TRAINING COURSE

ON

SOFTWARE FOR GROUNDWATER DATA MANAGEMENT

UNDER

WORLD BANK FUNDED HYDROLOGY PROJECT

LECTURE NOTES
ON

GROUNDWATER BALANCE
(UNIT-2)

BY

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SEEPAGE FROM A CANAL

1. STEADY SEEPAGE FROM A CANAL

1.1 Introduction

Canals continue to be major conveyance system for delivering water for irrigation in the alluvial plains of India. But loss of water due to seepage from unlined irrigation canals constitutes a substantial percentage of the usable water. By the time the water reaches the field, it has been estimated that the seepage losses are of the order of 45 percent of the water supplied at the head of the canal (CBIP, Technical Report No.14, 1975). According to the India Standard (IS:9452,1980) the loss of water by seepage from unlined canals in India generally varies from 0.3 to 7.0 m^3 per second per 10^6 m^2. The transit losses are more accentuated in alluvial canals. It has been estimated (CBIP,Report No.14, 1975) that if the seepage loss is prevented, about six million hectares of additional area could be irrigated easily. The seepage losses from a canal may lead to water logging and soil salanisation. Canals in alluvium are lined for water conservation and prevention of water logging. Depending on the lining material seepage from a lined canal occurs at a reduced rate.

The process of seepage from a canal starts as soon as water is filled in it. As the time elapses, in the first stage, the soil layers around the canal get saturated. The saturated front, in the next stage, moves slowly downwards and after a certain period of time, it reaches the water table below the bed of the canal. During this downward propagation of seepage from the canal into the flow domain beneath the canal, the seepage water is used for the saturation of the wetted zone, where the pores were previously filled with air. After reaching the water table only part of the infiltrating water is stored within the extending saturated zone where as the remaining part recharges the ground water. It may be noted that seepage rate from the canal is not the recharge rate at the water table at all time. With wetting front position some where between the canal bed and initial water table position at very large depth and for initially dry soil, the seepage rate varies in time, but the recharge rate is constant and zero. If the water content behind the wetting front is close to saturation, recharge rate rises abruptly from zero to the prevailing seepage rate at the time the saturation front encounters the table.

The seepage loss from an unlined canal depends on the depth of water in the canal, depth to water table in the vicinity of the canal measured from the water surface in the canal, width of the canal at the water surface, side slope of the canal, distance of the governing drainage, and coefficient of permeability of the porous medium. In addition, flow velocity, soil and water temperature, atmospheric pressure and stratification of the underlying soil also affect the seepage rate. In case the canal is lined besides the above mentioned factors seepage loss depends on the permeability and thickness of the lining material. Initially the seepage losses are high due to steep hydraulic gradient, but as the sub-soil becomes saturated, the gradient flattens and ultimately stabilise if the channel runs continuously. The discharge given by the theoretical formulae mostly corresponds to steady state conditions which are difficult to attain in practice due to intermittent running of the canal. Thus, except for cases of canals where steady condition can be obtained early as for instance in soils of high permeability with main drainage located close to the canal, the discharge as computed from theoretical formulae may be poor estimate of the likely losses and only serve to indicate the order of seepage losses for deciding the necessity or otherwise of the lining.

Of the various factors which influence seepage loss from a canal, the very important are the boundary conditions of the flow domain and permeability of the medium. Only after a correct assessment of the coefficient of permeability and boundary conditions, the seepage loss can be estimated either by numerical method or by analytical technique. In order to avoid the difficult task of estimating the insitu coefficient of permeability and the prevailing subsurface boundary conditions, experimental techniques like ponding method and inflow and out flow method have been used for estimation of seepage loss. In recent years, tracer technique has been used to estimate seepage from canal because of its comparatively easy operation in respect to other experimental method.

1.2 Analytical Method

Steady state seepage from a canal in a homogeneous isotropic porous medium, when the water table is at large depth, has been analysed by a number of investigators. For solving the seepage problem rigorously hodograph method and conformal mapping technique are applied. Zhukovsky function has been used to map the curve linear phreatic lines to straight line boundaries and conformal mapping technique has been used for finding an approximate seepage loss from a canal.

1.3 Experimental Method

Currently accepted methods for direct measurement of seepage losses from existing canals are the ponding, inflow-outflow and seepage meter. In addition to these, there are special methods such as tracer technique, and piezometric survey.

The only direct method of doubtless reliability for computation of seepage is to isolate the suspected section of the canal and to determine the rate of disappearance of the impounded water. However this method is not easily practicable and often may not be desirable due to continuous irrigation requirements. The Indian standard code of practice(IS: 9452 Part-I, 1980) for measurement of seepage losses from canals by ponding method has been finalised in January 1980. The ponding and inflow-outflow methods are applicable regardless of canal or soil conditions.

According to the Indian Standard for determining seepage losses by ponding method a reach of the channel is isolated by constructing temporary bunds or by bulkheads on existing control structure. The method has been applied to canal with discharge capacity of about 150 m^3 per second. Length of the pond should be large enough so that the area of the end bunds is a small percentage of the total wetted area. The suggested length of the pond is about 100 times the bed width of the canal. Gravity flow or pumping may be used to fill the test pond depending on the conditions that prevail at the site and the size of the canal.

The evaporation losses from the pond surface may be significant as compared to seepage losses from lined canal. In order to apply corrections due to evaporation, an evaporation pan is used for measuring the evaporation rate during the ponding test. The rate of fall of the water surface within a few hours after the initial readings of the gauges shall provide an indication of loss rate. Seepage losses may be computed from the observation recorded after the steady state condition has been achieved. In the computation of seepage loss the effect of variable head is not considered.

According to the Indian Standard (IS:9452,Part II,1980), for determining seepage losses by inflow-outflow method, the quantities of water that flow into and out of a canal reach are measured and the difference of the water quantities flowing into and out of the canal reach is attributed to seepage. Evaporation from the from the canal water surface and precipitation are taken into consideration in the computation. The selection of the site is governed by the availability of the measuring device at the site of inflow and outflow measurement. The standard devices which are preferred are: standing wave flumes, V notches, rectangular notchest. In the absence of such facilities the canal discharges at the two sections can be measured by current meter (IS:1192-1959). The length of a reach should be such that the loss from the reach is of higher order compared to the accuracy of the measuring devices.

1.4 Computation of Seepage when Water Table is at Large Depth

The seepage loss from a ridge canal when the water table is at large depth can be computed either using Vedernikov formula or using Kozeny formula. According to Vedernikov (vide Harr, 1962) the seepage per unit length of the canal is given by:

$$q=k(B+AH)$$

in which B = the width of the canal at the water surface; H = the maximum depth of water in the canal; and A = a parameter which has been derived rigorously for a trapezoidal straight canal in a homogeneous isotropic porous medium of infinite depth with the assumption that the water table lies at large depth below the canal bed. The water table can be considered to be at large depth if it lies below the canal bed at a depth more than 1.5 B. The value of the parameter A for a given canal cross section can be obtained using the graph given in Fig.1. According to Kozney formula the value of the parameter A is equal to two.

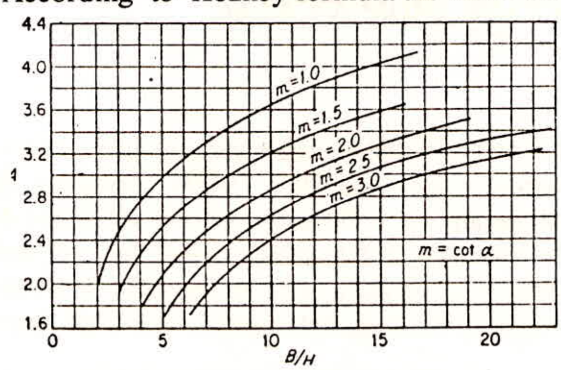


Fig.1 Vedernikov's Parameter A

The crux of computation of seepage depends on correct assessment of the hydraulic conductivity K. Knowing the percentage of sand silt and clay the hydraulic conductivity of undisturbed soil can be approximately known using Johnson's chart (1963).

Let us compute the seepage loss from a canal for various type of soils using Vederinkov's approach and the hydraulic conductivities values given by Johnson. Let the canal have the following dimension:

Bed width = 51.45m;
Water depth, H = 3.35m;
Side slope = 1.5:1 ; m = cot α = 1.5;
Wetted surface area of unit length of canal = 57.5m^2 ;
The width of the canal at the water surface, B = 61.5m;
For m=1.5, and B/H = 18.3, the parameter A=3.7.

The seepage losses for different type of soil are presented below.

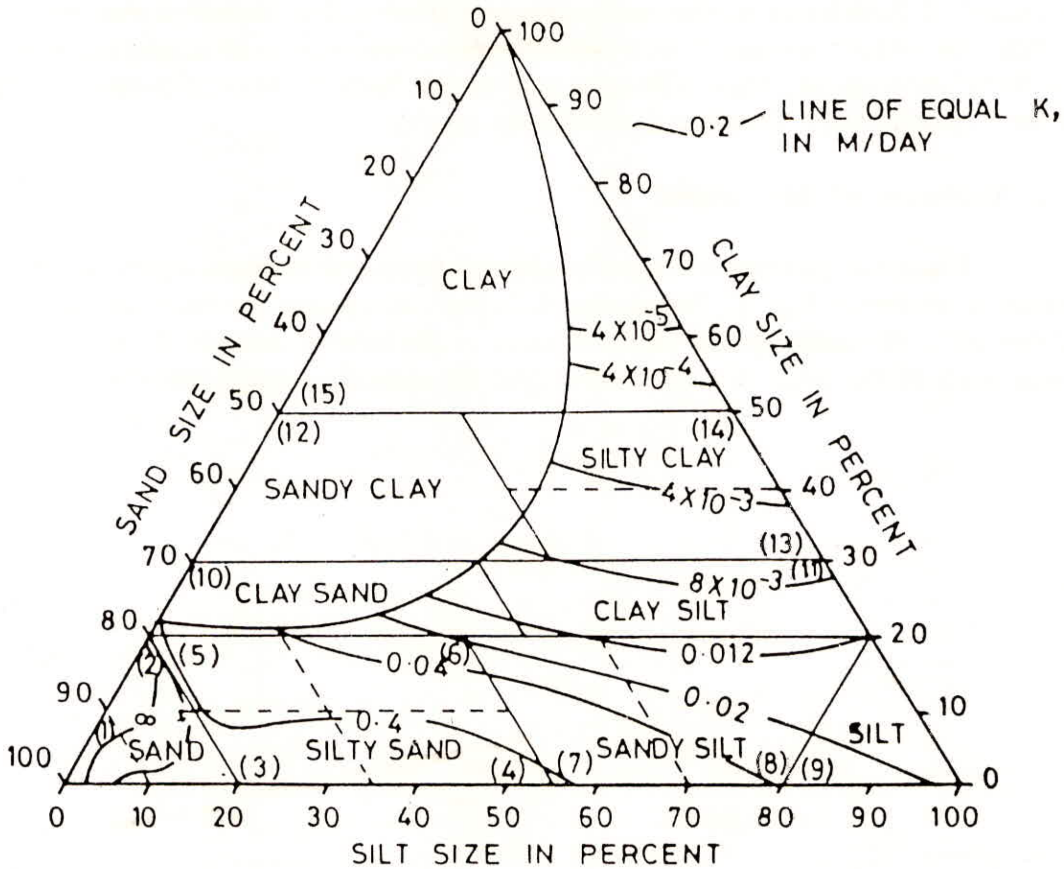
Soil type	Range of hydraulic conductivity(m/day)	Range of seepage loss(cumec/10^6 m^2) Vederinkov's approach	Kozney approach
Sand	8.0 to 4.0	119 - 59.5	109.8 - 54.9
Silty	4 to 0.04	59.5 - 0.59	54.9 - 0.549
Sandy	0.4 to 0.012	5.95 - 0.1785	5.49 - 0.1647
Clay silt	0.012 to 0.008	0.1785 - 0.119	0.1647 - 0.1098
Silty clay	0.008 to 0.004	0.119 - 0.059	0.1098 - 0.0549

It will be appropriate to compute seepage losses using the Vederinkov parameter A and the hydraulic conductivity given by Johnson's chart.

2. UNSTEADY SEEPAGE FROM A CANAL

2.1 Introduction

Interaction of a partially penetrating river and an aquifer for varying river stage has been analysed by Morel-Seytoux and Daly (1977). Assuming that the exchange of flow between the river and the aquifer is linearly proportional to the difference in the potentials at the periphery of the river and in the aquifer below the river bed and accordingly making use of the relation $Q(n) = \Gamma_r [S_a(n)-\sigma_r(n)]$, in which Γ_r is a constant known as the reach transmissivity, $\sigma_r(n)$ is the river stage measured from a high datum during time n, S_a (n) is



RELATION OF HYDRAULIC CONDUCTIVITY TO SAND SILT AND CLAY PERCENTAGES (JOHNSON, 1963) the depth to piezometric surface below the river at the time n measured from the same high datum, the influent seepage from a partially penetrating river to an aquifer has been derived by Morel-Seytoux and Daly. The same analysis has been extended to compute seepage loss from a hydraulically connected canal with an aquifer.

2.2 Statement of the Problem

A schematic section of a canal in a homogeneous and isotropic aquifer of infinite areal extent is shown in Fig. 2. The aquifer is initially at rest and the canal was dry. Water is conveyed in the canal and the depth of water in the canal is assumed to be constant over a long reach of the canal. It is required to find the unsteady seepage loss from the canal.

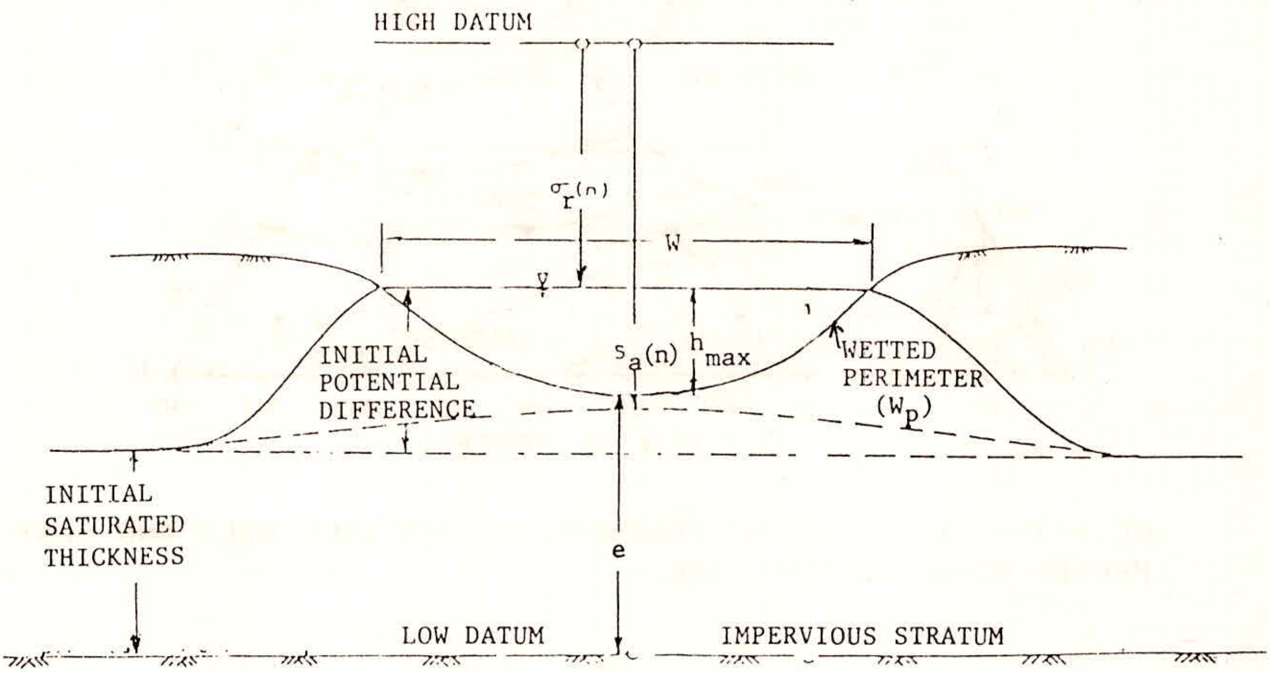


Fig. 2 Schematic Section of a Canal Hydraulically Conneted with Aquifer

2.3 Analysis

The following assumptions are made for the analysis:

(i) The flow in the aquifer is in the horizontal direction and is governed by one dimensional Boussinesq's equation.

(ii) The time parameter is discrete. Within each time step, the the seepage rate is a constant but it varies from step to step.

(iii) The seepage is linearly proportional to the difference in the potentials at the canal boundary and in the aquifer below the canal bed.

The differential equation which governs the flow in the aquifer is

$$T \frac{\partial^2 s}{\partial x^2} = \Phi \frac{\partial s}{\partial t} \tag{1}$$

in which

s = the water table rise in the aquifer,
T = transmissivity, and
Φ = storage coefficient of the aquifer.

The aquifer being initially at rest condition, the initial condition to be satisfied is :

$$s(x, 0) = 0. \tag{2}$$

The boundary conditions to be satisfied are:

$$s(\infty, t) = 0. \tag{3}$$

At the canal and the aquifer interface loss from the canal to the aquifer takes place in a manner similar to that from an overlying bed source to an underlying aquifer through an intervening aquitard. The canal resistance and the aquitard resistance are analogous. The recharge, which can be assumed to be linearly proportional to the potential difference between the canal and the aquifer under the canal bed, is to be incorporated at the canal boundary.

Solution to the problem has been obtained using a discrete kernel approach. The basic solution given by Polubrinova-Kochina for rise in piezometric surface due to continuous recharge from a strip source has been used in the analysis. If recharge takes place at unit rate per unit length of the canal and if the width of the canal at the water surface is W, the rise in piezometric surface at a distance x from the centre of the canal is:

$$s(x, t) = F(x, t) - (x^2 + 0.25\, W^2)/(2\, W\, T) \quad \text{for } x \le W/2;$$

$$= F(x, t) - \sqrt{(x^2)}/(2\, T) \quad \text{for } |x| \ge W/2 \tag{4}$$

in which

$$F(x, t) = t/(2\Phi W)\, [\, \text{erf}\{(x+0.5W)/\sqrt{(4\alpha t)}\} - \text{erf}\{(x-0.5W)/\sqrt{(4\alpha t)}\}\,]$$

$$+ 1/(4TW)\, [(x+0.5W)^2\, \text{erf}\{(x+0.5W)/\sqrt{(4\alpha t)}\} -$$

$$(x-0.5W)^2\, \text{erf}\{(x-0.5W)/\sqrt{(4\alpha t)}\}\,]$$

$$+ (\alpha t/\pi)^{1/2}/(2\, T\, W)\, [(x+0.5W)\exp\{-(x+0.5W)^2/(4\alpha t)\} -$$

$$(x-0.5W)\exp\{-(x-0.5W)^2/(4\alpha t)\}] \tag{5}$$

and $\alpha = T/\Phi$.

Let the rise in piezometric surface at x at the end of n^{th} unit time step due to recharge that occurred at unit rate from unit length of canal during the first time step only be designated as $\delta\,[x,\,n]$. The discrete kernel coefficients are related to the unit step response function as given below:

$$\delta\,[x,\,n] = F(x,\,n) - F(x,\,n-1); \qquad n > 2 \tag{6}$$

$$\delta\,[x,\,1] = F(x,\,1) - \sqrt{}\,(x^2)/(2T)\,; \qquad |\,x\,| > W/2 \tag{7}$$

$$\delta\,[x,\,1] = F(x,\,1) - (0.25\,W^2 + x^2)/(2\,T\,W)\,; \quad |\,x\,| < W/2 \tag{8}$$

Dividing the time span into discrete time steps, and assuming that, the recharge per unit length is constant within each time step but varies from step to step, the rise in piezometric surface below the centre of the canal due to time variant recharge taking place can be written as:

$$s(0,\,n) = \Sigma^{n}_{\gamma-1}\,q\,(\gamma)\,\delta(0,\,n-\gamma+1\,) \tag{9}$$

in which $q(\gamma)$ is the rate of seepage per unit length of canal per unit time which takes place during time step γ.

The recharge from the canal during n^{th} time step can be expressed as:

$$q(n) = \Gamma_r\,[H - s(0,\,n)] \tag{10}$$

in which H is the initial potential difference causing flow. Substituting s(0, n) in equation (10) and simplifying

$$q(n) = \Gamma_r\,[\,H - \Sigma^{n}_{\gamma-1}\,q\,(\gamma)\,\delta(0,\,n-\gamma+1\,)] \tag{11}$$

Splitting the temporal summation into two parts, one part containing the summation up to $(n-1)^{th}$ time step and the other part containing the term pertaining to n^{th} time step and rearranging

$$q(n) = [H - \Sigma^{n}_{\gamma-1}\,\{q(\gamma)\,\delta(0,\,n-\gamma+1)\}]/[1/\Gamma_r + \delta(0,\,1)] \tag{12}$$

In particular for the first time step

$$q(1) = H/[1/\Gamma_r + \delta(0,\,1)] \tag{13}$$

Making use of equation (12), q(n) can be solved in succession starting from the first time step .

2.4 Results

The reach transmissivity constant can be evaluated using the following relation given by Herbert (1970)

$$\Gamma_r = k\ \pi / \log_e\{0.5\ (e + h_{max})/r_r\} \tag{14}$$

where

r_r = W_p / π = radius of the equivalent semicircular canal cross section;
e = saturated thickness of the aquifer below the canal bed;
h_{max} = maximum depth of water in the canal.

A sample computation of seepage losses from a canal is presented for the following set of data.

(i) W = width of the canal at the water surface= 60 m.
(ii) Side slope of canal=1:1.
(iii) Depth to initial water table from the canal bed=6 m.
(iv) Depth of water in the canal, h_{max} = 3 m.
(v) Transmissivity, T = 1000 sq.m per day.
(vi) Specific yield, Φ = 0.1 .
(vii) Initial saturated thickness of aquifer =1000m.
(viii) Hydraulic conductivity, k =1m/day.

From above data, the initial potential difference H=9m. The wetted perimeter, W_p, = 62.5m. Equivalent radius r_r = 19.9 m. The reach transmissivity = 0.97m /day. The variation of seepage from the canal are as given below:

Time in day	Seepage loss in m /day
1	5.567898
2	5.447787
3	5.358006
4	5.283993
5	5.220059
6	5.163275
7	5.111897
8	5.064792
9	5.021166
10	4.980447

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