Equation**: Horton expressed the decay of infiltration capacity with time as an exponential decay given by

$$f_{ct} = f_{cf} + \left(f_{co} - f_{cf}\right)e^{-k_h \cdot t_d}$$

Where,



f_{ct} = infiltration capacity at any time t from start of the rainfall

f_{co} = initial infiltration capacity at t = 0

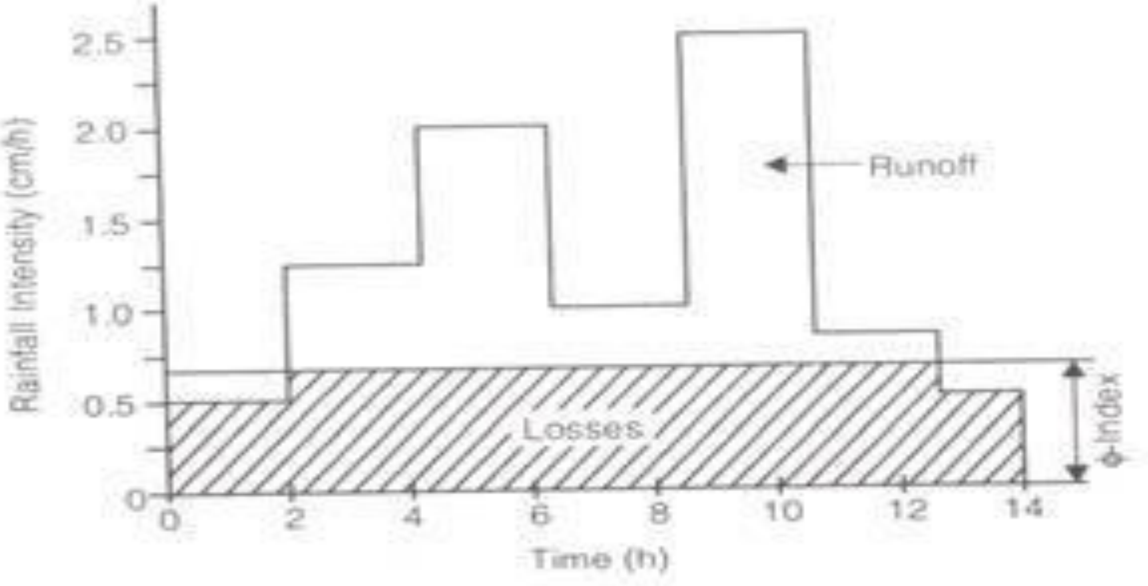
f_{ct} = final steady state value

t_d = Duration of rainfall

k_h = constant depending on soil.

Infiltration indices

In hydrological calculations involving floods it is found convenient to use a constant value of filtration rate for the duration of the storm. The defined average infiltration rate is called infiltration index and two types of indices are in common use



(i) **W-index**: In an attempt to refine the φ-index the initial losses are separated from the total abstractions and an average value of infiltration rate, called W-index, is defined as

$$W\text{-}index = \frac{P - R - I_a}{t_e}$$

Where, P = Total storm precipitation (cm)

R = Total storm runoff (cm)

I_a = initial losses (cm)

t_e = Duration of rainfall excess

W-index = Avg. rate of infiltration (cm/hr)

(ii) **φ-index**: The φ index is the average rainfall above which the rainfall volumes is equal to the runoff volume. The φ index is derived from the rainfall hyetograph with the edge of the resulting run- off volume.

$$-index = \frac{I - R}{24}$$

Where, R = Runoff in cm from a 24- h rainfall of intensity I cm/day

Runoff

Runoff means the draining or flowing off of precipitation from a catchment area through a surface channel. It thus represents the output from the catchment in a given unit of time.

Direct Runoff: it is that part of the runoff which enters the stream immediately after the rainfall. It includes surface runoff, prompt interflow and rainfall on the surface of the stream. In the case of snow-melt, the resulting flow entering the stream is also a direct runoff, sometimes terms such as direct storm runoff are used to designate direct runoff.

Base Flow: The delayed flow that reaches a stream essentially as groundwater flow is called base flow.

(i) Direct runoff = surface runoff + Prompt interflow

(ii) Direct runoff = Total runoff- Base flow

(iii) Form Factor $\frac{A}{l^2}$ where, A = Area of the catchment / Axial length of basin.

(iv) Compactness coefficient $= \frac{P}{2\pi r_e} r_e = \sqrt{\frac{A}{\pi}}$

r_e = Radius of equivalent circle whose Area is equal to area of catchment (A)

(v) Elevation of the water shed, (z)

$$z = \frac{A_1 z_1 + A_2 z_2 + ... + A_n z_n}{A_1 + A_2 + ...A_n}$$

Where, A_1, A_2 … Area between successive contours.

Z_1, z_2 … mean elevation between two successive contours.

Method to compute Runoff

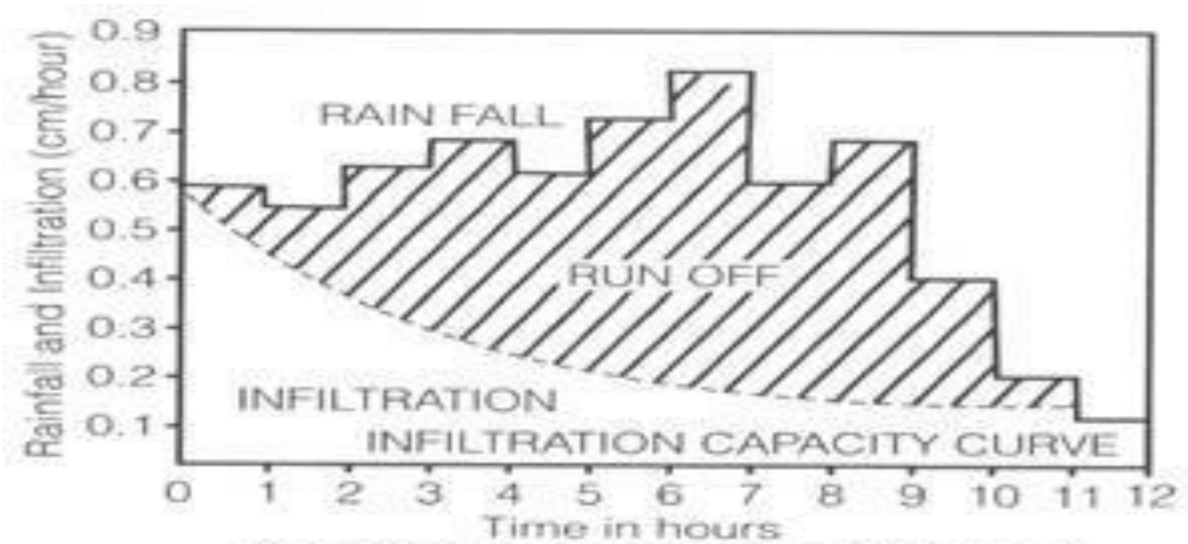
(i) By Runoff coefficient

Q = KP where, p = precipitation

K = Runoff coefficient

Q = Runoff

(ii) By infiltration Capacity Curve



Infiltrations capacity curve and run-off

(iii) By Rational Formula

$$Q_p = \frac{1}{36} \cdot kP_C A$$

Where, k = Runoff coefficient

P_C = Critical design rainfall intensity in cm/hr

A = Area of catchment in hectare

Q_p = Peak discharge in m^3/sec.

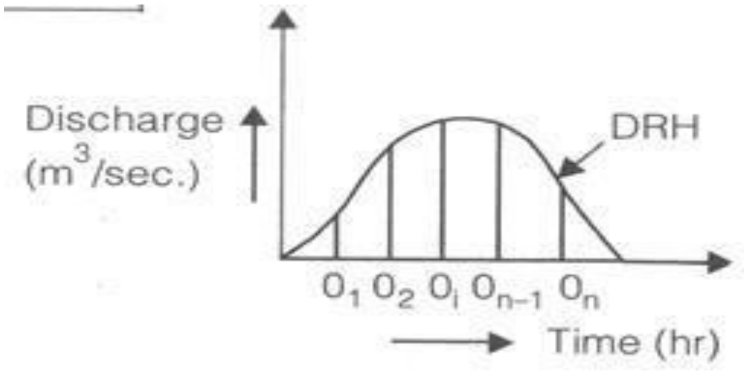
(iv) Direction runoff depath $= \dfrac{0.36\sum_{i=1}^{n}(0_i)t}{A} cm$

Where,

A = Area in KM^2

T = Time in hour

O_i = ordinate of i^{th} element i.e. discharge in m^3/sec.



Hydrograph

A plot of the discharge in a stream plotted against time chronologically is called a hydrograph.

Unit Hydrograph

This method was first suggested by Sherman in 1932 and has undergone many refinements since then.

A unit hydrograph is defined as they hydrograph of direct runoff resulting from one unit depath (1 cm) of rainfall excess occurring uniformly over the basin and at a uniform rate for a specified duration (D hours).

Time invariance: The first basic assumption is that the direct-runoff response to a given effective rainfall in a catchment is time invariant. This implies that the DRH for a given ER in a catchment is always the same irrespective of when it occurs.

Linear Response: The direct-runoff response to the rainfall excess is assumed to be linear. This is the most important assumptions of the unit-hydrograph theory. Linear response means that if an input X_1(t) cause an output y_1(t) an output x_2(t) causes an output y_2(t) then an input x_1(t) + x_2(t) gives an output $y_1(t) + y_2(t)$. Consequently if x_2(t) = rx_1(t), then y_2(t) = r_1y_1(t) thus, if the rainfall excess in a duration D is r times the unit depth, the resulting DRH will have ordinates bearing ratio r to those of the corresponding D-h unit hydrograph.

$(i)\ t'_B = t_B + (n-1)D$

Where, $t'_B =$ Base period of T hr U.H

t_B = Base period of D hr U.H

Also, T>D

T = n.D where 'n' is an integer.

Floods, Flood Routing and Flood Control

A flood an unusually high stage in a river, normally the level at which the river overflow its banks and inundates the adjoining area .The design of bridges, culvert waterways and spillways for dams and estimation of score at a hydraulic structure are some examples wherein flood-peak values are required. To estimate the magnitude of a flood peak the following alternative methods are available:

1. Rational method
2. Empirical method
3. unit-hydrograph technique
4. Flood- frequency studies

Rational Method

If $t_p \geq t_c$

$$Q_p = \frac{1}{36} \cdot k \cdot P_C \cdot A$$

Where, Q_p = Peak discharge in m^3/sec

P_C = Critical design rainfall in cm/hr

A = Area catchment in hectares

K = Coefficient of runoff.

t_D = Duration of rainfall

t_C = Time of concentration

Empirical Formulae

(a) Dickens Formula (1865)

$$Q_p = C_D \cdot A^{3/4}$$

Where, Q_p = Flood peak discharge in m^3/sec

A = Catchment area in km^2.

C_D = Dickens constant, $6 \leq C_D \leq 30$.

(b) Ryve's formula (1884)

$$Q_P = C_R \cdot A^{2/3}$$

Where,

C_H = Ryve's constant

= 8.8 for constant area within 80 km from the cost.

= 8.5 if distance of area is 80 km to 160 km from the cost.

= 10.2 if area is Hilley and away from the cost.

(c) inglis Formula (1930)

$$Q_P = \frac{124\text{A}}{\sqrt{A+10.4}} \simeq 123\sqrt{A}$$

Where, A = Catchment area in Km^2.

Q_P = Peak discharge in m^3/sec.

Flood Frequency Studies

(i) Recurrence interval or return Period:

$T = \frac{1}{P}$ where, P = Probability of occurrence

(ii) Probability if non-occurrence: q = 1-P

(iii) Probability of an event occurring r times in 'n' successive years:

$$^nC_r = \frac{n!}{(n-r)r!}$$

(iv) Reliability: (probability of non-occurrence /Assurance) = q^n

(v) Risk = 1-q^n → Risk = 1(1-P)^n

(vi) Safety Factory = $\frac{\text{Design values of hydrologic parameter adopted}}{\text{Estimated value of hydrological parameter}}$

(vii) Safety Margin = design value of hydrological parameter – Estimated value of hydrological parameter

Gumbel's Method

The extreme value distribution was introduction by Gumbel (1941) and is commonly known as Gumbel's distribution. it is one of the most widely used probability distribution functions for extreme values in hydrologic and meteorologic studies for prediction of flood peaks, maximum rainfall, maximum wind speed.

Gunbel defined a flood as the largest of the 365 daily flows and the annual series of flood flows constitute a series of largest values of flows.

Based on probability distribution.

$$P_{(x \geq x_0)} = 1 - e^{-e^{-y}}$$

$(i)\ X_T = \overline{X} + K \cdot \sigma$ Where, X_T = Peak value of hydrologic data

K = Frequency factor

$$(ii)\ k = \frac{y_T - \overline{y_n}}{S_\eta}$$

y_T = Reduced variate

$$y_T = -\log_e \log_e \left(\frac{T}{T-1} \right)$$

T = Recurrence interval in year

y_n = Reduced mean = 0.577

S_n = Reduced standard deviation.

S_n = 1.2825 for N → ∞

$$(iii)\ \sigma = \sqrt{\left(\frac{N}{N-1} \right) \left[\overline{X^2} - \overline{X}^2 \right]}\ \overline{X} = \frac{\Sigma x}{N}$$

$$And\ \overline{X^2} = \frac{\Sigma x^2}{N}$$

Confidence Limit

Since the value of the variate for a given return period, x_T determined by Gumbel's method can have errors due to the limited sample data used. An estimate of the confidence limits of the estimates is desirable the confidence interval indicates the limits about the calculated value between which the true value can be said to lie with specific probability based on sampling errors only.

For a confidence probability c, the confidence interval of the variate x_T is bounded by value x_1 and x_2 given by

$$X_2 / X_1 = X_T \pm f(c) \cdot S_e$$

Where, f(c) is a function of confidence probability 'C'.

C (in %)	50	68	80	90	95	99
F(C)	0.674	1.00	1.282	1.645		2.58

S_e = Probability error

Where, N = Sample size

B = factor

σ = Standard deviation

$$S_e = \frac{b\sigma}{\sqrt{N}} \quad b = \sqrt{1 + 1 \cdot 3k + 1 \cdot k^2}$$

$$k = \frac{y_T \overline{y_n}}{S_n} \quad k = Frequency\ factor.$$

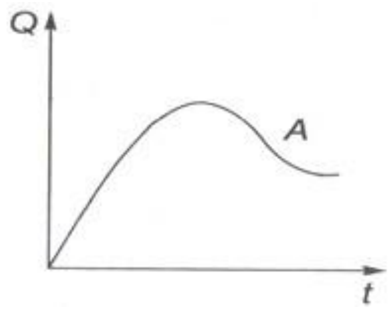
Flood Routing

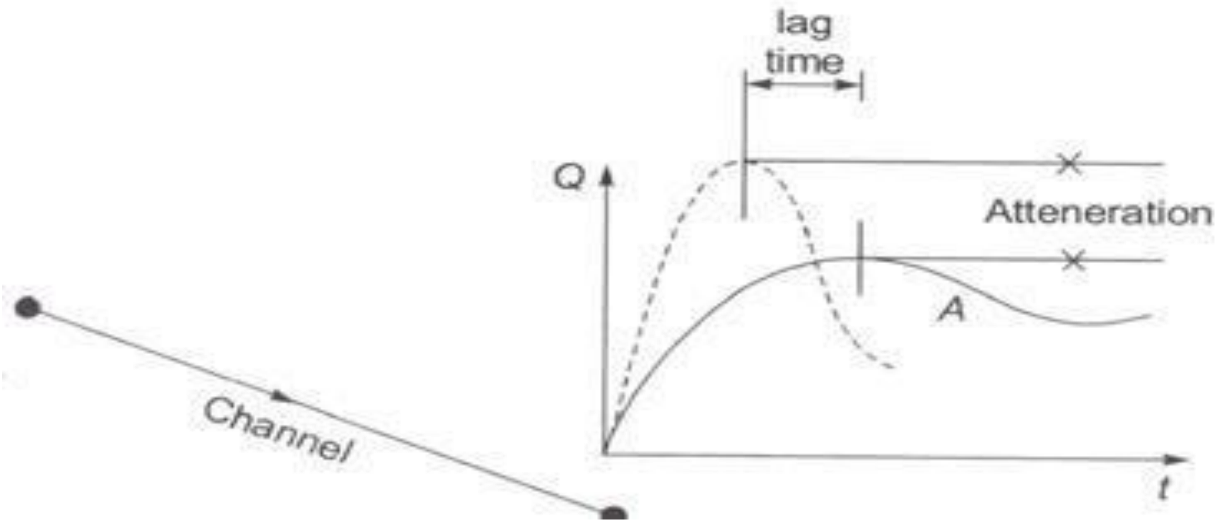
Flood routing is the technique of determining the flood hydrograph at a section of a river by utilizing the data of flood flow at one or more upstream sections. The hydrologic analysis of problems such a flood forecasting. Flood protection Reservoir design and spillway design invariable includes flood routing.

Prism Storage: it is the volume that would exist if the uniform flow occurred at the downstream depth. i.e., the volume formed by an imaginary plane parallel to the channel bottom drawn at the outflow section water surface.

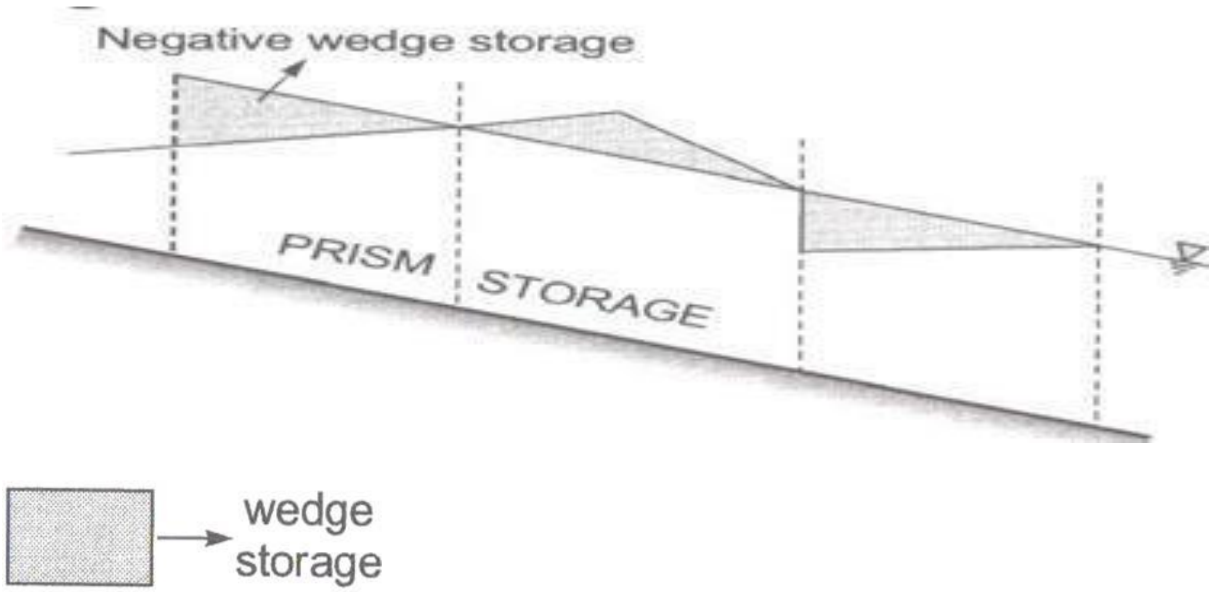
Wedge Storage: it is the wedge like volume formed between the actual water surface profile and the top surface of the prism storage.

Flood Routing





Muskingum Method



$S = S_p + S_w$

Where, S = Total storage in the channel.

S_p = Prism storage

= if (Q) = function of outflow discharge.

S_w = Wedge storage

= f(I) = function of inflow discharge.

$$S = f(Q) + f(I)\ S = k\left[XI^m + (1 - X)Q^m\right]$$

Where, X = Weighting factor

M = Constant = 0.6 for rectangular channels

= 1.0 for nature channels

K = storage time constant

Method of Channel Routing

Muskingum Method: Hydrologic channel Routing

$(i)\ \Delta S = \left(\bar{I} - \bar{Q}\right)\Delta t$ where, ΔS → Change in storage in time Δ*t*

Δ*t* → Time interval at which observations are taken. (Routing interval)

$\bar{I} \rightarrow$ Avg. in flow rate over the period Δ*t*

$\bar{Q} \rightarrow$ Average outflow rate over time period Δ*t*.

- $\Delta S = \left[\left(\frac{I_1 + I_2}{2}\right)\right] - \left(\frac{Q_1 + Q_2}{2}\right)\Delta t$
- $S_1 = k\left[XI_1 + (1 - X)Q_1\right]$
- $S_2 = k\left[XI_2 + (1 - X)Q_2\right]$
- $(S_2 - S_1) = k\left[X(I_2 - I_1) + (1 - X)(Q_2 - Q_1)\right]$
- $Q_1 = Q_1$
- $Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1$
- $Q_n = C_0 I_n + C_1 I_{n-1} + C_2 Q_{n-1}$

Where, C_0, C_1 and C_2, are Muskingum constant

$$C_0 = \frac{-kx + 0.5\Delta t}{k(1-x) + 0.5\Delta t}$$

$$C_1 = \frac{-kx + 0.5\Delta t}{k(1-x) + 0.5\Delta t}$$

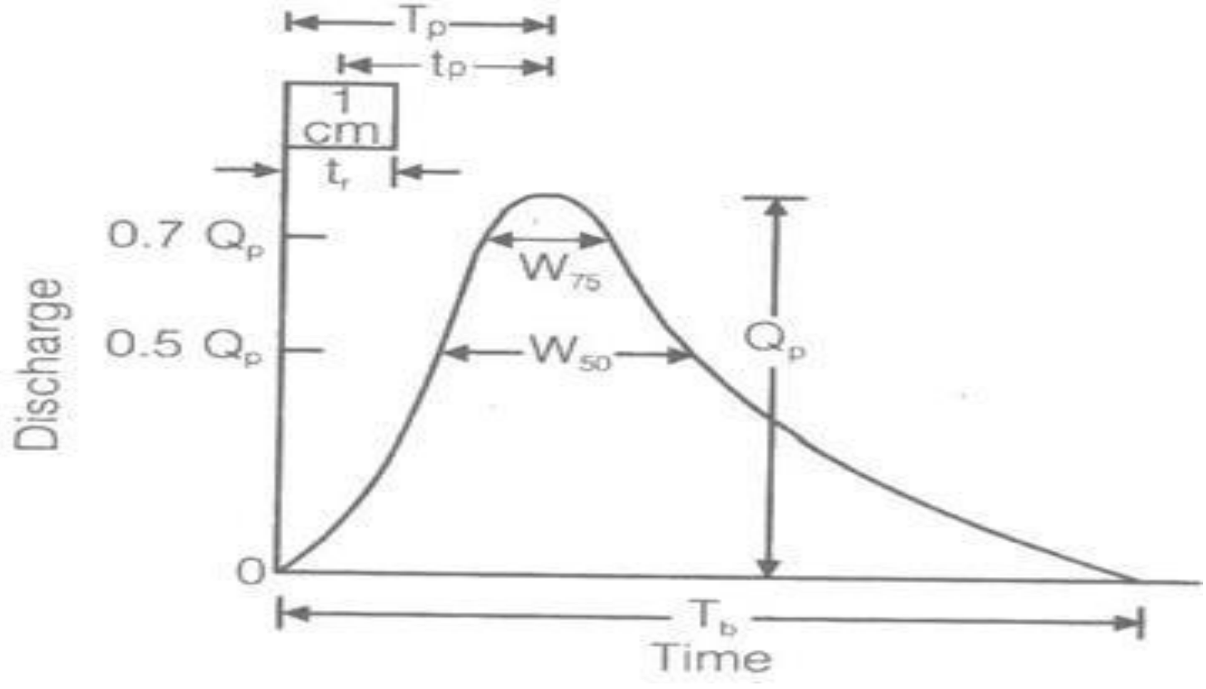
$$C_2 = \frac{k(1-x) + 0.5\Delta t}{k(1-x) + 0.5\Delta t}$$

$$C_0 + C_1 + C_2 = 1$$

- for best result, $\boldsymbol{2Kx < \Delta t < k}$

Synthetic Hydrograph

Synder's Method: Synder (1938), based on a study of a large number of catchment in the Appalachian Highlands of eastern United states developed a set of empirical equations for synthetic unit hydrograph in those areas. These equations are in use in the USA. And with some modifications in many other countries, and constitute what is known as Synder's synthetic unit hydrograph.



$$(i)\ t_p = C_t\left[L \cdot L_{Ca}\right]^{0.3}$$

Where, t_p = Time interval between mid-point of unit rainfall excess and peak of unity hydrograph in hour

L = Length of main stream

L_{Ca} = The distance along the main stream from the basin outlet to a point on the stream which is nearest to the centroid of basis (in KM)

C_t = Regional constant 0.3 < C_t 0.6

$$(ii)\ t_p = C_t\left[\frac{L\cdot L_{Ca}}{\sqrt{S}}\right]^n$$ S = Basin slope.

N = Constant = 0.38.

$$(iii)\ t_r = \frac{t_p}{5.5}$$ t_r = Standard duration of U.H in hour

$$(iv)\ Q_{PS} = \frac{2.78C_pA}{t_p}$$

Where, C_p = Regional constant = 0.3 to 0.92.

A = Area of catchment in km^2.

Q_{PS} = Peak discharge in m^3/s.

$$(v)\ t_p' = \frac{22}{22}t_P + \frac{t_R}{4}$$ where, t_R = standard rainfall duration.

t_p' = Basin lag for non-standard U.H.

$$(vi)\ Q_p = \frac{2.78C_pA}{t_p'}$$

$$(vii)\ t_B = \left(72 + 3t_p'\right)hour,$$ for a large catchment.

Where, t_B = Base time of synthetic U.H

$$t_B = 5\left[t_p' + \frac{t_R}{2}\right]hour,$$ for small catchment.

$$(viii)\ W_{50} = \frac{5.87}{(q)^{1.08}}$$ W_{50} = **width** of U.H in hour at **50%** peak discharge.

$$(ix)\ W_{75} = \frac{W_{50}}{7.15}$$ W_{75} = **Width** of U.H in hours at **75%** peak discharge.

$(x)\ q = \frac{Q_P}{A}$ where, Q_P = Peak discharge in m^3/sec.

A = Area in km^2.

Well hydraulics and Aquifiers

Aquifer

An aquifer is an saturated geological formation, underground layer of water-bearing permeable and porous or unconsolidated materials (gravel, sand, or silt) from which groundwater can be extracted using a water well.

Some Fundamental definitions:

1) Aquiclude

- These are the geological formations which, are highly porous but non-permeable. Hence water cannot be extracted from these types of geological formations.
  e.g. Clay

2) Aquitard

- These are the geological formations, which are porous but possess very less permeability. Hence water does not readily flow out of these formations, but instead water seeps out.
  e.g. Sandy Clay

3) Aquifuge

- These are geological formations, which are neither porous nor permeable.
  e.g. Granite

Type of aquifer

1. Un-Confined aquifer
2. Perched aquifer
3. Confined aquifer

1) Un-confined aquifer

- Boundary of Un-confined aquifer extended from water table (water surface which is under atmospheric pressure) to impermeable bed strata.
- Not subjected to any confining pressure and Water in Un-confined aquifer is under atmospheric pressure.
- Un-confined aquifer are recharged by directly rainfall over the surface and water body.
- This aquifer is also called **non-artesian aquifer**.

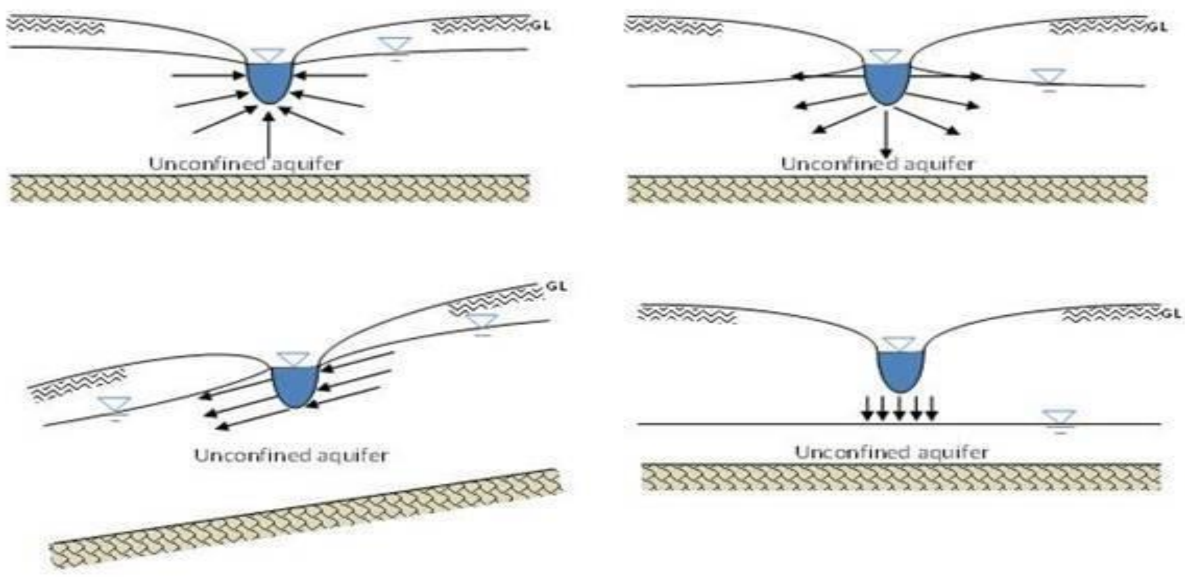


Figure Various type of Un-confined aquifer

2) Perched aquifer

- Perched aquifer is small water body which is situated in unsaturated zone of soil above the main ground water table or main unconfined aquifer, separated by ***impervious strata***.

3) Confined aquifer

- Confined aquifer bounded between two impermeable or very less permeable soil strata or rocks.
- In confined aquifer, water is ***under pressure or artesian pressure*** (pressure above the atmospheric pressure) because in that case water is sandwiches between to impermeable layer or rock.
- This is also called **artesian aquifer.**

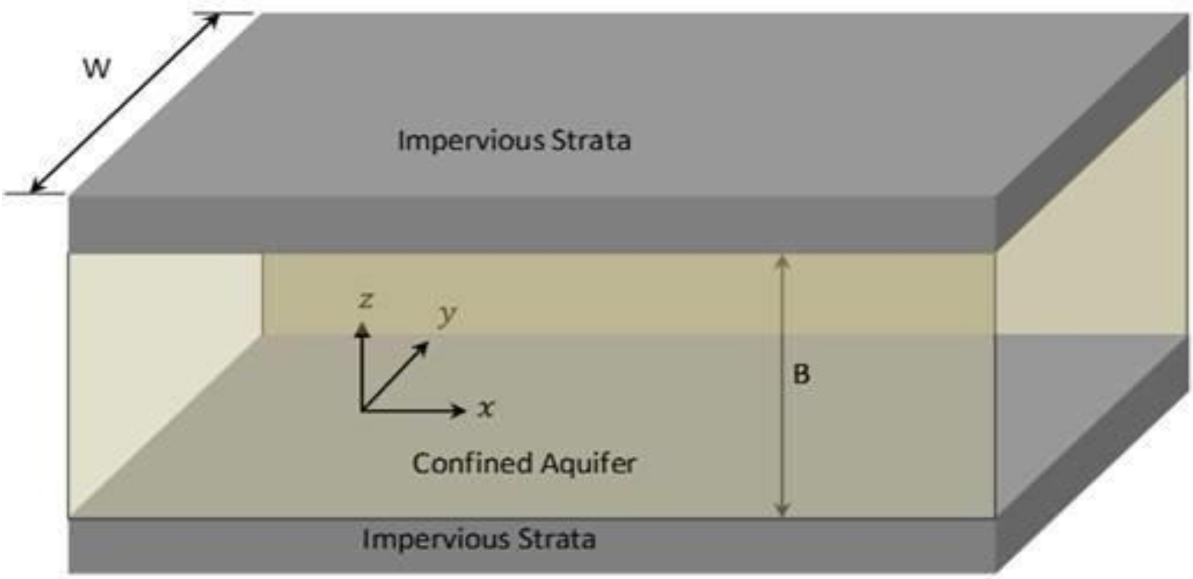


Figure Confined aquifer

Some important terminology used in well hydraulics

1) Cone of depression

- Cone of depression represent the water table during the drawdown of water with the help of *well* through *homogeneous* and *isotropic* aquifer.
- In un-confined aquifer cone of depression represent the drawdown water table but in confined aquifer it represent the *pressure drop* (change in piezometric head) around the well.
- Drop in water table from previous static water table is termed as ***drawdown depth*** or simply ***drawdown***.

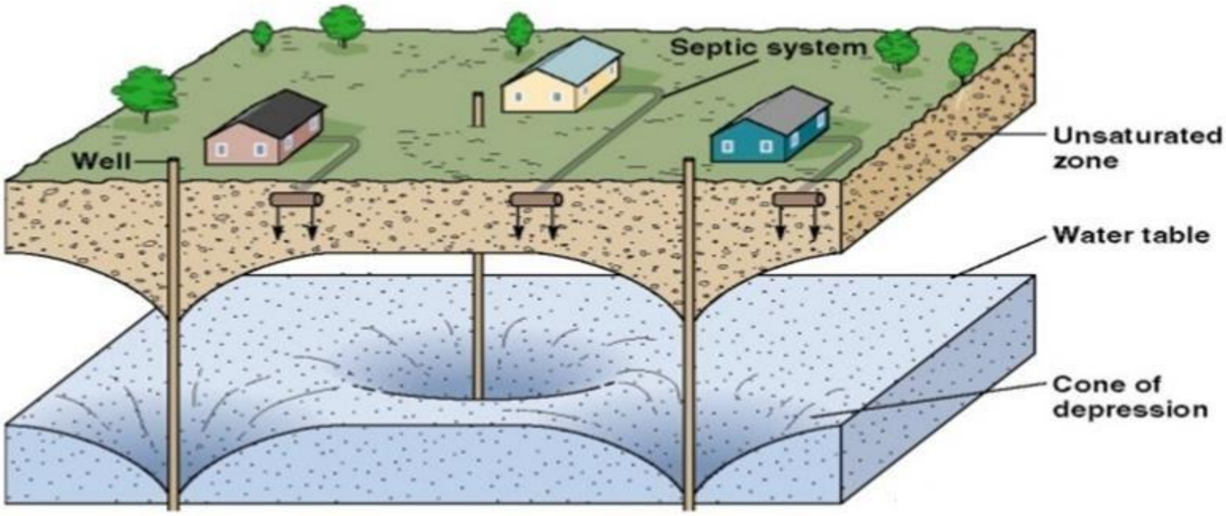


Figure 3-d view of cone of depression

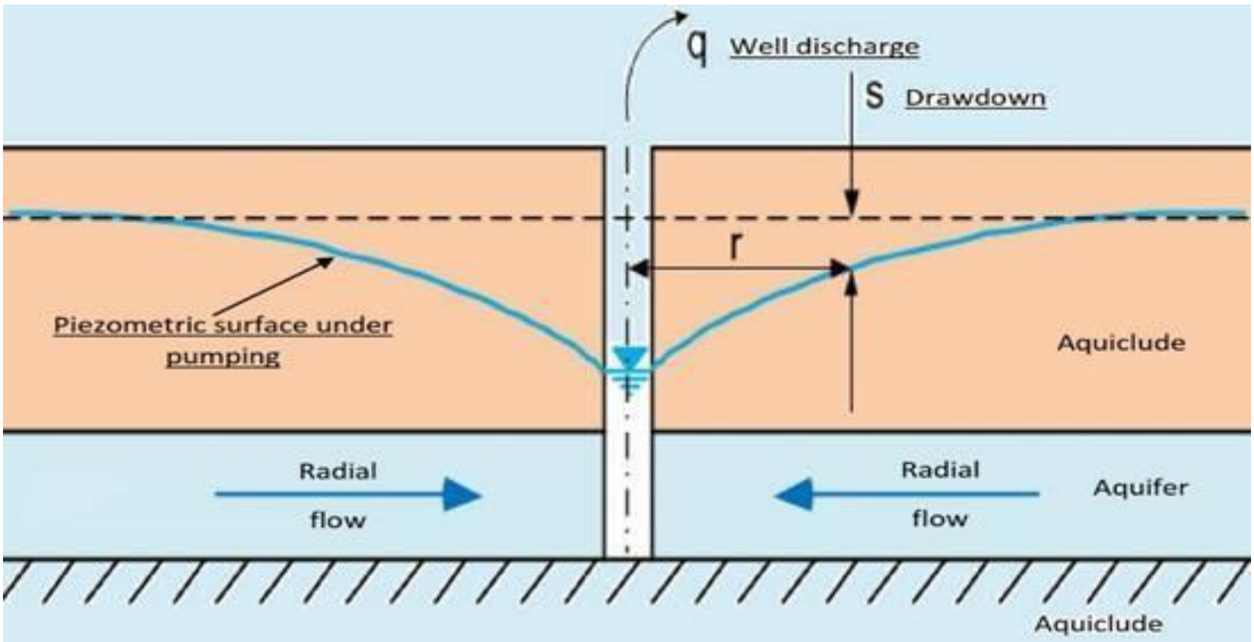
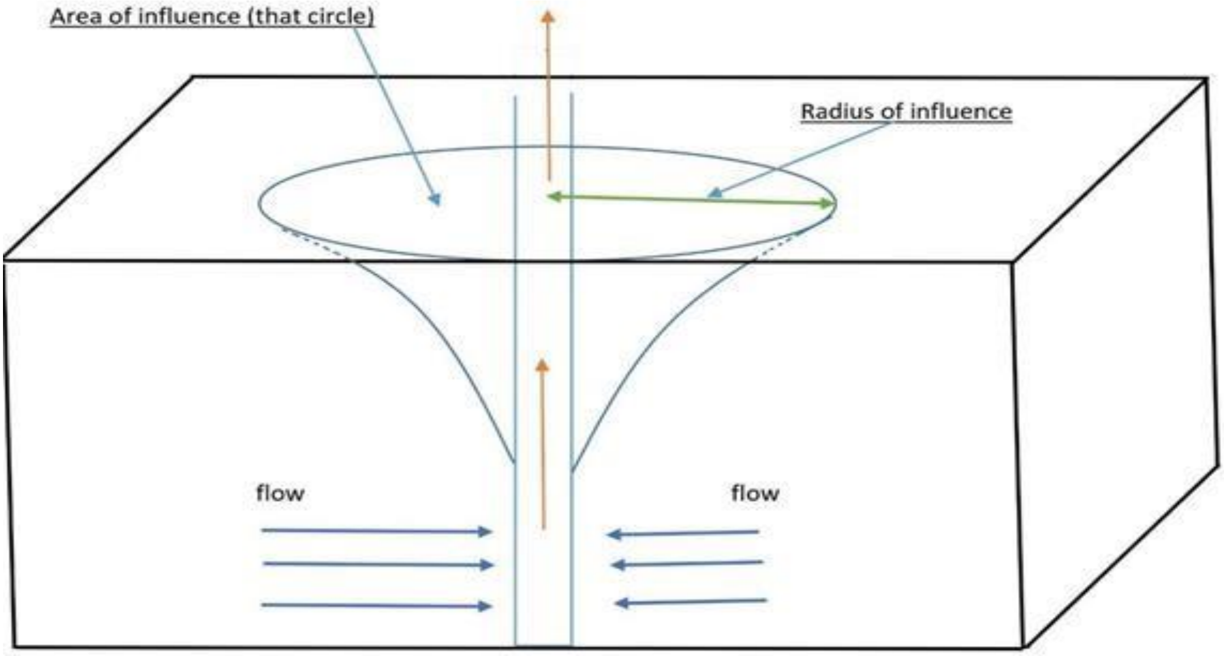


Figure Cone of depression in confined aquifer

2) Radius of influence

- It is the maximum distance up to the effect of drawdown is detected.
- In other word, radius of influence represent the radial extent of cone of depression. And areal extent represent by area of influence.

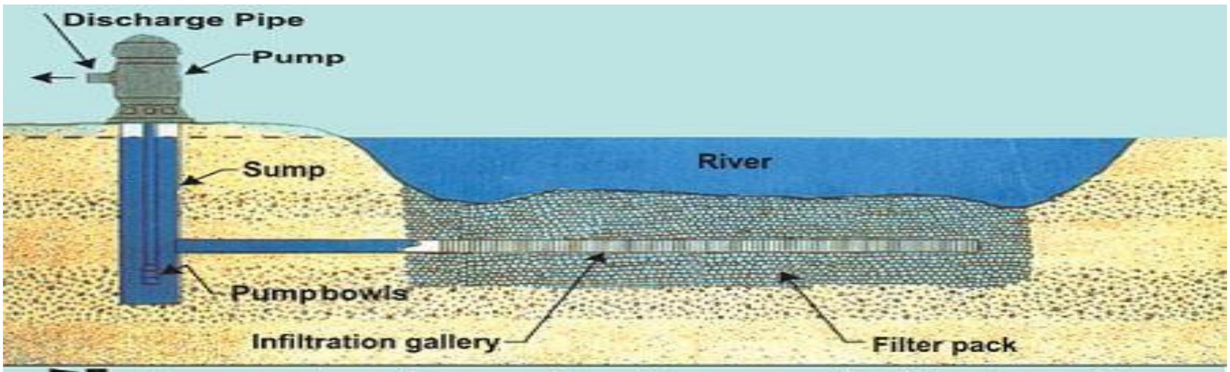


Note:

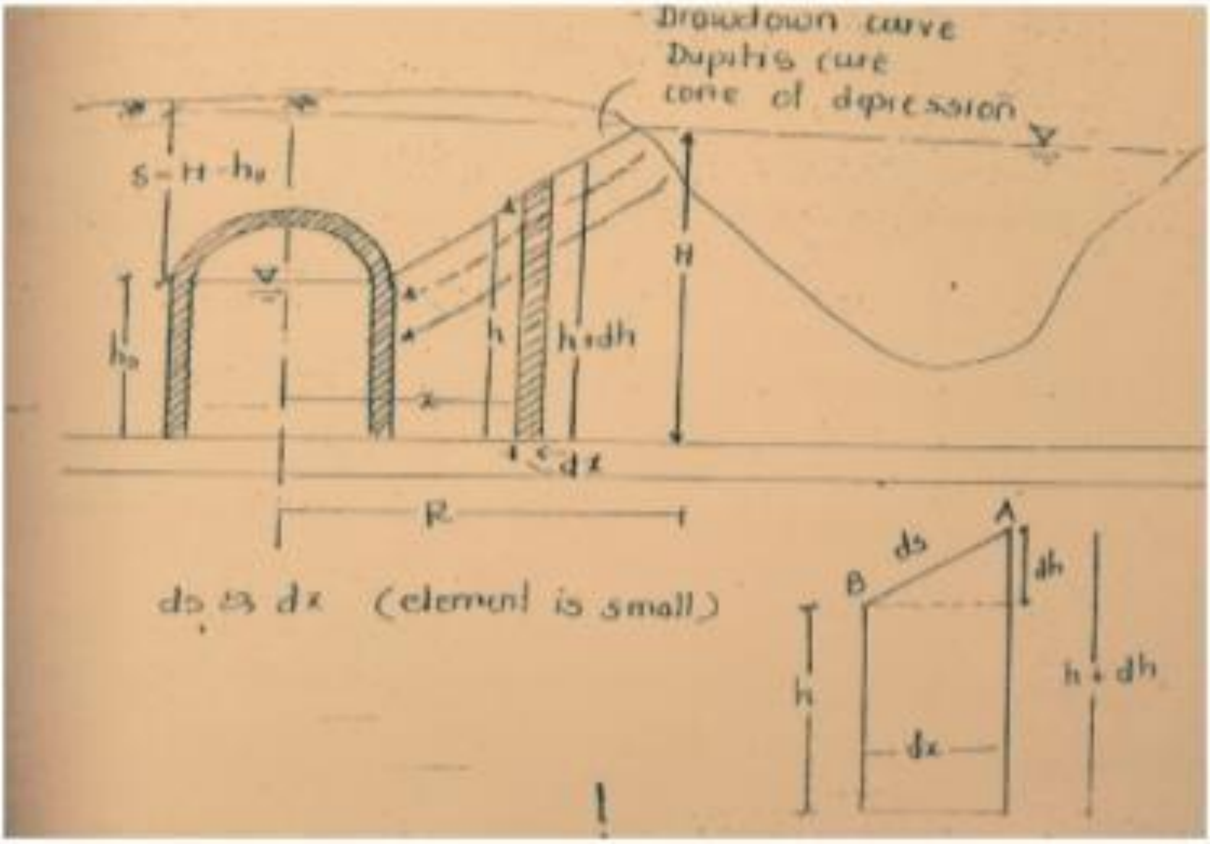
- When we start drawdown from well, initially the drawdown surface not constant and changes with time (due to unsteady flow). After sufficient time equilibrium state is reached and flow become steady.
- After attaining equilibrium state there is no change in drawdown surface, drawdown surface become constant with respect to time.
- And after stopping pumping, accumulation of water in influence zone started and this phenomenon termed as recuperation or recovery of well.

Different way of extracting water

1) Infiltration Galleries: These are horizontal tunnels constructed at shallow depth of the 3-5 m along the bank of river in water bearing strata.



Cross section of pump placed in sump of infiltration gallery.



Derivations:

Discharge through element

$q_x = a_x * v_x$

$= (h*L)*k*i_x$ (by Darcy's law $V = k*i_x$)

$= h*L*K*\ dh/dx$

Total Discharge

$Q = \int q_x$

$= \int h*L*k*dh/dx$

$$Q \int_{x=0}^{x=R} dx = k * L * \int_{h=h0}^{h=H} dh$$

$Q*R = k*L*\ [H^2 - h_0^2]$

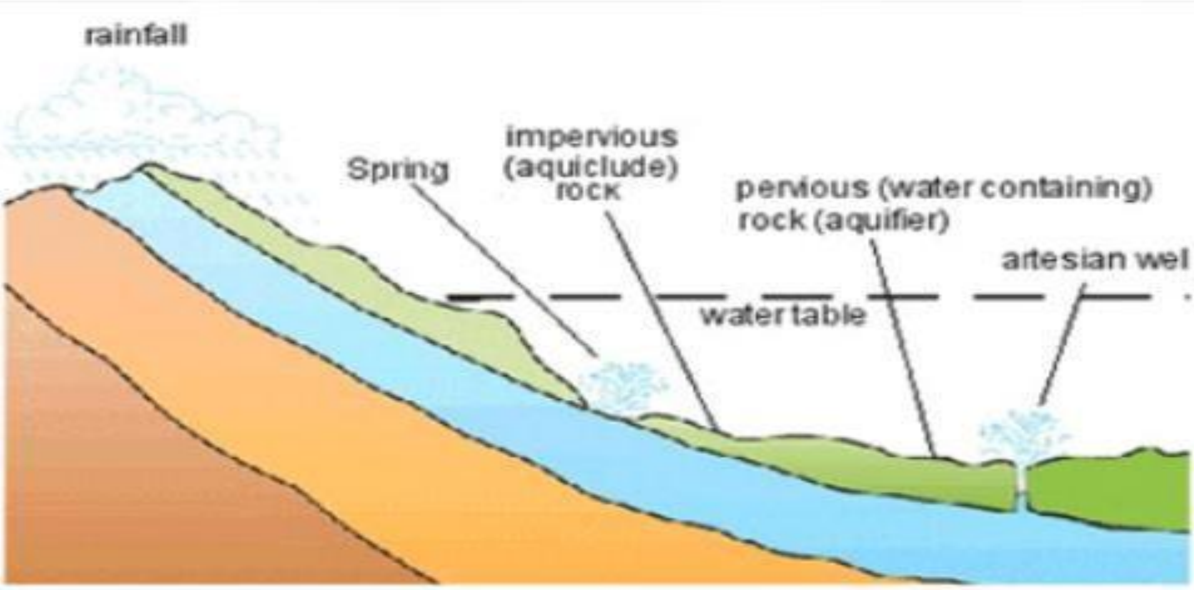
$Q = kL(H-h_0)(H+h_0)/(2R)$

2) Infiltration Well

- These are discontinuous structure constructed along bank of river in which water is collected through seepage from bottom.
- All such wells are connected through a common well known as Jack well from which water is pumped to treatment plant.

3) Artesian Spring

- Artesian spring have potential sources of raw water, while non-artesian spring are not potential sources. Because in summer water table may get depleted.



4) Well

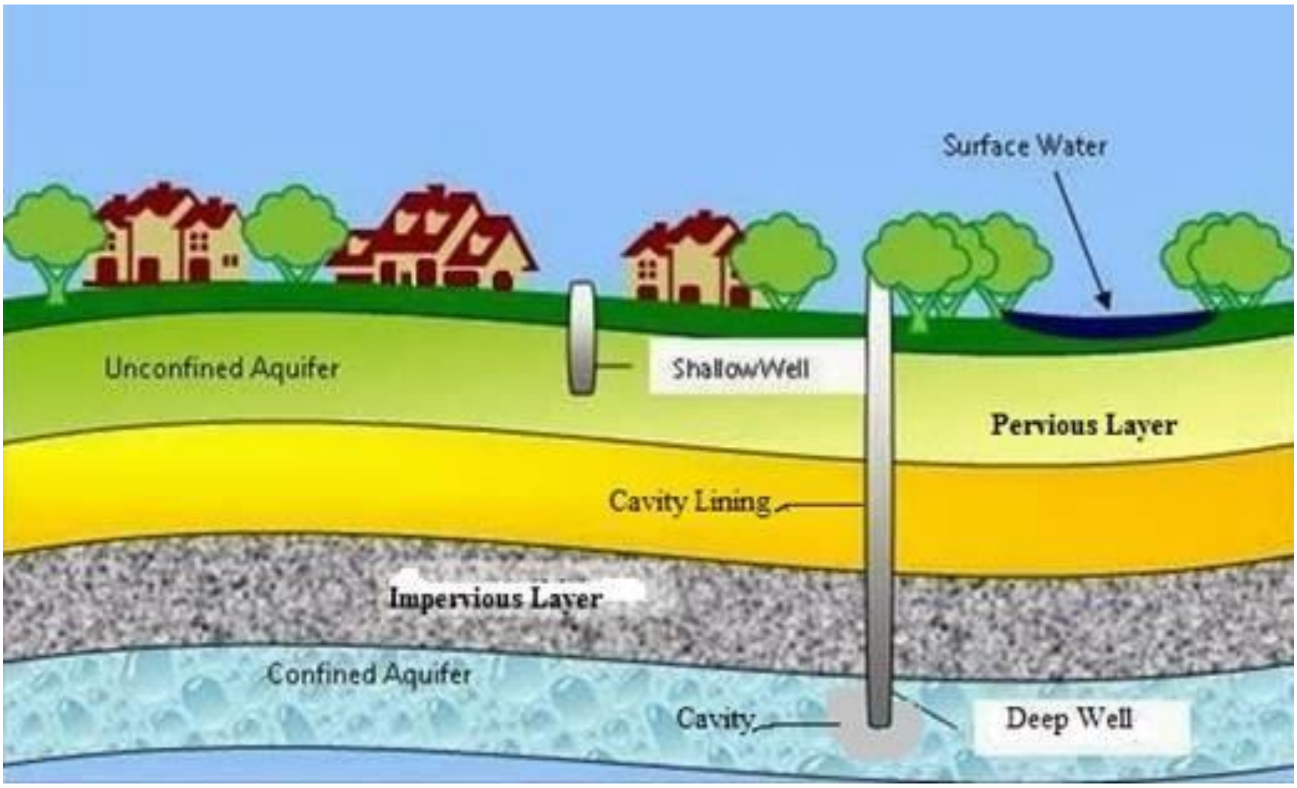
These are generally of two types

1. **Open Wells**
    a. Shallow well
    b. Tube well
2. **Tube Wells**
    a. Screen type
    b. Cavity type

1) Open Wells

- In **shallow well** water is drawn from top most water bearing strata, which is liable to be **contaminated**.
- Large quantity of water cannot be extracted from shallow well as with increase in discharge, velocity of flow through well increase and if this velocity exceeds critical velocity (velocity at which medium particles starts moving with flowing water settling velocity) leads to destabilisation of well lining and finally resulting in piping. This process is known as **"Piping"**. Sinking of well is consequences of piping.
- This problem does not occur in **deep well**, as with increase in discharge, when velocity through well increase resulting in the movement of medium particles from bottom of the well leading to increase area of flow from bottom. This process is known as cavity formation.

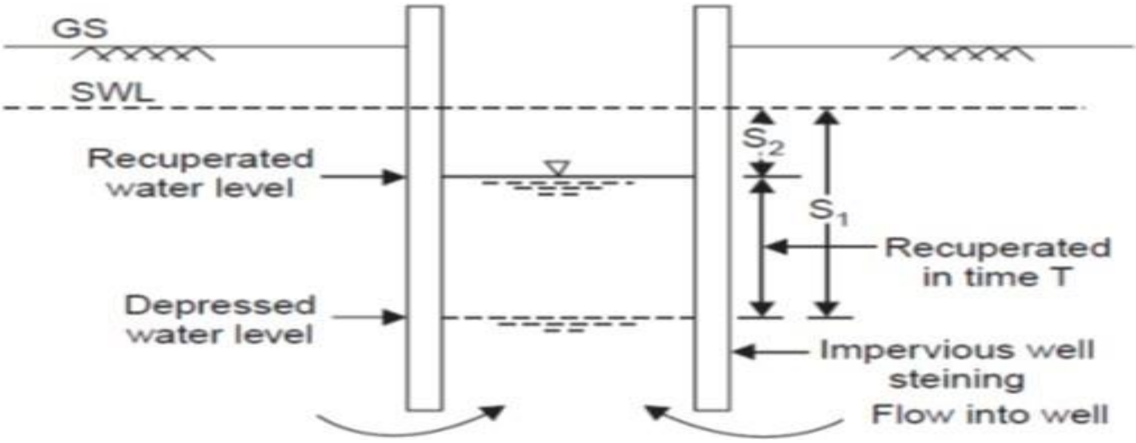
- Due to this increase area of flow velocity through the well again decrease which finally results in no movement of medium particles along with the water.
- In case of deep wells, **destabilisation of well lining** does not take place even after piping occurs as well lining is being supported by impervious layers.



Open Well Yield

Recuperating test

- Also known as equilibrium pumping test.
- This test is performed to get approximate yield from open well.
- In this test pumping is done up to working head (one third of critical depression head) for subsoil. Say S_1 Critical depression head- depression head at which loosening (quick sand phenomenon) of sand surrounding the well start.
- After that pumping is stopped and allowed to rise the water level in well or allow to recover the water head in well. And recuperation depth and corresponding time is noted for calculation of yield from well. Say recuperation depth = S, and corresponding time = T



Derivation Part-1

Let water level rises in well from s_1 to s_2 in T time

According to Darcy's law

"For laminar flow through saturated soil mass, the discharge per unit time is proportional to the hydraulic gradient".

Q = K.i.A ---------- (1)

i = Hydraulic gradient = s/L ------------ (2)

(Head s is lost in a length L of seepage path)

If ds is the water level rises in well in dt time than

Q dt = −A ds

Negative sign indicate the decrease in depression head with time during the recuperation of well.

From equation 1, 2 and 3

$$\frac{K.s.A}{L}dt = -Ads$$

Integrating them

$$\frac{K.A}{L}\int_0^T dt = \int_{S_1}^{S_2}\left(\frac{-ds}{s}\right)$$

$$\frac{K}{L} = \frac{2.303}{T}\log_{10}\frac{S_1}{S_2} --- (4)$$

Where $\frac{k}{L}$ a constant C and it is the specific yield of well. Dimension C is T^{-1}.

So equation 4 become

$$C = \frac{2.303}{T}\log_{10}\frac{S_1}{S_2}$$

Derivation Part-2

The yield of the well is

Q = CAH

Assumption: entire flow in well is from the bottom of well (impervious steining of masonry)

Where Q = safe yield of the well

A = area of cross section of the well

H = safe working depression head

C = specific yield of the well

Specific yield:-

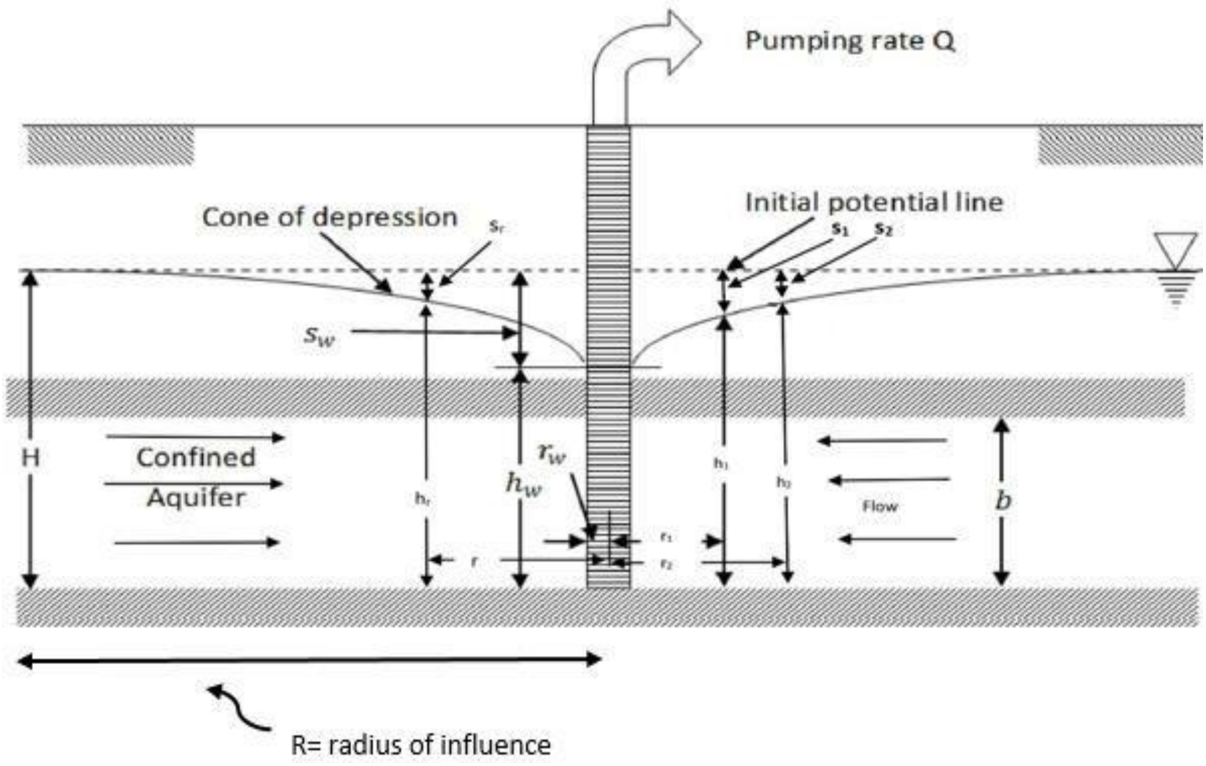
Specific yield soil is defined as discharge per unit area under a unit depression head (drawdown).

Steady flow into a well

Case-1: Well in Confined Aquifer. (Theim's theory)

Assumptions:

- Medium is assumed to be homogeneous and isotropic
- Flow of water in the vicinity of well is radial horizontal and laminar.
- The loss of head is directly proportional to tangent of hydraulic gradient (dh/dx) instead of (ds/dx) .



In this figure

r_w = radius of well, b = thickness of confined aquifer

s_w = Drawdown

h_w = Piezometric head at pumping well

H = original piezometric head or piezometric head before staring pumping

h_r, s_r = piezometric head and draw down of water table at distance r from centre of well

h_1, s_1 = piezometric head and draw down of water table at distance r_1 from centre of well

h_2, s_2 = piezometric head and draw down of water table at distance r_1 from centre of well

According to Darcy's low

Velocity of flow at radial distance r

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

Here $\frac{dh}{dr}$ is hydraulic gradient (dh is head loss over dr radial distance)

Discharge from well Q = V_r x A {A = cylindrical surface area through which water enter into well}

A= 2πrb

$$Q = \left(K\frac{dh}{dr}\right)(2\pi rb)$$

$$\frac{Q}{2\pi Kb}\frac{dr}{r} = dh$$

By integrating it

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

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

$$Q = \frac{2\pi Kb(h_2 - h_1)}{\ln\frac{r_2}{r_1}} \quad ...(5)\ (Thiem's\ equation)$$

if H is the original piezometric head, h_w = piezometric head at well,

R = Radius of influence, = $3000S_w\sqrt{K}$, K= Permeability of Soil,

r_w = radius of well

$$Q = \frac{2\pi Kb(H - h_w)}{\ln\frac{R}{r_w}} \quad (DUPIT'S\ equation)$$

Above equation represent the discharge from pumping well for steady flow condition.

s_1= H-h_1; s_2= H-h_2 -------------(6) (see above figure)

And T= Kb--------(7)

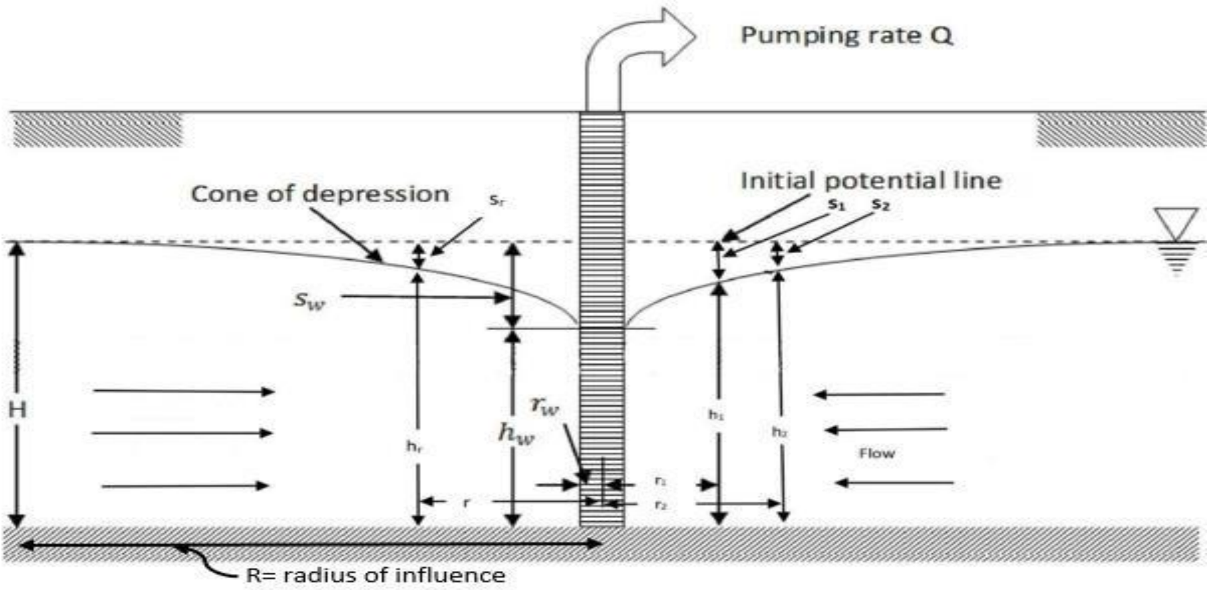
Note: T is **Transmissibility** and it defined as flow capacity or discharge of aquifer per unit width under unit hydraulic gradient. T has the dimension of [L^2/ T] .

From equation 5, 6 and 7

$$Q = \frac{2\pi T\left(s_1 - s_1\right)}{\ln\frac{r_2}{r_1}} \; ---(8)$$

Note: above equation is valid only for steady state flow condition and for well having complete penetration in aquifer

Case-1: well in unconfined aquifer.



In this figure

r_w = radius of well

s_w = Drawdown

h_w = Piezometric head at pumping well

H = original piezometric head or piezometric head before staring pumping

h_r, s_r = piezometric head and draw down of water table at distance r from centre of well

h_1, s_1 = piezometric head and draw down of water table at distance r_1 from centre of well

h_2, s_2 = piezometric head and draw down of water table at distance r_1 from centre of well

$$Q = \frac{\pi K}{\ln \frac{R}{r_w}} (H^2 - H_w^2) - - - (10)$$

Transmissibility T= Kb --------(13)

$$Q = \frac{\pi}{\ln \frac{R}{r_w}} (Ts_w) - - - (14)$$

$s_1 = H-h_1$; $s_2 = H-h_2$

Put the value of s_1 and s_2 in equation (9)

$$Q = \frac{2\pi T(s_1 - s_2)}{\ln \frac{r_2}{r_1}} (Theim's\ formula)$$

$$Q = \frac{\pi Kb(H2 - h2_w)}{\ln \frac{R}{r_w}} (DUPIT'S\ formula)$$

Note: above all equation of Q is valid only for steady state flow condition and for well having complete penetration in aquifer

Well losses and specific capacity

Head loss (drawdown) due to flow through soil pours, screen and in the well.

1) Drawdown (s_w) in a pumping well has three component like

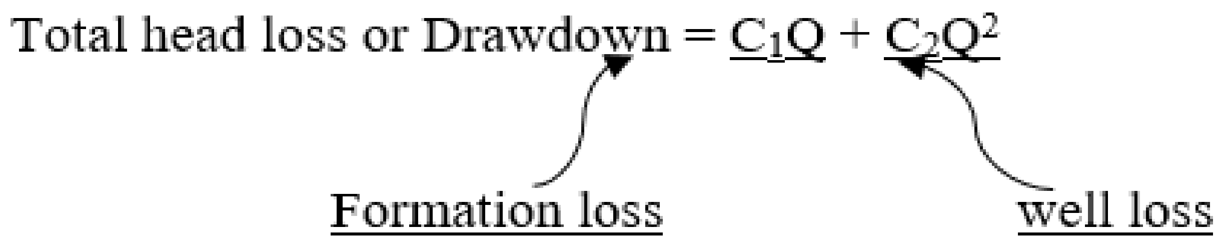
1) Formation loss- head loss due to flow through porous media. (s_{wL})

2) Head Loss due to turbulent flow near screen. (s_{wt})

3) Head loss due to flow through screen and casing. (s_{wc})

- Formation loss $s_{wL} \propto$ discharge Q
- $s_{wt} \propto Q^2$
- $s_{wc} \propto Q^2$

So



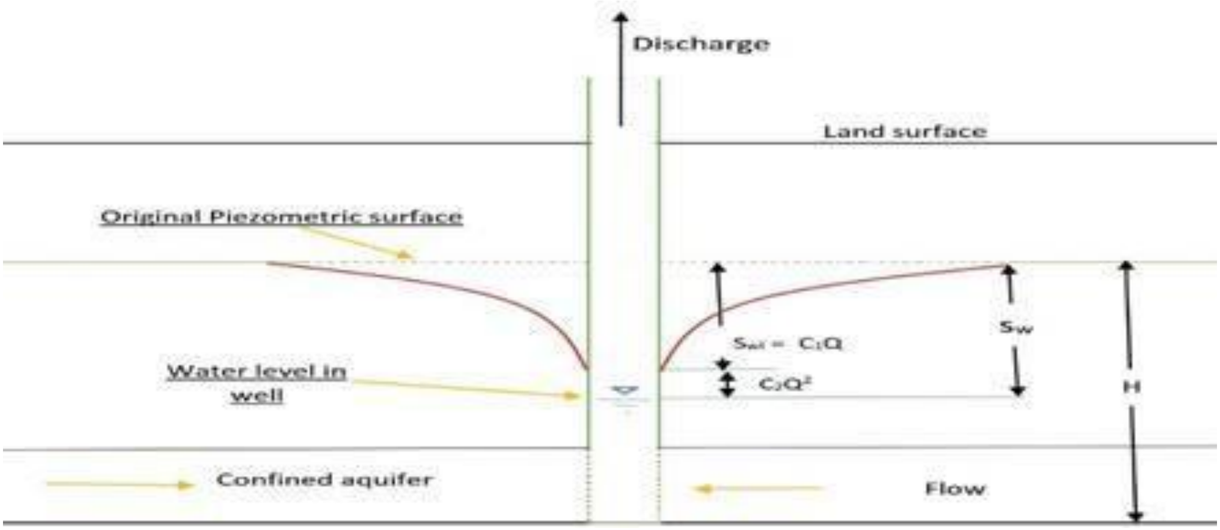


Figure well loss in confined aquifer

Specific capacity

- Discharge per unit drawdown known as specific capacity of well.
- Specific capacity indicates the performance of well.

So specific capacity (if well losses are ignored)

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

Example: ***(****from engineering hydrology by* ***k. subramanya)***

Given

1) Radius of pumping well r_w = 15 cm

2) Aquifer depth = 40 m

3) Steady state discharge = 500 lpm = $\frac{1500 * 10^{-3}}{60} = 0.025 m^3 / s$

4) Drawdown at two observation well

1. 25 m away from pumping well

r_1 = 25 m

s_1 = 3.5 m it means h_1 = (40.0 – 3.5) = 36.5 m

2. 75 m away from pumping well

r_2 = 75 m

s_2 = 2 m it means h_1 = (40.0 – 2) = 38 m

5) It is given that, the well is fully penetrated in aquifer.

Find

1. Transmissivity (T)
2. Drawdown at pumping well (s_w)

Solution:

1) Discharge from well

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

Put the known value in formula

$$Q = \frac{\pi K}{\ln \frac{75}{25}} \left\{ (38)^2 - (36.5)^2 \right\}$$

K = 7.823 x 10^{-5} m/s

We know that Transmissivity T = K x H

T = {7.823 x 10^{-5} m/s} {40 m}

T = 3.13 x 10^{-3} m^2/s

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

Put respective numerical values in above formula

$$0.025 = \frac{\pi * 7.823 x 10^{-5} m/s}{\ln \frac{25}{0.15}} \left((36.5)^2 - h_w^2 \right)$$

By solving that expression

h_w = 28.49 m and from that h_w, s_w = 40-28.49 = 11.51 m

Thank You

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