قسم الهندسة المدنية

المرحلة الثالثة

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Lecture - 1

Design of channels

Introduction

For carrying water from the canal head works to the field, a well-designed distribution system consisting of a network of canals is required. The canals taking off from a canal head works carry the water to far off place for various purposes, such as irrigation, hydropower and navigation.

The capacity of irrigation canals depends upon the water requirements of the corps and the area irrigated.

Canal can be defended as an artificial channel constructed on the ground to carry water from a river or another canal or a reservoir to the fields. Usually, canals have a trapezoidal cross-section.

Types of canals

The canals can be classified in to different types based on different criteria:

a) Classification based on size.
b) Classification based on canal surface.
c) Classification based on purpose
d) Classification based on alignment.

a) Classification based on size: based on the size, the canal can be divided in to the following types: -

1. Main canal.
2. Branch canal.
3. Major distributaries.
4. Minor distributaries.
5. Water course.

1- Main canal: main canal is the largest canal in the system. It takes off directly from canal headwork, which maybe a diversion headwork's, or storage headwork.

L.M.C أو R.M.C هناك أما

Sometimes there are two or more main canals on either side.

2- Branch canal: A branch canal takes off from a main canal or another branch canal. (carry discharge higher than $5m^3$/sec).

3- Major distributaries (0.25 to $5m^3$/sec).

4- Minor distributaries or minor (less 0.25 m^3/sec).

5- Water course (or filed channels or gulls): Watercourse are small channels which take water from a branch canal, a major distributaries or a minor distributaries and supply it to the agricultural fields.

B) Classification based on canal surface:

1. Lined canals.
2. Unlined canals.

1- Lined canals: A lined canal is the one which has it surface lined with an impervious material on its bed and sides to prevent seepage of water. Therefore, the seepage losses in a lined canal are small.

2- Unlined canals: An unlined canal is the one, which has the surface of the natural material though which it is constructed and it is not provided with a lining on its surface. The seepage losses are large.

c) Classification based on purpose: based on the purpose served:

1. Irrigation canal.
2. Power canal.
3. Navigation canal.
4. Water supply canal.
5. Feeder canal.
6. Carrier canal.
7. Multipurpose canal.

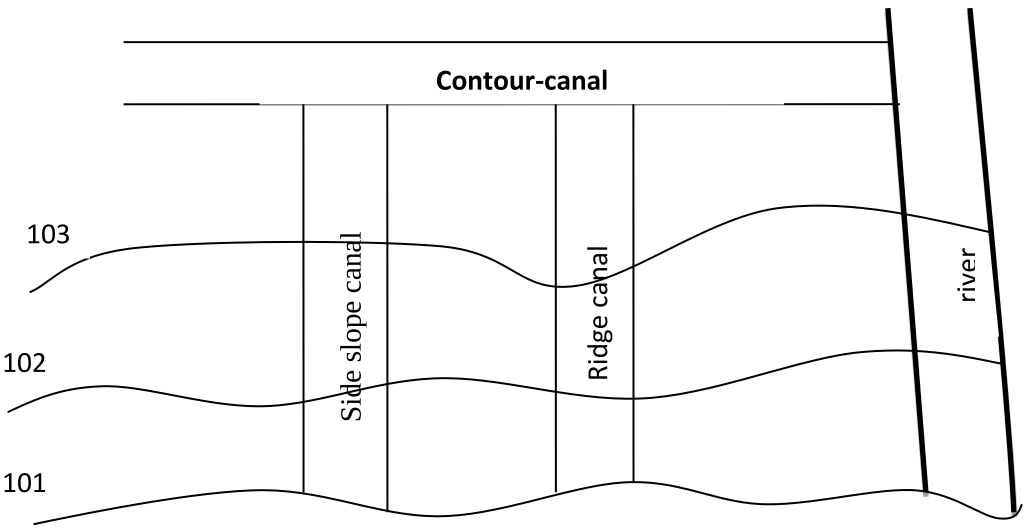
D) Classification based on alignment:

1. Watershed (or ridge canals).
2. Contour canals.
3. Side slope canals.

1- Water shed: the canal which is aligned along a watershed (or ridge) is called a watershed canal, it can irrigate on both sides of the ridge by gravity.

2- Contour canals: A contour canal is aligned almost parallel to the contours of area.

3- side-slope canals: a side-slope canal is aligned at right angles to the contour lines along the side slopes of the terrain.



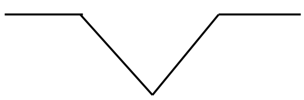
Design of channels

The design of the canal is mainly governed by the quantity of silt in the water and the type of boundary surface of the canal.

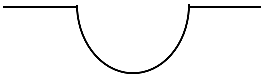
<u>Shape of cross-section of the canal.</u>

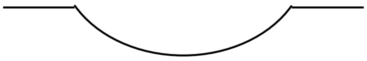
A. rectangle

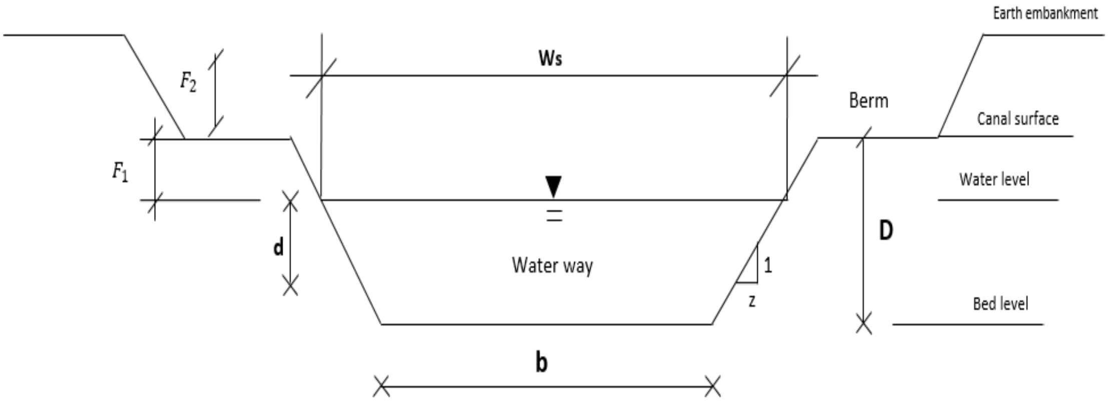
B. triangle



C. trapezoid 

D. semi-circle 

E. parabola 



Canal cross-section

b = bed width (m)

Ws = water surface width (m)

Y,d = water depth (m)

D = total depth of the canal (m)

1:z = side slope of the canal

F_1,F_2 = free board (m)

S = longitudinal slope

Water way: the part of the canal cross-section in which the water flow.

Berm: the area between the canal cross-section and the side embankment used as a road or for maintains.

F_1: the distance between the water surface and the canal surface used for the protection from overtopping (flooding).

F_2: the distance between the canal surface and the embankment level, used for protecting from flooding.

Basic design assumption (simplified assumption).

a. The flow is due to gravity.
b. The flow is uniform and steady state.

Lecture - 2

Depending upon the up factors, the irrigation channels can be broadly classified into the following types:

1. Rigid boundary channels.
2. Non- alluvial channels.
3. Alluvial channels.

1. In the rigid boundary channel, the surface of the cannels is lined .The quantity of silt transported by such channels remains more or less the same as that has entered the channel at its head. In such channels, relatively high velocity of flow is usually permitted which does not allow the silt to get deposited.

2. The non-alluvial channels are excavated in non-alluvial soils such as loam, clay, etc .Generally, there is no silt problem in these channels and they are relatively stable.

3. The alluvial channels are excavated in alluvial soils, such as silt. The silt content may increase due to scouring of bed and side of the channel.

Design of lined canals.

A lined canal is a rigid boundary channel. A lined canal decreases the seepage loss and, thus, reduces the chances of water logging.

A lined canal provides safety against breaches and prevents weed growth, reducing the annual maintenance cost of the canal. However, the only factor against lining is it cost.

Type of lining.

1. Concrete lining.
2. Precast concrete lining.
3. Brick lining, etc.

However, the maximum permissible velocity is relatively high. Table (1.1) gives the values of the maximum permissible velocity usually adopted in practice.

Lecture - 2

Table 1-1 maximum permissible velocity

No	TYPE of LINING	Max velocity(m/s)
1.	Boulder lining	1.5
2.	Brick tile lining	1.8
3.	Cement concrete lining	2.7

- Water logging when the pores of soil within the root zone of plant gets saturated and the normal growth of the plant is affect due to in sufficient air circulation .

Design By Trial And Error Solution.

The design of lined canal is usually done by Manning's formula. The value of Manning's coefficient (N) depends upon the type of lining. The higher values are for relatively rough surface and the lower, for smooth surface.

$$v = \frac{1}{n} R^{2/3} S^{1/2}$$

$Q = V * A$ $\qquad$ $R = \frac{A}{P} \quad (m)$

Where:

Q= max. discharge for a given (A) where the p is min. (m^3/s)

A: cross-sectional area (m^2) of flow.

P: wetted parameter (m).

S: longitudinal slope (m/m).

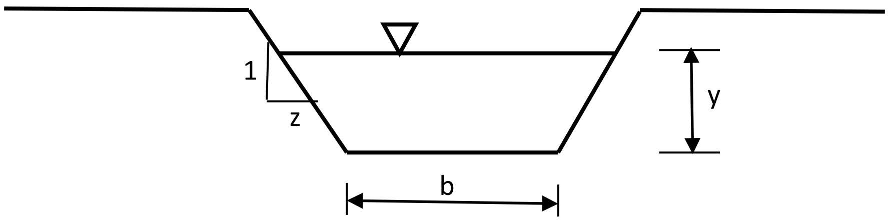
R: hydraulic radius (m). n: Manning's roughness coefficient.

n: 0.015 for lined canals with concrete.

For trapezoidal section

$A = (b + Zy) * y$ $\qquad$ *p=b+2y**$\sqrt{1+z^2}$

Lecture - 2



Cross-section of lined canals: for the most economical section, the hydraulic radius (R) should be a maximum. Theoretically, a semi- circular section is the best section for an open channel. However, it is not practicable to a depot this section. From the practical considerations, a channel of trapezoidal section or triangular section is usually selected. The corners of these sections are rounded to increase hydraulic radius

(a) <u>Side slope</u>

The side slopes depend on the properties of the material thorough which the channel is to pass.

Table (1-2) show Sui table side Slopes for channels excavated through different types of material

NO	*Material*	*Side slope (H:V)*
1	*Rock*	*N early vertical*
2	*Muck and Peat* سماد حيواني و فحم	*0.25:1*
3	*Stiff clay or earth with concrete lining*	*0.5:1 to 1:1*
4	*Earth with stone lining*	*1:1*
5	Firm clay	*1.5 : 1*
6	*Loose, sandy soil*	*2: 1*

Side slope for the canals should be assume 1V = 1.5 H

Note: Side slope for small lined canal, which has depth less than (0.7m), is taken (1:1) (for water course)

(b) Longitudinal Slope S

Assumption

- Uniform flow,
- longitudinal slope should be suitable to prevent the growth of grass and sedimentation of silt

$S_{\circ min}$ = 0.00015 $Q^{-0.2}$ ⎫ تؤخذ متوسط الناتج

$S_{\circ max}$ = 0.00025 $Q^{-0.2}$ ⎭

Q = full supply discharge in(m^3/s)Plus (10%)for over flow

Note: For water course the longitudinal slope should be (10-60 cm/km)

The slope of an irrigation canal is generally less than the ground slope in the head reaches of the canal, hence, vertical falls have often to be constricted. Power houses maybe constructed at these falls to generate power and, thus, irrigation canal can be used for power generation also.

(c) Minimum permissible velocity

- Velocity with sedimentation basin at the head of the system

$V_{min} = 0.33\, Q^{0.2}$ (m/s)

$$Fr = \frac{V}{\sqrt{gy}} < 0.6$$

Fr = Froude number.

- Velocity without sedimentation basin at the head of the system

$V_{min} = 0.5\, Q^{0.2}$ (m/s)

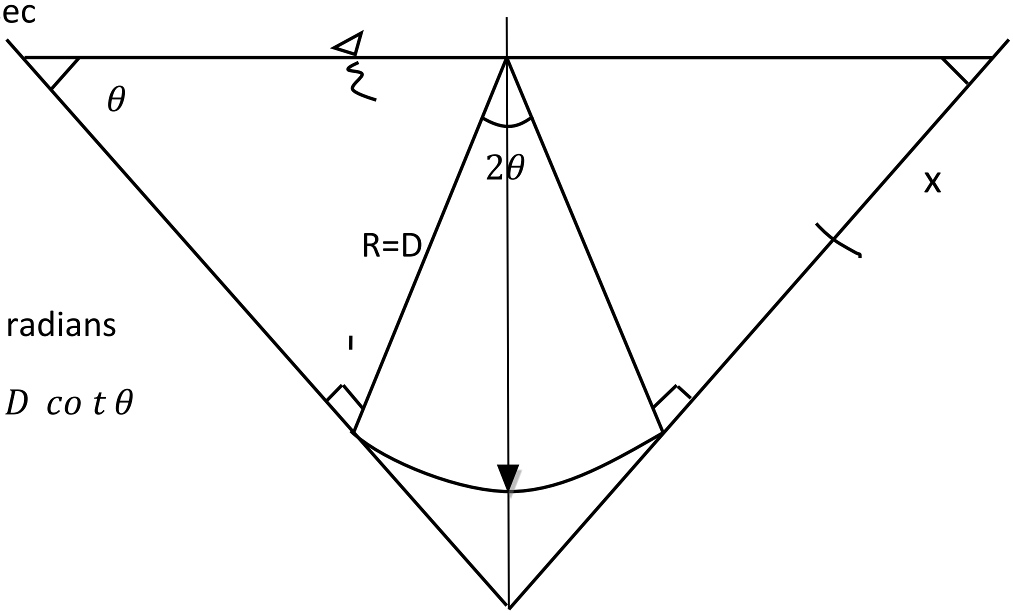
To avoid damage to the lining, the maximum velocity is restricted to (2m/sec). In general velocities of (0.7 -1 m/sec) will be adequate for prevent sedimentation as well as growth of vegetation if the sediment load is high.

(d) Bed-width and depth of water ratio (b/y)

$b/\mathrm{y} = (1-2)$ for discharges Less than (10m^3/s)

ملاحظات عامة

Triangular section is usually adopted for channels of discharge less than 50 m^3/sec



θ in radians

$X = D\ co\ t\,\theta$

The radius of the bottom is equal to the depth of water (D). The angle in the center is (2 θ)

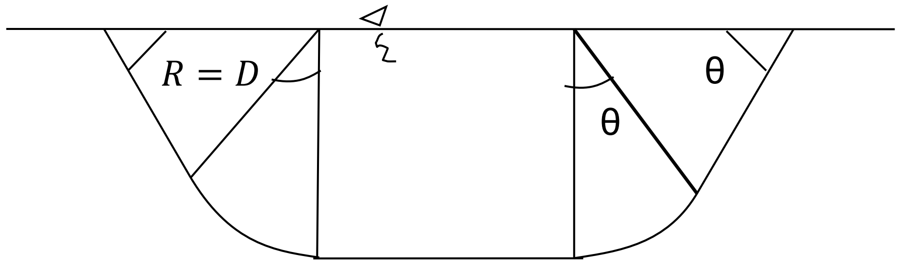
A= $(\pi D^2)\ (2\theta/2\pi) + 2(\frac{1}{2}D.\,Dcot\theta)$

A= $D^2(\theta + cot\theta)$

P= $2\pi D\ (2\theta/2\pi) + 2D\ cot\theta$

P= 2D $(\theta + cot\theta)$

Lecture - 2



The trapezoidal section for the lined canals with discharge greater than 50 m^3/sec.

A=BD+$\pi D^2(2\theta/2\pi) + 2\left(\frac{1}{2}\right) D.D\ cot\theta$

A= BD+$D^2(\theta + cot\theta)$

P=B+2D$(\theta + cot\theta)$

In the side slope is 1:1 $(\theta = 45^0 = \pi/4)$

A= BD+1.785D^2

P=B+3.57D

Ex: Design the cross- section of a concrete lined canal (trapezoidal section) to carry a discharge of 1.2 m^3/sec (By Manning Eq.)

Q=$\frac{1}{n}R^{2/3}S^{1/2}A$ $\qquad n = 0.015$

Assume 1:Z 1:1.5

S_{min} = 0.00015 $Q^{-0.2}$

$S_{min} = 1.446 \times 10^{-4}$

$S_{max} = 0.00025\ Q^{-0.2}$

$S_{max} = 2.41 \times 10^{-4}$

$S_{av.} = 1.928 * 10^{-4} \approx 2 * 10^{-4} = 0.0002$

Assume b=1m y=1m

$\therefore b/y = 1 \quad o.k \quad (1-2)$

$A = (b + zy)y \qquad = (1 + 1.5 \times 1) \times 1 = 2.5m^2$

P=b+2y $\sqrt{1+z^2} = 1 + 2 * 1 \sqrt{1.5^2 + 1} = 4.68m$

$R = \frac{A}{P} = 0.543\ m$

$\therefore Q = \frac{1}{0.015} * (0.543)^{\frac{2}{3}} (0.0002)^{\frac{1}{2}} * 2.5$

= $1.56 m^3/sec > 1.2 m^3/sec$

Assume b=1m y=0.9m b/y=1.111

$A = 2.115m^2$

$P = 4.242m$

: R=A/P =0.498m

V= 0.592 m/s check for velocity

$V_{min} = 0.5\ Q^{0.2}$

$V_{min} = 0.5\ (1.2)^{0.2}$

= 0.5186 m/sec

$\therefore V > V_{min} \qquad \text{o.k}$

$Q = V.A$

$= 0.592 \times 2.115$

= 1.252 m^3/sec $>$ 1.2 m^3/sec

Assume b=1m $y = 0.88m$ b/y=1.136

$A = 2.041m^2$

$P = 4.173m$

$R = 0.487m$

$V = 0.585 \quad > V_{min}$

Q= 1.194 m^3/sec $\approx$ $1.2\ m^3/sec$

Check Fr

$\text{Fr} = \frac{0.585}{\sqrt{9.81*0.88}} = 0.199 \ < 0.6$ o.k

Ex: Design a lined canal to carry discharge of 50 m^3. Assume bed slope as 1 in 8100, N as 0.015 and side slope as 45°

<u>Sol:</u> let us adopt a triangular section for $\theta = {}^{\pi}/_{4}$

A= 1.785 D^2

P= 3.570 D

Q=V.A $= \frac{1}{n}\ A.S^{1/2}R^{2/3}$

$50 = \frac{1}{0.015}\ (1.785\ D^2)\ .\ (\frac{1}{8100})^{\frac{1}{2}}\ .\ (\frac{1.785D^2}{3.570\ D})^{\frac{2}{3}}$

50 = 0.833 $D^{8/3}$ $\rightarrow$D=4.64m

Design by Section Factor Method by using the chart

1. Determine the value of (A $R^{2/3}$ / $B^{8/3}$).

2. Determine z.

3. Compute b/y.

Ex: Design the cross-section of a concrete lined canal for a discharge of 1.8m^3/sec, on slope of 20 cm/km by using section factor method.

Solution

Assume 1:Z 1:1.5 for lined canal

$Q = \frac{1}{n} R^{2/3}$ A $S^{1/2}$

$$\frac{Qn}{\sqrt{s}} = A\ R^{2/3}$$

$$\frac{1.8 * \ 0.015}{\sqrt{0.0002}} = 1.91 \ \approx 2$$

2= A $R^{2/3}$

Assume b = 1m $\therefore\ (AR\text{^}(2/3))/b\text{^}(8/3)\ = 2$

Z=1.5 $\longrightarrow$ from fig $\frac{b}{y} = 1$

$\therefore y = 1m \quad check\ for \quad Fr \quad no$.

$A = (1 + 1.5 \times 1) \times 1 = 2.5m^2$

$V_{min} = 0.5\ Q^{0.2} = 0.562\ m/s$ [Without sedimentation basin]

$$V = \frac{Q}{A} = \frac{1.8}{2.5} = 0.72\ m/\text{s} > V_{min}$$

$$Fr = \frac{V}{\sqrt{gy}} = \frac{0.72}{\sqrt{1 * 9.81}} = 0.23 < 0.6 \qquad o.k$$

Design of unlined canal

- Design by Manning Equation

$$Q = \frac{1}{n} R^{2/3} S^{1/2}\ A$$

(a) Side slope

Side slope in an un lined canal depend mainly on the nature of geological formations through which the canal is excavated. Side slopes in an un lined canal should be flatter than the angle of repose of saturated bank soil.

Initially, flatter slopes are provided for reasons of stability. Later, with the deposition of fine sediments, this side slope become steeper

and attain a value of (0.5 H: 1V) irrespective of the initial side slope provided. These steeper side slopes are stable and the design is usually based on these slopes. Table below show the side slopes for un lined canals in different types of soil.

Use (1.5 to 2 H): 1V For Large canal

for water course use (1.5 H:1V)

Type of soil	H:V
Loose sand to average sandy soil	1.5:1 to2:1(in cutting) 2:1 to 3:1 (in filling)
sandy loam and black cotton soil	1:1 to 1.5:1 (in cutting) 2:1 (in filling)
Gravel	1:1 to 2:1
Murum or hard soil	0.75:1 to 1.5:1
Rock	0.25:1 to 0.5:1

Lecture - 2

<u>(b) Bed width and water depth ratio (b/y)</u>

$$\frac{b}{y} = (2-3) for\ discharege\ less\ than\ (10m^3/s)$$

*b=0.4m usually for watercourse.

$y = 0.75\ Q^{0.33}$

Where

Q: Design discharge (m^3/s)

(c) Permissible velocities

$V\ min = C_2\ y^{0.64}$

Where:

V min: minimum permissible velocity to prevent sediment depositions (m/s)

C_2: constant

Type of suspended material	C_2
- Light loam and very fine sand	0.4
- Find sand (Dia.=0.4mm)	0.55
-Moderate Coarser sand	0.63
- Coarser Sand	0.67
- Very Coarser Sand	0.90

<u>(d) Maximum permissible Velocity</u>

The design of the canal should be to prevent scouring, there by, the design should be based on the concept of tractive force. Scour on a channel bed occurs, when the tractivc force on the bed exerted by the flow is adequate to cause the movement of the bed particles. If the tractive force acting on the bed or the resultant of the tractive force and the component of the gravitational force both acting on the side slopes is larger than the force resisting the movement of the particles erosion starts.

i- Empirical equation

V max = $C_1\ y^{0.64}$

Type of bed material C_1

Fine , light sandy loam	0.55
Coarser , light sandy loam	0.60
sandy , loamy silt	0.66
coarser silt	0.71

ii- Tractive force method

tractive force $\tau_{\circ}$ is calculated and it should be less thanτ_c(the max. permissible tractive force, critical shear stress) depending on bed material, maximum shear stress on the bed.

In uniform flow the average tractive stress, $\tau_{\circ}$ is given as:

$$\tau_{\circ} = w\ .R\ .S \qquad KN/m^2$$

$where$:

w: density of water (ρg) (KN/m^3)

R: the hydraulic radius (m)

S: slope of water surface (m/m)

If $\tau_{\circ} < \tau_c$ the bed will not scour

τ_c *اما تعطى او من جداول خاصة*

Lecture - 2

Serval investigators have given the expression for the critical tractive stress. Some of the commonly used expressions are given below:

1. **Shield's equation** According to shield, the critical stress (τ_c) is operational to gain diameter and the submerged unit weight of the sediment, and is given by

$$\tau_c = 0.06\, w\, (G - 1)\, d$$

Where τ_c is the critical share stress (KN/m^2). w is the specific weight of water (KN/m^2). *G* is the specific gravity of the sediment and *d* is the gain diameter (m).

Taking $G = 2.65\ and\ w = 9.81\ \ KN/m^2.\ Eq. 22.42\ becomes$

$$\tau_c = 0.98\, d$$

2. **White equation** According to white. $\tau_c = 0.801\, d$

Where *d* is the grain diameter (m). and τ_c in KN/m^2.

3. **Lane's equation** lane gave the following equation. $\tau_c = 0.78\, d$

All the above equation are for fully-developed turbulent flow.

Lecture - 3

Ex: Design the cross-section for unlined canal to carry a discharge of $1.6m^3/s$, if the longitudinal slope of the canal is 30cm/km, water contain light loamy suspended material and the bad material is a loamy silt. (n=0.025, max.tractive force is 2.8 N/m^2).

Sol:

By trial and error solution with Manning equation

$Q=1/n\ R^{2/3}\ S^{1/2}\ A$

$Q=1.6m^3/s$, n=0.025, S=30cm/km=0.0003

Assume side slope 1:Z, 1:2 for unlined canal

B/y=2-3

Assume b=2m, y=1m ⟶ b/y=2 (2-3) o.k

$A = (b + zy)y \rightarrow A = (2 + 2 \times 01) \times 1 = 4m^2$

$P = b + 2y\,(1 + z^2)^{0.5} = 6.472m$

$R = A/P = 0.618m$

$Q = 1/0.025\ (0.0003)^{0.5}\ (4)\ (0.618)^{2/3}$

$Q = 2.011m^3/s > 1.6m^3/s$

$Assume\ b = 2, m\ y = 0.9m \rightarrow b/y = 2.222\ (2 - 3)\ o.k.$

$A = (2 + 2 \times 0.9)0.9 = 3.42m^2$

$P = 2 + 2 \times 0.9\sqrt{4 + 1} = 6.025m$

$R = 0.568m$

$Q = 1/0.025 \times (0.0003)0.5(3.42)\ (0.568)2/3$

$Q = 1.625m^3/s \approx 1.6m^3/s\ (\mp 4\%)\ o.k.$

Check for velocity:

$V = Q/A = 1.6/3.42 = 0.468m/s$

Lecture - 3

$Vmin\ = C_2 y^{0.64}\ .from\ the\ table\ C_2 = 0.4$

$Vmin = 0.374 m/s$

$Vmax\ = c_1 y^{0.64}\ .from\ table\ C_1 = 0.66$

$Vmax. = 0.617 m/s$

$Vmine < V < Vmax$

0.374 < 0.468 < 0.617

المقطع مناسب لمقاومة الترسيب والتعرية وإمرار التصريف

*Check for scouring by attractive force method.

$\tau = w * R * s = 9.81 * 1000 * 0.568 * 0.0003$

$\tau = 1.671\ N/m^2$

$\tau < \tau_c$ The bed will not scour.

Design values b=2m, y=0.9m

Lecture - 3

Design by lacey's method

Assumption:

a• the channel flow is uniform.

b•the characteristics and the discharge of the sediment are constant.

c• the water discharge in the channel is constant.

Lacey's eq:

$Dm=2.46V^2/F$

$Ws=4.83\ e\ Q^{1/2}$

$$S = 0.0003\ f^{5/3}\ e^{1/3}\ \frac{E}{Q^{1/6}}$$

$$f = 1.76\sqrt{d}$$

Where:

Dm=mean depth (m).

V=mean velocity (m/s).

F=silt factor (to account the size and density of sediment)

e= width factor or reduction factored, d=median size of sediment.

Ws=water surface width (m).

E=wetted parameter per water surface width (P/Ws).

Q=discharge (m^3/s).

S=water surface slope (m/m).

$$b = 0.8\ Ws$$

Dm=A/Ws

Lecture - 3

*for main and branch canal e=1

*for distributaries with discharge (1-2m^3/s), (e=0.75-0.85)

*sill factor f=0.4-1, f=1.0 for large discharge.

ويعتمد على قطر الحبيبات المرسبة وكثافة المادة المرسبة

f=0.7-1 at north Iraq

f=0.6 at middle of Iraq

f=0.5 south of Iraq

Ex: Design a stable channel for carrying a discharge of 1m^3/s using Lacey's Method assuming silt factor equal to 0.75.

Sol:

$Assume \quad e = 0.75$

$Dm = 2.46\, V^2 \quad ; \; Dm = 3.28V^2$

$Dm = A/Ws \rightarrow \; 3.28V^2 = A/Ws \; \rightarrow \; A = 3.28V^2 Ws$

$Ws = 4.83 \; e\, Q^{1/2}$

$Ws = 4.83(0.75)\,(1)^{1/2} = 3.622m$

$A = 3.28V^2\,(3.622)$

$A = 11.882V^2$

$Q = AV \; \rightarrow \; V = \dfrac{Q}{A} = \dfrac{1}{A}$

$A = 11.882/A^2$

A^3=11.882 $\longrightarrow$ A=2.282m^2

b=0.8 Ws=0.8×3.662

b=2.9m

Lecture - 3

$$A = \left(\frac{b + Ws}{2}\right) y$$

2.282= (2.9+3.622/2)/y

y=0.7m

S=0.0003 $f^{5/3}$ $e^{1/3}$ E/$Q^{1/6}$

E=P/Ws,P=4.475m

E=4.475/3.622=1.24

S=0.0003$(0.75)^{1/3}(0.75)^{5/3}\{1.24/(1)^{1/6}\}$=0.000209.

S= 20cm/km

KENNEDY SILT METHOD

Kenned'y collected data from 22 channels of upper Bari Doab canal system in Punjab. His observation let to following relation .known as Kennedy's equation

$V_{\circ} = 0.55\ h^{0.64}$

Where:

$V_{\circ}$: The critical velocity (m/sec) which defined as the mean velocity

Which will not allow scouring or silting in channel having depth to h (m).

This eq.is, obviously applicable to such channels which have the same

Type of sediment as was present in the upper Bari Doab canal system.

On recognizing the effect of the sediment size on the critical velocity, Kennedys modified the above equation to:

V =0.55 m $h^{0.64}$

Where:

(m) is the critical velocity ratio and is equal to m $\frac{v}{V_{\circ}}$.here. The velocity (V) is the critical velocity for the relevant size of sediment of any other silt grade while ($V_{\circ}$) is the critical velocity for the upper Bari Doab sediment. This means that the value of (m) is unity for sediment of the size of Upper Bari Doab sediment. For sediment coarser is greater than one, while for sediment finer, m is less than one. Kenned'y did not try to establish any other relationship for the slope of the regime channels in terms of either the critical velocity or the depth of the flow. He suggested the use of the Kutter's eq. along the manning roughness coefficient. The final results do not differ much if one uses the manning eq. instead of the Kutter's eq.

Kutter's equation

$$V = \left(\frac{\frac{1}{n} + (23 + \frac{0.00155}{s}}{1 + \frac{(23 + \frac{0.00155}{s}}{n\sqrt{R}}} \right) \sqrt{RS}$$

NOTE: this critical velocity should be distinguished from the critical velocity of flow in open canal corresponding to froud no. equal to unity. Table (1-5) show values of critical velocity ratio (m).

No	silt grade	CVR
1	light sandy silt, as in Upper Bari Do	1.0
2	coarse light sandy soil	1.1
3	sandy loam	1.2
4	coarse silt or debris of hard soil	1.3
5	silt of river indus	0.77
6	silt of river nile	0.68

CVR---CRITICAL VELOCITY RATIO.

Design of channels by kennedy's theory

The design procedure will depend whther the bed slope (S) is given or the (B/D) ratio is given. The center water commission, new Delhiloy (B/D) ratio for canals carrying discharge ranging from 0.3 to 300 m^3/sec we can use a useful chart of this method or for canals up to adischarge of 15 m^3/s the following empirical formula is also somtimes used (D=0.5$\sqrt{B}$) for alluvia canal.

Lecture - 3

Design procedure when the bed slope is given (given Q ,m,n and S).

Steps:

1. assume a trial value of the depth (h).

2. calculate the velocity from ($v=0.5m\ h^{0.64}$).

3. determine the cross sectional area($A=Q/V$)

4. assume aside slope of (1H:2V), (1:0.5) ,and determine the width B from the relation. eqs, $A=BD+0.5D^2$

5. calculate the actual mean velocity (v) from kutter's or mannings eqs.

IF the value of (v) is nearly the same as that found from kennedy's eq. the assumed depth is correct . if not, the procedure is repeated after assuming anther value of (h) till the two values of velovity are approximately equal.

Ex: Design achannel carrying adischarge of $30m^3/sec$ with critical ratio andd mannings (n) equal to 1.0 and 0.0225 , respectively. The bed slope is equl to 1 in 5000.

Sol: kennedy's method

Assume h=2.0m

$V=0.55m\ h^{0.64} = 0.55x1x(2)^{0.64} = 0.857m/s$

$A=Q/V=30/0.857=35.01m^2$

For a trapezoidal canal with side slope 1H:2V , $A=bh+h^2/2$

$35.01=2b+2 \longrightarrow b=16.51m$

$R=A/P$, $P=16.51+2x2\sqrt{1+0.5^2}=1.67m$

$V=1/n\ R^{2/3}\ S^{1/2} =0.885m/s$

Lecture - 3

Since the velocities obtained from the kennedy's equation and manning's equation are appreciably different , assume h=2.25m ,

V=0.924 m/s , A=32.47m^2

B= 13.31m , R=1.77m , V= 0.92m/s ≈ 0.924m/s

Design procedure when the B/D ratio is given (given Q,M,N and B/D ration)

1. calculate the area (A) in terms of (D) as followed

$A=BD+0.5D^2=D^2(B/D+0.5)$

Or $A=D^2(X+0.5)$

Where → X = B/D ratio

2. the cotinuity eq . and substitute kenndys eq . for the velocity.
Thuse → $Q=V.\ A=D^2(X+0.5)\ (0.55m\ D^{0.64})$

3. calculate the value of (D) from the above eq.

4. determine thr bed width B=XD

5. compute $\mathrm{R} = \mathrm{A}/\mathrm{P}$

6. determine the velocity (V)

7. compute the slope from Kutter's or Manning's eq.

The design of non-alluvial is usually don by Chezy's equation or Manning's formula.

Chezy's equation $V = C\sqrt{R\ S}$

Where C is Chezy's coefficient. The value of Chezy's coefficient is usually determined from Bazin's equation.

Lecture - 3

$$C = \frac{87}{1 + \frac{K}{\sqrt{R}}}$$

Where K is Bazin's coefficient, which depends upon the surface of the channel. R is hydraulic radius and S is the longitudinal slope.

Channel in which silting problems are anticipated should be designed to have some minimum permissible velocity or the non-silting velocity. However, this velocity is very uncertain and can be determined only by advanced theories of sediments transport. The minimum velocity of 0-5 m/s is usually taken in other words, the velocity should on be le than 5.5 m/s.

Procedure the following procedure is used for design of non-alluvial channel by Manning's formula. Similar procedure can be used for design by Chezy's equation.

Given the discharge (Q), the maximum permissible velocity (*V).*
Manning's *N.* bed slope*(S)* and the side slope (r : 1) are given or have been assumed.

Steps:

1. Determine the area of cross-section from the continuity equation:

   $Q = AV$ Or $A = Q/V$

2. *Determine hydraulic radius R from the Manning formula.*

$$V = \frac{1}{N} R^{2/3} \; S^{1/2} \quad or \quad R = \left(\frac{VN}{S^{1/2}}\right)^{3/2}$$

3. Determine the wetted perimeter from the relation. P = A/R
4. Determine the depth D and bed width B from the values of A and P obtained from Eqs. (a) and (c) by solving the equations below.

$$(B + rD)\, D = A$$

$$B + \left(2\sqrt{1 + r^2}\right)\ D = p$$

Example

Design an irrigation channel in a non-alluvial material to carry a discharge of 15 cumecs when the maximum permissible velocity is 0.8 m/s. Assume bed slope = 1 in 4000, side slope 1:1 and Manning's N = 0.025

Solution A = Q/V = 15/0.8 = 18.75 m^2-

$$R = \left(\frac{VN}{S^{1/2}}\right)^{3/2} = 1.42\ m$$

$$P = \frac{A}{R} = 18.75/1.42 = 13.20\ m$$

Now $(B + D)\ D = 18.75$

And $B + \left(2h\sqrt{1 + 2^2}\right) D = 13.20 \quad or \quad B + 2.828\ D = 13.20$

Substituting the value of B from Eq. (b) in Eq. (a).

$[(13.20 - 2.828\ D) + D]\ D = 18.75$ Or D=1.95 m

Now $B = 13.20 - 2.828 \times 1.95 = 7.69\ m$

Example

An earthen channel in good condition carries a discharge of 10.0 cumecs

with a mean velocity of 0.7 m/s. determine the bed slope. Assume the bottom width as twice the depth. Take Bazin's coefficient as 1.30 and side slope as 1.5:1

B=2D

Solution

A = Q/V = *10/0.7 =14.30 m*^{2}

Now $(2DB + 1.5\ D)D = A = 14.30 \quad or \quad 3.5D^2 = 14.30$

Or $D = 2.02\ m. B = 4.04\ m$

Now $P = B + \left(2\sqrt{1 + (1.5)^2}\right) \ d = 4.04 + 728 = 11.32\ m$

$R = A/P \qquad = 14.30/11.32 = 1.26\ m$

$$C = \frac{87}{1 + K\sqrt{R}} = \frac{87}{1 + 1.30\sqrt{1.26}} = 40.32$$

Now $V = C\sqrt{RS}$

Or $0.7 = 40.34\ \sqrt{1.26 \times S} \quad or \ \ S = 1\ in\ 4185.$

Lecture - 4

Ranga Raju and Misri's simplified method

They also assume the side slope of 1:0.5

$$A = BD + 0.5D^2 = D^2(B/D + 0.5)$$

$$A = D^2(X + 0..5)$$

Where X is equal to B/D ratio

$$P = B + 2D\sqrt{1 + 0.5^2}$$

$$= D\ (X + 2.236)$$

$$R = \frac{A}{P} \qquad = \frac{D(X + 0.5)}{X + 2.236}$$

$$V = \frac{Q}{A} = \frac{Q}{D^2(X + 0.5)}$$

$$V = \frac{1}{n} R^{2/3} S^{1/2} \quad or \quad \frac{Q}{A} = \frac{1}{n}\ R^{2/3} S^{1/2}$$

$$\left(\frac{Q}{A}\right)^2 = \frac{1}{n^2}\ R^{4/3} S$$

$$S = \frac{Q^2 n^2}{A^2 R^{4/3}} = \frac{Q^2 n^2 [(X + 2.236)]^{4/3}}{[D^2(X + 0.5)]^{10/3}}$$

$$S = \frac{Q^2 n^2 (X + 2.236)^{4/3}}{D^{16/3}(X + 0.5)^{10/3}}$$

From Kennedy's Eq.

$$Q = V.A = 0.55\ m\ \ D^{0.64} \quad .(D^2(X + 0.5))$$

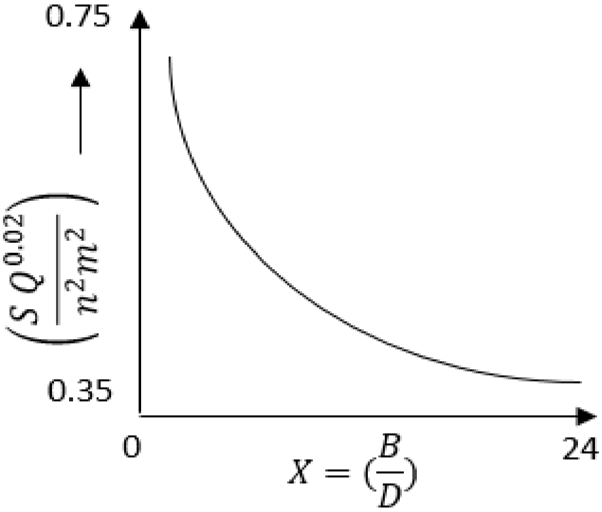
Or

$$D = \left[\frac{Q}{0.55\ m\ (X + 0.5)}\right]^{\frac{1}{2.64}} = \left[\frac{1.818\ Q}{m\ (X + 0.5)}\right]^{\frac{1}{2.64}}$$

$$S = \frac{Q^2 n^2 (X + 2.236)^{4/3}}{\left[\frac{1.818\,Q}{m\,(X + 0.5)}\right]^{2.02} (X + 0.5)^{10/3}}$$

$$\frac{S\,Q^{0.02}}{n^2 m^2} = 0.299 \quad \frac{(X + 2.236)^{4/3}}{(X + 0.5)^{1.313}}$$

:. For the given values of S, Q, n and m, the value of X can be found by trial and error or by using the fig.



Exercise: Design a canal carrying a discharge of 25 m^3 /s by Ranga Raju and Misri's method assume m = 1.0, n = 0.0255 and S = 1/5000

Solution

$$\frac{S\,Q^{0.02}}{n^2 m^2} = 0.299 \quad \frac{(X + 2.236)^{4/3}}{(X + 0.5)^{1.313}}$$

$$\frac{2 \times 10^{-4} \times (25)^{0.02}}{(0.0225)^2 (1)^2} = 0.299 \frac{(X + 2.236)^{4/3}}{(X + 0.5)^{1.313}}$$

$$0.421 = 0.299 \ \frac{(X + 2.236)^{4/3}}{(X + 0.5)^{1.313}}$$

Solving by trial and error X = 6.00 or by using fig.

$$for \ \frac{S\,Q^{0.02}}{n^2 m^2} = 0.421 \qquad\qquad we\ have\ X = 6$$

$$D = \left[\frac{1.818\ Q}{m\ (X + 0.5)}\right]^{\frac{1}{2.64}} = 2.09\ m$$

$$B = 2.09 \times 6 = 12.54\ m$$

$$V = \frac{Q}{D^2(X + 0.5)} = 0.88\ m/s$$

$$check\ V = 055\ m\ D^{0.64}$$

$$= 0.55 \times 1 \times (2.09)^{0.64}$$

= 0.88 m/s (o.k.)

Sediment transport theories

It has now been established that cross-section and bed slope of a true regime channel depend upon the following three independent variables:

1. Discharge (Q) carried by the channel.
2. Nature and grade of the sediment entering the channel such as the grain-size distribution, the shape of grains and the specific gravity of particles.
3. Quantity of sediment (or silt charge) entering the channel.

The silt theories discussed earlier consider only the first two variables and do not account for the third variable. The third variable, viz, the silt charge, is an important factor which considerably affects the channel design. For a satisfactory design, the silt charge should be considered. The various sediment theories consider the effect of silt charge on the design.

The sediment transported by a channel (or a stream) consists of the bed load and the suspended load.

1. The bed load is that portion of the sediment which moves on or near the bed of the channel. The movement of bed load is by rolling, sliding and saltation (i.e. small leaps).
2. The suspended load is that portion of the sediment which remains in suspension in the flowing water and does not touch the bed. The suspended load is kept in suspension by the turbulent eddies generated due to friction. The particles of the suspended load move freely through the flowing water.

Lecture - 4

Estimation of bed load. The bed load can be estimated by the following two methods:

1. Sampler method 2. Analytical method

1. Sampler method. In this method, the samples of stream water are taken with the help of various types of samplers, such as box-type sampler and slot-type sampler the bed load is obtained after drying the sample and by determining the mass of dry solids. However, the samplers do not give a reliable value of the bed load. It is the usual practice to determine the bed load from the suspended load. Generally, the bed load is taken as 3 to 25% of the total suspended load, depending upon the nature of the bed materials. An average value of 10% is quite common.

2. Analytical method. Varies investigation have given analytical methods for the determination of the bed load. A brief introduction of the following theories is given below:

(i) Meyer-peter's equation (ii) Einstein's equation

(i) Meyer-peter's equation Meyer-peter's equation, which is based on experimental work carried out at Federal Institute of Technology, Zurich, is

$$\left(\frac{Q_s}{Q}\right)(\frac{N^-}{N})^{3/2}\, wSD = 0.047(w_s - w)\, d + 0.25\, (w/g)^{1/3}\, (q_s)^{2/3} \quad \ldots (1)$$

Where Q_s is the actual discharge, Q is the discharge if the sides of the channel were frictionless, $N^-$is Manning's coefficient for plane bed, $N$is the actual value of the Manning's coefficient for rippled bed, w is the specific weight of water (kN/m^3), d is the mean grain diameter (m); S is the bed slope; D is the depth of flow (m), g is the acceleration due to gravity and q_s is the rate of bed load transport per unit width of the channel (kN/m/s).

The ratio Q_s/Q takes into account friction of the sides of the channel. If the sides friction is neglected, $Q_s/Q = 1.0$.

The values of N^- and N are obtained from Strickler's formula as follows:

$$N^- = (k_s)^{1/6}/24 \quad \ldots (2)$$

and

$$N = (k)^{1/6}/24 \quad \ldots (3)$$

Where $k_s$the effective is grain diameter (m) and k is the representative size (m) of roughness of the actual rippled bed. The values of N is also equal to Manning's coefficient. Its value normally varies from 0.020 to 0.025 for irrigation channels.

$$N = 0.0225\, f^{1/4} \qquad \text{... (4)}$$

Bed shear, $\tau_b = (Q_s/Q)\, wDS$

Critical share stress, $\tau_c = 0.047\ (w_s - w)d$

Therefore, Eq (1) can be written as

$$\tau_b (N^-/N)^{3/2} = \tau_c + 0.25\ (w/g)^{1/3}\ (q_s)^{2/3}$$

Or $q_s = 47500\ \left[\tau_b (N^-/N)^{3/2} - \tau_c\right]^{3/2}$ kN/m/hr … (5)

Eq.(5) can be written in MKS units as

$q_s = 4700\ \left[\tau_b (N^-/N)^{3/2} - \tau_c\right]^{3/2}$ kg(f)/m/hr

Where $\tau_b$and τ_c are in kg(f)/m^2.

Illustrative Example A channel is 50m wide, 2.5m deep and has a bed slope of 1 in 4000. Determine the bed load transported by the channel by Meyer-Peter's equations. Neglect the side friction and take Manning's $N = 0.02$. The mean diameter of the material is 0.30mm and the representative size of the bed material for unrippled bed is 0.50 mm.

Solution From Eq. (2),

$N^- = (k_s)^{1/6}/24 = (0.5 \times 10^{-3})^{1/6}/24 = 0.0117$

As side friction is neglected, $Q_s/Q = 1.00$

From Eq. (5) $q_s = 47500\ \left[\tau_b (N^-/N)^{3/2} - \tau_c\right]^{3/2}$

$\tau_b = wDS = 9.81 \times (1/4000) \times 2.5 = 6.13 \times 10^{-3}$

$N^-/N = 0.0117/0.02 = 0.585$

$\tau_c = 0.047\ (w_s - w)d = 0.047(2.65 - 1.00) \times 9.81 \times 0.3 \times 10^{-3}$

$= 2.28 \times 10^{-4}$

Subtracting the above values in Eq. (a).

$$q_s = 47500\left[6.13 \times 10^{-3}\ (0.585)^{3/2} - 2.28 \times 10^{-4}\right]^{3/2}$$

Or $\quad q_s = 5.98$ kN/m/hr

Total load, $\quad Q_s = 5.98 \times 50 =$ **299.00 kN/hr**

Lecture - 5

LINING AND MAINTENANCE OF CANALS

ADVANTAGES AND DISADVANTAGES OF LINING

a) **Advantages**: The following are the main advantages of lined canals over unlined canals:

1. **Prevention of loss of water:** Valuable water is saved by reducing seepage losses.
2. **Prevention of waterlogging:** the adjacent land is "prevented from waterlogging due to seepage of water from the canal.
3. **Low maintenance cost:** the maintenance cost of a lined canal is less compared to that of an unlined canal.
4. **Less breaches:** the possibility of breaching of the canal is considerably decreased, as the section is more stable and strong.
5. **Smaller cross-sectional area:** because higher velocity is permitted, the cross-sectional area of a lined canal is much smaller than that of an unlined canal.
6. **Saving in canal structures:** because of smaller cross-section of the canal, there is saving in the cost of earthwork, canal structures, and other allied works.
7. **Saving in land:** because of smaller bed widths, the cost of land is less.
8. **Less silting:** because of higher velocities, silting is less.
9. **Flatter slopes:** the bed slope of a lined canal is considerably less than that of an unlined canal. Because of flatter slopes in the bed, there is an increase of the commanded area. In the case of hydel channels, there is a larger useful head at the powerhouse.
10. **No scouring:** Because of hard lined surface, there is no scouring of the canal bed and sides, which normally occurs in an unlined canal.
11. **Reduction of weed growth:** there is a reduction of the weed growth. The transpiration losses are also decreased.
12. **Low evaporation loss:** Because of higher velocity and smaller exposed area, the evaporation losses are low.
13. **Increase of the value of land:** the value of the land is increased because the waterlogging problem is considerably reduced.
14. **Less salt problem:** the canal water does not come in contact with harmful salts present in the natural soil, and, therefore, the salt problem is reduced to some extent.
15. **Better operation:** the lined canal has a stable section, which is easy to operate.

b) **Disadvantages**: A lined canal has the following disadvantages as compared to an unlined canal.

1. **High initial cost** the initial cost of a lined canal is high.
2. **Difficult to repair** if the lining is damaged, it is difficult to repair.
3. **Difficult to shift the outlets** as the lining is strong and permanent; it is difficult to shift the canal outlets at a later stage.

4. **Less additional safety** as a lined canal does not have inside berms, the additional safety provided by the berms in the case of unlined to the vehicular and pedestrian traffic is absent.

TYPES OF LINING

Various types of lining can be grouped into the following categories:

a) **Exposed lining with a hard surface**. This type of lining has an exposed surface, which is quite hard. This is further classified into the following types:

1. In-situ concrete lining.
2. Precast concrete lining
3. Shot Crete lining.
4. Cement mortar lining
5. Lime concrete lining.
6. Brick tile lining or burnt clay tile lining
7. Stone block or undressed stone boulder lining.
8. Asphaltic concrete lining.

b) **Buried membrane lining**. This type of lining is buried below the channel surface. It is further divided into the following types:

1. sprayed-in-place asphalt membrane lining.
2. Prefabricated asphaltic membrane lining.
3. Polythene film and synthetic rubber membrane lining.
4. Bentonite and clay membrane lining.
5. Road oil lining.

c) **Earth lining**: This type of lining uses the soil as a lining material it is further divided into the following types:

1. Thin compacted earth lining.
2. Thick compacted earth lining
3. Loosely placed earth lining.
4. Stabilized soil lining
5. Bentonite soil lining.
6. Soil-cement lining.

d) **Porous lining**: This type of lining is made of a porous material a brief description of the various types of lining is given in the following sections.

IN-SITU CONCRETE LINING

In-situ concrete is the most commonly used lining. It consists of a layer of cement concrete placed on a well-prepared and compacted subgrade (soil) in the bed and sides of the channel.

Lecture - 5

Advantage of concrete lining

1. It is quite impermeable and seepage losses are considerably reduced.
2. It is quite strong and durable.
3. It has a low coefficient of rugosity and high hydraulic efficiency. Hence, high velocity can be permitted.
4. High velocities prevent silting tendency. Evaporation losses are also reduced.
5. It permits fast construction by mechanical equipment.
6. The maintenance cost is low.
7. It is immune to weed growth.
8. It can be used in different thicknesses according to the capacity of the channel.
9. There is no need of plastering the lined surface.
10. Economy can be effected by using lean proportions (1:4:8) and by partial replacement of cement with pozzolana (surkhi).

Disadvantages of concrete lining

1. The initial cost is quite high.
2. It is prone to cracking due to temperature changes and shrinkage.
3. Repair is costly and difficult. Alteration of the canal outlets is quite difficult.
4. Because of relatively small thickness, it has limited resistance to external hydrostatic pressure after rapid drawdown.
5. It is susceptible to adverse subgrade conditions.
6. Skilled labour, and elaborate concrete mixing plants and transportation equipment are needed
7. Strict quality control is required to ensure concrete of proper grade, consistency and strength.
8. Under very high velocities, the fine material in the concrete is eroded, leaving behind a coarse surface with a high rugosity coefficient.

PRECAST CONCRETE LINING

In precast concrete lining, precast concrete slabs of the sizes $50\ cm\ \times\ 50\ cm\ \times\ 5\ cm\ and\ 50\ cm \times\ 25\ cm\ \times\ 5\ cm$ are commonly used.

Advantages of precast concrete lining over in-situ concrete lining

1. Precast concrete slabs are manufactured under controlled conditions. Therefore, the quality of concrete is good. These slabs provide a better, more impervious and durable lining than in-situ concrete lining.
2. Precast concrete lining involves less site operations. Therefore, the speed of construction is quite fast.
3. Precast concrete slabs can be manufactured on a mass scale. The precast concrete lining is usually cheaper than the in-situ concrete lining.

4. Precast concrete slabs can be manufactured during non-working season. These slabs are sometimes placed all along the canal for curing purposes after casting. Thus the cost of curing is reduced.
5. The shrinkage cracks are less because the slabs are of small size.
6. If the lining is damaged due to settlement of subgrade, it can be easily repaired; whereas in the case of r a-situ concrete lining, it is very difficult to repair.
7. There the ground water table in high, water pressure is released through the joints between these labs.

Disadvantages

1. Seepage losses are generally more in the case of precast concrete lining.

2. Transportation of precast slabs is costly and time-consuming. Moreover, breakage of slabs also occurs during transportation.

SHOTCRETE LINING

In shotcrete lining, cement mortar is forced under pressure through a nozzle on the surface of the subgrade of the channel. Generally, cement mortar 1:4 is applied pneumatically (i.e. by compressed air) through a nozzle. The pneumatically applied mortar is called shotcrete and hence the lining is known as shotcrete lining

Advantages

1. There is no necessity of fine dressing of the subgrade because shotcrete lining can be placed even on irregular surfaces. This is specially useful in rock cuts.

2. The equipment required for shotcrete is quite light and mobile. It is particularly suitable for small and widely scattered jobs.

3. Shotcrete is useful for resurfacing of badly cracked and leaky but structurally sound old cement concrete lining.

Disadvantages

1. Shotcrete lining is costlier than concrete lining.

2. It is less durable than the concrete lining of the usual thickness. It gives satisfactory service only for 20 years or so.

CEMENT MORTAR LINING

In cement mortar lining, a layer of cement mortar (1:3) of uniform thickness is laid on a properly compacted subgrade. The usual thickness of the cement mortar lining is 2.50 cm.

Cement mortar lining is not commonly used, as it is quite expensive. Moreover, it is less durable than cement concrete lining. However, a 2.5 cm thick cement mortar lining is sufficient to reduce seepage losses by about 75%.

Lecture - 5

LIME CONCRETE LINING

Lime concrete lining consists of hydraulic lime, sand and coarse aggregate mixed in a suitable proportion.

Lime concrete lining is not as impervious as cement concrete lining. It is rarely used in practice.

BRICK TILE LINING

The brick tile lining, also called burnt clay tile lining, consists of either a single layer or a double layer of brick tiles laid in cement mortar. The size of the tiles is generally restricted to $30\ cm \times 15\ cm \times 5\ cm$ for convenience of handling.

Advantages of brick tile lining over cement concrete lining.

1. Brick tile lining is usually economical in initial cost.
2. No elaborate equipment is needed for laying tiles.
3. The tiles can be laid by ordinarily masons.
4. Transportation cost is usually small, as kilns for burning the tiles can be established near the site.
5. Expansion joints are not required, as the shrinkage is practically eliminated and the coefficient, of expansion of tiles is very small.
6. The thickness of lining is uniform because it is governed by the thickness of tiles.
7. The rounded sections of the channel can be easily lined without the use of a formwork.
8. If there is any settlement of the subgrade, numerous small cracks are developed in the mortar layer between the tiles, but the seepage loss is insignificant. In case of large settlement, the small damaged area can be easily repaired.

Disadvantages of brick tile lining over cement concrete lining.

1. Brick tile lining is relatively more previous than cement concrete lining.
2. The maintenance cost is high.
3. It has relatively lower resistance to abrasion.
4. It cannot be done mechanically. It is a relatively slow process.
5. It cannot be used where suitable materials for manufacture of tiles are not locally available. Brick title lining is commonly used at places where suitable clay for making brick tiles is easily available and it is economical.

STONE BLOCK LINING (OR BOULDER LINING)

Stone block lining consists of a layer of undressed stones (or boulders) set in 1:6 cement mortar

Stone block lining has better wearing resistance than brick tile lining. It is more suited to steep channels, especially in hilly areas. It is also more impervious than brick tile lining. It is quite economical where good quality stone is easily available near the site.

Lecture - 5

ASPHALTIC CONCRETE LINING

Asphaltic concrete lining is similar to cement concrete lining. The asphaltic concrete consists of a mixture of asphalt, cement and graded aggregates, and hence it is also known as *asphaltic* cement concrete.

Advantages

1. Asphaltic concrete lining is relatively flexible and has greater ability to withstand settlements in the subgrade.
2. It can be used in place of cement concrete lining wherever it is cheaper because of low cost of asphalt.
3. It can be used for the repairs of cement concrete lining by laying a resurfacing layer of asphaltic concrete.

Disadvantages

1. It has low resistance to external hydrostatic pressure developed due to seepage.
2. There is a danger of sliding or slipping during hot weather.
3. The velocity in asphaltic concrete lined channels is usually limited to 1.5 m/s
4. The coefficient of rugosity (N) is high if there is no special surface finish.
5. It permits certain type of weed growth which may cause its puncturing.

BURIED MEMBRANE LINING

Buried membrane lining are buried beneath the channel surface. The buried membrane linings are of the following types:

1. Sprayed-in-place asphaltic membrane lining
2. Prefabricated asphaltic membrane lining.
3. Polyethelene film and synthetic rubber membrane lining.
4. Bentonite clay membrane lining.
5. Road oil lining.

A brief description of these linings is given below:

1. **Sprayed-in-place asphaltic membrane lining:** This type of lining consists of a thin layer, about 6mm thick, of a special high softening-point asphalt sprayed in place at a high temperature of about 150°C to 20O°C on a properly prepared subgrade to form an impervious barrier. The asphaltic layer is covered with a 30 cm thick layer of earth and gravel to protect it from damage and weather.

Advantages

1. It provides an effective and relatively cheap method of seepage control in channels.
2. It can be easily laid even in cold and wet weather.
3. It is quite flexible and readily adjusts to the settlements in subgrade.

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Disadvantages

1. There is no decrease in the coefficient of rugosity because the exposed surface consists of earth.
2. High velocity cannot be permitted.
3. The useful life of lining is limited.
4. It requires special equipment and trained workers for spraying of hot asphalt.

2. Prefabricated asphaltic membrane lining: this type of lining consists of a prefabricated asphaltic membranes available in rolls. The membrane is spread directly on the prepared subgrade and covered with protective earth.

A prefabricated asphaltic membrane lining has the same advantages and disadvantages as those in sprayed-in-place asphaltic membrane lining. However, there is no need of special equipment and skilled workers in this case. Prefabricated asphaltic membrane lining is quite durable. The membranes can be easily-handled and transported.

3. Polyethylene film and synthetic rubber membrane lining. In this type of lining, a polyethylene film or a synthetic rubber membrane is laid on the subgrade and a protective cover of earth is place it.

4. Bentonite clay membrane lining. Bentonite is a type of clay which contains a large percentage of the mineral montomorrillonite
This type of lining is quite economical if bentonite is easily available.

5. Road oil lining. in this type of lining, road oil is sprinkled over the subgrade in a thickness of about 1.5 mm.

EARTH LINING

In earth lining, soil is used as a lining material. The following types of earth linings are sometimes used.

1. **Thin-compacted earth lining:** In this type of lining, a layer of clayey soil, 15 to 30 cm thick, is placed on the subgrade and thoroughly compacted.
2. **Thick compacted earth lining:** This-type of lining is similar to the thin-compacted earth lining, but the thickness of lining is more. The thickness varies from $30\ cm$ to $60\ cm$ at the bed and from $60\ cm$ to $90\ cm$ on the sides of the channel. The lining is thoroughly compacted in layers.
3. **Loosely placed earth lining:** In this type of lining, the clayey soil used as lining is not compacted. Suitable clayey soil is just spread over the bed and sides of the channel in layers up to a thickness of 30 cm.
4. **Stabilized soil lining:** In this type of lining, the stabilized soil is used in the bed and at the sides of the channel. The soil is stabilized and rendered impervious by the addition of specially treated resins and chemicals such as sodium silicate, sodium chloride, commercial resins, cement, lime, asphalt and petrochemicals.
5. **Bentonite clay lining:** In this type of lining, the bentonite clay is used as a lining material. The subgrade in the bed and at the sides of the channel is mixed in place with bentonite to form a $5\ to\ 10\ cm$ thick layer.

6. **Soil cement lining:** Soil cement is a mixture of soil and cement.

There are basically two types of soil cement lining.

(a) Compacted soil cement lining.

(b) Plastic soil cement lining.

POROUS LINING

Porous lining is usually provided in the head reaches of the main canal when the ground water table is higher than the bed level of the channel. The ground water in the subgrade passes throw the pores of the lining and, consequently, the external hydrostatic presser on the lining is released.

For laying the porous lining, the subgrade is properly prepared. The bed and sides are divided into suitable compartments. Not exceeding 15 m in any direction by constructing ribs of stone masonry or cement concrete. The ribs are generally rectangular in section. The thickness of ribs is equal to the combined thickness of the lining and filter.

Lecture - 5

Water Management (water losses)

1. Absorption Losses
2. Percolation (or seepage) Losses
3. Evaporation Losses
4. Transpiration Losses

ABSORPTION LOSSES

Absorption losses occur because of absorption of water by soil surface canal wetted perimeter.

When the water table is at a considerable depth below the canal bed, the
Infiltrating the soil below
The canal bed in unable to reach the ground water reservoir below the table.
Absorption losses are independent of the seepage head. These losses depend upon the water head hc form the water level of the canal to the bottom of the saturated zone and the capillary head hc, for the soil at the boundary of the saturated zone and the capillary head hc, for the soil at the boundary the saturated zone. In general, absorption losses depend mainly upon the depth of water in the canal **X** type of soil.

PERCOLATION (OR) SEEPAGE LOSSES

Percolation losses are usually much greater than absorption losses. They may be as high as 3 times or more of the absorption losses whether losses will be by absorption or by percolation will depend primarily on the nature of soil strata and the level of the water table.

Both the absorption loss & percolation are loss initially large because the water is utilized for filing the pores of the soil. With the passage of time, the losses decrease & an equilibrium is finally reached. Moreover the silt carried by the canal water gats deposited in the canal & reduces the soil permeability & hence the seepage

Absorption & percolation losses from the canal mainly depend upon the following factors.

(i) Permeability of soil

The greater is permeability of the soil in the bed & banks of the canal, the greater are the losses.

(ii) Depth of water

The greater is the depth of water, in the canal, the greater the losses.

(iii) Velocity of water

The losses decrease with an increase in the velocity of flow in the channel.

(iv) Amount of silt

The losses decrease with an increase in the amount of silt carried by the canal.

(v) Temperature of water

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The losses increase with an increase in temperature of water because permeability of the soil is increased.

(vi) Age of the channel

The losses are large in newly constructed channels and they reduce as silt deposited with the passage of time & a relatively impervious silt layer is formed.

(vii) Level of the channel bed

The losses depend upon the level of the channel bed w.r.t natural surface or ground level. The losses are more when the canal is in heavy filling.

(viii) Position of the water table

The losses depend upon the position of water table w.r.t the canal bed the
losses are more when the water table is high.

EVAPORATION LOSSES

Evaporation losses depend upon the water surface area of the canal, relative humidity, wind velocity, temperature & various other factors. In hot & dry summer months, the evaporation losses are high, but they seldom exceed 10% of the total losses. Generally, evaporation losses is less Than 1% of the total water entering the canal head.

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Ground Water Flow

Ground water represents that portion of water under the surface of the earth. It results from infiltration of rain, surface runoff and snow after melting into the soil. Subsequently, this water transports through the soil into the ground water level, where it eventually moves back to surface streams, lakes, rivers or oceans.

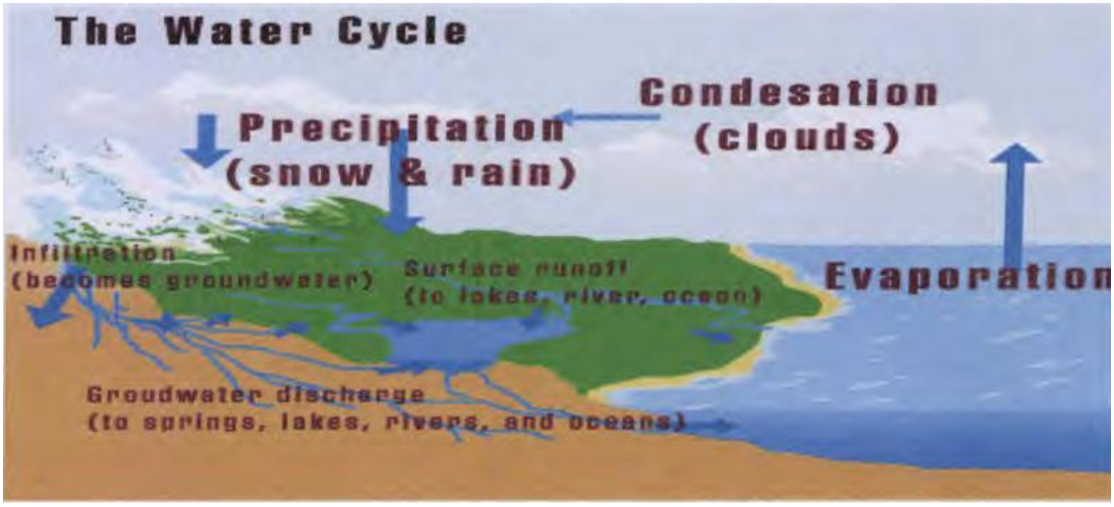

Figure 1: Water cycle in nature

Permeability:

It describes fluid ability to pass through the soil pores and voids. Mostly, permeability depends on the soil and fluid type. For most civil engineering applications, water is the applicable fluid. Therefore, it is important to describe the soil type. The course soil such as sand has high permeability, while the fine soil such as clay soil has low permeability.

Importance of Permeability:

Knowledge of the permeability properties of soil is necessary to:

1. Estimating the quantity of underground seepage.
2. Solving problems involving pumping seepage water from construction excavation
3. Stability analyses of earth structures and earth retaining walls subjected to seepage forces.

Permeability of porous medium:- depends on

1- The characteristics of the porous medium.
2- The characteristics of the flowing fluid.

The permeability of a medium is measured in terms of **hydraulic conductivity** (also known as **the coefficient of permeability).**

The coefficient of permeability is equal to the volume of water which flow in unit time through unit a cross-sectional area of the medium under a unit hydraulic gradient at the prevailing temperature.
The hydraulic conductivity therefore has the dimensions of (L/T).

Flow of water through Porous media:-

Ground water flows whenever there is an existing of a difference in head between two points. This flow can either be laminar or turbulent. Most often, ground water flows with such a small velocity that the resulting flow is laminar.

The rate of flow (discharge) is measured by using the **Darcy's Law:**

The fundamental premise for Darcy's law to work is:

1- The flow is laminar, no turbulent flows
2- Fully saturated
3- The flow is in steady state, no temporal variation.

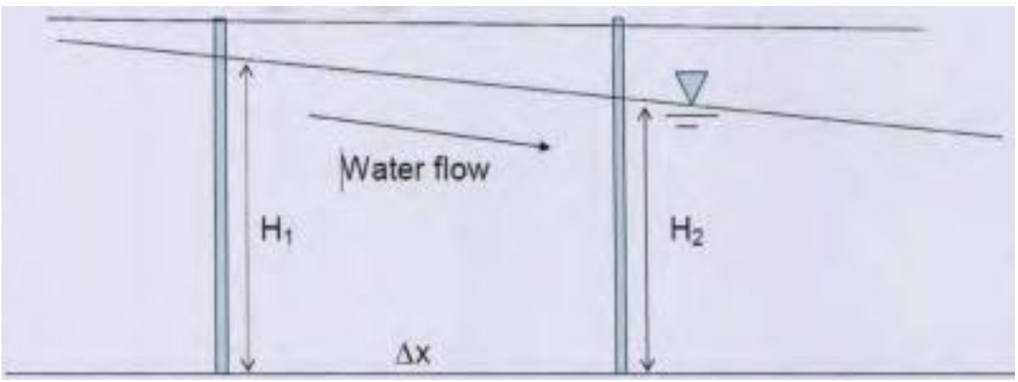

Figure 2: Water flow and hydraulic gradient between two wells

Q = K.A.$(\Delta h/L)$ *(1)*

Where:-

Q: is the rate of flow.

K:is the coefficient of permeability.

A: is the cross-sectional area.

Δh: is the difference in head between two points.

L:is the length between these two points (Δx).

Darcy's Law can be written also as:

$V = k.(\Delta h/L)$ *(2)*

Darcy's law states that how fast the groundwater flow in the aquifer which depends on two parameters:

1- How large is the hydraulic gradient of the water head *(i=dH/dx);* and
2- The parameter describing how permeable the aquifer porous medium.

<u>How to compute the hydraulic conductivity:-</u>

1- In the Lab.

a. <u>Constant Head permeameter:</u> **the coefficient of permeability** of relatively more permeable soils can be determined in laboratory by the constant — head permeability test. The test is conducted in an instrument known as Constant Head permeameter. Darcy's Law for flow of water through porous Medium (Soil) is applied for computing **the coefficient of permeability,** this method is used to compute the **coefficient of permeability** for granular soils (Sandy Soils , Gravel)

Where:
Q = K.A.$(\Delta h/L)$

Lecture - 6

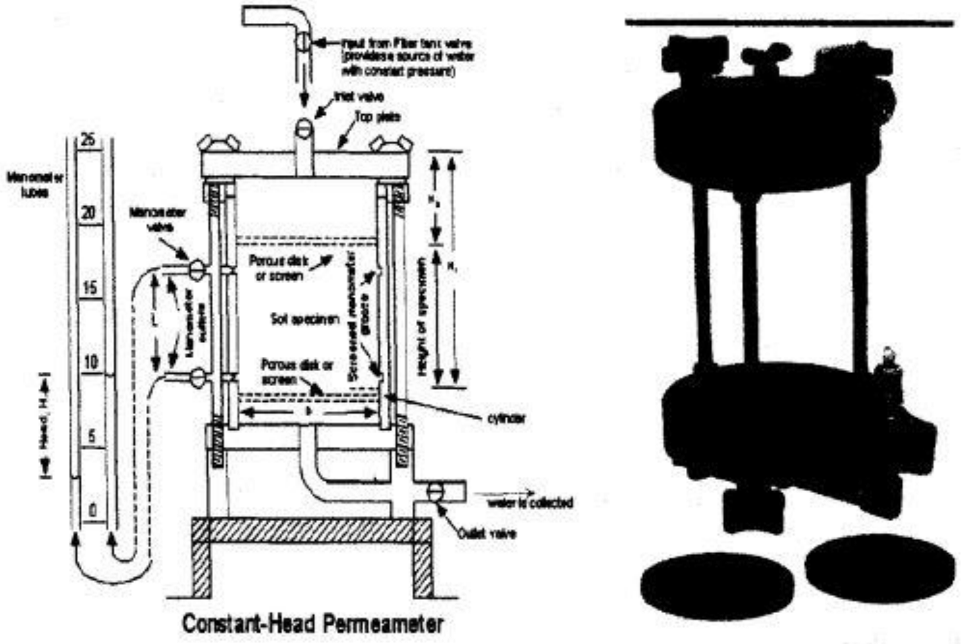

Figure 3: Constant- head permeameter test

b. Variable Head Permeameter: this method is used to compute the **coefficient of permeability** for Fine Texture- Compacted Soils and Low Permeability Soils (silty clay, clay soils).

The device consist of a cylinder attached to a vertical glass tube of small diameter, the cylinder pressed into the soil to a known depth then the whole apparatus filled with water, as the water percolate through the soil in the cylinder the water drop in the tube.

$$adh = -qdt$$

Where:
a: is the cross-sectional area of the tube

$$a\,dh = -(A.\ K.\ i)\,dt$$

Where:
A:is the cross-sectional area of the cylinder.
i=h/L: hydraulic gradient.
L: length of soil Specimen.

$$a\,dh = -A.\ K\,(h/L)dt$$

$$(A.\ k.\ dt)/\ (a\ L) = (dh/h)$$

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Integrating: $(\boldsymbol{A.k}/\ \boldsymbol{a\ L})\ \int \boldsymbol{dt} = -\,\boldsymbol{dh}/\boldsymbol{h}$

$$\boldsymbol{(A.\ K\ /a\ L)\ (t_2 - t_1) = \log (h_1/h_2)}$$

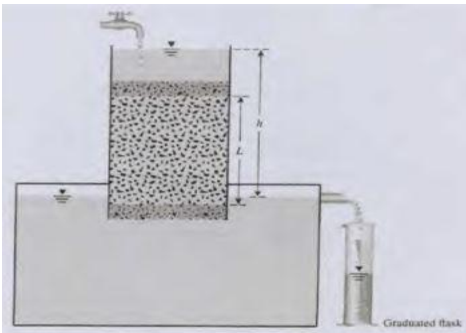
$$\boldsymbol{K\ =\ (a\,.L\ /A.t)\ \log\ (h1/h2)}$$

2- in situ

a. By using an auger to make a hole in the soil with determined diameter and height if the water table with the ground level, computing the time to fill the hole with water and then calculating the coefficient of permeability.

b. if the water table below the ground level (under the end of the hole filling the hole with water, and computing the time to percolate the water through the soil.

Example 1:



Referring to the shown figure, calculate the hydraulic conductivity (K) in m/s. The tap valve was switched on to allow water pass through the medium in the cylinder. Then, the passed water moved to the bottom tank. The collected water in the graduated cylinder was found to be365 cm^3every three min. The water level difference between the top cylinder and the bottom tank was 72 cm. the length of the permeable layer is 46 cm and the area of the cross sectional is 23 cm^2.

Solution:

Given:

Q = Vol./t= 365 cm^3/ (3min*60 sec) = 2.02 cm^3 /s.

i = dh/L

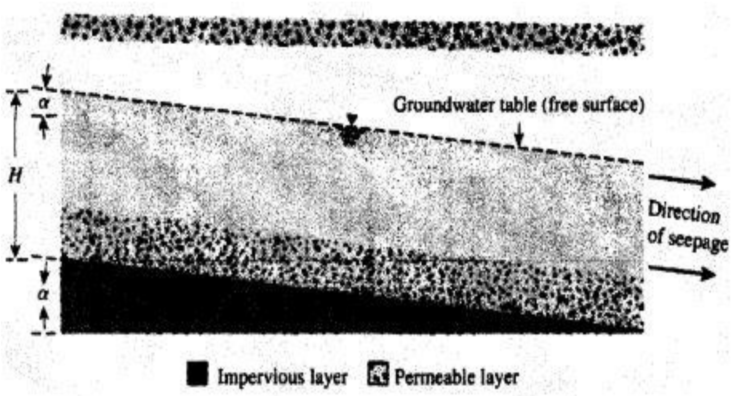
= 72/46 = 1.56

Q = KiA

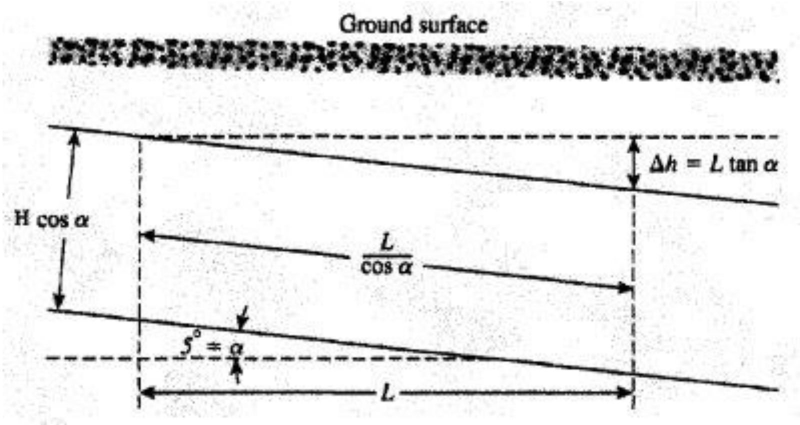
2.02 = K *1.56* 23

= 0.056 cm/s.

Example 2:



A permeable soil layer is underlain by an impervious layer as shown in the figure with K= $4.8*10^{-3}$ cm/s for the permeable layer. Calculate the rate of seepage through this layer cm^3/s/cm width if H is 3 m and α=5^0



Solution:

i= dh/L

= L tan α/ (L/cos α)

= sin α= sin 5

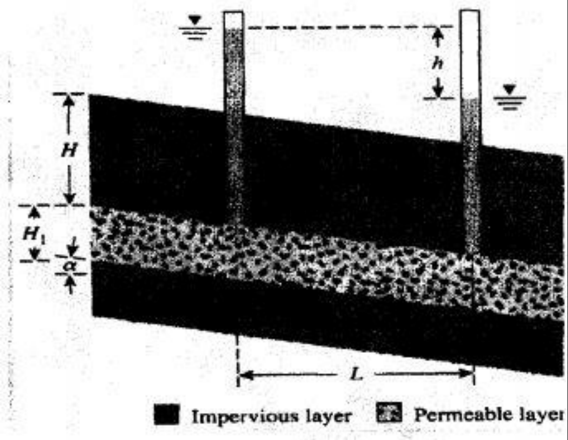
Q = KiA

$= 4.8*10^{-3} * \sin 5 * (3m*100\ cm)*COS\ 5*(lcm)$

$= 0.125\ cm^3/s/cm$

Example 3:

Find the flow rate in $m^3/s/m$ length (at right angles to the cross section shown in the figure through the permeable soil layer. Notably, the depth of the impervious layer is 8 m and the permeable layer is 3 m, the water level difference between the two wells is 4 m, the distance between the two wells is 50 m, α=8° and K= $8*10^{-4}$ m/s.



Solution:

$i = dh/L$

$= h/ (L/\cos \alpha)$

$= 4\ m/(50\ m/\cos 8^0) = 0.081\ m/s$

A of 1 unit length = $H1 * \cos \alpha$

$= 3\ m * \cos 8^0$

$= 2.97\ m^2/m$

$Q = KiA$

$= 8*10^{-4} *0.081 *2.97$

$= 1.93 * 10^{-4}\ m^3/s/m$

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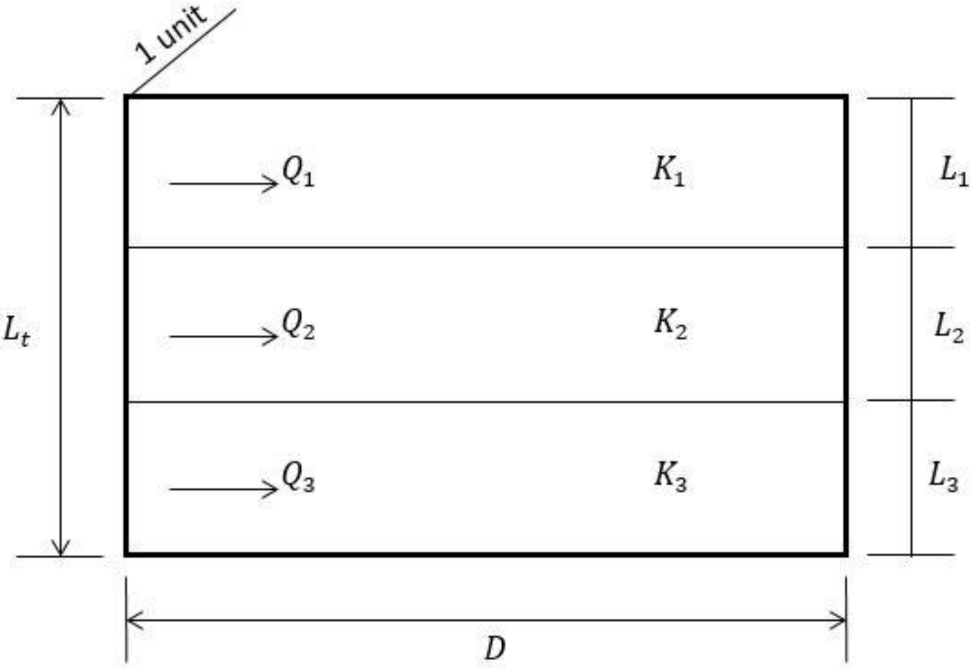
The hydraulic conductivity of multiple layers: -

1- multiple horizontal layers:



$Q = KiA$

$Q_1 = K_1 i_1 A_1$

$Q_2 = K_2 i_2 A_2$

$Q_3 = K_3 i_3 A_3$

$Q_t = Q_1 + Q_2 + Q_3$

$A = (L) \times (1)$

$A_t = (L_t) \times (1)$

$A_1 = (L_1) \times (1)$

$A_2 = (L_2) \times (1)$

$A_3 = (L_3) \times (1)$

$L_t = L_1 + L_2 + L_3$

$A_t = A_1 + A_2 + A_3$

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$i_t = i_1 = i_2 = i_3$

$\Delta \mathrm{h_t}/\mathrm{D} = \Delta \mathrm{h_1}/\mathrm{D} = \Delta \mathrm{h_2}/\mathrm{D} = \Delta \mathrm{h_3}/\mathrm{D}$

$\mathrm{Q_t} = \mathrm{K_h}.\mathrm{A_t}.(\Delta \mathrm{h_t}/\mathrm{D})$

$\mathrm{Q_1} = \mathrm{K_1}.\mathrm{A_1}.(\Delta \mathrm{h_1}/\mathrm{D})$

$\mathrm{Q_2} = \mathrm{K_2}.\mathrm{A_2}.(\Delta \mathrm{h_2}/\mathrm{D})$

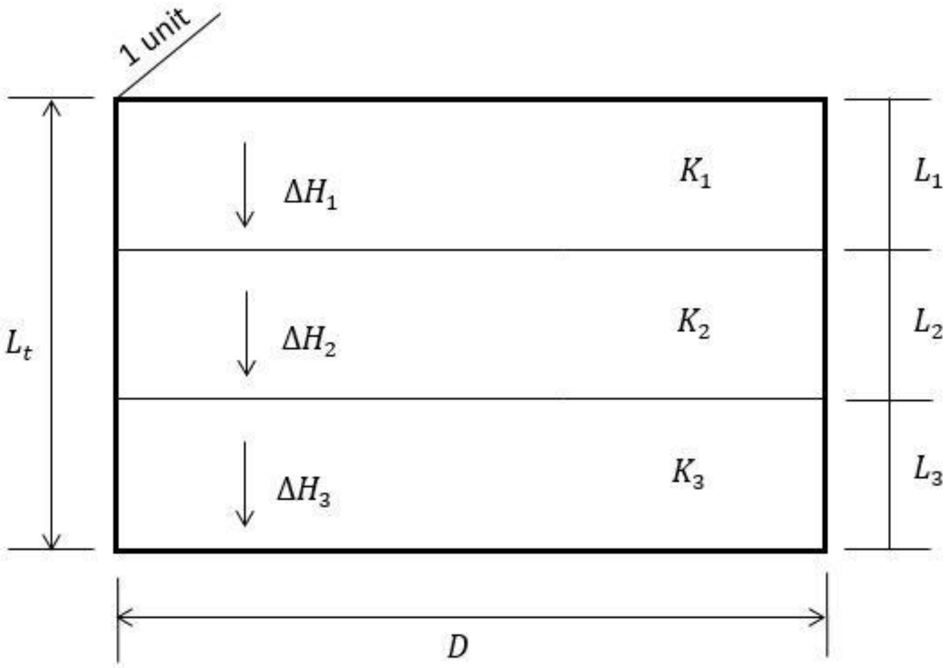
$\mathrm{Q_h} = \mathrm{K_3}.\mathrm{A_3}.(\Delta \mathrm{h_3}/\mathrm{D})$

$$\mathrm{K_h}.\mathrm{A_t}.(\Delta \mathrm{h_t}/\mathrm{D}) = \mathrm{K_1}.\mathrm{A_1}.(\Delta \mathrm{h_1}/\mathrm{D}) + \mathrm{K_2}.\mathrm{A_2}.(\Delta \mathrm{h_2}/\mathrm{D}) + \mathrm{K_3}.\mathrm{A_3}.(\Delta \mathrm{h_3}/\mathrm{D})$$

$\mathrm{K_h}.\mathrm{A_t}. = \mathrm{K_1}.\mathrm{A_1} + \mathrm{K_2}.\mathrm{A_2} + \mathrm{K_3}.\mathrm{A_3}$

$$\mathrm{K_h} = \frac{\mathrm{K_1}.\mathrm{L_1}.+\mathrm{K_2}.\mathrm{L_2} + \mathrm{K_3}.\mathrm{L_3}}{\mathrm{L_1} + \mathrm{L_2} + \mathrm{L_3}} = \frac{\sum \mathrm{KL}}{\mathrm{L}}$$

2- Multiple vertical layers:



$\Delta \mathrm{h_t} = \Delta \mathrm{h_1} + \Delta \mathrm{h_2} + \Delta \mathrm{h_3}$

$Q_t = Q_1 = Q_2 = Q_3$

$\mathrm{Q_t} = \mathrm{K_v}.\mathrm{A_t}.(\Delta \mathrm{h_t}/L_t)$,

$\mathrm{Q_1} = \mathrm{K_1}.\mathrm{A_1}.(\Delta \mathrm{h_1}/L_1)$,

$Q_2 = K_2. A_2. (\Delta h_2/L_2)$,

$Q_3 = K_3. A_3.(\Delta h_3/L_3)$,

$$A_t = A_1 = A_2 = A_3$$

$$L_t = L_1 + L_2 + L_3$$

$$Q_t.\frac{L_t}{K_v. A_t.} = Q_1.\frac{L_1}{K_1. A_1.} + Q_2.\frac{L_2}{K_2. A_2.} + Q_3.\frac{L_3}{K_3. A_3.}$$

$$\frac{L_t}{K_v} = \frac{L_1}{K_1} + \frac{L_2}{K_2} + \frac{L_3}{K_3}$$

$$K_v = \frac{L_1 + L_2 + L_3}{\frac{L_1}{K_1} + \frac{L_2}{K_2} + \frac{L_3}{K_3}}$$

Example 1

Three layered soil of (200 cm) thickness each, with total horizontal permeability of (0.35 cm/hr), the permeability of second layer is equal to twice the permeability of the first layer, and the permeability of the third layer is equal to the half of the permeability of the first layer. If the three layers are confined by two impervious layers, find the permeability of each layer and the total flow assuming that the hydraulic gradient is equal to one unit?

Solution

$$K_h = \frac{K_1. L_1. + K_2. L_2 + K_3. L_3}{L_1 + L_2 + L_3} = 0.35\ cm/hr$$

$$K_2 = 2\ K_1$$

$$K_3 = 1/2\ K_1$$

$$L_1 = L_2 = L_3 = 200\ cm$$

$$0.35 = \frac{K_1.L_1. + 2\,K_2.L_2 + 1/2\,K_3.L_3}{L_1 + L_2 + L_3}$$

$K_1 = 0.3\ cm/hr$, $K_2 = 0.6$, $K_3 = 0.15$

$Q_t = K_h.A_t.(\Delta h_t/L_t)$

$Q = 0.35.(3 \times 200 \times 100).(1) = 21000\,cm^3/hr$

Example

Find the vertical permeability of the three layered soil (100 cm) thickness each, the permeability of the first layer is (1.27 cm/hr), the second layer is (0.127 cm/hr), and the third one is (12.70 cm/hr) respectively. If the water pounded on the surface is (350 cm), find the flow and the head losses between the layers?

Solution

$$K_v = \frac{(L_1 + L_2 + L_3)}{\left(\frac{L_1}{K_1} + \frac{L_2}{K_2} + \frac{L_3}{K_3}\right)}$$

$$K_v = (3 \times 100)/\left(\frac{100}{1.27} + \frac{100}{0.127} + \frac{100}{12.7}\right)$$

$Q_t = K_v.A_t.(\Delta h_t/L_t)$

$$Q_t = (0.343)(100 \times 100)(\frac{350}{300})$$

$Q_t = 4004.0\ cm^3/hr$

$Q_1 = K_1.A_1.(\Delta h_1/L_1)$

$4004.0 = 1.27(100 \times 100)\ (\Delta h_1/100)$

$\Delta h_1 = 31.53\ cm$

$Q_2 = K_2.A_2.(\Delta h_2/L_2)$

$4004.0 = 0.127(100 \times 100)\ (\Delta h_2/100)$

$\Delta h_2 = 315.30\ cm$

$Q_3 = K_3.A_3.(\Delta h_3/L_3)$

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$4004.0 = 12.7(100 \times 100)\ (\Delta h_3/100)$

$\Delta h_3 = 3.15\ cm$

Some Definition:-

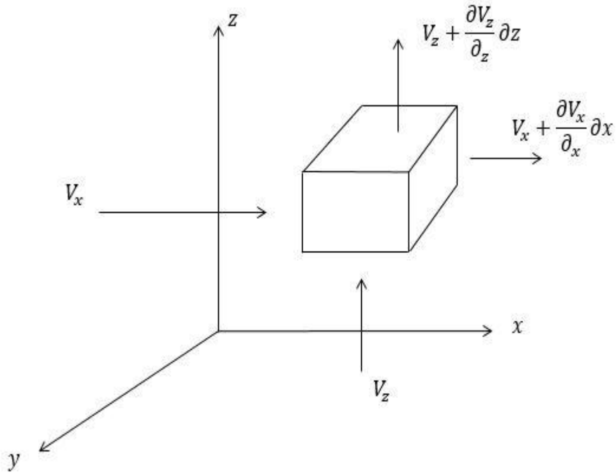
1- Aquifer: A saturated permeable geological unit that can transmit significant quantity of water under ordinary hydraulic head. Types of Aquifers:-

a- Unconfined aquifer: the aquifer that occurs near the ground surface and bounded from the bottom with impervious layer, using (Dupuit-Forchheimer Eq. to compute discharge and the head losses in any point throw the aquifer).

b- Confined aquifer: the aquifer that occurs between two impervious layers and in higher depth, using (Darcy's law to compute discharge and head losses at any point throw the aquifer).

Steady State Saturated flow (Lap lace's Equation):-

Consider an element of soil of size dx, dy. Where $(dx = dy)$ through which the flow is taking place (the third dimension along z-axis is large, it is taken as unity)



Let the velocity at the inlet faces equalV_x, and V_y respectively, and the outlet faces equal $V_x, +(\partial V_x/\partial x).dx$, and $V_y + (\partial V_y/\partial y).dy$

As the flow is steady and the soil is incompressible the discharge entering the

element is equal to that
leaving element.

Thus $V_x\,dy + V_y\,dx = (V_x + (\partial V_x/\partial x).dx).dy + (V_y + (\partial V_y/\partial y).dy).dx$

Or $\left((\partial V_x/\partial x) + (\partial V_y/\partial y)\right).dx.dy = 0$

Or $(\partial V_x/\partial x) + (\partial V_y/\partial y) = 0 \ldots\ldots\ldots\ldots$ ① (the continuity equation for two-dimensional flow)

Let h be the total head at any point, the horizontal and vertical components of the hydraulic gradient are respectively:

$i_x = -(\partial \mathrm{h}/\partial x)$ $\quad i_y = -(\partial \mathrm{h}/\partial y)$

The minus indicates that the head decreases in the direction of flow.

From Darcy's Law: $V_x = -K_x(\partial \mathrm{h}/\partial x)$ $\quad V_y = -K_y(\partial \mathrm{h}/\partial y)$

Substituting in Eq,①: $K_x(\partial^2 h/\partial x^2) + K_y(\partial^2 h/\partial y^2) = 0$

As the soil is isotropic $K_x = K_y$ therefore.

$(\partial^2 h/\partial x^2) + (\partial^2 h/\partial y^2) = 0 \ldots\ldots\ldots\ldots$ ② (Lap lace's Eq. in terms of head)

<u>Methods for Lap lace's Equation Solution:-</u>

a- Approximation method (Simplified) method by Dupuit — Forchheimer.
b- Electrical Analogy.
c- Numerical method (Relaxation & Iteration method).

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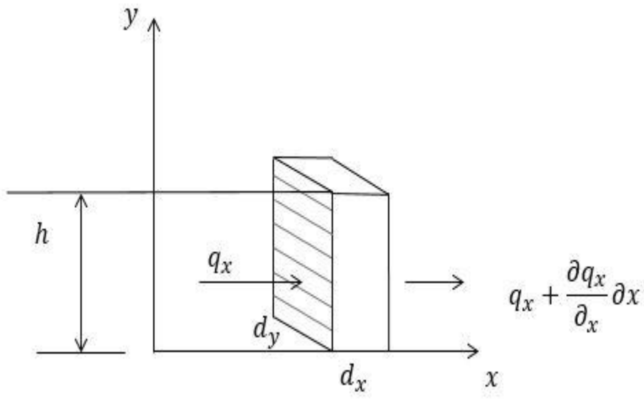
Dupuit-Forchheimer Equation (theory of free surface):-

Simplified Assumptions:



1- Neglect curvature of water surface (neglect the vertical flow component).
2- Stream lines are straight and parallel.
3- Hydraulic gradient equal to the slope of free surface.
4- Velocity is uniform across the section :

$V_x = -K_x(\partial \mathrm{h}/\partial x)$

$q_x =$ the discharge per unit width in the X-direction.

$q_x = V_x \,.\, A = -K_x(\partial \mathrm{h}/\partial x).(h) = -K_x(\partial \mathrm{h}^2/\partial x)$

Where $q_x =$ constant, then: $\partial q_x/\partial x = 0$

Or $-K_x(\partial^2 h^2/\partial^2 x) = 0$

Or $(\partial^2 h^2/\partial^2 x) = 0 \ldots\ldots\ldots\ldots$ 1

Integrating Eq. 1: $h^2 = Ax + B \ldots\ldots\ldots\ldots$ 2

Boundary conditions:

$At \qquad x = 0 \qquad h = y_1$

$At \qquad x = L \qquad h = y_2$

Substitution into Eq. 2:

$A = ({y_2}^2 - {y_1}^2)/L \quad , \quad B = {y_1}^2 \quad$, then:

$h^2 = ({y_2}^2 - {y_1}^2/L).x + {y_1}^2 \ldots\ldots\ldots\ldots$ 3

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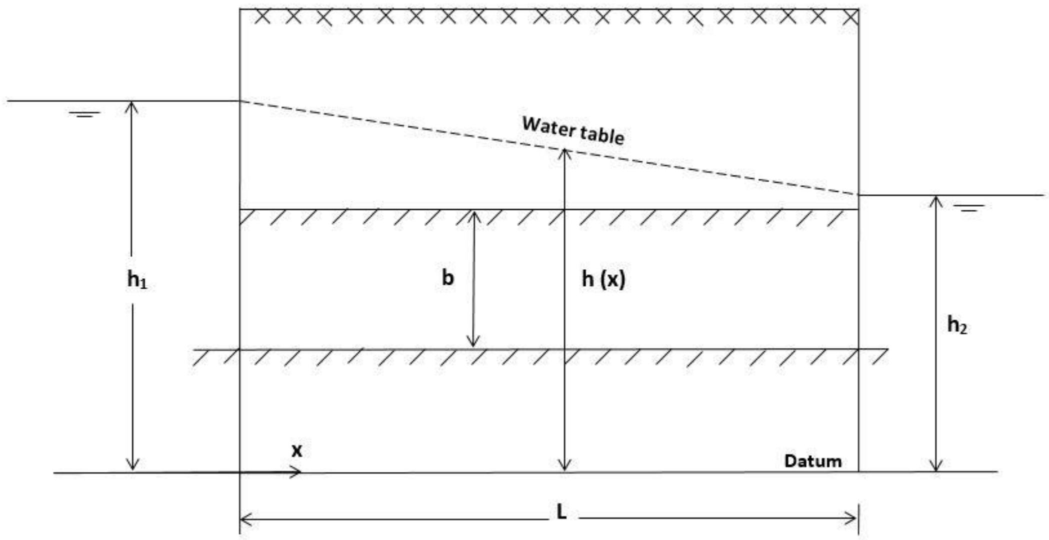
Flow between tow line sources in a confined aquifer of uniform thickness



$$\frac{d^2 h_{(x)}}{d\,x^2} = O$$

Subject to the condition

$h(O) = h_1 \qquad at \quad x = O$

$h(L) = h_2 \qquad at \quad x = L$

the hydraulic head in the flow system is given by solution the eq.

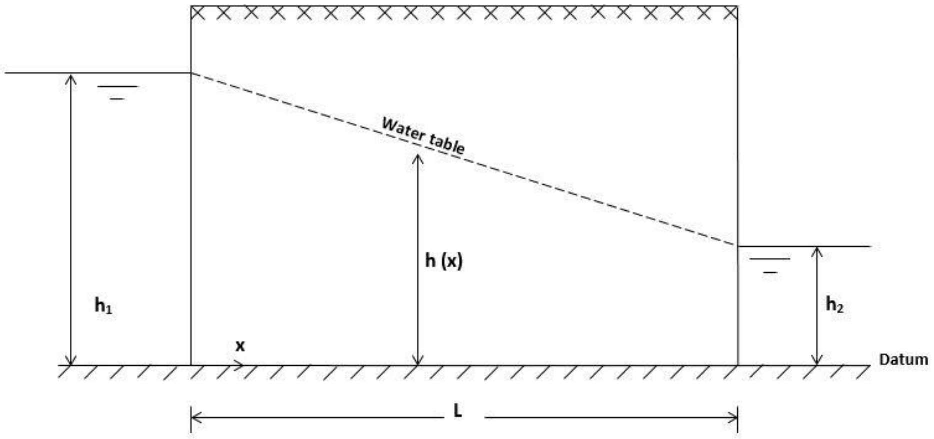
$$h(x) = h_1 - \left(\frac{(h_1 - h_2)}{L}\right) \times x$$

$Q = V_x . A \Rightarrow q_{(x)} = Q \times 1unit$

$$q_{(x)} = V_x . b = -k . b \left(\frac{dh}{dx}\right) = k . b . \left(\frac{h_1 - h_2}{L}\right)$$

Lecture - 6

Flow between the line source in a horizontal unconfined aquifer



$$\frac{d^2 h^2{}_{(x)}}{d\,x^2} = O$$

$$h(O) = h_1 \qquad at \quad x = O$$

$$h(L) = h_2 \qquad at \quad x = L$$

$$h^2(x) = {h_1}^2 + ({h_2}^2 - {h_1}^2)(x/L)$$

$$V_{(x)} = -k.\frac{dh}{dx} = k.\frac{\left({h_1}^2 - {h_2}^2\right)/L}{2h_{(x)}} \Rightarrow$$

$$q_{(x)} = V_{(x)}.h_{(x)} = k\left({h_1}^2 - {h_2}^2\right)/2L$$

Example: Given $h_1 = 50\ m\ , h_2 = 20\ m\ , L = 100\ m\ , k = 5\ m/d$. Plot the variation of $V(x)$. $dh/dx\ .\ q$? unconfined aquifer.

Solution:

$\boldsymbol{X}$	$\boldsymbol{h(x)}$	$\boldsymbol{V(x)}$	$\boldsymbol{\frac{dh}{dx}}$
20	45.6	1.15	-0.23
40	40.74	1.28	-0.257
60	35.2	1.49	-0.298
80	28.6	1.83	-0.367
100	20	2.625	-0.525

Lecture - 6

$$h^2(x) = (50)^2 + \left[\frac{(20)^2 - (50)^2}{100}\right] X$$

$$h(x) = \sqrt{2500 - 21\,X}$$

$$V(x) = 5 \times \frac{((50)^2 - (20)^2)/100}{2\,h(x)} = \frac{52.5}{h(x)}$$

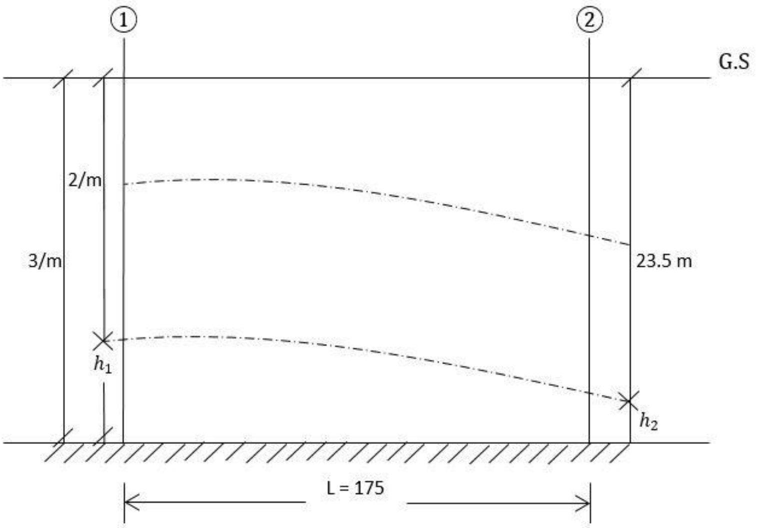
$$\frac{dh}{dx} = \frac{((20)^2 - (50)^2)/100}{2\,h(x)} = -\frac{10.5}{h(x)}$$

Example: water table flowing in sandy aquifer with hydraulic conductivity of (0.002 cm/s) and the aquifer thickness is (31 m), at well (1) water level (21 m) below ground surface, at well (2) water level (23.5 m) below ground surface. The distance between the two wells is (175 m).

Find:-

1- The discharge per unit width.
2- The hydraulic head at distance of (100 m).

Solution:



1) $q = k\ \left({h_1}^2 - {h_2}^2\right)/2L$
$k = 0.002\, cm/s$
$k = 0.002\,(3600 \times 24)/100 = 1.7\ m/day$
$L = 175\, m$

$h_1 = 31 - 21 = 10\ m\ , h_2 = 31 - 23.5 = 7.5\ m$
$q = 1.7\ ((10)^2(7.5)^2)/(2 \times 175) = 0.212\ m^3/day$

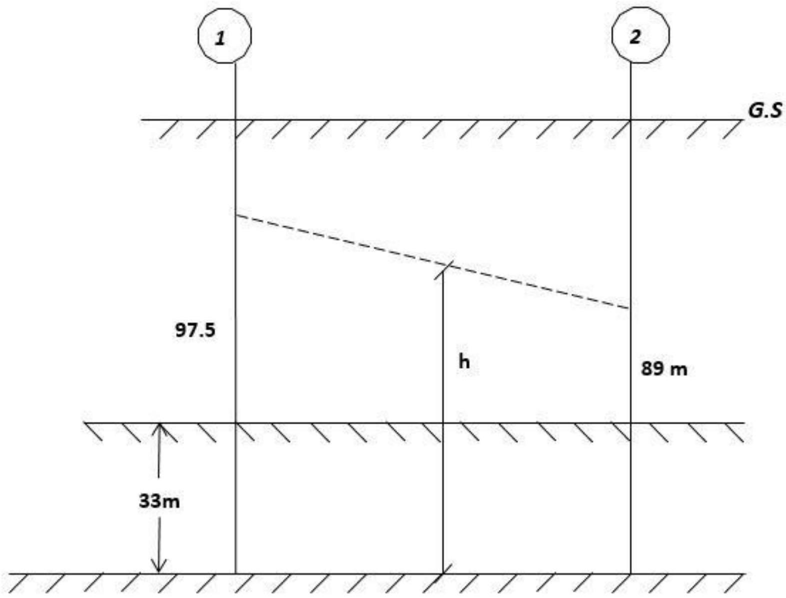
2) $h^2(x) = {h_1}^2 + ({h_2}^2 - {h_1}^2)(x/L)$
$x = 100\ m\ , L = 175\ m$
$h^2(x) = (10)^2 + ((7.5)^2(10)^2) \times \frac{100}{175} = 75\ m$

$h(x) = 8.66\ m$

Example: (33 m) thick confined aquifer, (7 km) wide, for two observation wells (1.2 km) apart head reading at well (1) was (97.5 m) and at well (2) was (89 m). if (k) equal to (1.2 m / day)

Find
1) The total daily flow.
2) The hydraulic head of an intermediate distance (x) between the wells.

Solution:



Lecture - 6

1) $Q = K\ i\ A$

$$= (1.2) \times \frac{(97.5 - 891)}{1200} \times (33 \times 7000)$$

$$= 1963.5\ m^3/day \leftarrow$$

2) $I = \frac{Q}{K\ A}$

$$i = \frac{1963.5}{1.2\ (33 \times 7000)} = \frac{(97.5 - h)}{600}$$

$$h = 93.25\ m$$

Lecture - 6

Electrical Analogy.

Analogy between the flow of ground water and electricity:-

(1) <u>For ground water:</u>

$Q = K.A. (\Delta h/L)$

$Q = K\ (BD).(\Delta h/L)$

Let $V = Q/(BD)$ and $\nabla h = (\Delta h/L)$

$V = K.\ h$ -------- (1)

Where:-

$\nabla = (\partial/\partial x) + (\partial/\partial y) + (\partial/\partial z)$

And $(\partial v_x/\partial x) + (\partial v_y/\partial y) + (\partial v_z/\partial z) =$ ------- (2) (continuity Eq.)

Sub. (1) in (2):-

$(\partial^2 h/\partial x^2) + (\partial^2 h/\partial y^2) + (\partial^2 h/\partial z^2) =$ $\nabla^2 h = 0$ (Lap Lace's Eq.)

(2) For Electricity:

$\Delta V = R.I$ --------- (1)

Where:-

R: Resistance (ohm)

I: Current (Ampere)

ΔV: Potential difference (volts)

$R = (1/\delta).(L/BD)$ -------- (2)

Where:-

δ: Specific Conductivity (1 / ohm .m)

Combining (1) and (2):

$I = \delta.(\Delta V/L).\ (BD)$

Let: $J = (I/BD)$ and $\nabla V = -(\Delta V/L)$

Where:-

J: Current density (Ampere / m^2)

Lecture - 6

$J = \nabla V$ --------- (3)

$(\partial J_x/\partial x) + \left(\partial J_y/\partial y\right) + (\partial J_z/\partial z) = 0$-------- (4) (Continuity Eq.)

Sub. (3) In (4):-

$(\partial^2 V/\partial x^2) + (\partial^2 V/\partial y^2) + (\partial^2 V/\partial z^2) = \nabla^2 V = 0$ (Lap Lace's Eq.)

<u>Corresponding Elements between Ground flow and Electricity:-</u>

Ground Water	Electricity
1- Hydraulic head differences ($\Delta \boldsymbol{h}$) (m)	1- potential difference (ΔV) (volt)
2- Hadronic conductivity ($\boldsymbol{K}$) (m /day)	2- Specific conductivity (δ) (1 / ohm.m)
3- flow rate ($\boldsymbol{Q}$) (m^3 / day)	3- Current (I) (Ampere)
4- Specific discharge ($\boldsymbol{V}$) (m/s)	4- Current density (J) (Ampere / m2)
5- Darcy's Law: ($\boldsymbol{v} = \boldsymbol{K}. \quad \boldsymbol{h}$)	5- ohm's Law: $J = \delta. \quad V$)
6- Lap lace's Eq. ($\boldsymbol{\nabla^2 h = 0}$)	6- Lap lace's Eq. ($\nabla^2 V = 0$)

Lecture - 7

Infiltration

Infiltration: A portion of the precipitation reaching the ground percolates into ground and is called infiltration. A part of the infiltrated water is held by capillarity at or near the ground surface and is ultimately evaporated from surface. another portion is used by vegetation and returned to the atmosphere as transpiration. Some portion percolates deep in to the ground and joins the water table as ground water. Another portion may drain in to the ocean. The remaining small portion may percolate great depth and appear at distant place as artesian wells. Infiltration is the process which water enters the soil from the ground surface.

حركة مرور الماء من سطح التربة راسيا الى اسفل داخل مسام التربة ويعبر عن معدل الترشح (infiltrating rate) من المياه المعطاة للأرض بوحدات عمق مكافئ من المياه المترشحة أي mm/hr (وحدات العمق لوحدة الزمن) اما سعة الترشيح (infiltration capacity) هي اقصى معدل للترشح يمكن ان تستوعبه التربة المحددة في ظروف معينة ولها نفس وحدات معدل الترشيح

ان القوى الأساسية المسببة لعملية الارتشاح هي قوى الشد الشعري المتأتية أصلا من قوى تلاصق جزيئات الماء بأسطح حبيبات التربة (قوى الشد السطحي) وقوة الجذب الأرضي نحو الأسفل.

- التغلغل percolation هو ظاهر حركة الماء داخل او خلال جسيم ترابي ليس بالضرورة ان يكون مشبعا بالماء.
- اما النفاذية للتربة فهي باختصار قابلية التربة المشبعة على انتقال حركة الماء بداخلها وتعد من صفات التربة.
- اما الارتشاح فهو ليس صفة ثابتة وانما يتأثر بعدد كبير من العوامل.

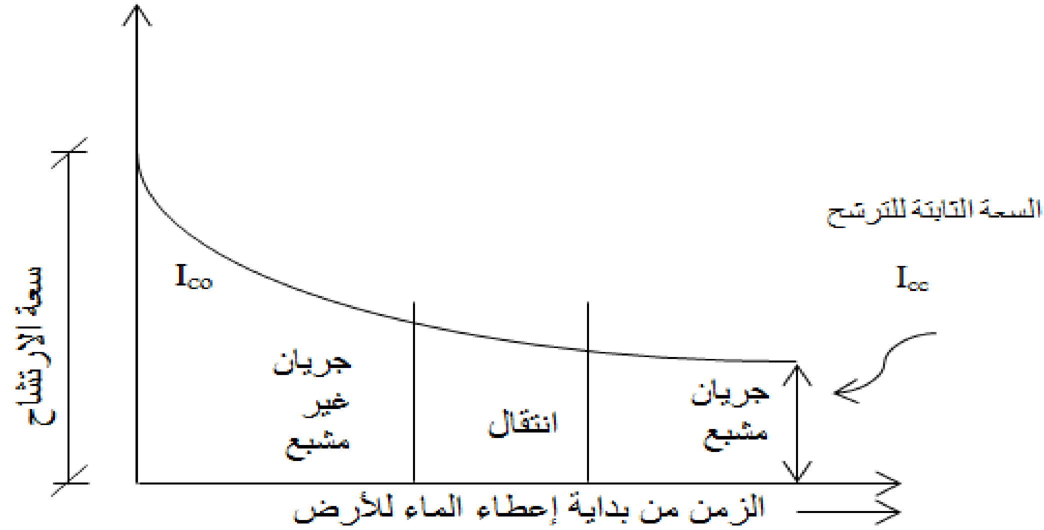
Factors affecting infiltration rates

1. Soil moisture: when the soil is dry, the infiltration rate is high because there is a strong eapillary attraction for the moisture which acts in the same direction as gravity. As the soil becomes saturated the capillary attraction is reduced and the infiltration rate decreased.
2. Type of soil medium: the infiltration rate depends upon the type of soil, it’s texture, the amount of clay and colloids in the soil, and the thickness and depth of permeable layers.
3. Permeability: the infiltration of rate depends upon the permeability (or the transmission capacity) of the soil. Infiltration will continue only if the infiltrated water is transmitted by the soil.

4. Vegetal cover: the dense vegetal cover over the surface of the soil increase the infiltration rate. The vegetal cover provides protection to the soil surface against the compaction due to impact of rain drops. Moreover, it also provides a layer of decaying matter with cavities which increase infiltration.
5. Surface fines: if the soil surface is sealed with soil fines, the infiltration rate is reduced.
6. Compaction of soil: caused by impact of rain drops or that caused by men and animals reduced the infiltration rate.
7. Available storage in soil stratum: the available storage in soil depends upon the thickness of the stratum, porosity and the water content of the soil. The infiltration is more available storage is large.
8. Depth of surface detention: after satisfying the interception and depression storage loss, the rain water collects over the ground surface as surface detention. The rate of infiltration increases as the depth of surface detention increases because the head causing flow is increased.
9. Temperature of water: an increase in temperature cased a reduction in the viscosity of water and consequent increase in the rate of infiltration.
10. Other factors: a large number of other factors such as entrapped air in the soil pores, quality of water, turbidity of water salt content, and freezing characteristics of soil also affect the rate of infiltration.

عندما تتدفق المياه من احدى فتحات الري لتلامس سطح التربة الجافة فان سعة الترشح تتناقص تدريجيا مع مرور الوقت الى ان تثبت قيمتها وتسمى في هذه الحالة سعة الترشح الثابتة (constant infiltration capacity) كما في الشكل ادناه. ويعزى تناقص سعة الترشح بمعدل سريع في المرحلة الأولى من بداية إعطاء المياه للتربة الى شد (مص) المياه من اعلى الى أسفل أي في اتجاه حركة الماء المنجذب وخلال هذه المرحلة التي تعرف بمرحلة جريان غير المشبع يتم بواسطة خاصية الشعرية ملئ المسام الدقيقة للتربة بالمياه مما يقلل من استيعاب التربة بعد ذلك للترشح، مع استمرار عملية الترشح تستمر حركة المياه خلال المسام الكبيرة وعندما يزداد المحتوى الرطوبي للتربة عن حاجتها (سعتها الحقلية) ويتلاشى تأثير الضغوط الشعرية، تثبت قيمة سعة الترشح وتسمى بمرحلة الجريان المشبع.

وبين مرحلتي الجريان المشبع والجريان الغير مشبع مرحلة انتقال تتناقص خلالها سعة الترشح بمعدل بسيط نسبيا.



وقد وجد العالم هورتن (Horton) ان منحني تغيير سعة الترشح مع الزمن يمكن تمثيله بالمعادلة التالية:

$I_c = I_{cc} + (I_{co} - I_{cc}).\ e^{-kt}$

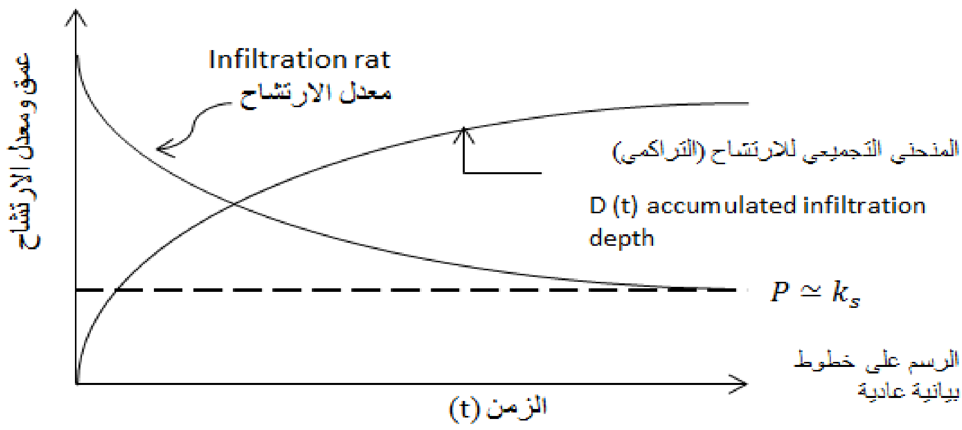
$I_c =$ سعة الترشح بعد (t) من الزمن

$I_{co} =$ سعة الترشح العظمى

$I_{cc} =$ سعة الترشح الثابتة

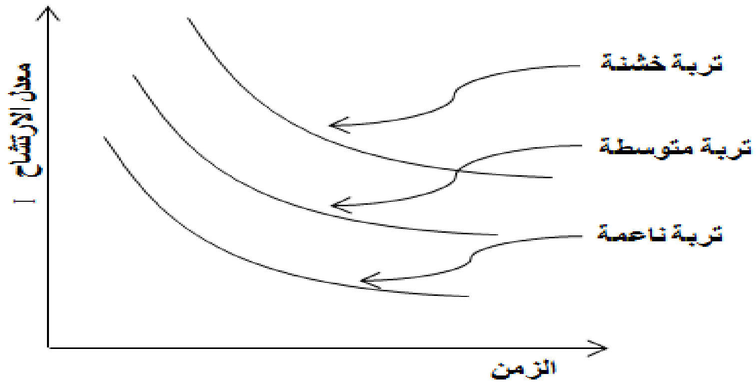
ويشترط في استعمال المعادلة أعلاه ان يكون معدل إعطاء المياه للأرض أكثر او يساوي سعة الترشح.

- من المفيد أحيانا دراسة المنحني التجميعي للترشح لتحديد كمية المياه التي ترشحت داخل الأرض بعد أي فترة زمنية من بداية إعطاء الماء للأرض ويمثل هذا المنحني واقع تغيير سعة الترشح مع الزمن على النحو التالي.



الشكل يمثل تغيير معدل الارتشاح I (t) والارتشاح التراكمي D (t) مع الزمن.

إذا استمرت عملية الارتشاح لفترة زمنية طويلة فان معدل الارتشاح يقترب من قيمة ثالثة (P). وعموما فان قيمة الثابت P يجب ان يكون مساوياً للإيصالية المائية للتربة المشبعة (K_s)، الا ان انحباس وانحصار الهواء بين دقائق التربة يمنع التربة من الوصول الى حالة الشبع الكامل التي عندها يكون المحتوى الرطوبي للتربة (على أساس الحجم) يساوي مسامية التربة porosity ولهاذا السبب فان P تكون اقل من K_s بقليل.



الشكل يمثل تأثير قوام التربة على معدل الارتشاح.

تعد معادلة كوستاكوف (Kostiakov) من المعادلات الوضعية لوصف ارتشاح الماء في التربة:

$D = ct^m$

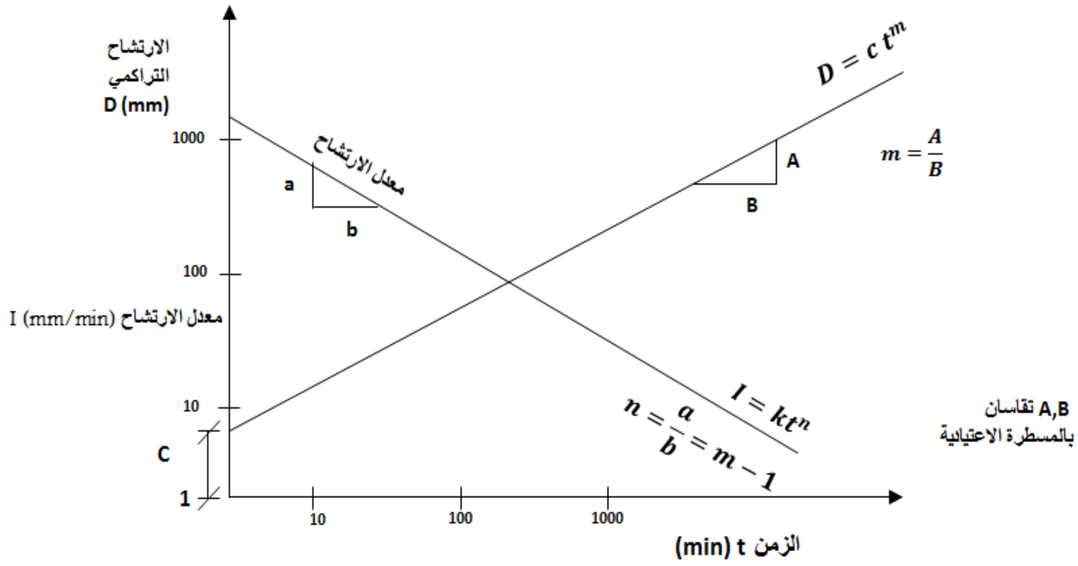
Where

D = accumulated infiltration depth (mm) عمق الارتشاح التراكمي

t = time accumulated Infiltration (min) زمن الارتشاح التراكمي

C,m = constant ثوابت

المعادلة أعلاه لوغاريتمية. فاذا رسم Dمقابل t على ورق لوغاريتمي log-log paper نحصل على خط مستقيم مائل تقاطعه مع D عند الزمن دقيقة واحدة (t = 1 min)تمثل قيمة معامل الارتشاح (C) وميله الحسابي هو $m\ \ (m = \frac{A}{B})$



المعادلة على ورق لمقياس لوغاريتمي

وللحصول على معادلة لوصف معدل ارتشاح الماء في التربة نشتق المعادلة أعلاه بالنسبة للزمن كالتالي:

$$\frac{dD}{dt} = I$$

$$D = ct^m$$

$$I = cm\,t^{m-1}$$

$$I = kt^n$$

Where

k = constant = cm

n = constant = m – 1

I = infiltration rate (mm/min) معدل ارتشاح الماء في التربة

n = m – 1

وبتحليل بسيط للمعادلتين يمكن اثبات ان قيمة m تقع بين الصفر والواحد وان قيمة n تقع بين الصفر وناقص واحد وكالاتي:

بما ان D تزداد تراكميا مع الزمن (t) فان قيمة (m) يجب ان تكون موجبة $(m > 0)$ وبما ان معدل الارتشاح I يقل بزيادة الزمن فان قيمة (n) يجب ان تكون سالبة $(n < 0)$ لذا لتحقيق الشروط الثلاثة لابد ان يتحقق الاتي:

$$m > 0 \quad and \quad n < 0$$

$$n = m - 1$$

$0 < m < 1$

$-1 < n < 0$

وعلى الرغم من نجاح المعادلة السابقة في وصف معدل الارتشاح خلال الساعات الأولى من عملية الارتشاح الا ان هناك ضعفا من الناحية النظرية في هذه المعادلة لأنه عندما يزداد الزمن (t) بشكل كبير تقترب قيمة معدل الارتشاح (I) الى الصفر. وهذا غير صحيح لان اقل قيمة لمعدل ارتشاح الماء في التربة هي النفاذية على أساس ان التربة تصبح مشبعة بالماء اذا كان سطحها مغطى بالماء لفترة طويلة جداً.

Exercise: The data below is from field tests of soil infiltration:

time (min) Accumulated infiltration time (min)	5	120
Accumulated infiltration depth (mm)	13	52

Find

a) The infiltration rate.
b) The infiltration function of depth.

Solution

$D = ct^m$

$13 = c(5)^m$

$52 = c(120)^m$

بحل المعادلتين

$c = 6.4$

$m = 0.44$

$\therefore D = 6.4\ t^{0.44}$

$\frac{dD}{dt} = I$

$I = 6.4\ (0.44)t^{+0.44-1}$

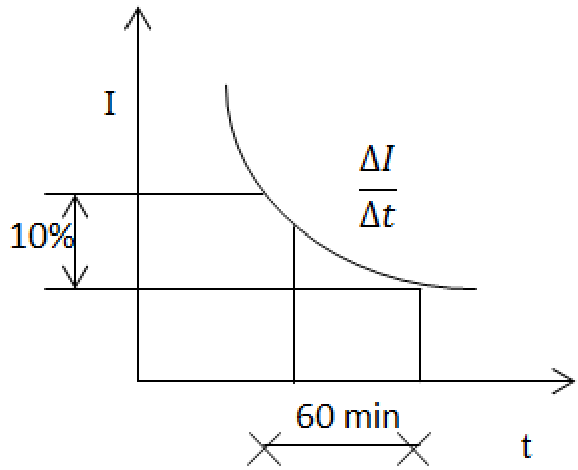
$I = 2.8\ t^{-0.56}$ (mm/min)

Lecture - 7

Basic infiltration rate. معدل الارتشاح الأساس

هو تلك القيمة على منحني الارتشاح التي يكون عندها التغيير في معدل الارتشاح خلال ساعة واحدة ولا يزيد عن (10%) ويمكن ايجاده من تقديره من زمن الارتشاح الأساسي. ويعرف عموما بمعل الارتشاح الثابت تقريبا الذي يحصل بعد مرور فترة زمنية منذ بداية عملية الارتشاح او الارواء. وقد لا يصل الارتشاح الثابت الى هذه القيمة النسبية اثناء ارواء الترب الخشنة لان عملية الارتشاح تحصل خلال زمن قصير عادة بينما في حالة الترب المتوسطة والناعمة القوام حيث يكون زمن الارواء او الارتشاح طويلا نسبيا لذا فان مفهوم معدل الارتشاح الأساس يكون أكثر أهمية ووضوحاً

$$T_b = |600n|$$



$$\frac{\Delta I}{I} \le 0.1$$

$$\Delta I = 0.1\, I$$

$$I = kt^n$$

$$\frac{\Delta I}{\Delta t} = kn\, t^{n-1}$$

$$\Delta I = kn\, t^{n-1}\,(\Delta t)$$

$$0.1\, I = kn\, t^{n-1}\,(\Delta t)$$

$$0.1kt^n = kn\, t^{n-1}\,(60)$$

$$0.1\, t^n = n\;\; t^n.\;\, t^{-1}(60)$$

$$(60)\frac{1}{t}(n) = 0.1 \Rightarrow \qquad -1 < n < 0$$

$$t = 600\, n$$

$$\therefore\;\; T_b = |600n|$$

وبعد معرفة الزمن (T_b) الذي يحصل عنده معدل الارتشاح الأساس يمكن تخمين قيمة معدل الارتشاح الأساس بتعويض قيمة الزمن (T_b) في معادلة الارتشاح العامة $I = kt^n$.

ويعد البعض معدل الارتشاح الأساسي من صفات التربة التي تعكس قوامها وتركيبها ويمكن تقسيم التربة على أساس قيم معدل ارتشاحها الأساس. وتبرز أهمية هذا المفهوم في دراسة وتصاميم نظم الري بالرش التي تحدد معدل الارواء (شدة الرش) المناسب للتصميم

للسؤال السابق

$$\mathrm{T_b} = |600\mathrm{n}|$$

$$\mathrm{T_b} = |600\;\times\; -(0.56)|\;\; = 366\;\mathrm{min}$$

$I_b = 2.8\,(366)^{-0.56} = 0.108$ mm/min

$I = 6.465$ mm/hr.

Measurement of infiltration capacity(flooding – type infiltrometer)

A simple flooding type infiltrometer consists of a metal cylinder open at both ends and about (22.5 cm) in diameter and (60 cm) long. The cylinder is driven in to the ground with a driving plate and hummer such that about (10 cm) length projects above the ground surface. Water is filled in the cylinder to maintain the water depth of about 55 cm in it. A pointer is set to mark the water level. As the infiltration take place, the water level goes down. The water level is mainted constant by adding water from burette. Reddings of the burette are taken at a regular time intervel to determine the rate of infiltration is obtained. The experiment is generally continued till a constant rate of infiltration is obtained, which usually accurse after 2-3 hrs deeping upon the type of soil. A plot is made between the infiltration capacity and time.

Lecture - 8

Drainage networks   شبكات البزل

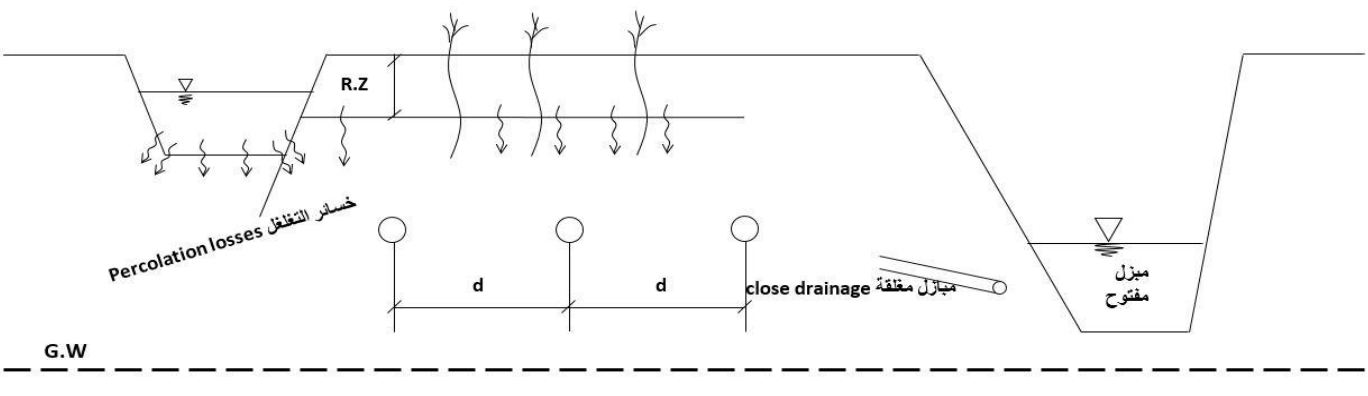
- **Drainage:** is the process of which the excess water removed from the soil.
  عملية إزالة الماء الفائض عن حاجة التربة
- **Reclamation:** is the adequate lowering of the water table.
- **Benefits of drainage in general:**
  1. Improves the soil structure. تحسين هيكل التربة
  2. Increase the productivity of soil. زيادة إنتاجية التربة
  3. Reclamation of saline and al kali soils. استصلاح الترب القوية والمالحة

Sources of excess water in the soil. **مصادر الماء الزائد (الفائض) عن حاجة التربة**

1. Seepage losses from reservoirs.
   الخسائر الناتجة عن النضخ (التسرب) في القنوات والخزانات
2. Deep percolation losses from irrigated lands.
   خسائر التغلغل العميق الناتجة عن مياه الارواء للأرض
3. Flooding of low lands due to overflow of rivers.
   فيضانات الأراضي المنخفضة نتيجة طفح الأنهار
4. Up word flow from an artesian aquifer. ارتفاع المياه من الابار الارتوازية

Field investigation **الفحوصات الحقلية**

1. Topography → topographic survey and area maps.
2. Soil → permeability, location. النفاذية للتربة والموقع للتربة
3. Water table → water table depth. عمق مستوى الماء
4. Water sources → quantity and quality. نوعية وكمية المياه الصالحة للتربة



Lecture - 8

Depth and spacing of drains

The position of the water table will depend upon the following factors:

1. The rain fall rate or the rate of irrigation water applied. معدل الامطار والارواء
2. Soil hydraulic conductivity. التوصيل (النفاذية)
3. Depth and spacing of drains. (ارتفاع منسوب الماء وعددها الجوفي)
4. The depth of the impermeable layer. عمق الطبقة الصماء (غير النفاذة)

- المبازل أما مفتوحة والتي تكون غالباً أما على شكل شبه منحرف ذي انحدار جانبي يتراوح ما بين (1:1⇒1:2) حسب الخواص الفيزيائية للتربة او مبازل مغطاة وهي المبازل التي تدفن تحت سطح التربة للمحافظة على منسوب المياه الجوفية والتخلص من الماء الزائد. وقد توجد المبازل المغطاة مع المبازل المفتوحة جنباً الى جنب في شبكات البزل المنفدة (كما في العراق). ويفضل البزل المفتوح في المناطق التي تكون فيها كميات المياه السطحية كبيرة وبصورة دائميه وكذلك في المناطق التي قد تتعرض فيها المبازل المغطاة للانسداد بفعل الترسبات الكيميائية وخاصة في المناطق ذات الترب العضوية. ان الكلفة الانشائية الأولية للمبازل المغطاة هي بحدود ثلاثة اضعاف كلفة المبازل المفتوحة. ومن المعروف ان تصريف المبازل لا يكون منتظما وان معرفته ذات أهمية لكونه يدخل ضمن المعادلات في حساب الأبعاد ما بين المبازل. وكلما زاد عمق المبازل ازدادت قدرة التربة على مسك الماء فوق المبزل مقارنة بالمبازل الضحلة، ونتيجة لذلك فان المبازل العميقة تعطي تصريفا بفترة أطول بين الريات في المناطق الجافة وشبه الجافة المروية، وان المبازل العميقة يكون تصريفها قليلاً.

Drainage system **نظم البزل**

أ- البزل السطحي. Surface drainage

ب- البزل تحت السطحي. Subsurface drainage

ويمكن تقسيم المبازل الى

أ- المبازل المفتوحة. Opened drainage

ب- المبازل المغلقة. Closed drainage

Lecture - 8

المبازل المفتوحة

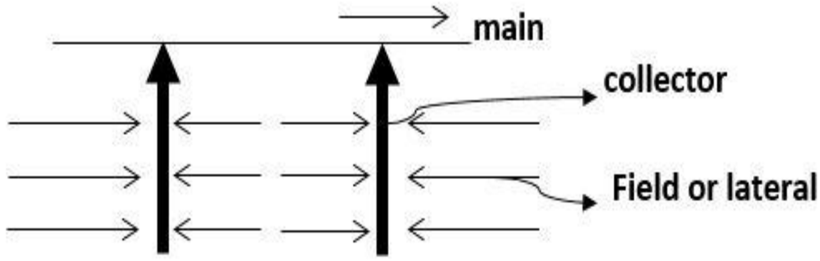
اشكالها / مربع ، مستطيل ، شبه منحرف ، ،

وتتميز هذه المبازل بـــــ

1. كبير السعة.
2. تجمع مياه R-o.
3. في حالة زيادة العمق تخفض W-T.

In any system of drains one may distinguish between:

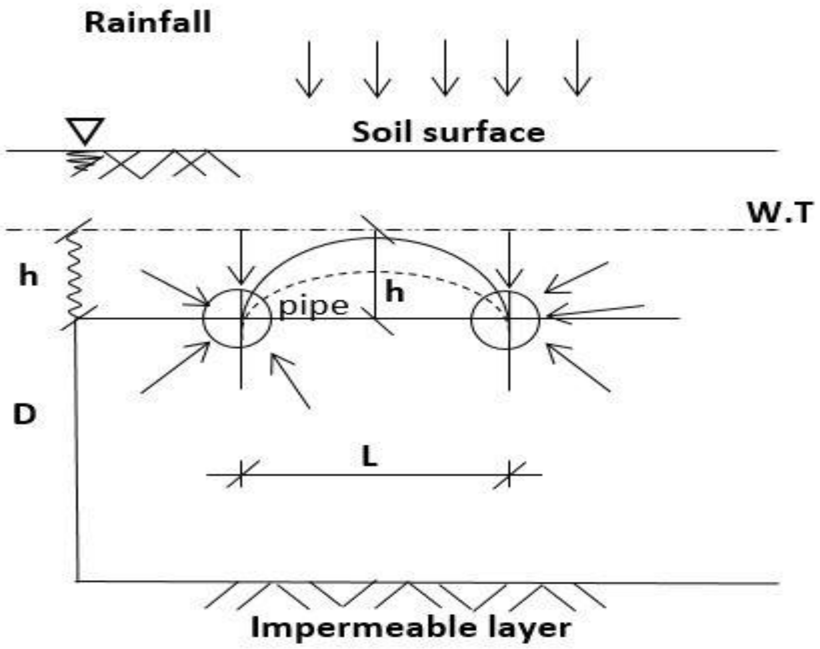
- Field drains or field laterals, usually parallel drains whose function is to control the ground water depth.
- Collector drains, whose function is to collect water from the field drains and to transport it to the main drains.
- Main drains, whose function is to transport the water from out of the area.



The factors which influence the height of the water table are:

- Precipitation and other sources of the recharge.
- Evaporation and other sources of the discharge.
- Soil properties.
- Depth and spacing of the drains.
- Cross-sectional area of the drains.
- Water level in the drains.

Lecture - 8



Steady state drainage equation

There are two simplified assumptions:

1. Two-dimensional flow. جريان ثنائي الابعاد
2. Uniform distribution recharge. معدل تغذية الماء الجوفي ثابتة مع الزمن

Lecture - 8

Hooghoudt's Equation For The Water Table In Equilibrium With Rainfall Or Irrigation Water

$$L^2 = \frac{4kh^2}{q} + \frac{8khd}{q} \qquad (based\ on\ Darcy's\ low)$$

From above drain level

كمية الجريان باتجاه المبزل من المنطقة التي تقع فوق المبزل

From below drain level

كمية الجريان باتجاه المبزل من المنطقة المحصورة بين المبزل والطبقة الصماء

Where:

q = discharge per unit length of drain (the recharge) (m/d معامل البزل (كمية التغذية

K = hydraulic conductivity (m/d).

L = drain spacing (m).

D = depth of the impermeable layer. (m) عمق الطبقة الصماء عن المبزل

h = hydraulic head between two drains. (m) ارتفاع الماء الجوفي عن المبزل عند منتصف المسافة بين المبزلين

$$q = \frac{deep\ percolation\ losses\ \text{خسائر التغلغل}\ (m)}{time\ intervale\ between\ two\ irrigation\ \text{الزمن بين سقيتين}}$$

اشكال معادلة *Hooghoudt's* تحت حالات مختلفة

A. For shallow aquifer.

$$L^2 = \frac{4kh^2}{q} + \frac{8khD}{q}$$

- التربة متجانسة ويقع المبزل فوق الطبقة الصماء بمسافة معينة (الطبقة الصماء قريبة من سطح التربة) (العمق تقريبا اقل من *3m*)

B. For deep aquifer

$$L^2 = \frac{4kh^2}{q} + \frac{8khd}{q}$$

Lecture - 8

ـ التربة متجانسة والطبقة الصماء على بعد كبير عن المبزل تعدل قيمة D الى ما يعادلها وهنالك مرتسمات او جداول خاصة لاستخراج العمق المكافئ للطبقة الصماء وفضلا عن المرتسمات والجداول الخاصة بتحديد العمق المكافئ (الفعال) للطبقة الصماء يمكن استخراجها بالمعادلة التالية

$$d = \frac{D}{\left[\left(\frac{8D}{\pi L}\right) 1n \left(\frac{D}{u}\right)\right] + 1}$$

Where

d = equivalent depth of the impermeable layer (m). العمق المكافئ للطبقة الصماء عن المبزل

u = wetted circumference = half of drain circumference.

نصف محيط الانبوب

u = $r°\pi$

$r°$ = drain radius (m). نصف قطر أنبوب المبزل

C. For layered aquifer التربة متجانسة ويقع المبزل فوق الطبقة الصماء بمسافة معينة

$$L^2 = \frac{4k_1h^2}{q} + \frac{8k_2dh}{q}$$

k_1 = *النفاذية للطبقات المشبعة بالماء فوق المبزل*

k_2 = النفاذية للطبقات الفاصلة بين المبزل والطبقة الصماء

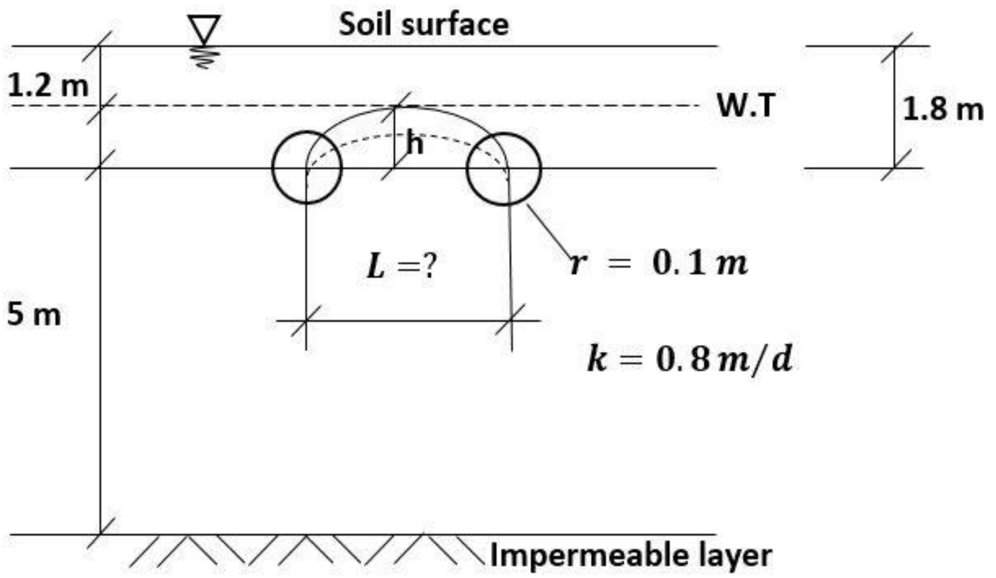
D. The drain at the impermeable layer. يقع المبزل فوق الطبقة الصماء مباشرة

$$L^2 = \frac{4kh^2}{q}$$

Lecture - 8

Exercise:

For the derange of an irrigated area a drain pipe of radius of (0.1 m) was used at depth of (1.8 m) below soil surface the impermeable layer was found at depth of (6.8 m) below surface of soil. The hydraulic conductivity was (0.8 m/d), irrigation was applied once every twenty days. The irrigation losses which recharge the ground water table is (40 mm) from each irrigation. What drain spacing must be applied when the average water table is (1.2 m) below soil surface.



Solution:

$$D = 6.8 - 1.8 = 5\ m$$

$$h = 1.8 - 1.2 = 0.6\ m$$

$$q = \frac{40 \times 10^{-3}}{20\ day} 0.002\ m/d$$

الطبقة الصماء عميقة لذا يستعمل

$$L^2 = \frac{4kh^2}{q} + \frac{8kdh}{q}$$

$$L^2 = \frac{4(0.8)(0.6)^2}{0.002} + \frac{8(0.8)(0.6)d}{0.002}$$

$$L^2 = 576 + 1920\ d \qquad \dots\dots\dots\dots 1$$

Lecture - 8

طريقة الحل

اما باستعمال الجداول او *trail & error*

$assume\ L = 80\ m \qquad for \quad r = 0.1\ m$

$d = 3.55\ m \quad (table)$

check

$use\ L = 85\ m \qquad \therefore d = 3.61\ m$

$(85)^2 \cong 576 + 1920(3.61) \qquad O.K.$

$\therefore L = 85\ m$

طريقة أخرى

Or

$$d = \frac{D}{\left[\left(\frac{8D}{\pi L}\right) |n \left(\frac{D}{u}\right)\right] + 1}$$

$\mathrm{u} = r^{\circ}\pi$

$= 0.1\pi$

$= 0.314$

$$d = \frac{5}{\left[\left(\frac{40}{\pi L}\right) |n \left(\frac{5}{0.314}\right)\right] + 1} \dots\dots\dots\dots\dots (2)$$

وبحل المعادلتين (1) و(2) ذاتا المجهولين L,d نجـد

$L = 85\ m \qquad d = 3.61\ m$

- لا يفضل تقليل المسافة بين المبازل لان هذا غير اقتصادي.
- المسافة L من مضاعفات الرقم (5).

Lecture - 8

Van Beers Approach

$$L_o^2 = \frac{4kh^2}{q} + \frac{8kDh}{q} \quad (Hooghoudt\ Eq.)$$

$$L_o^2 = \frac{4kh^2}{q} + \frac{8kdh}{q} \quad (Van\ Beers\ approach)$$

عندما يكون *Equifir* ضحل او *W.T (G W)* عميق.

ويمكن الاستغناء عن المعادلات باستخدام بعض المرتسمات

$$L = L_o - C$$

$$C = D\ 1n\left(\frac{D}{u}\right)$$

Where:

L = actual drain spacing (m).

L_o = approximately drain spacing (m).

C = correction factor. *معامل التصحيح*

Exercise

Given h = 0.6 m , D = 5 m , k = 0.8 m/d , q = 0.002 m/d , r^o = 0.1 m

Solution

$$L_o^2 = \frac{4 \times 0.8 \times (0.6)^2}{0.002} + \frac{8 \times 0.8 \times 5 \times 0.6}{0.002}$$

$$L_o^2 = 576 + 9600 = 10176$$

$$L_o = 100.876 \simeq 100\ m$$

$$u = r^\circ \pi$$

$$= 0.1\pi$$

$$= 0.314\ m$$

$C = 5\ 1n\left(\frac{5}{0.314}\right) = 13.836\ m$

$\therefore L = 100 - 13.836 = 86.164\ m$

$use\ L = 85\ m$

Principle of the Kirkham equation.

$h = \frac{qL}{k} F_k$ (1958) ………… 1 $(steady\ state\ flow)$

and

$$F_k = \frac{1}{\pi}\left[|n\ \frac{L}{\pi ro} + \sum_{n=1}^{\infty} \frac{1}{n}\left(\cos\frac{2n\pi ro}{L} - \cos n\pi\right)\left(\coth\frac{2n\pi D}{L} - 1\right)\right]$$

Value of F_K are given in the table below.

L/D	(the flow above the drain is ignored)
$\frac{D}{2ro}$	

In the solution represented by eq. 1 the flow is in the upper region has been neglected in the later paper Kirkham (1960) reported region [the flow in the layer above the drain level] the general equation for a two-layer problem is

$h = \frac{qL}{k_b}\ \frac{1}{1 - q/k_a} F_k$

k_a= soil permeability for the above drain level. نفاذية التربة اعلى المبازل

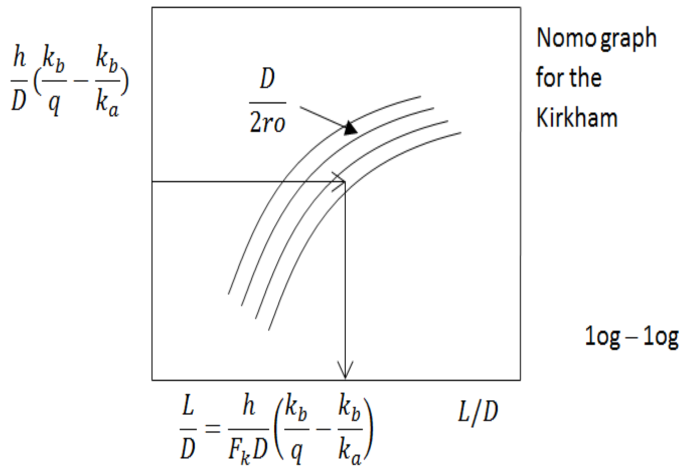
k_b= = soil permeability for the below drain level. نفاذية التربة أسفل المبازل

إذا التربة نوع واحد $k_a = k_b = k$

Lecture - 8

Exercise

$k_a = k_b = 0.8/day$



$r_o = 0.1\ m$

$h = 0.6\ m$

$q = 0.002\ \ m/day$

$k_a = k_b = k$

, if the vertical flow is assumed in this region

$$\frac{k_a}{k_b} = 1$$

Solution

$$\frac{h}{D}\left(\frac{k}{q} - 1\right) = \frac{0.6}{5}\left(\frac{0.8}{0.002} - 1\right) = 48$$ بدون وحدات

$$\frac{D}{2ro} = \frac{5}{2(0.1)} = 25$$

$$\therefore \frac{L}{D} = 17 \Rightarrow L = 17(5) = 85\ m$$

Principles and applications of Dagan equation

The Dagan equation, is a form similar to the Hooghoudt and Kirkham equations:-

$$h = \frac{qL}{k} F_D$$

Where

$$F_D = \frac{1}{4}\left(\frac{l}{2D} - \beta\right)$$

Where

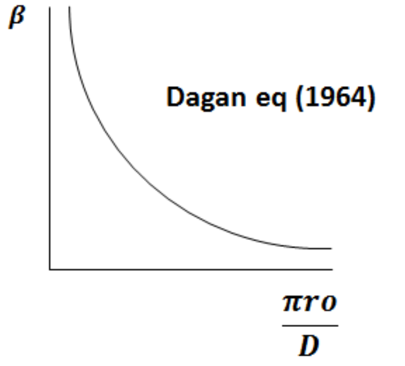
$$\beta = \frac{2}{\pi}\ ln\ \left(2\cosh\frac{\pi ro}{D} - 2\right)$$

Lecture - 8

In the fig below the term β has been represented as a function of

$\frac{\pi ro}{D}$

- Note: that β -values are negative.



Exercise

The same example before

Solution

$$\frac{\pi ro}{D} = 3.14 \times \frac{0.1}{5} = 0.06 \Longrightarrow \beta = -2.1$$

$$F_D = \frac{1}{4}\left(\frac{l}{2D} - B\right) = \frac{1}{4}\left(\frac{L}{2D} + 2.1\right)$$

$$h = \frac{qL}{k} F_D = \frac{qL}{4k}\left(\frac{L}{2D} + 2.1\right)$$

$$0.6 = \frac{0.002L}{0.8 \times 4} \times 4\left(\frac{L}{2 \times 5} \times 5 + 2.1\right)$$

$$L^2 + 21L - 9600 = 0$$

$$L = \frac{-21 \mp \sqrt{(21)^2 + 4 \times 9600}}{2} \Longrightarrow \qquad L = 88\ m$$

Lecture - 9

Daring coefficient معامل البزل

كمية المياه المبزولة في مساحة معينة خلال وحدة الزمن وغالبا ما يؤخذ الزمن 24 ساعة ويمكن التعبير عنه بالصيغ الاتية

1. عمق مكافئ من المياه المبزولة في وحدة الزمن مثلا mm/d
2. وحدات تصريف ثابتة من المياه المبزولة في وحدة المساحة مثلا m^3/d/donam
3. وحدات المساحة التي تعطي تصريف قدره وحدة واحدة لمدة 24 hr

Donam من مساحة m^3/s/day

يقسم معامل البزل الى

1. معمل البزل السطحي.
2. معامل البزل تحت السطحي (الباطني).

معامل البزل الكلي = معامل البزل السطحي + معامل البزل الباطني

العوامل التي تؤثر على قيمة معامل البزل

1. المساحة المبزولة daring area.
2. العوامل المناخية.
3. كمية مياه الري.
4. نوعية التربة والنبات.
5. عمق المبازل.
6. المسافة بين المبازل.

Hydraulic Design Of Open Drain

The hydraulic characteristic of the materials used for daring purposes must be known because they are used to carry the drainage water out of the filed. The size of the drains must be adequate to carry the water of the drainage water at the design slope.

Lecture - 9

أعماق المبازل المفتوحة

يتوقف عمق المبزل المفتوح على عدة عوامل:

1. نوعية التربة وصفاتها الطبيعية ونفاذيتها للمياه.

- الترب الرملية تحتاج الى مبازل عمقها اقل من الترب الغرينية التي تحتاج الى عمق اقل من الترب الطينية.

2. نوع النبات.
3. العوامل المناخية (العمق الحرج للمياه الجوفية critical depth) والذي يعتمد على درجات الحرارة العالية وكميات سقوط الامطار.
4. العوامل الاقتصادية.

- Note: if W.T is low (sandy soil) we will not need for drains.

لأنه الغرض من المبزل خفض منسوب الماء الجوفي W.T

العمق الحرج :- هو عمق الماء الجوفي الذي يحدث عنده الخاصية الشعرية فيرتفع الماء الى الأعلى مستصحبا معه الاملاح التي تتراكم على سطح التربة.

العمق ا = عمق الماء الجوفي المراد الحصول عليه.

العمق ب = مقدار الانخفاض منسوب الماء الجوفي باتجاه المبازل الحقلية ويؤخذ عادة من اعلى نقطة ف منحني سطح الماء الحر وتقع بين منتصف المسافة بين المبزلين الى سطح الماء داخل المبزل.

العمق ح = عمق الماء داخل المبزل ويلاحظ ان العمق ا يجب ان يكون كبيرا اذا كانت التربة متأثرة بالأملاح وذات نسجة ناعمة وذلك للابتعاد عن تأثير العمق الحرج للماء الجوفي وفي العراق يوصي بان يكون العمق ا ما بين (1.25 – 1.75 m) اما العمق ب فيعتمد على المسافة بين المبازل وفي العراق يكون حوالي (0.5 m) وهو مقدار متوسط الانخفاض في المنسوب للماء الجوفي اما (ج) فيتم احتسابه باستخدام المعادلات الهيدروليكية الخاصة بتصميم المقاطع.

- ان اعماق المبازل المفتوحة تتراوح غالبا ما بين (1.8 – 3.5 m)

Lecture - 9

Hydraulic design

By using Manning Equation: -

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

1. By trial and error solution.
2. Section factor method.
3. By using design chart for open drain. تخص فقط المبازل ولا تستخدم في القنوات المفتوحة لمعامل خشونة محدد وانحدار جوانب محدد

- Note: check Fr.no. for line drain. Check silt factor for unlined drain.

$$Fr = \frac{V}{\sqrt{gy}} \qquad Fr < 0.6$$

$$f = 1.76\sqrt{d}$$

$$or \qquad Dm = 2.46\frac{V^2}{f} \qquad f = 0.4 - 1$$

4. By using conveyance factor. طريقة معامل النقل

   $Q = K\, S^{1/2} \Longrightarrow \qquad V = c\sqrt{RS}$

   K = conveyance factor معامل النقل والذي يمكن حسابه من خلال معرفة كمية المياه بوحدة الزمن وانحدار القناة.

واهم الطرق في حساب K هي ايجاد جداول تبين نتيجة مساحة القطع A ومعامل النقل وعمق الماء Y وعرض القناة (b) لمعامل خشونة محدد ولانحدار جوانب محددة. والاعتماد هنا في التصميم على السرعة المحسوبة والمحددة بالسرعة المسموح بها والمعتمدة للتصميم.

Exercise

باستخدام conveyance factor method

Given Q = 0.15 m^3/s

S = 0.001

$$\begin{bmatrix} n = 0.025 \\ 1 : Z : 1 : 1 \end{bmatrix}$$

n = 0.025

Z = 1:1

Solution

$K = \frac{Q}{\sqrt{S}} = 4.743$

A	$b = 0.2\ m$	$y = 0.48$ $y = 0.5$	$K_1 = 4.608$ $K_2 = 5.054$
$A = 0.34\ m^2$	$b = 0.3\ m$	$y = 0.44$ $y = 0.46$	$K_1 = 4.193$ $K_2 = 5.071$

$y = 0.45$

$A = 0.33\ m^2$	$b = 0.4\ m$	$y = 0.4$ $y = 0.42$	$K_1 = 4.508$ $K_2 = 4.903$

$y = 0.41\ m$

$A = 0.33\ m^2$	$b = 0.6\ m$	$y = 0.34$ $y = 0.36$	$K_1 = 4.44$ $K_2 = 4.939$

$y = 0.35\ m$

$A = 0.33\ m^2$	$b = 0.8\ m$	$y = 0.3$ $y = 0.32$	$K_1 = 4.517$ $K_2 = 5.068$

$y = 0.31\ m$

$if \quad n = 0.02$

- ***Note:***

اذا كانت قيمة معامل الخشونة اقل مما هي عليه في الجدول مثلا n = 0.02 *فان قيمة* K *تتغير لتصبح*

$$K = K \times \frac{0.02}{0.025} = 3.794$$

وباستخدام الجدول يمكن إيجاد الاحتمالات التالية لأبعاد القناة

$\therefore K_{n0.025} = K_{0.02} \times \frac{0.02}{0.025}$

$= 4.743 \times 0.8 = 3.7944$

$b = 0.3\ m \qquad y = 0.4\ m \qquad A = 0.28\ m^2$

$b = 0.4\ m$ $\quad$ $y = 0.37\ m$ $\quad$ $A = 0.28\ m^2$

$b = 0.6\ m$ $\quad$ $y = 0.31\ m$ $\quad$ $A = 0.28\ m^2$

$b = 0.8\ m$ $\quad$ $y = 0.27\ m$ $\quad$ $A = 0.28\ m^2$

$b = 0.3\ m$

$b = 0.4\ m$

$b = 0.6\ m$

Exercise

Find the dimension of earth drain carrying discharge 0.45 m^3/s, n = 0.03, S = 0.00024

1.5 H : 1 V.

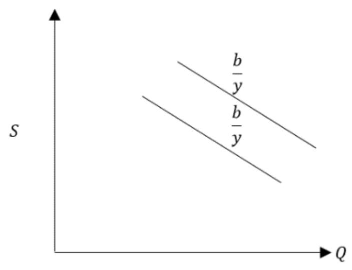
Solution

By using design chart

1. $b/y = 3$ $\quad$ $y = 0.6\ m \Rightarrow b = 1.8\ m$
2. $b/y = 2$ $\quad$ $y = 0.7\ m \Rightarrow b = 1.4\ m$

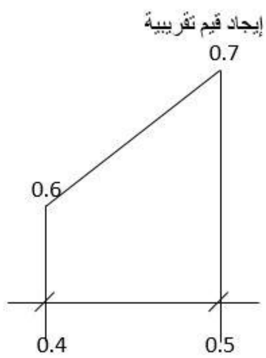
$b/y = 3$

$b/y = 2 \quad \Rightarrow$



تستخدم الجارت في الحالات التالية

1. قطع شبه منحرف
2. *Z = 1.5*
3. *n = 0.03*



Lecture - 9

ادامة المبازل المفتوحة Open drains maintenance

1. السيطرة على الترسبات في المبزل المفتوح.
2. السيطرة على انجراف كتوف المبازل.
3. السيطرة على انحدار الجوانب للمبزل.
4. السيطرة على نمو الادغال.

- ان المبازل الحقلية (الفرعية) المغطاة يمكن ان تصب في مبازل مجمعة مفتوحة وبذلك تسمى بالنظام المفرد *single system* او تصب في مبازل مجمعة مغطاة وبذلك تسمى بالنظام المركب *composite system*.

ان الاختيار ما بين النظامين يعتمد على بعض الاعتبارات وهي:

1. ان المبازل المجمعة المفتوحة توفر أحسن وسيلة للتخلص من المياه السطحية الزائدة.
2. ان الخسارة في الأرض باستخدام النظام المفرد قد تصل 2-3%.
3. سهولة فحص المبازل الحقلية مع النظام المفرد وكذلك ليس هناك تخوف من انسداد المبازل المجمعة في هذه الحالة.
4. المبازل المجمعة المفتوحة تحتاج الى صيانة أكثر من المغطاة.
5. ان كلفة النظام المفرد هي اقل من كلفة النظام المركب.

ومع كل الاعتبارات السابقة يفضل النظام المفرد في لمناطق ذات الطبوغرافية المستوية وذات المناخ الرطب. في حين يلائم النظام المركب المناطق ذات الطبيعة المنحدرة. وكذلك فان النظام المركب يفضل في لمناطق الاروائية, اذ ان المبازل المجمعة لا تتعارض مع قنوات الري.

المعايير التصميمية للمبازل المفتوحة *Drainage wells*

a. Permissible velocity *السرعة المسموح بها*

To avoid sedimentation of material *ترسب المواد العالقة*

To avoid scouring of canal *انحراف المقطع و تعريته*

وتعتمد السرعة المسموح بها

- *الصفات الهيدروليكية للقناة.*
- *حالة التربة (نسيج التربة وتركيبها).*
- *عمر القناة.*
- *وجود الحشائش والادغال في القناة.*

Lecture - 9

- *وجود العوالق في ماء البزل.*

$$V = Cy^x$$

V = velocity (ft/s)

y = water depth(ft)

x = constant (0.5 – 0.64) *يعتمد على وجود عالق او عدمه في الماء*

C = constant (0.84 – 1.09) *يعتمد على نسيج التربة من التربة الناعمة حتى الخشنة .*

- *تزداد السرعة في المبزل في حالة وجود الغطاء النباتي لأنه يعمل على تثبيت التربة للمبزل مما يقلل تأثير السرعة على تعرية تربة المبزل وانجرافها .*

b. Discharge *التصريف*

ان التصميم الناجح للمبزل المفتوح هو الذي يستوعب التصريف العالي وغير الاعتيادي دون ان يؤثر على نمو النباتات والذي يجعل من العمق الحر free board *يستوعب ما لا يقل عن 50% من السعة التصميمية للمبزل.*

السعة التصميمية للمبزل تعتمد على

- *كمية الامطار وبشكل خاص التي تبزل سطحيا .*
- *متطلبات الغسيل والمعتمدة على ملوحة التربة وملوحة ماء البزل.*
- *الايصالية المائية ومعدل الفيض.*

c. Side slope *انحدار الجوانب*

يجب ان يكون انحدار الجوانب ثابتا ومستقرا مع مرور الزمن عند تنفيذ مبازل مفتوحة او قنوات ري. والتي تعتمد على نسيج التربة

1:1

V:H

d. Open drains slope *انحدار المبازل المفتوحة*

$$V \propto \sqrt{S}$$

يتحدد انحدار المبازل بنحو عام بانحدار الأرض

0.03 – 0.15% at collector and main drain

$Iraqi \rightarrow 0.025 - 0.08\%$

e. Cross-section *المقطع العرضي للمبزل*

شبه منحرف

تختلف ابعاد المبازل المفتوحة باختلاف الغاية في تنفيذها . فالمبازل المفتوحة المستخدمة للبزل السطحي (أي للتخلص من الماء الزائد على سطح التربة) تكون غير عميقة مقارنة بالمبازل المستخدمة على مستوى ماء

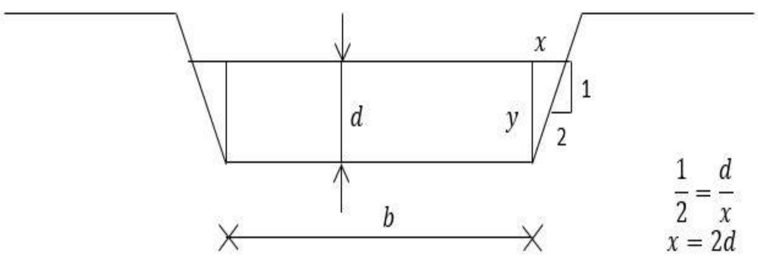
محدد للماء الأرضي. وكذلك المبازل الرئيسية المفتوحة تختلف عن المبازل المجمعة في المبازل الرئيسية تكون أكثر عمقا وعرضا من المبازل المجمعة. وان عرض المبزل المجمع يتراوح ما بين 0.8-1 m وقد يصل الى أكثر في المبازل الرئيسية.

f. Drain spacing *المسافة بين المبازل*

تعتمد أساسا على K ايصالية التربة ونسجتها تتراوح ما بين 30-40 m وقد تصل الى أكثر من 100 m للترب الرملية

Exercise

Design an open drain carrying a discharge of 3m^3/s, the permissible velocity is 1 m/s side slope 1:2 Use C = 55 (economical section)



$Q = V.A$

$\therefore A = \frac{3}{1} = 3m^2$

$p = W = b + 2d\sqrt{z^2 + 1}$

$p = W = b + 2d\sqrt{5}$

$R = \frac{A}{W} = \frac{3}{b + 2d\sqrt{5}} = \frac{3}{b + 4.472\, d}$

$\therefore R = \frac{d}{2} \Longrightarrow \frac{d}{2} = \frac{3}{b + 4.472\, d}$

$6 = bd + 4.472\, d^2 \ldots\ldots\ldots (1)$ $\quad\quad \Longrightarrow A = \frac{2b + 2(2d)}{2} \times d$

$A = 3 = bd + 2d^2$

$bd = 3 - 2d^2 \ldots\ldots\ldots (2)$ بالتعويض في (1)

$6 = 3 - 2d^2 + 4.472\, d^2$

$3 = 2.472\, d^2 \Longrightarrow \ d = 1.101\, m$

$b(1.1) = 3 - 2(1.1)^2$ بالتعويض في (2)

$\therefore b = 0.527\, m$

Lecture - 9

$$R = \frac{d}{2} = 0.55\ m$$

$$V = C\sqrt{RS}$$

$$1 = 55\sqrt{0.55}\ S^{1/2}$$

$$\therefore S = 6 \times 10^{-4}\ m/m$$

Lecture - 9

Drainage layout **أنماط توزيع شبكة البزل**

ان اشكال شبكة البزل تعتمد بالدرجة الرئيسية على طبوغرافية الأرض المراد بزلها. ان الشبكة تكون فعالة في حالة وضعها او مرورها في المناطق الواطئة والمناطق التي توجد فيها المياه الزائدة.

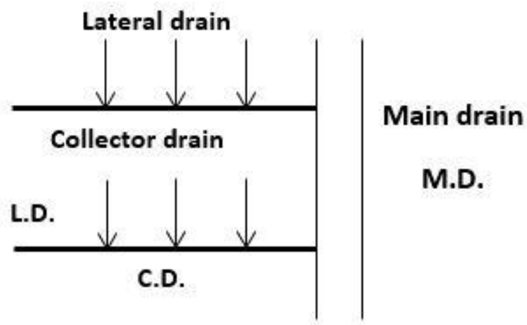
There are two types of drainage layout:

1. Parallel grid system.

- According to the topographic factor.

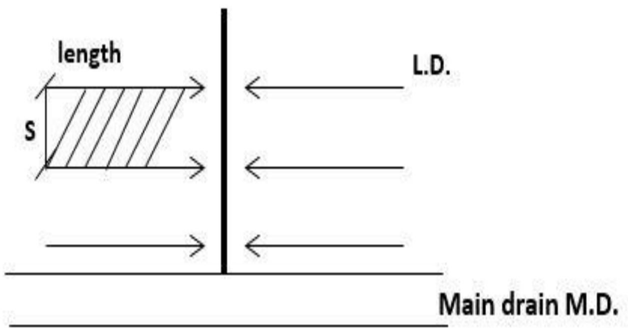
    a. Single slide collector

    تجمع المياه من المبازل الفرعية باتجاه واحد (يخدم جهة واحدة)



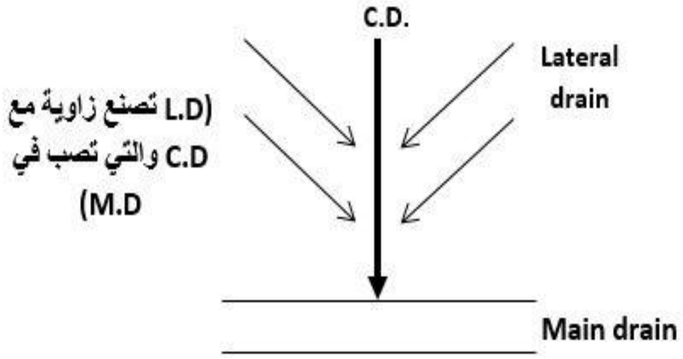
    b. Dc

    تجمع المياه من جهتين



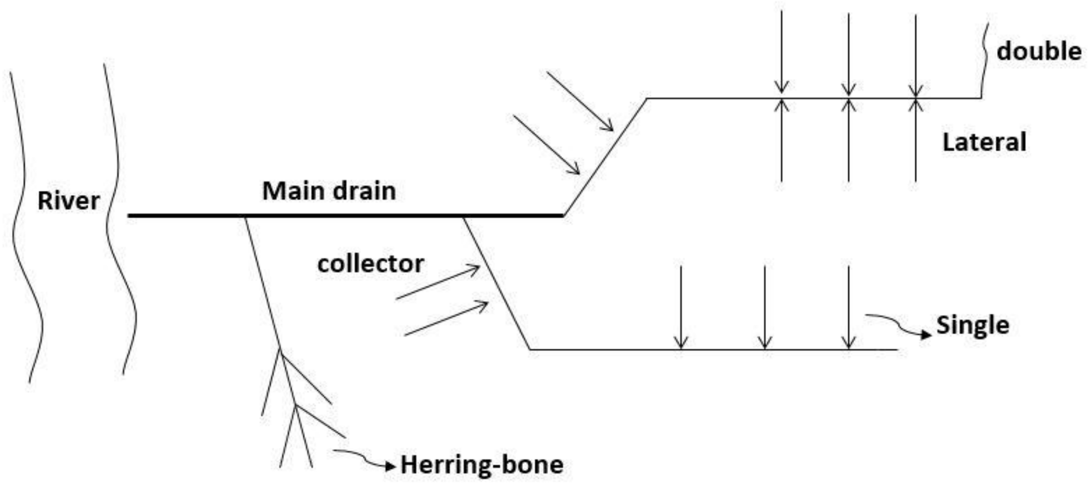
    c. Herring bone system

    التوزيع على شكل هيكل السمكة والذي يكون فيه المبزل المجمع موازيا للانحدار والمبازل الفرعية مقاطعة له.



2. Random pipe drainage system *النظام العشوائي*

- According to the field area. *مساحة الحقل*
- Lateral drain designed with angle with the collector. *عندما تكون الأرض غير منتظمة*



<u>Advantage and disadvantage of open and closed drains system.</u>

Open drains system ***مبازل مفتوحة***

1- Advantages

a) Receive excess surface and ground water.

b) Need small hydraulic gradient about (0.01%). *انحدار هيدروليكي قليل*

c) Easy in monitoring and maintenance. *سهولة الصيانة والمراقبة*

2- Disadvantages.

a) Caused losses in area.

b) Growing plants and grasses in the canals causes ponds of water. *نمو الحشائش و الأعشاب تؤدي الى تجمع الماء*

c) Difficulties in machine and human movement. *صعوبة استعمال المكائن وحركة الأشخاص لان الأرض مقطعة الى عدة مساحات*

Closed drain system مبازل مغلقة

1- Advantages

a) Receive excess ground water only. تستقبل المياه الزائدة من الماء الجوفي فقط

b) Need more hydraulic gradient about (0.1%). انحدار هذا النظام أكبر

c) Monitoring need dug hole in soil. المراقبة تحتاج حفر في التربة

2- Disadvantages

a) No losses in area. الانابيب تحت الأرض

b) No growing of plants and grasses.

c) No difficulties in movement.

Lecture - 10

Drainage wells **ابار البزل**

ان مشكلة ارتفاع مناسيب الماء الجوفي يمكن ان تحدث في المناطق التي تحتوي على طبقة حاملة للمياه تحت منطقة الجذور ذات ايصالية مائلة عالية وطبوغرافية مستوية ولا تحتوي على منفذ طبيعي للمياه الزائدة. تحت هذه الظروف لا يمكن الاعتماد على إقامة شبكية بزل افقية للتخلص من المياه الزائدة، لكن يمكن لكن يمكن التخلص من هذه المياه بالبزل العمودي (الضخ عن طريق الابار). ان البزل العمودي يكون فعالا في المناطق التي تكوف فيها الطبقة الحاملة للمياه ذات قدرة عالية جداً اعلى نقل الماء(*Transmissivity*) ويفضل هنا استخدام قابلية او قدرة النقل T بدل عن الايصالية المائية (K) لان الأول (T) يأخذ بنظر الاعتبار سمك الطبقة الحاملة للمياه (*D*) حيث ان:

$T = KD$

$T = transmissivity$ معامل الامرار

$$= \frac{m}{d} m = \frac{m^2}{d} \frac{L^2}{T}$$

وتستخدم الابار لغاية البزل والسيطرة على منسوب الماء الجوفي وان البزل العمودي يكون اقتصاديا من البزل الافقي في الحالات التي تكون فيها المياه الجوفية ذات نوعية جيدة وذلك لإمكان استخدامها في عمليات الري. لذلك فان مياه البزل في هذه الحالة تكون مياه ذات قيمة اقتصادية. ومن الناحية الأخرى ان صيانة هذه الابار تكون اقل كلفة من المبازل المفتوحة و المغطاة في حالة البزل الافقي.

Objectives

- The use of drainage wells for the purpose of drainage lands.
- The use of drainage wells for controlling the water table.

Advantages of drainage wells

1. Drainage wells more economical for unflate area.
2. The amount of discharge that pumped can be increased and lowering of the ground water table increased too because of the continues flow due to the pumping.
3. If the quality of ground water is good, the discharge can be used for industrial and agriculture purpose.

Lecture - 10

Disadvantages of drainage wells

1. More complex in design criteria.
2. More cost due to the use of fuel and electricity.
3. Need maintenance and monitoring.
4. The drainage wells caused management and legal problems. مشاكل إدارية واقتصادية
5. Drainage wells need soil of certain properties of high permeability and low drainable porosity. مسامية منخفضة وذات نفاذية عالية

The main points of designing a well are:

1. Choice of well locations:
   (soil conditions) [deep sandy aquifer fractured rock]
2. Geological and geophysical logging, water quality sampling and test pumping can be carried out in a satisfactory way:
   [●pumping test from trail wells can be used to estimate the spacing that is necessary. ●piezometers should be located within about 10, 20, …….. , 100 ft from the a well]
3. Selection of a appropriate drilling method. [proper dimensional factors of bore hole and well structure.]
4. Selection of appropriate construction materials, including pump specification.
5. Well design should be such that pollutants from land surface or other sources cannot enter the well.
6. The pumping rate should satisfy the demand for water.
7. A well with a long life. [more than 25 years and pumping from aquifer of 80 - 300 ft beneath soil surface.]
8. Materials used in the well should be resistant to correction and with sufficient strength to prevent collapse.
9. Well design should be based on low installation and running coast while not affecting well performance.

Lecture - 10

There are two types of wells

1- When the water table is located in un confined aquifer. The wells remove water directly from root zone. It may be either shallow or deep wells. [steady – state well].
2- When water table located in artesian aquifer the water may be free flow (confined aquifer). [artesian wells].

Information required for well design:

1- Aquifer location [depth of water bearing strata, and aquifer thickness]
2- Aquifer nature [consolidated or unconsolidated material, confined or unconfined]
3- Aquifer parameters [hydraulic conductivity, transmissivity, grain size]
4- Aquifer recharge characteristics
5- Location of aquifer boundaries.
6- Nature of formation above aquifer.

Design of drainage wells

1. Steady-state wells

يطلق على الجريان بانه مستقر إذا لم يحصل أي انخفاض في منسوب الماء الجوفي على مسافة *r* من البئر وتسمى هذه المسافة بالقطر المؤثر للبئر *re*

re = radius of influence.

ومن الناحية النظرية يمكن الحصول عليها بفترة زمنية قصيرة جدا.

- الذي يساعد على حصول الجريان المستقرهو

- الطبقة الحاملة للمياه تكون بحجم كبير جدا او مصدر تمويلها يكون بشكل ثابت و على هذا الأساس فان الجريان المستقر لأغراض تحليل هذه الابار يعطي نتائج مقبولة.
- بعض الفرضيات في التصميم

أ- منسوب الماء الجوفي يمثل حدود الجريان العليا.

ب- الجريان يكون باتجاه افقي horizontal وشعاعي radial.

ح- الجريان متناظر حول البئر (cylindrical sink).

د- ليس هناك تغذية للمياه في منطقة القطر المؤثر.

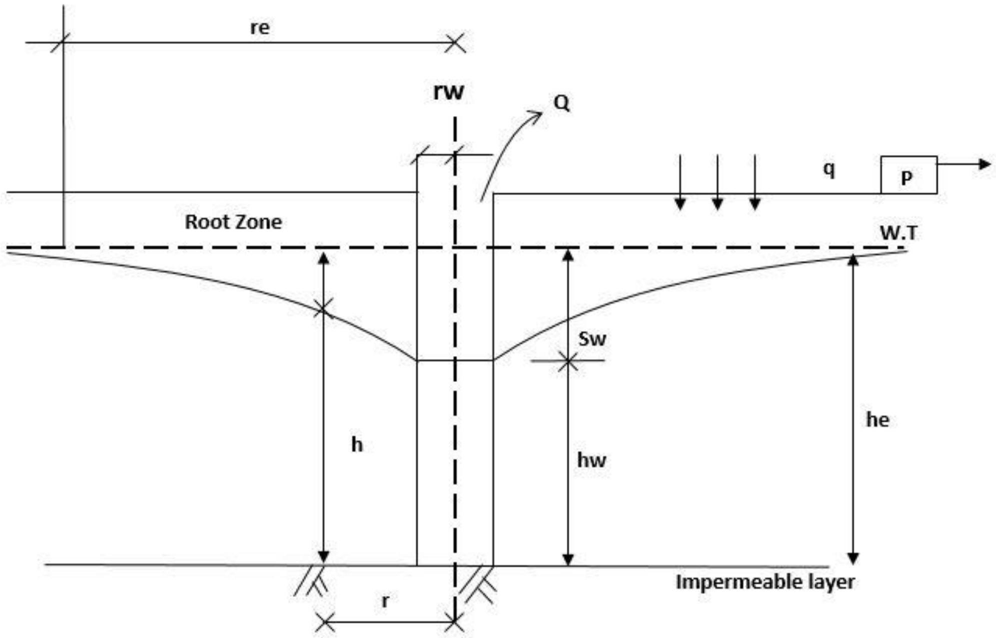
ه- الجريان يميل الى حالة الثبات.

و- البئر يخترق الطبقة الكاملة للمياه الى عمق الطبقة الصماء.

ز- حدود الطبقة الحاملة للمياه واسعة جدا.

وهذا هو الأساس لتجديد تصريف البئر الواحد والمسافة بين الابار، وذلك لغرض خفض منسوب الماء الجوفي الى العمق الذي يسمح به لنمو النبات بشكل جيد.

Lecture - 10



لحساب تصريف البئر *Q (Vol/time)* واعتمادا على دارسي:

$$Q = KiA$$

$$= 2\pi rh\, K\frac{dh}{dr} \ldots\ldots\ldots\ldots (1)$$

$Q, h, \dfrac{dh}{dr}$ $\quad unknown$ كمية غير معلومة

الكمية المبزولة والمحسوبة من المعادلة (1) هي نفسها الكمية النازلة كرشح عميق يمكن التعبير عنه بالمعادلة التالية

$$Q = \pi[(re^2 - rw^2)]q \ldots\ldots\ldots\ldots (2)$$

(1) = (2)

$$\therefore\ 2\pi r\ h\ \ k\frac{dh}{dr} = \pi(re^2 - rw^2)q$$

$$\frac{q}{k}\int_{rw}^{re}\frac{(re^2 - rw^2)}{r}\ dr = 2\int_{hw}^{he} h\ \ dh$$

ويمكن اهمال قيمة rw نصف قطر البئر لان قيمتها قليلة مقارنة مع re

$$Q = \pi re^2 q \ldots\ldots\ldots\ldots (3)$$

Lecture - 10

$$\frac{q}{k}\int_{rw}^{re} re^2\frac{dr}{r} = 2\int_{hw}^{he} h\ dh$$

$$\frac{q}{k}re^2\quad |n\,r]\begin{matrix} re \\ rw \end{matrix} = h^2]\begin{matrix} he \\ hw \end{matrix}$$

$$\frac{q}{k}\ re^2\ |n\frac{re}{rw} = he^2 - hw^2$$

$$\frac{Q}{k\pi}\ |n\frac{re}{rw} = he^2 - hw^2$$

$$\frac{Q}{k\pi} = \frac{he^2 - hw^2}{|n\frac{re}{rw}} \dots\dots\dots\dots 4$$

$$\frac{Q}{k\pi} = \frac{(he + hw)(he - hw)}{|n\frac{re}{rw}}$$

$$\frac{Q}{k\pi} = \frac{Sw(he + hw)}{|n\frac{re}{rw}} \dots\dots\dots\dots 5$$

When the draw down Δh or lowering (i.e the change in head due to pumping) is small in compare with the thickness (D) of the standard aquifer, we may write

$$h + hw = 2D$$

يمكن كتابة معادلة 4 عند مسافة من البزل r معطاه

$$\frac{Q}{k\pi} = \frac{h^2 - hw^2}{|n\frac{r}{rw}} \dots\dots\dots\dots 6$$

eq 4 $= eq$ 6.

$$\frac{he^2 - hw^2}{|n\frac{re}{rw}} = \frac{h^2 - hw^2}{|n\frac{r}{rw}} \dots\dots\dots\dots 7$$

$$\frac{Q}{k\pi} = \frac{(h - hw)(h + hw)}{|n\frac{re}{rw}}$$ المعادلة 6

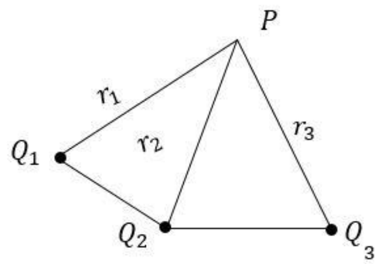
$h + hw = 2D \qquad \Rightarrow since\ \ \Delta h = S_w$

$$\frac{Q}{k\pi} = \frac{Sw \times 2D}{\ln \frac{r}{rw}}$$

$$\therefore \quad \Delta h = \frac{Q}{2\pi KD} \qquad \ln \frac{r}{rw} \ldots\ldots\ldots\ldots (8)$$

(methods of super-position) طريقة التراكب

$\because S = he - h$



ان الهبوط عند نقطة ما مثلا (P) يمكن ان تحسب من خلال جمع مستوى الهبوط نتيجة للضخ من الابار المجاورة والمؤثرة وعندما يكون الهبوط في منسوب الماء الجوفي صغيرا عند المقارنة بسمك الطبقة الحاملة للماء وتحت ظروف الجريان المستقر يمكن حل مشكلة جريان الماء الجوفي باتجاه الابار بالشكل التالي

$$(he^2 - h^2) = \sum_{i=1}^{n} \frac{Qi}{\pi k} \ \ln \left(\frac{rei}{ri}\right)$$

Where

Qi = constant discharge for the well.

ri = distance from known point (I,m).

h = hydraulic head at (P) during pumping.

Lecture - 10

$$Q_T = \sum Q$$

Q_T = مجموع تصاريف الابار

$he - h$ = lowering in water depth الانخفاض في الماء الجوفي

n = عدد الابار

و المعادلة تصبح بالشكل التالي [اذا كان تصريف الابار اقتصاديا ولنفس نصف القطر المؤثر]

$$(he^2 - hw^2) = \frac{Q_T}{\pi k} 1n \left(\frac{re}{\bar{r}}\right)$$

$\bar{r}$ = equivalent distance from (P) البعد المكافئ للنقطة

$$\bar{r} = (r_1 . r_2 . r_3 . \ldots \ldots \ldots \ldots . r_n)^{1/n}$$

re = نصف القطر المؤثر للآبار

وعندما يراد حساب المسافة بين الابار عادة تؤخذ نضر الاعتبار نصف القطر الفعال ومنسوب الهبوط المطلوب للحصول على منسوب ماء جوفي محدد غير مؤثر على نمو النباتات.

Exercise

5 wells located at random distribution fully penetrated in unconfined aquifer each well discharge 3600 m^3/d and has an influence radius of 300 m, the hydraulic conductivity of the aquifer is 10 m/d and its saturated thickness is 75 m. the distance from point (P) (which wont the lowering) to the wells are (250, 175, 90, 150 and 200). What is the drawdown at point P if it reach the steady state – P.

ماما مقدار الانخفاض في الحاصل في النقطة P.

Solution

$$he^2 - h^2 = \frac{Q_T}{\pi k} 1n \left(\frac{re}{\bar{r}}\right)$$

$$he = 75\ m$$

$Q = 3600\ m^3/d$

$k = 10\ m/d$

$re = 300\, m$

$\bar{r} = [r_1 . r_2 . r_3 . r_4 . r_5]^{1/5}$

$\bar{r} = [250 \times 175 \times 90 \times 150 \times 200]^{1/5}$

$= 163.9\, m$

$$(75)^2 - h^2 = \frac{5 \times 3600}{\pi \times 10}\ 1n\left(\frac{300}{163.9}\right)$$

$h = 72.7\, m$

$\therefore S = lowring\ at\ point\ \ p = 75 - 72.7$

$= 2.3\, m$

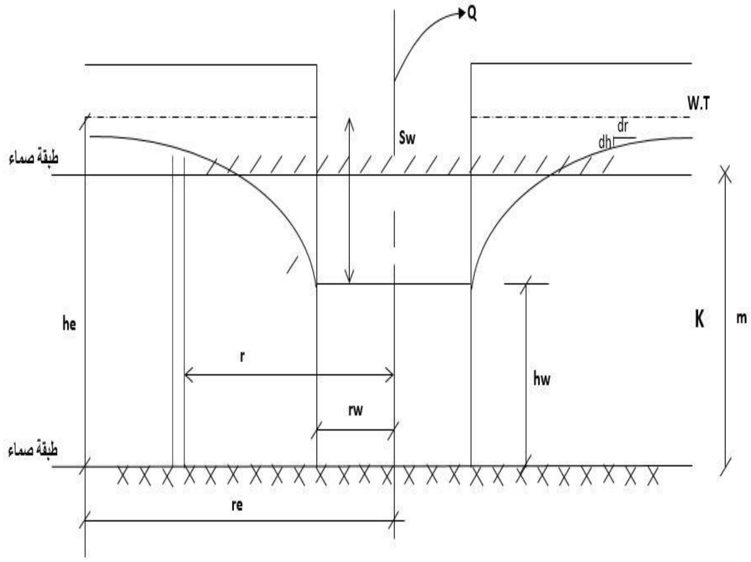
أي ان عمق الماء فوق الطبقة الصماء عند النقطة P هو 72.7 ومقدار الهبوط هو $2.3\ m$.

<u>Artesian wells الابار الارتوازية</u>

confined aquifer *المكمن المائي المحصور*

ويمكن كتابة قانون دارسي بالصيغة التالية

Lecture - 10



(تعمل ثقوب رأسية بالطبقة العليا لتوصيلها بالطبقة السفلية العالية المسامية وملئ هه الثقوب بمواد عالية المسامية

مثل الحصى لتوصيل الماء الجوفي في الطبقة العليا بالمياه في الخزان الجوفي)

$Q = \text{kiA}$

$Q = k.\frac{dh}{dr} \times 2\pi r \times m$

$\frac{Q}{2\pi\ km} \int_{rw}^{re} \frac{dr}{r} = \int_{hw}^{he} dh$

$\frac{Q}{2\pi\ km} [1n\ r]_{rw}^{re} = [dh]_{hw}^{he}$

$\frac{Q}{2\pi\ km}\ \ 1n\frac{re}{rw} = he - hw$

$\frac{Q}{2\pi\ km} = \frac{he - hw}{1n\frac{re}{rw}} \ldots\ldots\ldots\ldots (2)$ ويمكن كتابتها عن اي قيمة معطاة لـــ r

$\longrightarrow \frac{he - hw}{1n\frac{re}{rw}} = \frac{h - hw}{1n\frac{r}{rw}}$

$$h = \frac{he - hw}{1n\frac{re}{rw}} \times 1n\ \frac{r}{rw} + hw$$

∴ الضغط الهيدروليكي متغير خطياً مع لوغاريتم المسافة الافقية

وبما ان التصريف ثابت eq ① = eq ②

$$\frac{he - hw}{1n\frac{re}{rw}} = \frac{h - hw}{1n\frac{r}{rw}} \ldots\ldots\ldots\ldots ③$$

$T = km\ \left(\frac{L^2}{T}\right)$ معامل الامرار

Where

m = (m) سمك الطبقة الحاملة للمياه والتي نفترضها ذات سمك منتظم

$$h = (he - hw)\frac{1n(\frac{r}{rw})}{1r_1(\frac{re}{rw})} + hw$$

الخط الهيدروليكي يتغير خطياً مع لوغاريتم المسافة الافقية

<u>EX</u>

For confined aquifer, if the transmissivity factor is 1000 m^2/d and the average discharge allowed from the well is 1200 m^3/d with influence radius of 300 m and the well radius is 0.3 m. find the max drawdown lowering of the well if the height of the water inside the well is 30 m. And then fined the lowering at a distance of 30 m from the center of the well?

Solution

$$\frac{Q}{2\pi\ km} = \frac{he - hw}{1n\frac{re}{rw}} \ldots\ldots\ldots\ldots ①$$

$$he - hw = Sw$$

$$\frac{1200}{2\pi(1000)} = \frac{Sw}{Ln\frac{300}{0.3}}$$

$$Sw = 1.31\ m$$

$$Sw = he - hw$$

$$he = 1.31 + 30 = 31.31\ m$$

استخدام المعادلة التالية على أي مسافة r

$$r = 30\ m$$

$$\frac{Q}{2\pi\ km} = \frac{h - hw}{Ln\frac{r}{rw}} \ldots\ldots\ldots\ldots ②$$

$$\frac{1200}{2\pi(1000)} = \frac{h - 30}{Ln\frac{30}{0.3}}$$

$$he = 30.7\ m$$

$$Sww\ 30\ m = he - hw\ 30\ m$$

$$= 31.31 - 30.7 = 0.61\ m$$

Lecture - 11

تباعد المبازل الحقلية المكشوفة:

عندما يتم شق مجموعه من المبازل الحقلية المتوازية فان السطح العلوي للمياه الجوفية بين كل مبزلين متجاورين يتخذ شكل الناقص الناقص.

P = drain spacing (m)

K = hydraulic conductivity of the soil (m/d)

H = height above the previous layers of the ground water table Mid way Between Two drains (m)

h = height above the impervious layer of the water level in the drain = thickness of aquifers below drain level (m)

r = recharge rate per unit surface area (m/d)

Q = discharge peer unit length (m^3 /d/m)

Q = p.r.1

الانحدار الهيدروليكي لسطح الماء ذو قيمة ثابتة مع عمق المقطع

التصريف ثابت مع الزمن [steady state] , dy/dx

- المياه تتحرك الى المبازل في خطوط افقية

ندرس تصريف المياه لمتر طول من المبزل عند اي مقطع عمودي على اتجاه حركة المياه المقطع (I-I)

مساحة المقطع المائي A = y . 1

ومن معادلة دارسي:

V = K . I

= K . dy/dx

وبذلك يكون التصريف للمياه في جهة واحدة للمبزل خلال المقطع (I-I)

$$Q_{I.I} = 2AV = 2yk \quad \frac{dy}{dx} \dots\dots\dots\dots ①$$

ان تصريف المياه في منتصف المسافة بين المبزلين = صفر ويزداد التصريف في اتجاه المبزل حيث تبلغ اكبر قيمة له عندما تؤول قيمة (x) الى الصفر.اي عند جانب المبزل

ويمكن ربط التصريف الكلي عند جانب المبزل Q بتصريف مياه البزل عند المقطع (I.I)

$$\frac{Q_{I.I}}{Q} = \frac{P/2 - x}{P/2}$$

Or

$$Q_{I.I} = Q.\frac{2}{P}\left(\frac{P}{2} - x\right) \dots\dots\dots\dots ②$$

وبحل المعادلتين 2 و 1 نحصل على الاتي

$$2\,ky\frac{dy}{dx} = Q\,.\frac{2}{p}\left(\frac{p}{2} - x\right)$$

Or

$$ydy = \frac{Q}{pk}\left(\frac{p}{2} - x\right)dx \;\ldots\ldots\ldots\ldots 3$$

وبأجراء عملية التكامل للمعادلة رقم 3 نحصل على الآتي:

$$\frac{Q}{p.k}\,.\left(\frac{px - x^2}{2}\right) = \frac{y^2}{2} + c$$

ولتجد قيمة c فانه عندما x = 0 فان (y = h)

$$\therefore\, c = \frac{-h^2}{2}$$

وعندما x = p/2 فأن (y = H) ونحصل على المعادلة الآتية:

$$p = \frac{4k(H^2 - h^2)}{Q}$$

Or

$$p^2 = \frac{4k(H^2 - h^2)}{r} \qquad\qquad Donnan\ Eq.$$

والتي استنتجها ايضاً Hooghoudt وهي معادلة قطع ناقص وتعطي نتائج مقبولة كلما اقتربت الطبقة الصماء عن قاع المبزل [اي كلما اقتربنا من الغرض الذي نص عل افقية حركة المياه ناحية المبزل].

$$p^2 = \frac{4k(H - h)(H + h)}{r}$$

$$S = H - h$$

$S =$ The water table height above drain level at mid poin d,i.e. the hydraulic head for subsurface flow into drains (m).

$$p^2 = \frac{4k\,S(H + h)}{r}$$

Or

$$p^2 = \frac{8k\,S(h + \frac{S}{2})}{r}$$

Where:

h + S/2 = the average thickness of the soil layer throw which the flow take place (aquifer). [h⁻]

$$p^2 = \frac{8k\ S\ h^-}{r}$$

($S.h^-$) = transmissivity of the aquifer (m^2/day).

وعندما ينعدم وجود المياه داخل المبزل (h = 0)

$$p^2 = \frac{4k\ S^2}{r}$$

This equation represents the horizontal flow above drain level.

Or

$$Q_a = \frac{4k\ S^2}{p}$$

ومن ناحية اخرى فانه يمكن اهمال قيمة S/2 اذا كانت بالنسبة لقيمة (h) صغيرة

If h is large compared with S, the equation become

$$Q_b = \frac{8k\ Sh}{p}$$

وفي حلة وجود تباين في قيمة المستوصلية الهيدروليكية للطبقتين المفصولتين عند منسوب المياه بالمبزل

The above consideration permit the conception of two-layered soil with interface at drain level

$$Q = \frac{4k_a\ S^2 + 8k_b\ Sh}{p}$$

Where:

K_a = hydraulic conductivity of the layer above drain level (m/d)

K_b = hydraulic conductivity of the layer below drain level (m/d)

Lecture - 12

Salt problems in soil and water

Salinization and drainage

Irrigated soil receive considerable quantities of dissolved salts, supplied partly by irrigation water itself, and partly by inflowing ground water, irrigation water, even if it is of excellent quality, it’s a major source of soluble salts. If these salts are not removed from root zone, salinization is inevitable.

ان السيطرة على ملوحة التربة هي واحدة من الاهداف الرئيسية لمشاريع البزل في المناطق الجافة وشبه الجافة فضلا عن السيطرة على المياه الزائدة في التربة. وعلى الرغم من ان هذه الاهداف ليست مترابطة بصورة كلية، الا ان ظهور احدهما قد يؤثر على الاخر بشكل او اخر. اذ ان ارتفاع منسوب الماء الجوفي في المناطق الجافة اوشبه الجافة ليصاحبه في كثير من الاحيان ظهور مشكلة تجمع الاملاح على سطح التربة. ان الترب الحاوية على نسبة معينة من الاملاح تؤثر سلبيا على انتاج هذه الترب [يمكن ان تكون هذه الترب ملحية او قلوية].

- Source of soluble salts

  ان تملح التربة يمكن ان يعرف بصورة عامة على انه زيادة تركز الاملاح الذائبة في التربة الى درجة انها تؤثر في نمو النباتات.

  ان المصدر الرئيسي لزيادة تركيز الاملاح في التربة المستغلة زراعياً هو مياه الري، الماء الجوفي، عملية الذوبان من الصخور الغنية بالاملاح وعمليات التسميد.

  Source of soluble salts :

  a. Heavy annual of rainfall of humid regions cause water percolate through the soil and carry to the streams, rivers and oceans large amount of soluble minerals.
  b. Lack of percolation through arid region and the excessive evaporation of water gives rise of soluble salts in the soil.
  c. Insufficient application of water during irrigation. كمية مياه غير كافية اثناء فترة السقي.
  d. Minerals soils are drived from weathering of rocks. تأثير التجويه على الصخور الملحية (الكلسية).
  e. The use of irrigation water containing salts. استخدام مياه الري المالحة
  f. The use of drainage water. استخدام مياه البزل التي تكون ملوحتها عالية

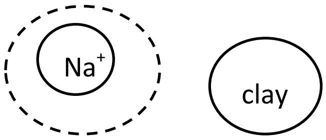
Saline soils **الترب الملحية**

The soils having excess amount of soluble salts that make the soil solution concentrated and reduce soil productivity.

Alkaline soils التربة القلوية (القاعدية)

The soils which have excess amount of soluble sodium.

ان زيادة عنصر الصوديوم في التربة يعمل على تحطيم بناء التربة من خلال شحنته الموجبة الواحدة والغلاف المائي الكبير فيسبب تفرقة الدقائق الاولية للتربة وبذلك خلق ظروف غير جيدة لحركة المياه وتوزيعها مما يؤثرعلى توفير الظروف الملائمة لنمو النبات.



Lecture - 12

ولغرض تصنيف التربة يجب الحصول على مستخلص التربة او مستخلص العجينة المشبعة (soil extract) واجراء التجارب المختبرية لتحديد ما ياتي:

a. Electrical conductivity (E_c) الايصالية الكهربائية
in (mm hos /cm) @ $25c^o$ by using conductivity meter.
b. pH
c. Ionic composition in $(mg/\ell)ppm$
Positive ions (C_a^{++},N_a^{+},M_g^{++},K^{+})
(mille equivalent / liter) ℓ/مليمكافئ meq/ ℓ ويمكن تحويلها الى
d. Negative ions (CL^-,$CO_3^=$,HCO_3^-,NO_3^-,$So_4^=$)

$$\text{Meg } /\ell = \frac{ppm \leftsquigarrow \text{mq } /\ell}{\frac{sum\ of\ atomic\ wt}{ionic\ charge}\ \frac{\text{الوزن الذري}}{\text{الشحنة الايونية}}}$$

$$Ec = \frac{C}{12}$$

Where:
C = salt concentration of water in (meq $/\ell$) or salt content
Ec = electrical conductivity in mm hos / cm @ $25C^o$.

Evaluation in water

لتقييم نوعية المياه هناك بعض الفحوصات التي يجب اجراءها:

1- Drinking water
   a. Physical properties: color, pH, turbidity…
   b. Chemical properties: acidity, hardness, alkalinity, total dissolved soils...
   c. Bacteriological-coliform, bacteria…

2- Irrigation water
   a. Salinity الملوحة $\begin{matrix} TDS \\ Ec \end{matrix}$
   b. Sodicity الصودية $\begin{matrix} SAR \\ ESP \\ SSP \end{matrix}$
   c. Toxicity السمية $\begin{matrix} NO_3^- \\ Cl^- \\ B \\ check \end{matrix}$

Sodicity الصودية

انواع التقيمات لمياه الري بالنسبة للصوديوم

1. SAR : the sodium adsorption ratio. قابلية التصاق ايونات الصوديوم

Lecture - 12

$$SAR = \frac{Na}{\sqrt{\frac{Ca + Mg}{2}}}$$

كلما كان ايون الصوديوم ذو تركيز عالي يعني قابلية التصاقه بالتربة اكبر و العكس صحيح

Where

Na, Mg, Ca are concentration in (meq $/\ell$)

S_1	S_2	S_3	S_4
$0 - 10$	$10 - 18$	$18 - 26$	$26 <$

2. ESP = Exchangeable sodium percentage

$$ESP = \frac{100 \times (-0.0126 + 0.01475\ SAR)}{1 + (-0.0126 + 0.01475\ SAR)}$$

نسبة الاستبدال للصوديوم (نسبة الصوديوم المتبادل)

3. SSP = soluble sodium percentage نسبة ذوبان الصوديوم

$$SSP = \frac{Na}{Na + Ca + Mg + K} \times 100$$

4. RSC = residual sodium carbonate كاربونات الصوديوم المتبقية

$$RSC = [(CO_3^{=} + HCO_3^{-}) - (Ca^{++} + Mg^{++})]$$

المعايير المستخدمة لتصنيف الترب المتاثرة بالاملاح

1- الترب الطبيعية

$SAR < 13$
$ESP < 15$
$ECe < 4$ mmhos/cm

2- الترب الملحية saline soil

$SAR < 13$
$ESP < 15$
$ECe > 4$
$PH < 8.5$ mm hos/cm

3- الترب الملحية الصودية saline alkali soils

$SAR > 13$
$ESP > 15$
PH $(8.5 - 10)$
$ECe < 4$ mm hos/cm

Irrigation water standard مقاييس مياه الري

a) According o its electrical conductivity

Irrigation water class	Ec (microm hos/cm)
Class 1 (C_1): Excellent	0 – 250
Class 1 (C_2): Good	250 – 750
Class 1 (C_3): Permissible	750 – 2250
Class 1 (C_4): Doubtful	> 2250

b) According to sodicity (SAR)
Low S_1 0 – 10
Medium S_2 10 – 18
High S_3 18 – 26
Very high S_4 > 26 (26-100)

c) For RSC in (meq/ℓ)
RSC (<1 . 25) low effects.
RSC (1.25 – 2.5) medium effects.
RSC (>2.5) high effects.

ان الاتحاد بين الصوديوم والكاربونات ضعيفاً لان الاواصر تكون ضعيفة.

مشكلة الملوحة معبراً عنها بالتوصيل الكهربائي Ec (c) michos/cm

Sodium Hazard (sodicity)

The structure of the soil is depend on the type of exchangeable cations. In general, bivalent cations such as Ca^{++} and Mg^{++} promote a good soil structure, where as monovalent cation such ass K^{+}, and especially Na^{+}, have a deteriorating effect causing, amongst other things, a poor permeability. In normal soils Na and K occupy only about (5%) of the exchange capacity, the remainder occupied mainly by Ca and Mg ions. If the percentage of adsorbed Na rise above (10), sodium problem may be expected. The adverse effect of (Na) is the more pronounced as more clay of the swelling type is present and as the total salt concentration in the soil moisture is less.

ECe = the electrical conductivity of soil sample is usually determined in their saturation extract

$$Ece = \frac{W_{fc}}{We}\, E_{c_{fc}}$$

wfc and We are the moisture content percent at the field and in the saturated past respectively.

For moderately saline soils

Ece = 6 – 8